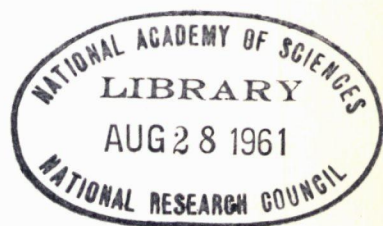


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Bulletin 282

***Influence of Stabilizers on  
Properties of Soils and  
Soil-Aggregate Mixtures***



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***Influence of Stabilizers on  
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Soil-Aggregate Mixtures***

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# Evaluation of Strength Properties of Several Soils Treated with Admixtures

RADNOR J. PAQUETTE and JAMES D. McGEE, respectively, Professor of Civil Engineering and Research Assistant, Engineering Experiment Station, Georgia Institute of Technology

This paper is a study of soil stabilization with various admixtures. Using compressive strength to evaluate stability, five selected soils of widely varying physical properties were stabilized with portland cement, a lime and fly ash mixture, phosphoric acid, and asphaltic cutback (RC-3).

Moisture-density tests were made with the soils and admixtures to determine the effect of the admixtures on standard Proctor maximum density and optimum moisture. Using this density and moisture data, samples 2.8 in. in diameter by 5.6 in. in height were statically compacted. After curing for 7 and 28 days, the samples were tested by an unconfined compression test and the triaxial test using a confining pressure of 20 psi. Additional tests were made on the soil-portland cement mixture for determining the effect of that stabilizer on the angle of internal friction and cohesion.

Results indicate that phosphoric acid slightly increased the density in all soils. Portland cement, lime-fly ash and RC-3 increased the density in the uniformly-graded soils. There was little effect from the addition of portland cement or RC-3 in the well-graded soils whereas lime-fly ash caused a marked reduction in density in these soils.

Strength tests indicated that portland cement was the most effective stabilizer in all soils giving high strength gains. The addition of portland cement also increased the angle of internal friction and cohesion. The lime-fly ash admixture and phosphoric acid caused slight increases in all soils. Some soils had a negligible strength increase with the addition of RC-3, whereas other soils indicated a reduction in strength.

●SOIL STABILIZATION with portland cement and other admixtures has become of great importance in recent years. The tremendous increase in vehicles and vehicle-miles has brought about a need for more highways and the ever-increasing truck traffic has created a need for more stable roads. With the available supply of high quality soil for base construction rapidly diminishing in many areas, the highway engineer is confronted with the problem of transporting suitable soils to the area or artificially producing a high quality soil by mixing the available soil with an admixture.

The use of various admixtures combined with soil has been widespread in many areas in recent years. This has been particularly true in the construction of secondary-type roads. Stabilization has proved its worth, but a problem which remains in many areas is the type and amount of admixture necessary to transform this unsuitable soil into a load-supporting base material. This paper concerns the comparative effectiveness of stabilization of several soils with various admixtures.

## MATERIALS

The soils chosen for this study are typical of some of the roadbuilding soils available

at various locations throughout Georgia. A description of these soils is given in Table 1 with the grain-size distribution shown in Figure 1. Soil I is a brownish, well-graded, clayey, silty sand; Soil II is a reddish-brown, uniform, silty, clayey sand; Soil III is a greyish-white uniform sand; Soil IV is a red, well-graded, silty, sandy clay; and Soil V is a yellowish-brown, well-graded, clayey, silty sand.

According to the Georgia Highway Department classification and usage, only Soil II would be suitable for base construction without treatment with aggregate or an admixture. Soils I, III and IV would be suitable for subgrade construction without treatment while Soil V would require treatment before using as a subgrade and would not normally be used for base construction even with treatment.

The admixtures studied were Type I normal portland cement, asphaltic cutback RC-3,

TABLE 1  
DESCRIPTION OF SOILS

Soil No. Location by County	I Carroll	II Effingham	III Camden	IV Fulton	V Fulton
Textural analysis					
% retained by wt.					
Sieve No. 10	3	0	0	3	2
Sieve No. 40	14	54	2	19	24
Sieve No. 60	37	68	7	28	36
Sieve No. 100	44	74	53	37	46
Sieve No. 200	62	83	92	46	55
Total Silt Sizes, %	21	2	3	22	24
Total Clay Sizes, %	6	11	—	27	14
Specific Gravity	2.67	2.63	2.69	2.70	2.69
Liquid Limit	13	14	—	29	37
Plastic Limit	—	—	—	23	—
Plastic Index	NP	NP	NP	6	NP
AASHO Classification	A-2-4(0)	A-2-4(0)	A-3-(0)	A-4-(4)	A-4-(2)
Ga. Hwy. Dept. Classification	C-1 Topsoil	A-1 Topsoil	A-1 Subgrade	1-B Embankment	II-A Embankment

a combination of line and fly ash, and phosphoric acid. An analysis of the portland cement and the fly ash is given in Table 2.

#### METHOD OF TESTING

The soils used were air-dried to a uniform moisture content and sieved through a No. 4 sieve with only the minus 4 material used in the tests.

Moisture-density tests conforming to standard ASTM and AASHO specifications were performed on each soil and each soil combined with the test increments of stabilizer. An exception was Soil III where no moisture-density tests were made with phosphoric acid.

Mixing was done with a mechanical mixer using a total mixing time of 10 min. The dry admixtures and soil were proportioned and mixed dry; then optimum water content for standard Proctor maximum density was added and mixed for the remainder of the 10 min. For the mixture with phosphoric acid, the acid was combined with the water before adding to the soil for mixing. The mixture with RC-3 was first mixed for 3 min with the optimum amount of water, then the asphalt was added and mixing continued for 7 more minutes.

Molding of all the soils and mixtures was done immediately after mixing except when RC-3 was used as the admixture. The soil and RC-3, after mixing, was allowed

to stand in the open air until it had a "tacky" feel before molding. Molding was done by static compaction in a 2.8-in. diameter mold compacting the sample to a height of 5.6 in. A predetermined weight of material to give the standard Proctor maximum density, as determined from the moisture-density curve, was placed in the mold and rodded before compaction. Compaction was accomplished by forcing the pistons in each end of the mold together until the 5.6-in. height was attained.

After molding, the samples were placed in polyethylene freezer bags and sealed to prevent any change in moisture conditions. Curing was done for 7 and 28 days in a

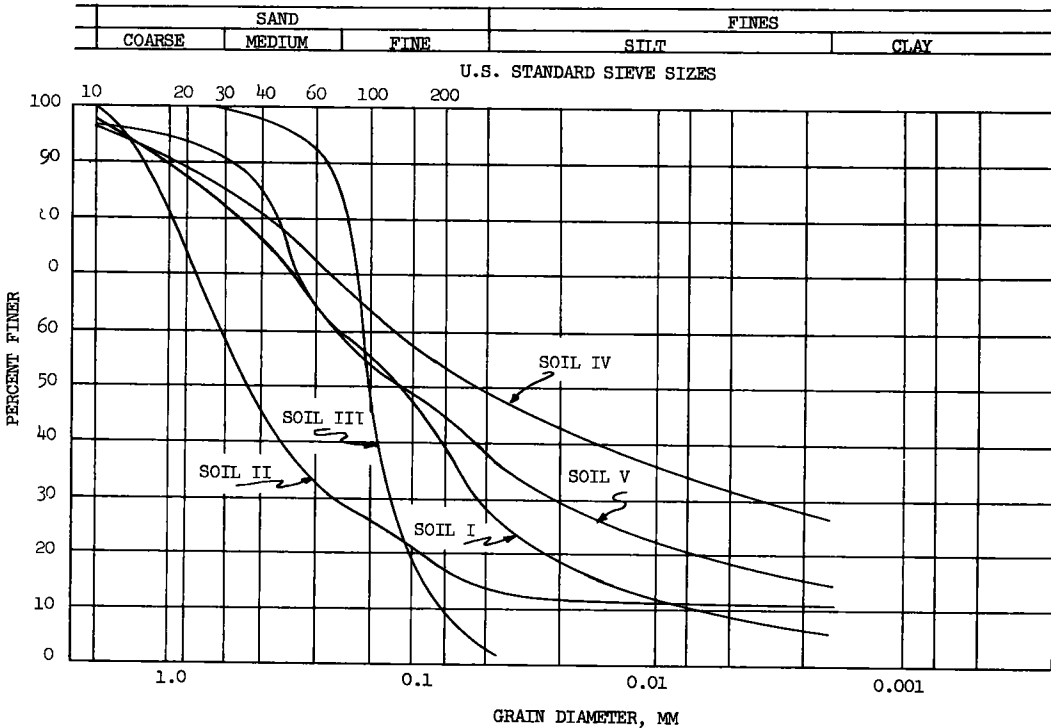


Figure 1. Grain-size distribution.

**TABLE 2**  
**ANALYSIS OF PORTLAND CEMENT AND FLY ASH**

	Portland Cement	Macon, Ga. Fly Ash	Columbia, S. C. Fly Ash
Chemical composition %			
Silicon dioxide, SiO <sub>2</sub>	20.46	41.40	45.92
Ferric oxide, Fe <sub>2</sub> O <sub>3</sub>	2.44	8.65	16.50
Aluminum oxide, Al <sub>2</sub> O <sub>3</sub>	5.90	21.05	32.00
Sulphur trioxide, SO <sub>3</sub>	2.08	1.16	0.84
Calcium oxide, CaO	62.87	—	—
Magnesium oxide, MgO	4.18	5.36	1.40
Carbon, C	—	1.66	2.32
Loss on ignition	1.38	3.12	2.24
Specific surface area			
Blaine (sq cm/gm)	3,464	3,427	1,760

moisture room with approximately 70 F and 90 percent relative humidity.

Compressive strength determinations were made by both the unconfined compression test and the triaxial test using a lateral confining pressure of 20 psi. All samples were tested in a moist condition as they were removed from the plastic bags. Twenty-eight clay samples of each soil with no admixture and with 6, 9, 12 and 15 percent portland cement were also tested triaxially using a confining pressure of 50 psi. Loading was at a rate of 0.05 in. per minute.

## EVALUATION OF TEST RESULTS

Testing of the soils with the various admixtures involved determining maximum dry density and optimum moisture and compressive strength. Compressive strength data of the soils and soil-portland cement mixtures were also evaluated to determine the cohesion and angle of internal friction. Various increments of each of the four admixtures were combined with each of the five soils. Portland cement was added in increments of 2 percent ranging from 2 to 12 percent; lime-fly ash was used on a basis of 75 percent soil and 25 percent lime-fly ash with the ratio of lime to fly ash varying by 1:1, 1:2, 1:5 and 1:9 ratio; phosphoric acid was added at 1 and 2 percent; the amount of RC-3 used was 3, 5 and 7 percent. All percentages of admixtures were based on the dry weight of the soil. The phosphoric acid was an 85 percent concentration and the percentages used were based on this concentration. Figures 2 through 6 show the relationship of admixtures and density.

### Effects of Admixtures on Maximum Dry Density and Optimum Moisture

The addition of phosphoric acid caused an increase in density in all soils. Optimum moisture of the soil-acid mixture remained approximately the same or decreased slightly as compared to the soils with no admixture. A part of the increased density may be attributed to a replacement of a portion of the moisture by the acid which has a higher specific gravity.

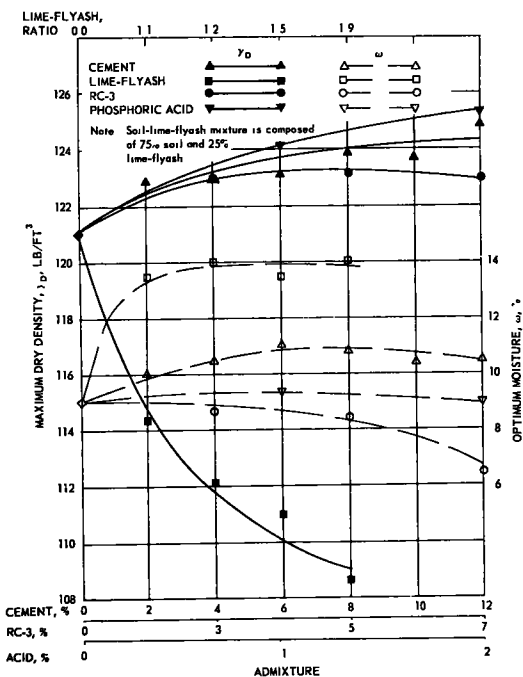


Figure 2. Relationship of maximum dry density and optimum moisture versus admixture for Soil I.

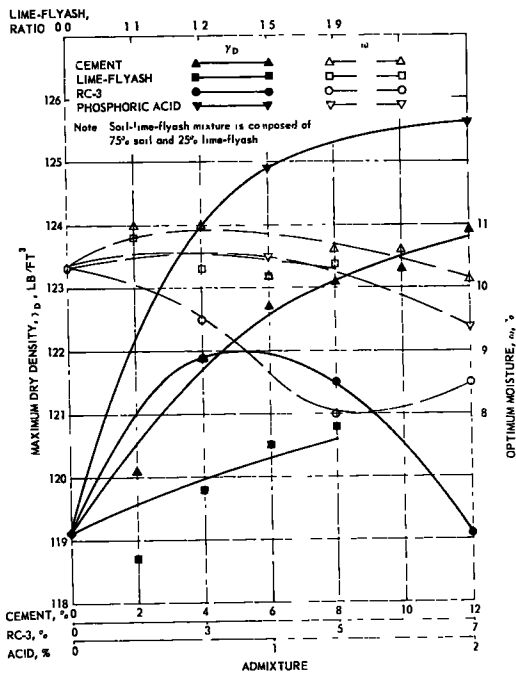


Figure 3. Relationship of maximum dry density and optimum moisture versus admixture for Soil II.

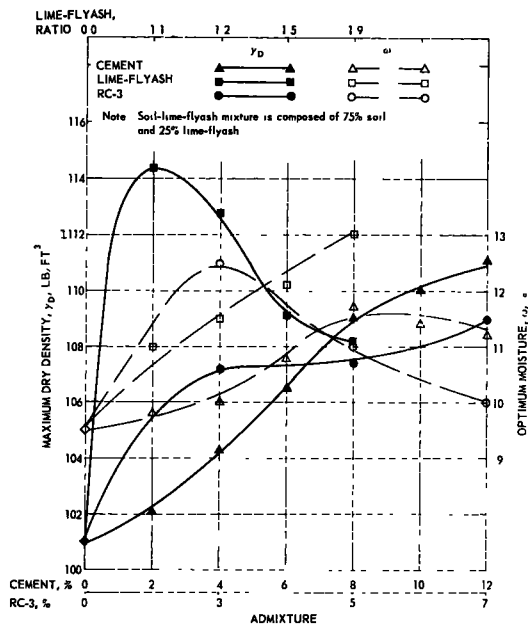


Figure 4. Relationship of maximum dry density and optimum moisture versus admixture for Soil III.

In Soils II and III, both uniform in gradation, the addition of portland cement or lime-fly ash increased the density. This increase in density was greater in Soil III which was the more uniform soil. The increased density is probably caused by the better gradation afforded by the addition of the finer particles of the admixtures. The only explanation for the higher density obtained in Soil III with the addition of lime-fly ash as compared to the addition of cement is that a greater amount of admixture was used in the case of lime-fly ash. Due to the much higher specific gravity of the cement, it would be expected to obtain the greatest density from this admixture. Optimum moisture was not critical in Soil III and varied only slightly with the addition of these two admixtures but in Soil II, optimum moisture increased with the addition of each of the admixtures. The greater amount of finer particles, as found in Soil II, would necessarily require a greater amount of moisture for compaction. Change in density of these two soils by the addition of RC-3 also can be attributed to better

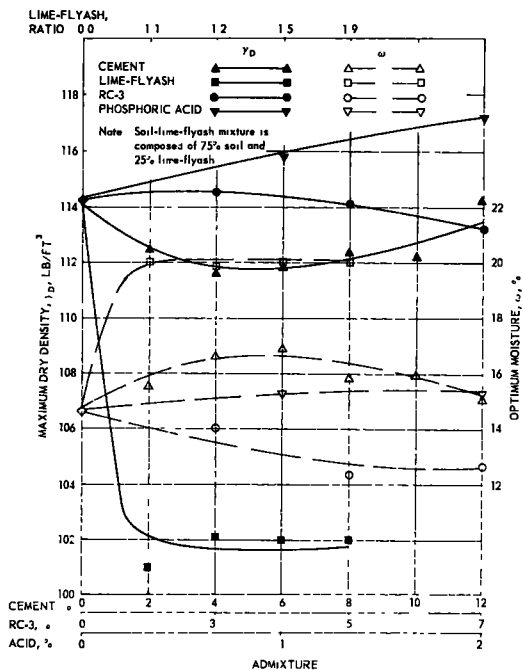


Figure 5. Relationship of maximum dry density and optimum moisture versus admixture for Soil IV.

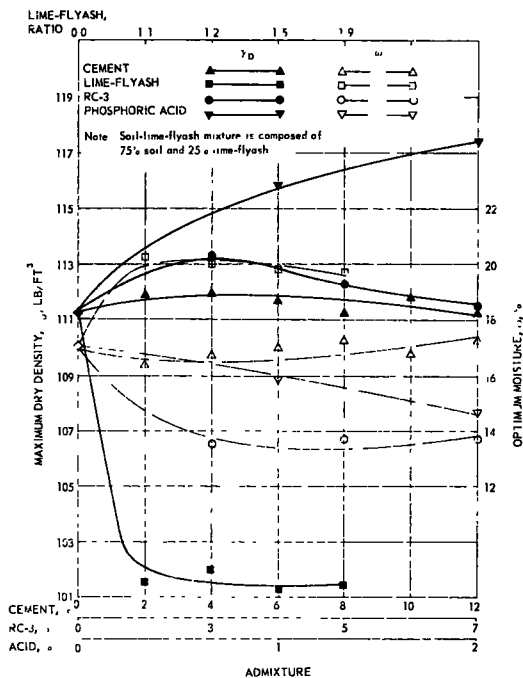


Figure 6. Relationship of maximum dry density and optimum moisture versus admixture for Soil V.

filling of voids and in the case of Soil II, where the density was reduced with the higher asphalt content, an overfilling of voids caused displacement of some soil particles.

Soils I, IV and V are all well-graded and the addition of portland cement should be of less benefit in increasing the density. This is substantiated from the density figures of these soils where it is shown that the cement had little effect on the density. From a gradation standpoint, lime-fly ash should have little effect on these soils but it is noted that a marked decrease in density was obtained. Probably this admixture created a greater amount of voids and the 25 percent addition of lime-fly ash caused a replacement of some soil particles.

The little change produced by the addition of RC-3 was probably due to the replacement of some of the moisture by the RC-3 as noted in the reduced optimum moisture for the well-graded soils.

### Effects of Admixtures on Compressive Strength

In evaluating the molded samples, only samples molded within 1 percent of optimum moisture were used. For each test, 4 samples were molded for unconfined compression at 7 and 28 days and 4 samples for triaxial testing at 7 and 28 days. For compressive strength evaluation, only the values which were within 10 percent of the average of the other samples were used. In most instances the results were consistent and represent the average of 4 samples tested.

Figures 7 through 16 depict the variation in compressive strength as brought about by the addition of the different admixtures. In all of the soils tested, portland cement produced the greatest improvement in strength. All soils reacted favorably with the portland cement and no problem was encountered with the soil-cement mixture

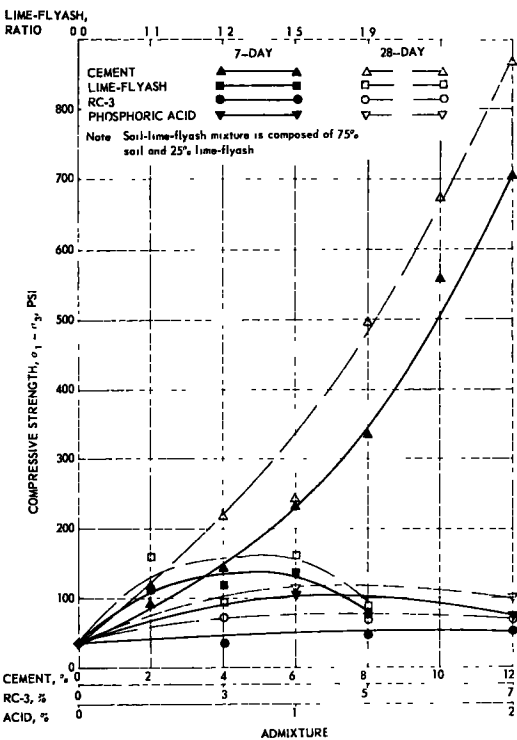


Figure 7. Relationship of confined compressive strength and admixture for Soil I.

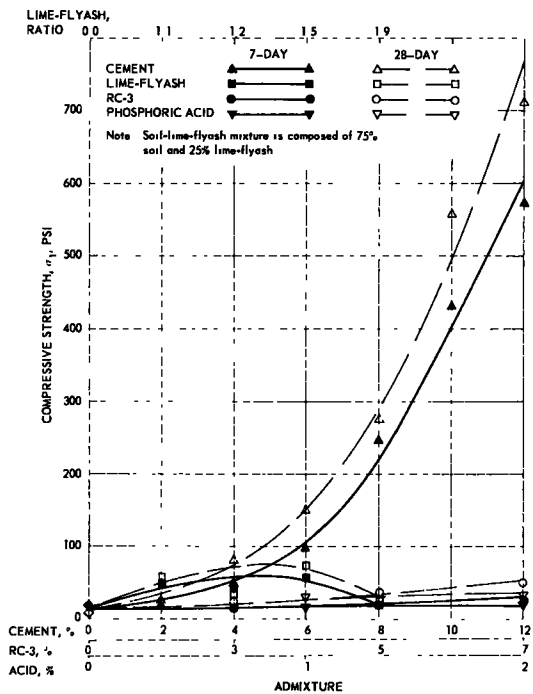


Figure 8. Relationship of unconfined compressive strength and admixture for Soil I.

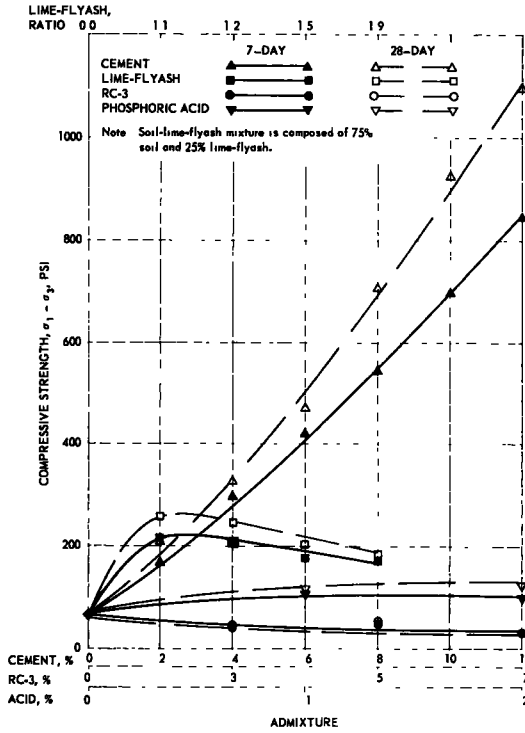


Figure 9. Relationship of confined compressive strength and admixture for Soil II.

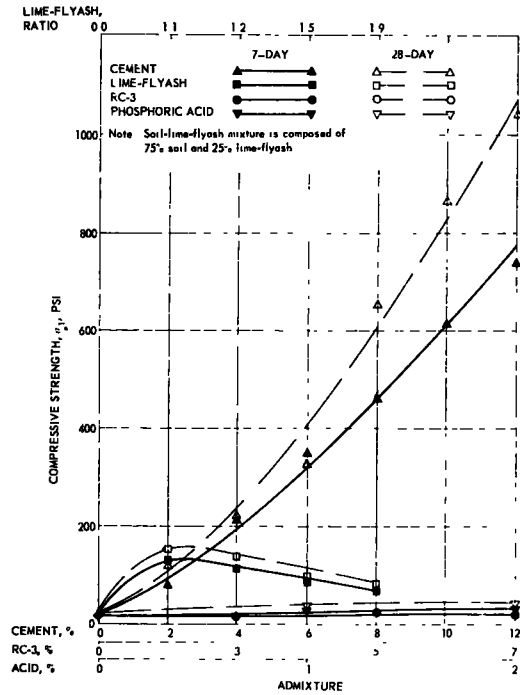


Figure 10. Relationship of unconfined compressive strength and admixture for Soil II.

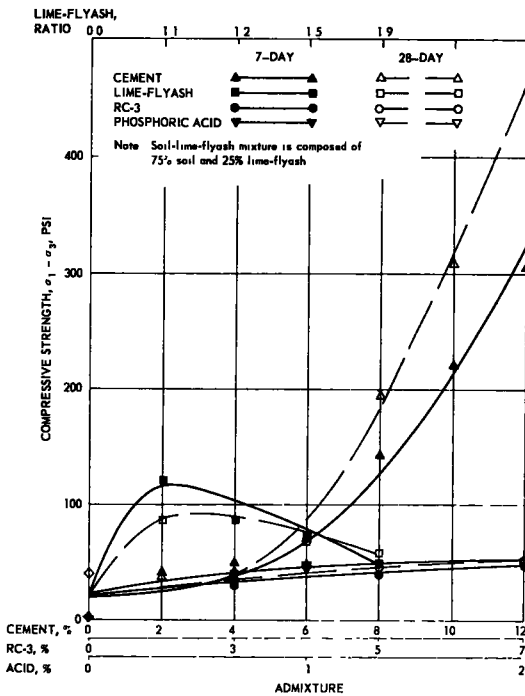


Figure 11. Relationship of confined compressive strength and admixture for Soil III.

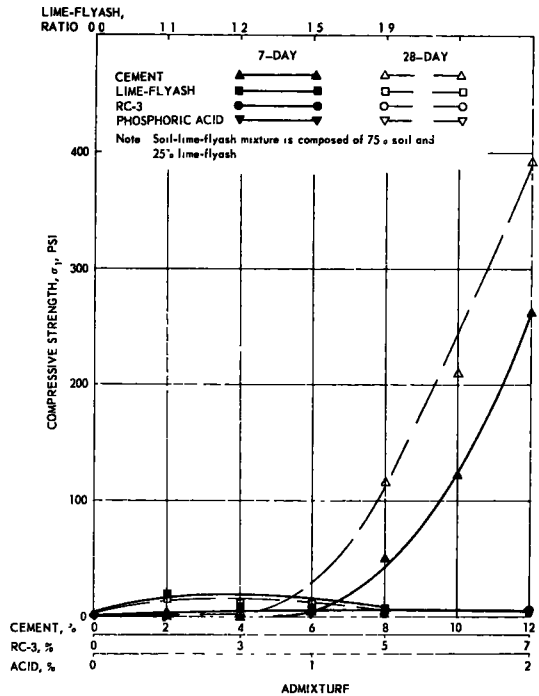


Figure 12. Relationship of unconfined compressive strength and admixture for Soil III.

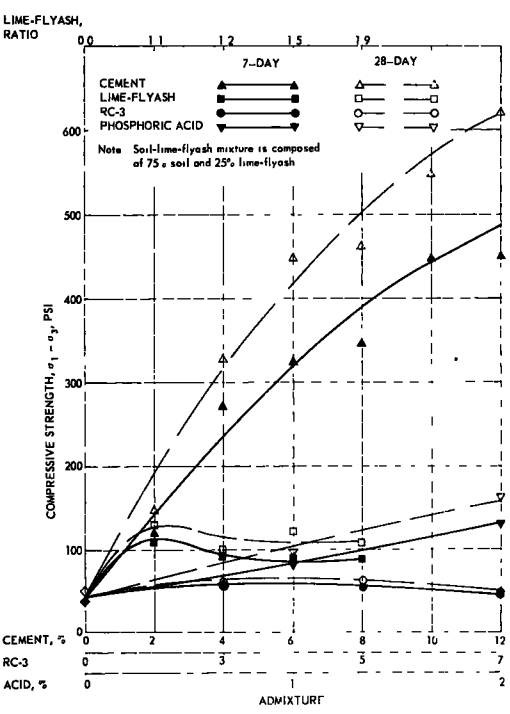


Figure 13. Relationship of confined compressive strength and admixture for Soil IV.

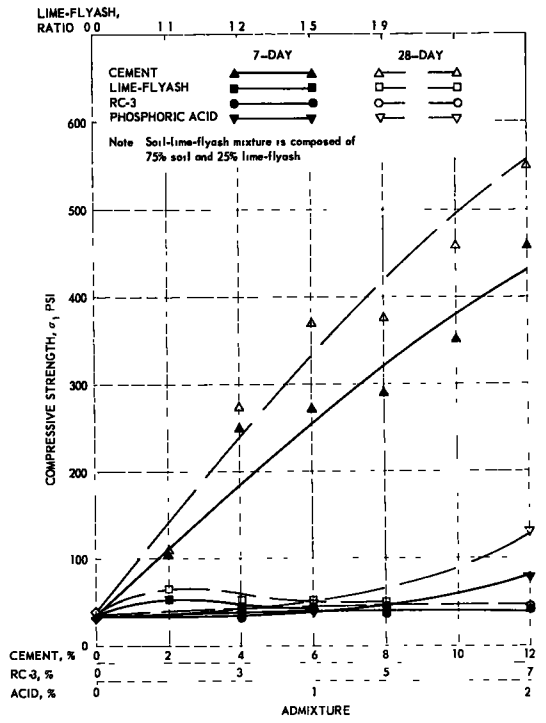


Figure 14. Relationship of unconfined compressive strength and admixture for Soil IV.

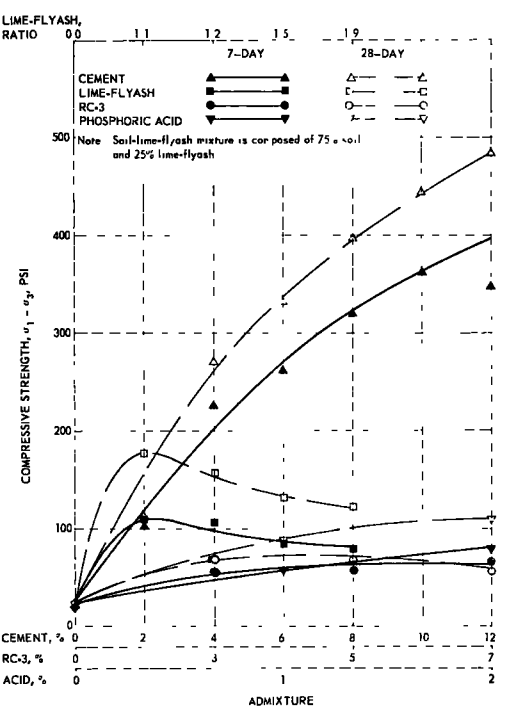


Figure 15. Relationship of confined compressive strength and admixture for Soil V.

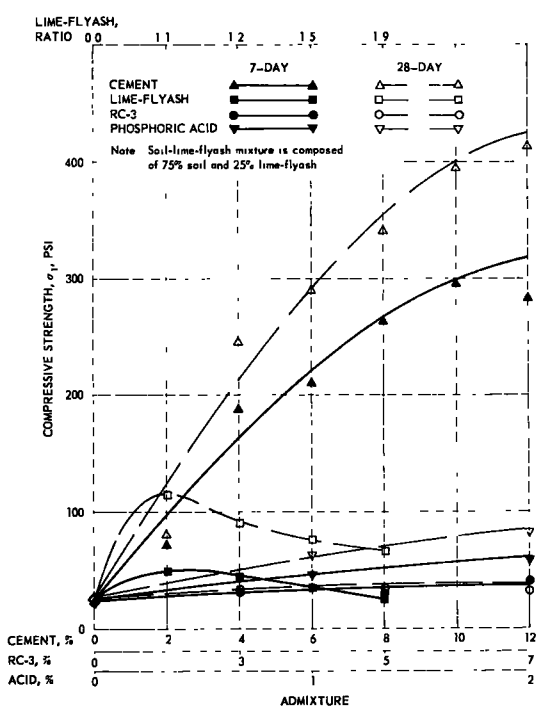


Figure 16. Relationship of unconfined compressive strength and admixture for Soil V.



hardening. The lime-fly ash mixture improved the strength of all soils approximately 300-400 percent. Phosphoric acid caused an increase in strength in the finer-grained soils with the greatest benefit in the soil with the highest clay content. From a dry strength standpoint, RC-3 gave little or no benefit, even causing a decrease in strength in one soil.

### Effect of Amount of Admixture on Strength

Increasing the percentage of portland cement caused an increase in strength. Strength gains were noted in the soil-cement mixtures with as low as 2 percent cement. The amount of increase appears to vary with the fineness of the soil with the finer soils showing greater improvement at lower percentages while the coarser materials had a greater rate of strength increase at the higher percentages. Soil III differed somewhat in that little strength improvement was noted below 6 percent cement. Above 6 percent cement the strength of the soil increased very rapidly.

The highest gain in strength by using lime-fly ash occurred with a ratio of 1:1 lime to fly ash. The silty soil, Soil I, differed in that little change was noted in the strength gain until a 1:9 lime-fly ash ratio was used, where this strength gain decreased. The addition of 2 percent phosphoric acid was of greater benefit than 1 percent except in Soil I where the lower percentage gave slightly higher strength.

Varying the amount of RC-3 had only a slight effect on the strength. Where a strength increase occurred the 3 and 5 percents were usually the most beneficial.

### General Discussion of Strength Results

The improvement in compressive strength brought about by the addition of portland cement was as expected. Of the admixtures used, the hydration of portland cement should produce the best cementing bond of the soil particles. Although the lime-fly ash

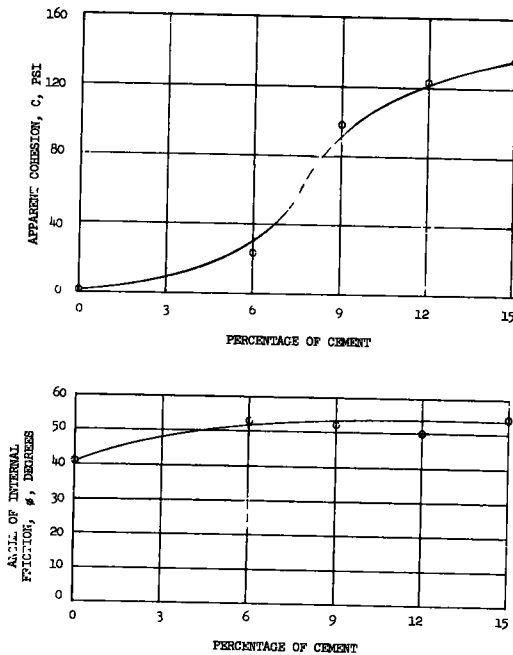


Figure 17. Apparent cohesion and angle of internal friction versus cement content for Soil I.

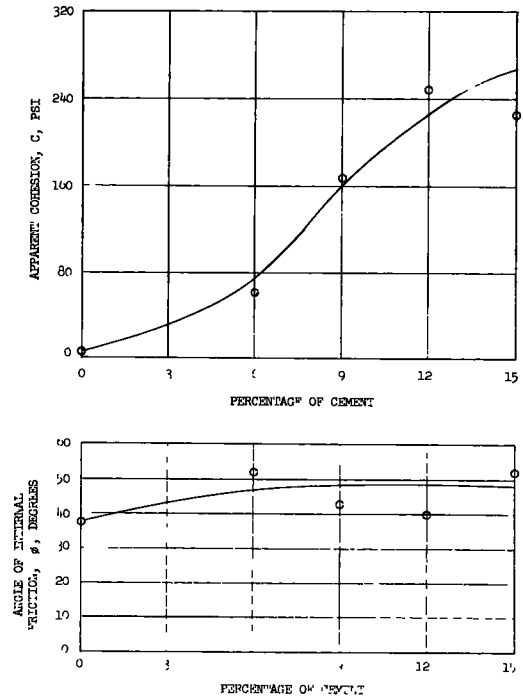


Figure 18. Apparent cohesion and angle of internal friction versus cement content for Soil II.

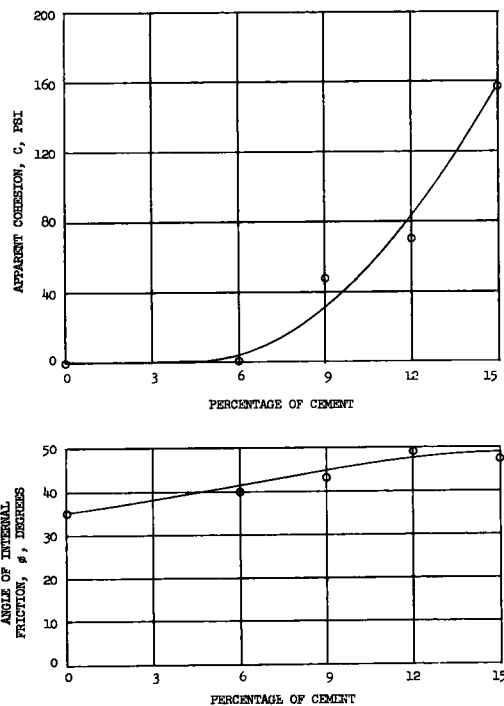


Figure 19. Apparent cohesion and angle of internal friction versus cement content for Soil III.

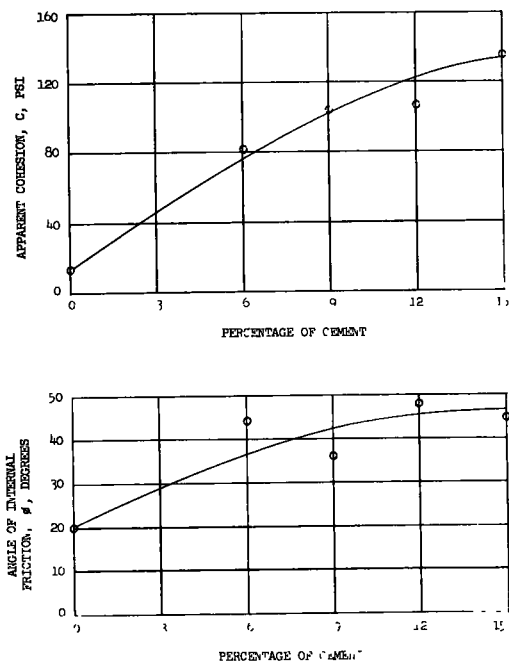


Figure 20. Apparent cohesion and angle of internal friction versus cement content for Soil IV.

admixture produces a cementing action similar to portland cement, certainly this is not as strong a bond and this is substantiated by the test results. It may be expected then, in any soil which contains no elements detrimental to the "hardening" of the cement, to obtain a greater strength with portland cement as compared to lime and fly ash. Another problem in the use of fly ash is the vastly dissimilar cementing properties obtained from fly ash produced at different plants. The non-linear increase in strength brought about by the increasing percentages of cement is explained by the gradation of the soil grains, especially in the sand range. A greater rate of increase in strength at the higher cement contents was noted in Soils I, II and III, where Soils II and III are uniformly graded and Soil I, although well-graded in its entirety is fairly uniform in the sand range. A soil with strong grains should become stronger with a greater amount of cementing agent to bond these particles together.

In Soils IV and V, the non-linearity in strength increase is also evident but the

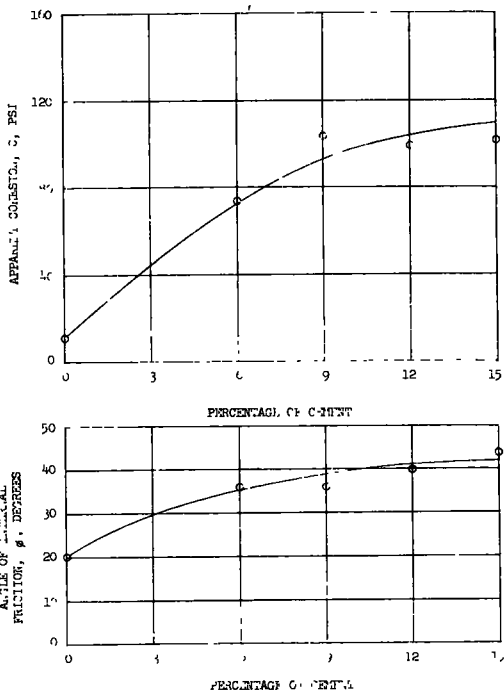


Figure 21. Apparent cohesion and angle of internal friction versus cement content for Soil V.

greatest rate of strength increase is obtained at the lower cement contents. This is probably due to the breaking of the softer grains of these two soils at the higher cement contents.

The low strengths obtained with RC-3 as an admixture could be attributed to the low "cohesion" afforded by asphalt. In the instances where the strength was reduced with RC-3, the cohesive bond of the soil was probably destroyed and replaced by a lower cohesion from the asphalt.

It has been proposed that the stabilizing properties of phosphoric acid are due to a molecular attraction of the phosphorous ions with certain ions in the soil, possibly aluminum. This has not been investigated in this study and no further explanation is given.

### Effects of Portland Cement on Cohesion and Angle of Internal Friction

A Mohr's envelope was plotted for each soil and the soils combined with 6, 9, 12 and 15 percent portland cement. The variation in cohesion and angle of internal friction are shown in Figures 17 through 21.

As shown in these figures, the angle of internal friction and cohesion increases sharply with the addition of portland cement with the exception of Soil III, the fine uniform sand, where the increases are less noticeable at the low percentages. An interesting point here is the almost constant angle obtained as the cement content increases. While the cohesion continues to increase with increasing cement content up through 15 percent, the rate of increase begins to decrease at about 9 percent. Soil III is an exception here but it would be expected to have the same tendency at a higher cement content.

These findings would substantiate the changes obtained in physical properties of a soil with a small addition of portland cement such as reduced PI, LL and volume changes. The cementation is acting to produce a larger soil grain from several small grains thereby altering the properties of the soil.

### CONCLUSIONS

The following conclusions have been reached as a result of this study:

1. The effects of admixture on maximum dry density and optimum moisture vary with the soil gradation: (a) the effects of admixtures on density are more pronounced in the uniform soils where an increased density can be expected; and (b) less variation occurred in the well-graded soils with the exceptions of a marked reduction in density due to the addition of lime-fly ash and a substantial increase in density with the addition of phosphoric acid to the more clayey soil.
2. The improvement in strength varied with the soil and particular admixture: (a) portland cement was by far the most beneficial stabilizing agent producing a large strength gain in all soils; (b) the addition of 25 percent lime-fly ash improved the strength of all soils; (c) phosphoric acid caused a nominal increase in strength of all soils with the greatest benefit in the more clayey soil; and (d) the addition of RC-3 caused negligible strength increases and in some instances caused a strength reduction.
3. The variation in strength in most cases depends on the amount of admixture used: (a) increasing the percentage of portland cement increased the strength non-linearly; (b) the smallest percent of fly ash in the lime-fly ash mixture (that is, the 1:1 ratio), gave the greatest strength improvement except in one soil where little change was noted from a 1:1 to 1:5 ratio; (c) the higher percentage of phosphoric acid produced a greater benefit; and (d) the amount of RC-3 used was negligible.
4. Cohesion and the angle of internal friction were increased by the addition of portland cement: (a) the angle of internal friction increased sharply with small amounts of cement to approximately a constant angle with increasing cement content while the cohesion increased as the cement content increased; and (b) as the rate of increase in cohesion decreases at higher cement contents and the angle is approximately constant, the efficiency of high cement contents to obtain higher strength may be considerably reduced.

### ACKNOWLEDGMENTS

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# An Investigation of Soil Waterproofing And Dustproofing Materials

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The capabilities of a number of materials to waterproof and dustproof a lean clay soil are evaluated by means of field and laboratory studies. The field tests were conducted on a test section composed of 16 adjoining panels, 13 of which were surfaced with a 3-in. layer of soil mixed with one of the selected materials; the other three, used as controls, comprised two untreated soil panels and one provided with a bituminous surface. Traffic was applied to the test section at intervals during a 13-month period of exposure to weather; the amount of material abraded by the traffic was measured to determine dustproofing effectiveness of the various treatments. Observations of moisture changes in the compacted soil base under each of the surfaces indicated the degree of waterproofing provided. Several materials were found to be fairly effective both as waterproofers and dustproofers. In general, the materials displaying superior waterproofing ability also were the most effective in controlling dust. Results of laboratory tests to determine waterproofing ability of the various materials were found to correlate well with field data.

● **MOST NATURAL** fine-grained soils, when compacted to a relatively high density at an appropriate water content, are capable of providing a firm soil surface that is quite satisfactory for construction roads, secondary roads, and military roads and airfields for combat operations. The usefulness of such construction is limited, however, in that a muddy, slippery, and often impassable condition can occur during wet weather, and excessive dust may develop in dry weather. Even relatively small amounts of dust can greatly increase maintenance requirements for engines and other mechanisms and, although the dust condition seldom becomes so severe as to prohibit operations completely, it may reduce visibility to the point that operations become hazardous.

An agent that could be applied readily to the soil to render it immune to the deteriorating effects of water, desiccation, and traffic abrasion would be of material value. The essential requirement for such a material is that it be capable of retaining in a naturally stable or artificially stabilized soil an adequate stability condition; improvement of soil strength characteristics, although advantageous, is not a primary objective. It is desirable that a treatment with such a waterproofing and dustproofing material be effective in a relatively thin layer, preferably not exceeding 6 in. Ideally, the additive should be effective when applied in quantities of 5 percent (by weight of dry soil) or less and should retain its effectiveness under the expected traffic for at least one full cycle of seasonal change without additional treatment. The capability of being applied as a penetration treatment, although extremely advantageous, is not considered to be an essential requirement.

In 1956, prior to the initiation of a laboratory and field investigational program, all available reports of soil waterproofing and/or dustproofing studies conducted by the U. S. Army Corps of Engineers and by other agencies were studied and a summary review of this literature was prepared (1). The information from this review and from soil stabilization studies made by the Corps of Engineers provided the basis for selec-

tion of the materials evaluated in the study reported herein.

The primary purposes of this investigation were to evaluate, by means of field tests, several materials as waterproofing and dustproofing agents for soil under varied weather conditions and a controlled amount of traffic over a 1-yr period, and to develop correlative laboratory evaluation tests to supplement the field studies.

### FIELD TEST PROGRAM

The field test program consisted of traffic tests and related measurements and ob-

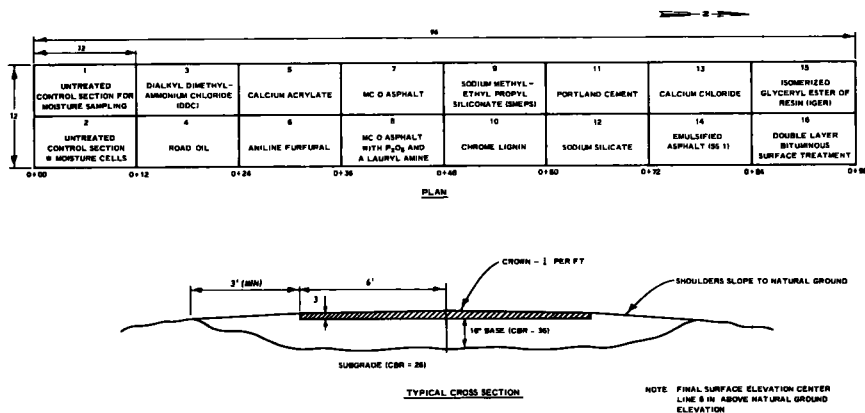


Figure 1. General layout of test section.

servations on a test section composed of 16 adjoining panels of soil arranged in two traffic lanes (Fig. 1). A loessial soil typical of those found in the lower Mississippi Valley was used in all phases of construction of subgrade, base, and treated surfaces. It is an inorganic lean clay and is classified as CL according to the Unified Soil Classification System. It has an average liquid limit of 38, an average plasticity index of 16, and a specific gravity of 2.70. The maximum dry density of the soil resulting from the standard Proctor compaction effort is 108 pcf at an optimum water content of 17 percent. Grain-size data indicate approximately 97 percent of the soil to be finer than 0.074 mm (No. 200 sieve) and approximately 25 percent finer than 0.005 mm.

Each of 13 panels was treated with a different waterproofing and dustproofing agent. The additives and a description of each are given in Table 1. Two panels were of untreated soil, and the remaining panel consisted of a compacted soil surface on which was applied a conventional double-layer bituminous surface treatment. The latter three panels were used as controls which were anticipated to be representative of minimum and maximum waterproofing and dustproofing effectiveness.

The topsoil in the test area was removed to a depth of 15 in. below natural ground surface. The soil at the bottom of the excavation had a CBR value of 26. The excavation was refilled with soil placed in four lifts, each compacted by means of a rubber-tired roller. This resulted in a 16-in. base having an average CBR of 36 at 10.8 percent water content and 101.9 pcf dry density. The subgrade and base were considered strong enough to assure no failures under the intended traffic unless the surface permitted the ingress of excessive amounts of water.

Nylon electrical-resistance moisture units (2) were installed in the untreated base under each panel to detect trends of moisture change. The units were installed approximately 4 and 12 in. below the top of the final treated surfaces. Temperature recording units also were installed to obtain soil temperature readings by which the measured moisture-unit resistances could be corrected to a common temperature.

The preparation of all the treated surface panels, with the exception of the road oil (panel 4), was similar. The additives were blended with previously processed and stockpiled soil by means of a garden rotary tiller. The mixing was accomplished off-site. The liquids content of the admixtures were controlled to obtain optimum for compaction. The treated soil was placed and compacted on the test section base course in the form of panels 12 ft long by 6 ft wide by 3 in. deep. In the construction of panel 4, a light-grade road oil was applied as a penetration treatment on a 3-in. compacted, untreated-soil panel. The control panels were of compacted, untreated soil, on one of which (panel 16) was constructed a double-layer bituminous surface. A view of the completed test section is shown in Figure 2.

Traffic was applied to the test section by a military 5-ton truck having a load of 16,000 lb on each of the two rear axles. The dual 11.00-20 by 12-ply tires were inflated to a pressure of 50 psi. The truck was operated at a speed of 5 to 7 mph with the center line of each wheel path coincident with the center line of each row of panels. All passes of the vehicle followed the same track.

Traffic was first applied to the test section two weeks after completion of construction. During the 13-month exposure period, seven traffic tests were conducted. The dates of these were 26 July, 10 August, 20 September, and 27 December 1956, and 17

TABLE 1  
AGENTS AND CHEMICALS USED IN FIELD TESTS

Panel No.	Additive	Description	Quantity Used in Percent of Dry Soil Wt
3	Dialkyl dimethyl-ammonium chloride (DDC)	Surface-active organic salt, dispersible in water	0.5
4	Road oil	Blend of a medium volatile distillate and a nonasphaltic, viscous, petroleum base	— <sup>a</sup>
5	Calcium acrylate (precatalyzed)	A water-soluble monomer that polymerizes in the presence of a suitable catalyst system	7.0
6	Aniline-furfural	Organic compounds which when combined interact to form a resin	3.34 <sup>b</sup>
7	MC-O asphalt	A blend of a high-penetration asphalt and kerosene	10.0 <sup>c</sup>
8	Modified MC-O asphalt (with phosphorus pentoxide and a lauryl amine)	A blend of a high-penetration asphalt and kerosene A highly deliquescent inorganic compound A surface-active, fatty, nitrogenous, water-dispersible compound	10.75 <sup>d</sup>
9	Sodium methyl-ethyl propyl silicate (SMEPS)	An aqueous solution of a sodium salt of a silicone	1.0
10	Chrome-lignin	A combination of a hexavalent chromium compound and a resinous waste product of the sulfite process of the paper industry	5.0
11	Portland cement	Commercial Type I	3.0
12	Sodium silicate	An aqueous solution of 30 percent concentration of sodium silicate	14.5
13	Calcium chloride	A deliquescent, crystalline, inorganic salt	1.0
14	Emulsified asphalt (SS-1)	A colloidal dispersion of a high-penetration asphalt in an emulsifying agent	8.4 <sup>e</sup>
15	Isomerized glyceryl ester of resin (IGER)	A hydrophobic, polymer, emulsion-type resin	5.0
16	Bituminous surface	Asphalt cement, 100-150 penetration	—

<sup>a</sup>Applied as penetration treatment at rate of 1 gal/sq yd. <sup>b</sup>2.14 percent aniline to 1.20 percent furfural. <sup>c</sup>50 percent asphalt content. <sup>d</sup>10 percent MC-O asphalt; 0.50 percent phosphorus pentoxide; 0.25 percent lauryl amine. <sup>e</sup>60 percent asphalt content.

January, 17 June, and 7 August 1957. Each traffic test consisted of 40 passes of the vehicle; thus, the panels were subjected to a total of 280 passes during the test period. It should be noted that the time interval between traffic tests was not uniform. The decision to conduct a traffic test was dependent on the weather conditions, the ideal situation consisting of a wet period followed by a sufficiently long dry period for the treated surfaces to have dried to the extent that dust would be created by the traffic. Immediately prior to each traffic test, the test section was swept clean of all loose material. This was done to remove any material that might have been eroded by weathering between traffic tests as well as any material abraded during preceding tests.

## TEST RESULTS

The data-collection program included visual observations of the behavior of the

surface during traffic, collection of abraded material after each traffic test for dust analysis, periodic moisture-unit readings to determine changes in the subgrade water content, and occasional direct sampling of the surface and compacted soil base for density and water content determinations. In addition, daily rainfall data were obtained with a standard rainfall gage located a short distance from the test site.

#### Observations of Panel Behavior During Test Period

During the two-week period between the completion of construction and the initial application of traffic, no rain fell on the test section. Some surface cracking due to shrinkage had occurred during this interval but was not severe in any of the panels. Close observation of the test section during the first few passes of the test vehicle in the first traffic test revealed only slight surface deflection under the load for all panels except the modified MC-O asphalt. Here, the deflections were greater, and shoving and rutting of the surface were evident. The early failure of this panel was found to be the result of incomplete curing of the treated surface. The panel was repaired with unmodified asphalt-treated soil and withstood subsequent traffic without distress. The second traffic test followed a two-week period during which a total rainfall of about 1 in. occurred. The only panel showing signs of distress at the end of this test was the one treated with calcium chloride (panel 13). Some deflection and cracking were noted at the outer edges of the tire path.

However, rutting of this panel was not severe and repairs were not necessary. No additional failures or distress were observed on any of the test panels during the five subsequent traffic tests.

From visual observations, it was evident that some panels resisted abrasion better than others. Similarly, progressive deterioration of the surfaces was evident during the course of the test program, apparently as a result of exposure to the weather. Observations of the panels immediately after periods of rainfall and during prolonged dry

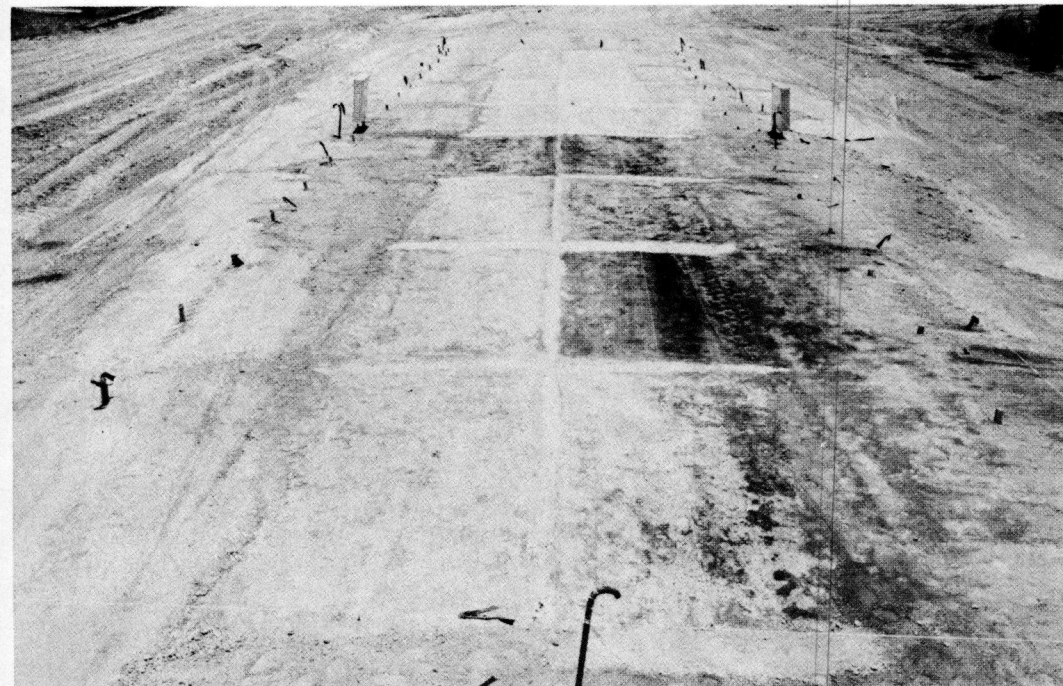


Figure 2. Completed test section, untreated panels in foreground.



periods showed considerable differences in response to alternate wetting and drying. Certain panels retained a dry appearance following a heavy rainfall, with little or no apparent swelling or reduction in strength of the surface. Other panels appeared wet; some of these retained a fairly high surface strength, whereas the remainder had softened considerably.

**Data Collection**

The material abraded from each panel during each traffic test was collected by means of a common, tank-type, home vacuum cleaner. To afford a uniform area of pickup, a canvas template with a 4- by 2.5-ft rectangular section cut out of it was used. The template was positioned directly on the traffic path, and the abraded material was collected as shown in Figure 3. The material was weighed and its water content and particle-size distribution determined. The quantities of abraded material obtained from the various panels after each traffic test are given in Table 2. The over-all average water content of the abraded material was 1.7 percent, ranging from a low of 0.9 percent to a high of 3.1 percent.

The electrical resistivities of the nylon moisture units installed under each panel were read at frequent intervals during the test period. Also, actual water contents were determined on soil samples taken from the 4- and 12-in. depths in the compacted soil base. The direct-sampling data are given in Table 3. A field calibration curve for each moisture unit was developed from the direct-sampling data. Using these curves, the water contents in the base under each test panel were estimated from the moisture-unit resistance readings. These data and the direct sampling results for the compacted soil base are plotted in Figures 4a through 4c. Direct sampling of the treated surface layers for water content and density determination was accomplished three times during the field investigation. The surface water contents and densities are given in Table 4.

Daily rainfall data are plotted in each of Figures 4a through 4c. The total rainfall for the 13-month period of the test was 54.93 in. The distribution of rainfall by months indicated a slightly below-normal amount for the months of January, February, and March 1957, whereas a record high rainfall was measured in June 1957.

**ANALYSIS OF FIELD DATA**

The analysis of data and subsequent evaluation of the various treated surfaces tested during this field investigation consist primarily of a comparison of the capabilities of each treatment in (a) resisting

TABLE 2  
AMOUNT OF ABRADED MATERIAL COLLECTED FROM TEST PANELS AFTER EACH TRAFFIC TEST

Panel No.	Additive	26 July 1956			10 Aug. 1956			20 Sept. 1956			27 Dec. 1956			17 Jan. 1957			17 June 1957			TOTAL WT.		
		Total Abraded Than No. 200 Sieve	Mt Finer Ml (Dry)	Total Abraded Than No. 200 Sieve	Mt Finer Ml (Dry)	Total Abraded Than No. 200 Sieve	Mt Finer Ml (Dry)	Total Abraded Than No. 200 Sieve	Mt Finer Ml (Dry)	Total Abraded Than No. 200 Sieve	Mt Finer Ml (Dry)	Total Abraded Than No. 200 Sieve	Mt Finer Ml (Dry)	Total Abraded Than No. 200 Sieve	Mt Finer Ml (Dry)	Total Abraded Than No. 200 Sieve	Mt Finer Ml (Dry)	Total Abraded Than No. 200 Sieve	Mt Finer Ml (Dry)	Total Abraded Than No. 200 Sieve	Mt Finer Ml (Dry)	
1	Untreated soil	1,148	690	3,481	1,855	2,783	1,410	3,058	1,710	1,989	1,355	7,824	3,890	3,970	2,235	3,890	3,970	2,235	3,890	3,970	2,235	
2	Untreated soil	239	140	3,103	1,440	2,342	1,092	3,000	1,473	1,831	1,473	7,836	3,550	4,631	2,400	3,550	4,631	2,400	3,550	4,631	2,400	
3	DDC	166	92	267	138	582	292	644	262	539	284	1,065	572	453	208	572	453	208	572	453	208	
4	Road oil	390	148	582	224	631	295	710	202	676	229	1,200	480	1,083	424	480	1,083	424	480	1,083	424	
5	Calcium acetate	134	40	299	103	1,731	865	2,288	897	2,016	1,024	6,357	2,555	2,662	1,091	2,555	2,662	1,091	2,555	2,662	1,091	
6	Calcium chloride	150	148	503	236	958	492	874	263	481	234	489	278	241	110	234	489	278	241	110	234	
7	MC-O	186	86	284	115	282	155	368	170	608	307	722	346	516	221	307	722	346	516	221	307	
8	Modified MC-O asphalt	202	79	183	112	435	250	510	—	973	626	2,350	1,304	654	312	626	2,350	1,304	654	312	626	
9	SMEPS	448	180	1,465	665	1,400	745	1,400	135	945	386	1,016	509	459	239	386	1,016	509	459	239	386	
10	Caromex-lignin	225	109	774	394	869	509	1,025	476	544	302	3,002	1,025	2,249	1,030	3,002	1,025	2,249	1,030	3,002	1,025	
11	Portland cement	905	452	1,237	600	2,012	1,012	2,730	1,143	2,243	1,265	5,161	3,955	2,138	1,480	3,955	2,138	1,480	3,955	2,138	1,480	
12	Sodium silicate	239	126	3,148	1,560	708	372	724	363	1,493	1,500	5,866	2,400	3,164	1,370	2,400	3,164	1,370	2,400	3,164	1,370	
13	Calcium chloride	252	128	532	286	708	372	724	363	1,016	570	7,626	3,480	3,700	1,700	3,480	3,700	1,700	3,480	3,700	1,700	
14	Emuls. asphalt (SS-1)	392	184	908	382	1,089	497	1,190	445	1,228	542	3,372	1,406	1,951	681	542	3,372	1,406	1,951	681	542	
15	IGER	5	2	6	6	66	52	63	90	72	53	100	53	62	43	53	62	43	53	62	43	
16	Bit. surf. treat. c	5	2	6	6	66	52	63	90	72	53	100	53	62	43	53	62	43	53	62	43	

No dust collection made in first four tests because repaired condition following failure of panel in first traffic test was not considered representative of originally designed panel. Dust was collected after last three tests for comparative purposes because panel was bearing uniformly to MC-O asphalt (unmodified) panel during this period. No dust collection following third and fourth traffic tests because it appeared that excessive abrasion was occurring as a result of an approximate failure tests because failure was more severe than that on either of the untreated panels during these tests. Collection was resumed for last 2 quantities of abraded material for bituminous-surfaced panel represent material picked up from turnaround area or preceding panel. (unmodified asphalt) and designated by vehicle tires.

the abrading action of traffic and subsequent dust formation; and (b) protecting the base and, to a certain extent, the surface itself from ingress of water. Because there are no criteria by which the adequacy of a given treatment can be defined in specific terms, the use of a comparative-capability approach appears to be the most logical. In this regard, reasonable standards for comparison in a somewhat more quantitative sense are provided by the untreated and the double-layer bituminous-treated control panels.

#### Dustproofing Effectiveness

In a broad sense, "dust" may be defined as soil and/or other material which has become airborne. In this connotation, no attempt is made to establish a limit of particle size that may be considered dust. In this paper the term "abraded material" is used in place of "dust." Abraded material refers to the total amount of loose material worn away or otherwise eroded from the test panel surface by the action of the test vehicle. For surfaces constructed predominantly of silt- and clay-size particles, such as those in the test section under discussion, virtually all the material that is broken away from the surface can be considered a "dust" potential. Even though the loose material may consist of agglomerations of small particles held together as a result of either treatment or compaction, the action of repetitive traffic will eventually reduce them to a fineness approaching that of the individual soil grains.

The data given in Table 2 indicate significant differences in the amount of material abraded from the various surfaces by the imposed traffic. The quantities shown represent only the material abraded by traffic and do not include material eroded by weathering between traffic tests, inasmuch as the test section was swept clean of loose material immediately prior to each test. The quantities of abraded material collected from each panel generally fall between the amounts collected from the untreated and the double-layer bituminous-treated control panels. A small quantity of material was recovered from the double-layer bituminous surface panel. This was not material abraded

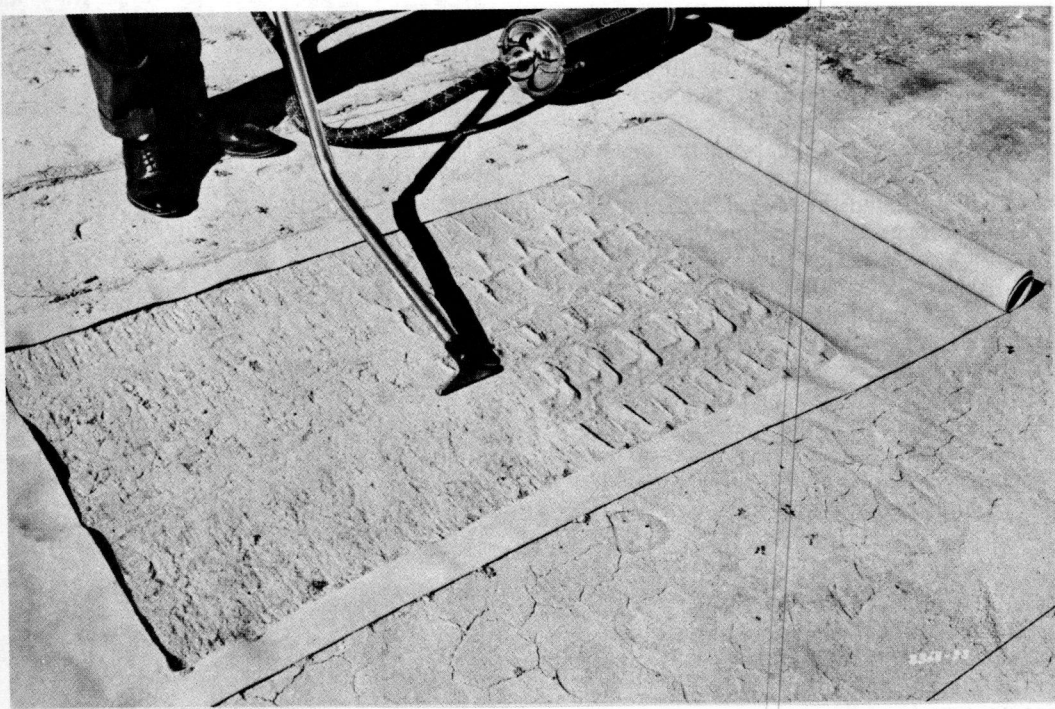


Figure 3. Collection of abraded material from traffic lane.

from the panel, but material that had been picked up and carried from the adjacent panel and the north turnaround area by the vehicle's tires. Inasmuch as this amount of material was small, and because the tendency for carry-over in all panels was similar, no correction for this occurrence was made.

The conditions under which the traffic tests were performed were constant with respect to the number of passes per traffic test, and relatively similar with regard to surface water contents at the time of the tests. Therefore, it is considered that differences in abrasion among the various panels for a given traffic test are primarily a function of the differences in the quality of the surfaces provided by the several treatments. Similarly, the differences indicated for any given surface from one traffic test to another must be a result of some change in a property of the treated surface layer that has occurred between traffic tests. Assuming that curing of a surface had been essentially completed by the time the traffic tests were begun, any change taking place is probably a result of weathering. Because no freezing occurred during the test period weathering effects are primarily those resulting from wetting and drying cycles.

To examine the possible influence of weather on abrasion, total rainfall from the time of completion of test-section construction to the time of each traffic test has been plotted against total abraded material collected from the beginning of the tests for each panel (Fig. 5). It is seen that the relation of total rainfall to abraded material is nearly rectilinear for each panel beginning with the third traffic test, with the general slope probably dependent on the effectiveness of the surface treatment. The deviations from a constant slope observed in these plots are believed to be due to a nonuniformity of rainfall patterns for the intervals between successive traffic tests.

To provide a uniform basis for comparing the relative abrasion-resistant characteristics of the various surfaces, a statistically averaged general slope was determined for each panel, using the data shown in Figure 5 and beginning with the third traffic test. From a comparison of these slopes, it was considered that the most effective dustproofing material tested was aniline-furfural (panel 6), followed in close order by MC-O asphalt (panel 7), SMEPS (panel 9), DDC (panel 3), and road oil (panel 4). Quantitatively, the accumulative amounts of abraded material from all traffic tests on these

TABLE 3  
COMPACTED SOIL BASE WATER CONTENTS (4- and 12-in. Depths)

Panel No.	Additive	Water Content (% on Dry Weight Basis)							
		28 Feb 1957		5 Apr 1957		8 May 1957		12 Aug 1957	
		4 in.	12 in.	4 in.	12 in.	4 in.	12 in.	4 in.	12 in.
1	Untreated soil <sup>a</sup>	20.4	19.7	18.3	19.5	14.6	14.8	—	—
2	Untreated soil	18.7	19.2	18.9	19.0	14.8	16.8	10.3	13.9
3	DDC	7.5	9.3	10.3	8.7	12.1	11.7	9.4	13.1
4	Road oil	14.2	14.2	7.6	9.7	7.9	10.0	8.3	9.8
5	Calcium acrylate	14.4	15.1	15.7	14.3	14.8	15.0	14.5	14.7
6	Aniline-furfural	9.5	11.3	8.9	11.0	8.2	10.6	6.8	9.8
7	MC-O asphalt	8.6	9.8	9.5	10.1	8.1	9.0	11.0	13.7
8	Modi. MC-O asphalt	9.0	10.5	9.1	9.9	8.3	9.4	8.8	11.3
9	SMEPS	9.0	9.4	7.5	10.8	9.1	12.9	8.1	11.6
10	Chrome-lignin	10.1	9.6	11.7	11.6	13.5	15.2	10.4	11.1
11	Portland cement	18.4	18.2	18.7	18.9	16.3	17.0	14.3	16.6
12	Sodium silicate	14.7	15.8	19.0	16.8	18.3	15.9	13.0	13.2
13	Calcium chloride	13.0	18.9	21.1	19.4	16.0	17.7	16.1	16.3
14	Emuls. asphalt (SS-1)	15.0	16.7	19.2	18.5	15.9	16.5	15.2	15.9
15	IGER	13.7	11.8	15.6	15.0	15.5	15.6	12.7	13.7
16	Bit. sur. treat.	9.4	10.3	13.1	12.0	14.4	12.9	12.4	15.3

<sup>a</sup> During first seven months sampling was accomplished only in panel 1 (untreated surface without moisture units); Water Content as follows: 27 July 1956, 4 in. = 8.0 percent, 12 in. = 10.8 percent; 29 Nov 1956, 4 in. = 13.8 percent, 12 in. = 11.2 percent; 26 Dec 1956, 4 in. = 18.2 percent, 12 in. = 17.4 percent.

five surfaces range from 12 to 22 percent of the total amount collected from the untreated control surfaces. The aniline-furfural panel was rated the most effective on the basis of the slope analysis, even though the total amount of abraded material (all traffic tests) collected from this panel slightly exceeded that from three other panels. The remaining materials, in the order of decreasing dustproofing effectiveness, were chromelignin (panel 10), IGER (panel 15), portland cement (panel 11), sodium silicate (panel 12), emulsified asphalt (panel 14), calcium acrylate (panel 5), and calcium chloride (panel 13). The modified MC-O asphalt data (panel 8) are omitted from Figure 5 because of the failure of this panel during the first traffic test. On the basis of the data collected during the final traffic tests this panel, had it cured properly, probably would

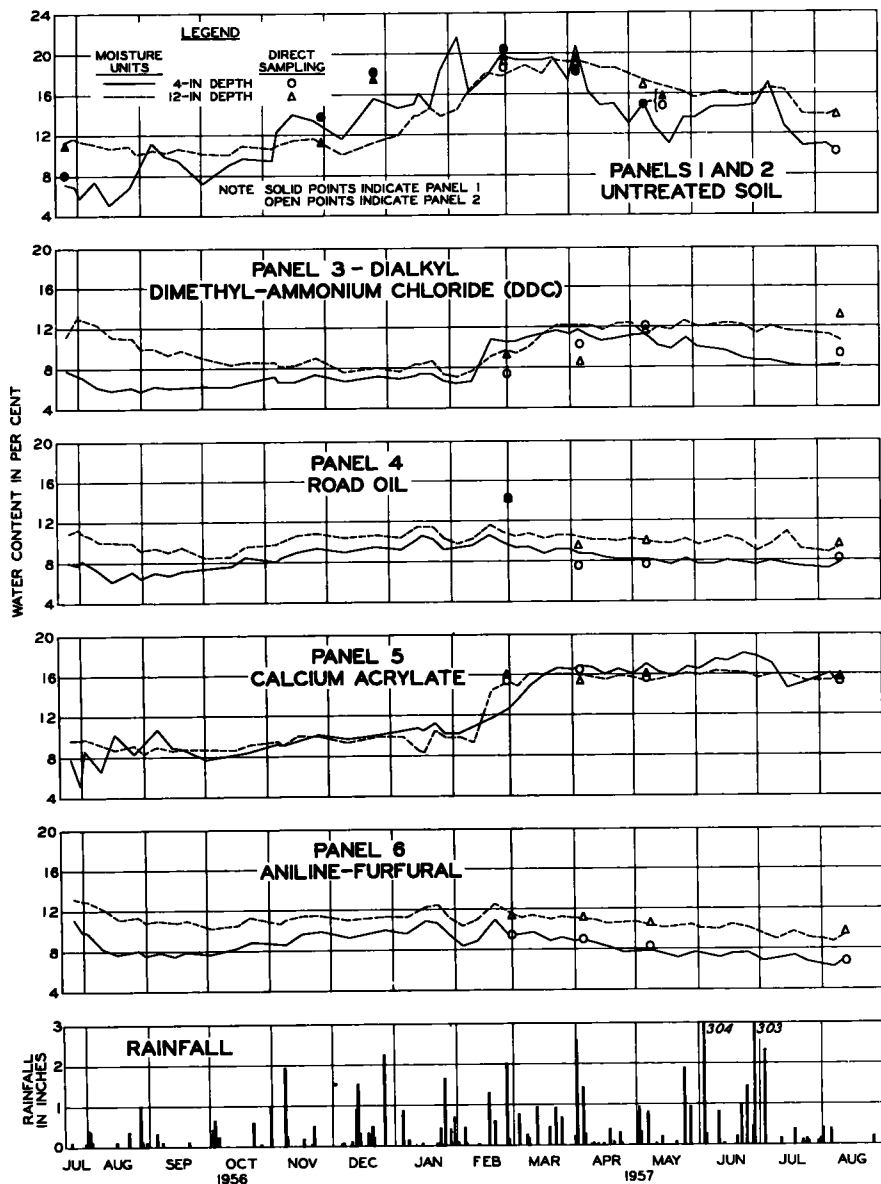


Figure 4a. Moisture trends in compacted soil base.

have performed about the same as the unmodified MC-O asphalt panel.

In evaluating these results, it should be noted that the calcium chloride treatment consisted of a single initial application with no subsequent re-treatment or sprinkling during dry periods, although this is the usual practice. The data showed that calcium chloride was more effective than several other treatments during the first traffic test. However, it appeared to be more adversely affected by weathering than the other materials. Similarly, it should be noted that the portland cement panel contained only 3 per cent cement. The low percentage was selected because it was desired to examine a cement-modified soil rather than a soil-cement.

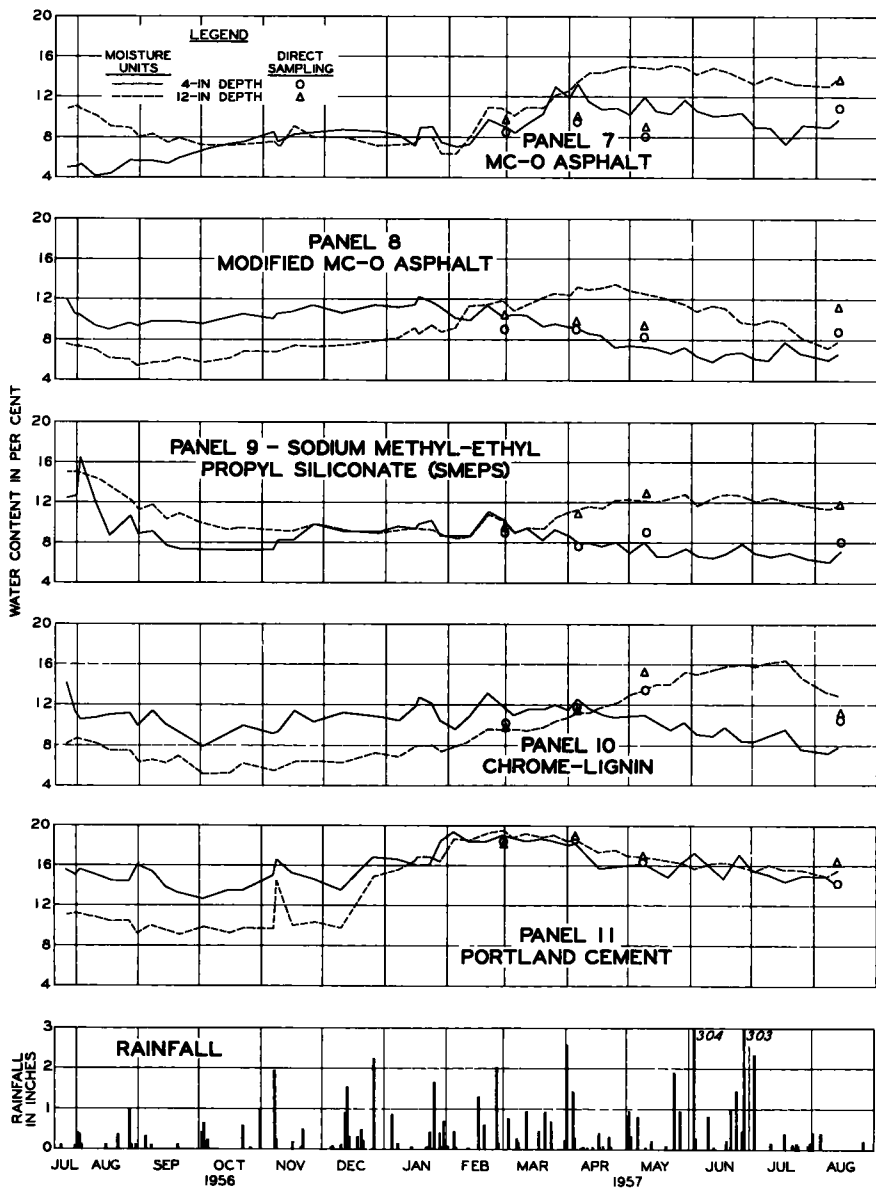


Figure 4b. Moisture trends in compacted soil base.

## WATERPROOFING EFFECTIVENESS

Determination of the comparative capabilities of the various treated surfaces in preventing surface water from entering the soil base was based on the maximum moisture contents measured at the 4- and 12-in. depths. Although it is believed that the moisture-unit resistance data are indicative of the over-all trends in the compacted-base water content, their validity in terms of absolute water content is questionable. The direct-sampling data (Table 3) were therefore used as the principal criterion in determining the relative waterproofing capabilities of the various surfaces. To provide a uniform basis for comparing the different surfaces, a representative maximum water content under each surface was determined by averaging all the values at the 4- and 12-in. depths

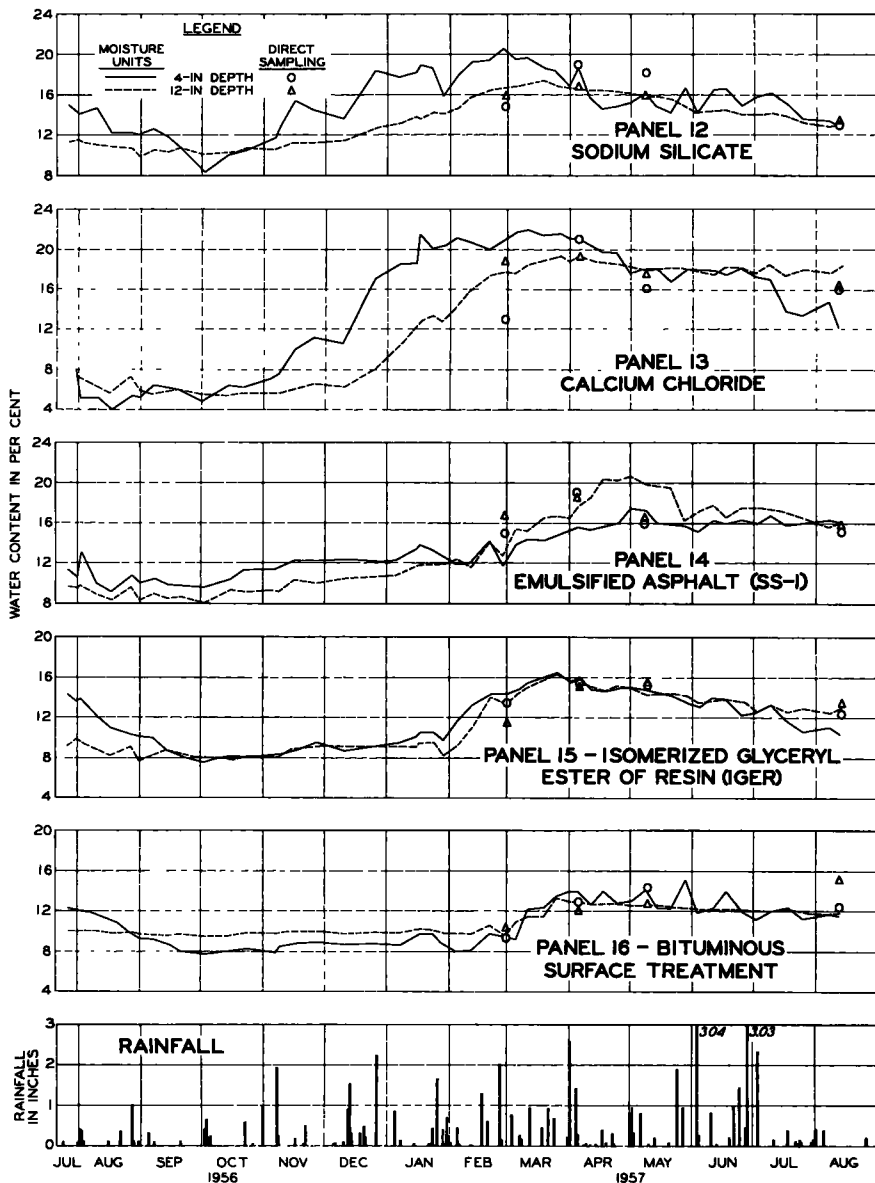


Figure 4c. Moisture trends in compacted soil base.

TABLE 4  
SURFACE WATER CONTENT AND DENSITY (0- to 3-in. DEPTH)

Panel No.	Additive	28 Feb 1957		5 Apr 1957		8 May 1957	
		Water Content (%)	Dry Density <sup>a</sup> (pcf)	Water Content (%)	Dry Density <sup>a</sup> (pcf)	Water Content (%)	Dry Density <sup>a</sup> (pcf)
1	Untreated soil	18.4	104.7	19.3	103.5	14.4	104.6
2	Untreated soil	17.1	110.9	18.5	107.7	14.8	105.3
3	DDC	7.3	109.6	7.6	108.0	6.7	106.0
4	Road oil	6.0	113.7	6.2	110.0	4.3	103.5 <sup>C</sup>
5	Calcium acrylate	16.5	106.7	19.1	102.8	17.5	104.3
6	Aniline-furfural	6.4	105.0	5.2	108.8	5.8	104.4 <sup>C</sup>
7	MC-O asphalt	5.9	109.4	6.9	111.4	6.8	110.6
8	Mod. MC-O asphalt	5.1	110.0	7.2	113.0	5.4	104.0 <sup>C</sup>
9	SMEPS	6.9	112.8	11.3	107.5	6.4	114.1
10	Chrome-lignin	12.7	100.2	13.1	100.5	10.9	101.6
11	Portland cement	17.9	106.5	18.6	105.9	15.5	100.4 <sup>C</sup>
12	Sodium silicate	18.2	102.9	19.8	97.1	16.9	93.0
13	Calcium chloride	16.6	107.1	19.1	105.9	13.8	103.4
14	Emuls. asphalt (SS-1)	10.9	114.1	15.3	107.8	12.0	112.1
15	IGER	13.3	105.0	15.4	103.6	13.0	104.0
16	Bit. surf. treat. <sup>d</sup>	9.6	110.1	10.4	108.8	11.2	110.6

<sup>a</sup> Percent by dry solids weight.

<sup>b</sup> During first seven months sampling was accomplished only in panel 1 (untreated surface without moisture units); on 27 July 1956 water content<sup>a</sup> was 5.7 percent, dry density 107.7 pcf; on 29 Nov. 1956 water content was 10.3 percent; on 26 Dec. 1956 water content was 14.7 percent.

<sup>c</sup> Samples very poor for density determination.

<sup>d</sup> Represents 3-in. soil depth immediately beneath 3/4-in. double-layer bituminous surface.

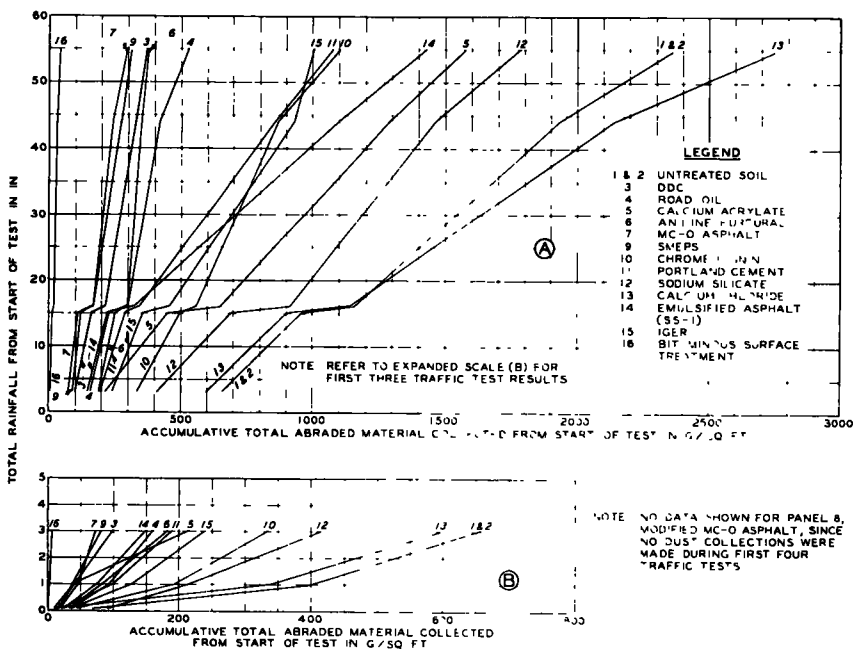


Figure 5. Relation of abraded material collected to rainfall.

TABLE 5  
RESULTS OF LABORATORY CAPILLARY RISE TESTS

Panel No.	Additive	As Molded			After 4-Day Air-Drying			After 4-Day Rewetting			Remarks
		% Additive by Dry Soil Wt	% WC by Total Wt Dry Solids	Dry Density in lb Total Solids per cu ft	% WC by Total Wt Dry Solids	Dry Density in lb Total Solids per cu ft	% WC by Total Wt Dry Solids	Dry Density in lb Total Solids per cu ft			
1 & 2	Untreated soil	—	16.9	110.5	2.7	115.3	25.3	96.1	Uniformly wet, very soft		
3	DDC	0.5	18.0	108.8	3.0	115.1	6.0	113.4	Bottom 0.5 in. wet, top dry		
4	Road oil (dipped)	5.5	16.0	109.5	2.8	115.7	4.5	114.9	No apparent absorption		
5	Calcium acrylate	7.0	16.0	110.9	4.2	116.3	10.9	116.3	Uniformly wet, firm		
6	Aniline-furfural <sup>a</sup>	3.3	16.5	106.8	2.0	110.1	3.9	109.2	Bottom 1.0 in. wet, top dry		
7	MC-O asphalt <sup>b</sup>	10.0	13.9	108.8	4.1	110.2	5.2	109.2	No apparent absorption		
8	Mod. MC-O asphalt <sup>c</sup>	10.0	13.7	107.7	4.2	110.0	4.8	109.7	No apparent absorption		
9	SMEPS	1.0	17.5	109.5	2.8	116.0	9.4	111.0	Bottom 1.0 in. wet, top dry		
10	Chrome-lignin	5.0	16.8	110.4	3.3	118.4	12.7	112.5	Bottom 2.0 in. wet, top dry		
11	Portland cement	3.0	15.8	109.9	2.2	111.6	19.2	107.8	Uniformly wet, soft		
12	Sodium silicate <sup>d</sup>	14.5	16.7	103.0	3.8	105.7	25.0	95.8	Bottom 2.0 in. wet, very soft		
13	Calcium chloride	1.0	16.5	111.9	3.6	117.4	28.5	95.1	Uniformly wet, very soft		
14	Emuls. asphalt (SS-1) <sup>e</sup>	8.4	19.9	103.9	2.5	112.4	20.0	100.7	Uniformly wet, soft		
15	IGER	5.0	15.2	105.6	2.8	109.0	23.3	96.4	Uniformly wet, soft		

<sup>a</sup>2.1 percent aniline and 1.2 percent furfural.

<sup>b</sup>50 percent asphalt content.

<sup>c</sup>Modified with 0.5 percent phosphorus pentoxide and 0.25 percent of a lauryl amine.

<sup>d</sup>30 percent solution.

<sup>e</sup>60 percent asphalt content.

for the three sampling dates of 28 February, 5 April, and 8 May 1957. This average water content value was considered indicative of the waterproofing afforded and was employed to obtain a relative comparison of treatments. On this basis, the panels were rated in the order of decreasing waterproofing effectiveness as follows: MC-O asphalt (panel 7), SMEPS (panel 9), aniline-furfural (panel 6), DDC (panel 3), road oil (panel 4), double-layer bituminous surface (panel 16), chrome-lignin (panel 10), IGER (panel 15), calcium acrylate (panel 5), sodium silicate (panel 12), emulsified asphalt (panel 14), calcium chloride (panel 13), portland cement (panel 11), and the untreated surfaces (panels 1 and 2). It is to be noted that the modified MC-O asphalt (panel 8) was not rated although the data for this treatment are included in the tables and figures. For purposes of analysis, this panel was not evaluated because it had been repaired early in the tests with unmodified MC-O asphalt and its performance was not significantly different from the unmodified asphalt panel.

The impervious double-layer bituminous-treated surface (panel 16), which originally was expected to produce a surface of maximum effectiveness, showed a higher water content in the soil base than did several other surfaces. This moisture buildup is believed to have been a result of the surface acting as a highly effective vapor barrier that prevented evaporation of sub-grade water even during dry periods. There did not appear to be a great deal of difference in the respective capabilities of the five materials rated more effective than the double-layer bituminous surface. However, it is significant that, with the exception of a single high value recorded for road oil on 28 February 1957 (Fig. 4a), the water contents beneath the better surfaces were either lower than or about equal to the initial water content (10.8 percent) of the soil when the base was originally compacted. In contrast, maximum water contents ranging from 14 to more than 20 percent were observed under the remaining panels.

Examination of the surface water content data (Table 4) indicates that the bet-

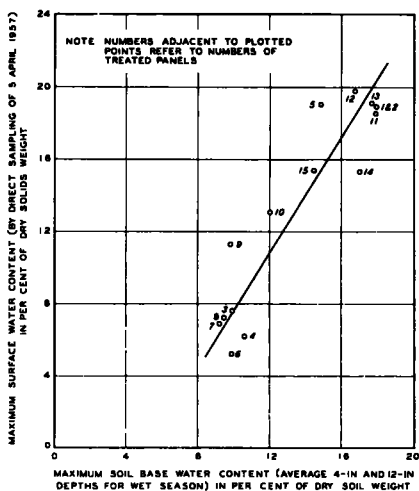


Figure 6. Comparison of maximum water content of soil base and maximum surface moisture.



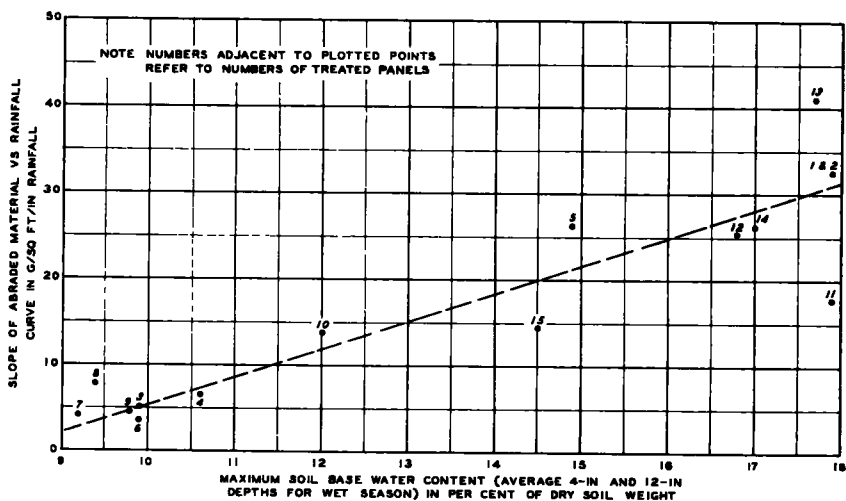


Figure 7. Relation of maximum water content of soil base and slope of abraded material vs rainfall curve.

ter base-protective surfaces are those which appear themselves to absorb the least water. A comparison of the moisture content of the base with that of the surface for all the panels during the wettest period is shown in Figure 6. From this plot, it is seen that there is a general relation between the moisture that may be absorbed by the surface and the water present in the base. An interesting observation in this respect is that the water contents in the base under the more effective surfaces are somewhat greater than those in the surfaces themselves, whereas the reverse appears to be true for the less effective surfaces. The existence of the surface-base moisture relation suggests the possibility of determining the waterproofing capability of a specific treatment by convenient laboratory techniques, which are discussed later.

### Combined Dustproofing and Waterproofing Effectiveness

Because the purpose of the treatments was to provide both waterproofing and dustproofing capabilities, the effectiveness of the various treatments in accomplishing the combined purpose was examined. It has been stated previously that the ability of a surface to resist abrasion is apparently a function of its ability to resist changes as a result of weathering. Thus, it might be expected that a surface achieving high water contents during a wet period would show both poor abrasion-resistant qualities and little waterproofing protection for the soil base or subgrade. Also, it would appear that the better waterproofing materials are the better dustproofers.

In Figure 7, the maximum water content measured at the 4- and 12-in. depths in the soil base during the wet season (average direct-sampling data of 28 February, 5 April, and 8 May 1957) was plotted against the general slopes of the "ab-

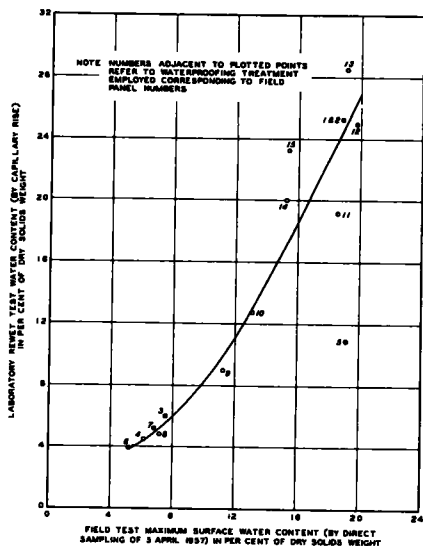


Figure 8. Comparison of maximum field surface water content and laboratory rewet test results.

graded material versus rainfall" curve for each panel (Fig. 5). It is apparent from Figure 7 that a general relation does exist between the waterproofing and abrasion-resistant characteristics of a given surface. Although the points plotted are not without scatter, the trend is sufficiently consistent to be considered significant.

Quantitatively, five materials of those tested were considered sufficiently effective as combination soil dustproofers and waterproofers to warrant additional investigation. These materials are: aniline-furfural (panel 6), MC-O asphalt (panel 7), SMEPS (panel 9), DDC (panel 3) and road oil (panel 4). Of particular interest are DDC and SMEPS, because they were found to be effective in such small quantities; that is, 0.5 and 1.0 percent, respectively, by dry soil weight. The MC-O cutback asphalt is of interest because it is so commonly available, although the quantity required (10.0 percent by weight of soil) is considered rather high. The road oil has an advantage in that it was effective when applied as a penetration treatment, thus reducing construction requirements. Aniline-furfural, although highly effective, has the disadvantage of being a two-chemical system, one highly toxic, thus requiring special handling and storage precautions as well as complicating the construction operation.

### CORRELATION OF LABORATORY AND FIELD RESULTS

Concurrently with the field test program, comprehensive tests were made in the laboratory to assist in evaluating the materials used in the field investigation. Furthermore, it was intended that these laboratory tests assist in developing suitable laboratory methods and procedures for determining the effectiveness of other proposed materials without extensive field investigation. The same type of loess soil and the same additives and quantities were used in all phases of the laboratory investigation as had been used in the field tests. Test specimens were prepared using the Harvard miniature compaction apparatus and cured under ambient laboratory conditions for at least four days before any tests were performed. A variety of tests were made, including rewet tests, strength tests, and abrasion tests. Of these, the most satisfactory correlation with field results was obtained with a capillary-rise rewet test. Laboratory abrasion test results were not satisfactory because none of the test procedures devised truly duplicated the kneading-abrading action of a rolling, pneumatic-tired wheel on a weathered soil surface.

In the capillary-rise rewet test, air-dried specimens were inserted in a membrane that was open at both ends, and placed in an upright position on a  $\frac{3}{8}$ -in. porous stone in an evaporating dish. Water was placed in the dish and maintained approximately  $\frac{1}{8}$  in. below the bottom of the specimens for a period of four days. Following this, the specimens were removed from the membrane and the densities and water contents determined. The results of these tests are given in Table 5.

In the discussion of field results, it was shown that the maximum water content of the base, as well as observed abrasion, was related to the maximum field surface water content. In Figure 8, maximum field surface water content is plotted against capillary-rise rewet test data for each treatment. With the exception of calcium acrylate (panel 5), a good correlation is observed. It may be noted that the materials indicated to be better by the capillary test show a smaller water pickup than was evident in the field surfaces, whereas for the poorer materials, the water pickup exceeded that observed in the field panels.

### SUMMARY

Field tests of a number of soil waterproofing and dustproofing materials have been described. Analysis of the results of these tests showed that all the test surfaces were adversely affected by weathering. The better treatments reduced surface abrasion to about  $\frac{1}{5}$  that of the untreated soil and did not allow the water content of the soil base to exceed the as-constructed value. Those materials that displayed superior waterproofing ability also were found to be the most effective in controlling abrasion of the treated surfaces. Five of the 13 materials evaluated were markedly superior as water-proofers and dustproofers for the lean clay soil tested. These were: aniline-furfural, MC-O asphalt, sodium methyl-ethyl propyl silicate, a quaternary ammonium chloride,

and a road oil. Laboratory tests conducted concurrently with the field tests showed that the waterproofing ability of the materials tested could be related to the results of a laboratory rewet test.

#### ACKNOWLEDGMENTS

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# Phosphoric Acid Stabilization of Fine-Grained Soils: Improvements with Secondary Additives

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Several techniques for improving, or reducing the cost, of phosphoric acid stabilization of a clayey silt and a heavy clay have been examined.

With the clayey silt, treatment with mixtures of calcium phosphate, or pulverized Florida phosphate rock, and sulfuric acid yields humid cure strengths greater than with the equivalent amount of phosphoric acid, but lower immersed strengths. Using phosphate rock, however, the reduction in cost of stabilization is so great that higher concentrations of this mixture can be used to produce satisfactory stabilization at a cost competitive with conventional stabilizers.

Granular, orthorhombic phosphorus pentoxide can be used to stabilize clayey silt as effectively as phosphoric acid or ordinary  $P_2O_5$ , and with potentially greater field-convenience. Trace amounts (0.1 percent on soil) of this compound function as an excellent soil-dispersant, and when used in conjunction with phosphoric acid or phosphate rock-sulfuric mixtures, greatly increase compacted density and strength.

Ferric chloride, at 1.6 percent on soil, is as effective a water-proofer for phosphoric-acid-stabilized heavy clay as 0.5 percent octylamine. Although this substitution brings about a substantial reduction in stabilization cost, the materials costs for very heavy clay stabilization are still too high to be attractive for all but emergency situations. Development of an inexpensive, highly effective waterproofer may broaden the usefulness of phosphoric acid in such soils.

● USE OF PHOSPHORIC acid for stabilization of fine-grained soils has become a subject of growing interest in recent years; this interest stems from the experimental observations that (a) phosphoric acid exhibits impressive stabilizing ability in a wide variety of aluminosilicate soils, (b) acid requirements for effective stabilization are low, and (c) strength development in phosphoric acid-treated soils occurs rapidly under humid conditions (1, 3, 4). These characteristics of phosphoric acid make it potentially competitive with conventional soil stabilizers, such as portland cement, asphalt, and lime.

Despite its effectiveness and breadth of utility as a stabilizer, phosphoric acid suffers several limitations which have deterred its field evaluation. The most important of these are:

1. The cost of phosphoric acid stabilization of moderately fine-grained soils is considerably higher than that of conventional stabilization methods. Although the acid is uniquely effective in heavy clays (relative to conventional stabilizers), the cost of amine waterproofer required for this purpose is far too high to make the method attractive except in special situations.

2. Phosphoric acid stabilization (as is true with most other stabilization methods) is quite sensitive to initial soil moisture content, and to the degree of soil compaction prior to cure.

3. Phosphoric acid, in concentrated form, presents problems in the field regarding transportation and handling with conventional soil-processing equipment. A safe, granular solid product would be more attractive from this standpoint.

The object of this investigation has been to explore a number of possible solutions to the aforementioned problems. These include studies of (a) the use of calcium phosphate-sulfuric acid mixtures as alternatives to phosphoric acid, (b) the use of granular orthorhombic phosphoric anhydride as an acid substitute, and (c) the use of polyvalent metal salts as substitutes for amines for the waterproofing of acid-stabilized heavy clay

### PROCEDURE

The two soils studied were Massachusetts clayey silt (MCS), a moderately fine-grained illitic soil (PI=6) and Vicksburg buckshot clay (VBC), a heavy clay containing about 25 percent of the expanding-lattice montmorillonoid minerals (PI=32). The physical properties and mineralogical compositions of these soils have been previously published (3).

Chemicals used in this work were as follows:

Phosphoric acid (86.1%  $H_3PO_4$ ) — Reagent grade

Orthorhombic phosphoric anhydride ( $P_2O_5$ ) — Victor Chemical Works

Sulfuric acid (98%  $H_2SO_4$ ) — Reagent grade

Tricalcium phosphate ( $Ca_3(PO_4)_2$ ) — Reagent grade

Calcium monohydrogen phosphate ( $CaHPO_4$ ) — Reagent grade

Calcium dihydrogen phosphate ( $Ca(H_2PO_4)_2$ ) — Reagent grade

Sodium fluosilicate ( $Na_2SiF_6$ ) — Reagent grade

Ferric chloride ( $FeCl_3 \cdot 6H_2O$ ) — Reagent grade

Aluminum chloride ( $AlCl_3 \cdot 6H_2O$ ) — Reagent grade

Magnesium chloride ( $MgCl_2 \cdot 6H_2O$ ) — Reagent grade

Chromic chloride ( $CrCl_3 \cdot 6H_2O$ ) — Reagent grade

Pulverized Florida phosphate rock — supplied by International Minerals and Chemicals Co., analyzing 29.6% CaO, 32.27%  $P_2O_5$ , and 2.72 - 4.08% F; 93% finer than 60 mesh, 63% finer than 200 mesh (5).

The preparation and testing of samples were similar to those used in previous studies (3, 4). Liquids content at molding was usually maintained at 12.5 cc total liquids volume per 100 gm dry soil for MCS, and 17.0 cc per 100 gm soil for VBC; maximum compacted density was achieved under these conditions. Methods of additive incorporation depended on the type of additive, and in some cases variations in procedure were studied; therefore the method of addition will be specified in the discussion. In general, chemicals were added to the soil and blended for several minutes in a finger prong mixer. Compression test samples were then prepared by two-end static compaction in a Harvard miniature mold. Compaction pressure was always approximately 1,000 psi. Samples were cured at about 70 F and 100 percent relative humidity for various periods of time, and in some cases subsequently subjected to 24-hr water immersion, then tested to failure in unconfined compression and dried at 110 C for 24 hr. They were weighed and measured after molding, testing, and drying.

### RESULTS AND DISCUSSION

#### Stabilization of Massachusetts Clayey Silt with Phosphate-Sulfuric Acid Admixtures

Calcium Phosphate-Sulfuric Acid Stabilization. — Because calcium phosphates react with aqueous sulfuric acid (in sufficiently high concentration) to form phosphoric acid and hydrated calcium sulfate, it was believed that if this reaction would proceed in the presence of soils, successful stabilization would result. To explore this possibility, moist Massachusetts clayey silt was blended with the various calcium phosphates, then mixed with a stoichiometric quantity of concentrated sulfuric acid, compacted, and cured.

Table 1 gives the compressive strengths obtained with MCS treated with each of the

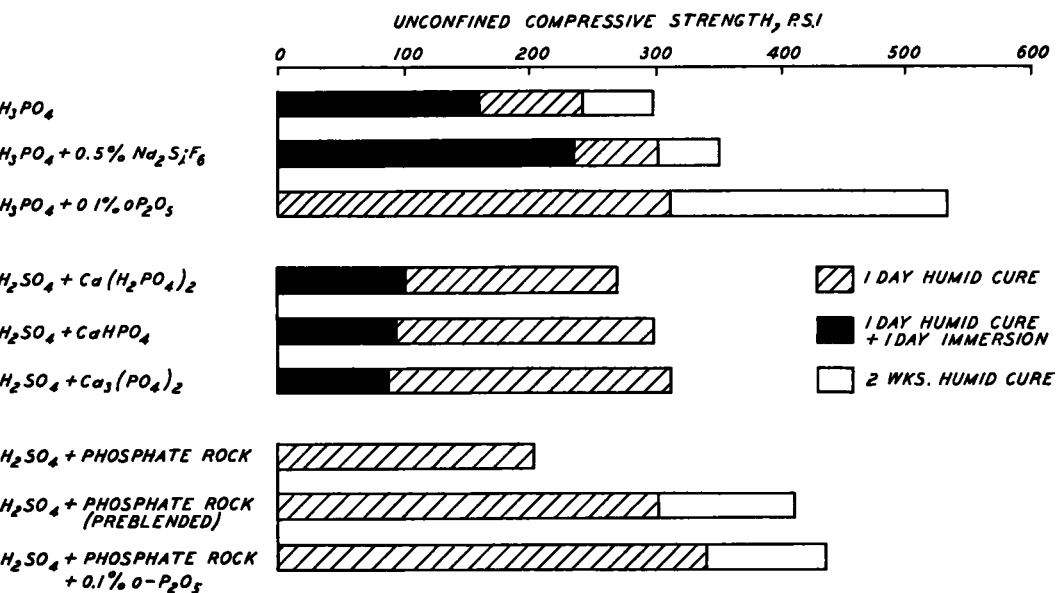


Figure 1. Comparison of various phosphorus-based stabilizing systems (equivalent to 2 percent by weight  $H_3PO_4$ ) on properties of Massachusetts clayey silt.

three calcium phosphates and sulfuric acid at various volatiles contents after 1- and 7-day humid cure, and after 1-day humid cure followed by 1-day immersion. For comparison, results with phosphoric acid are also included.

At equal densities, the samples prepared with calcium phosphates have considerably higher 7-day humid cure strengths than samples with  $H_3PO_4$ , but immersed strengths after 1-day humid cure are significantly lower. There is a significant downward trend in compressive strength as the ratio of calcium to phosphate in the salts is increased, even though the phosphate content was held constant.

Because the sulfuric acid will, in part at least, be consumed by reaction with alumina in the soil, the observed trends are perhaps to be expected: the more basic calcium phosphates are less soluble and more refractory to acid attack, and therefore are more slowly converted to phosphoric acid. Considering the competition between soil

TABLE 1  
STABILIZATION OF MCS WITH MIXTURES OF CALCIUM PHOSPHATES AND SULFURIC ACID  
EQUIVALENT TO 2 %  $H_3PO_4$

%	Phosphate	100% $H_2SO_4$ %	Humid Cure				24-Hr Humid Cure & 24-Hr Immersion		
			Compressive Strength (psi)		Dry Solids Density at Test (pcf)	% Volatiles at Test	Comp. Strength (psi)	Dry Solids Density at Test (pcf)	% Volatiles at Test
			24 Hr	7 Days					
2	$H_3PO_4$	—	242±9	300	131.0	11.0	160±8	131.0	11.4
2	$H_3PO_4$	—	184±9	271±15	128.4	11.3	—	—	—
2.07	$Ca(H_2PO_4)_2$	0.85	270±30	410±110	131.3±0.6	10.1±0.1	100±0	129.5±0.7	11.1±0.1
2.07	$Ca(H_2PO_4)_2$	0.85	263±22	—	131.3±0.2	10.0±0.1	77±13	129.5±0.1	11.9±0.2
2.36	$CaHPO_4$	1.70	300±10	375±45	129.5±0.3	10.6±0.1	95±10	127.2±0.7	12.6±0.1
2.72	$Ca_3(PO_4)_2$	2.58	248±28	—	124.3±0.4	8.9±0.2	—	—	—
2.72	$Ca_3(PO_4)_2$	2.58	235±15	333±47	125.8±1.2	9.7±0.2	—	—	—
2.72	$Ca_3(PO_4)_2$	2.58	313±3	—	127.3±0.3	10.5±0.1	87±8	124.9±0.1	12.8±0.0
2.72	$Ca_3(PO_4)_2$	2.58	257±23	—	126.9±0.7	10.7±0.3	—	—	—
2.72	$Ca_3(PO_4)_2$	2.58	287±8	—	128.1±0.1	11.5±0.2	—	—	—

Note: All percentages are based on dry soil weight.

and phosphate for sulfuric acid, it is remarkable that these mixtures exhibit such pronounced stabilizing action. The observation that the immersed strengths of samples treated with phosphate-sulfuric acid mixtures are considerably lower than those obtained with phosphoric acid appears to support the belief that less than the stoichiometric quantity of acid is generated in the process.

Another factor which is undoubtedly of importance is precipitation in the soil of the slightly soluble calcium sulfate formed in the reactions. The small amounts of precipitate formed in the dihydrogen phosphate appear to increase solids density and humid cure strength. Larger quantities of sulfate, such as with tri-calcium phosphate, appear to reduce density and strength. In any case, the increase in density would not be expected to improve inter-particle cementation and, indeed, the sulfate may tend to hinder cementation; this factor may contribute to the lower immersed strength obtained with the phosphates.

The data in Table 1 also show the effect of variations in molding water content on strength and density. Strength and density increase with water content up to about 10.5 percent, beyond which both decrease. This behavior is similar to results with pure phosphoric acid, and suggests that optimum water content for phosphate-sulfuric acid stabilization is about the same as for pure phosphoric acid.

**Phosphate Rock-Sulfuric Acid Stabilization.** — In light of the established stabilizing action (on MCS) of calcium phosphate-sulfuric acid mixtures, a series of experiments was conducted using finely ground Florida phosphate rock (consisting predominantly of fluorapatite —  $\text{CaF}_2 \cdot 3\text{Ca}_3(\text{PO}_4)_2$ ) and sulfuric acid to treat this soil. Inasmuch as fluorapatite is considerably more refractory to acid attack than the fluorine-free calcium phosphates, it was expected that less phosphoric acid would be generated in situ in the soil with this mineral. To clarify this point, samples were prepared (a) by mixing rock and soil and then adding sulfuric acid, and (b) by preblending rock and acid and then mixing the blend with soil. Results are given in Table 2.

Consistent with predictions, soils stabilized by mixing with rock followed by acid addition are substantially weaker than those stabilized with equivalent concentrations of phosphoric acid, or of pure phosphate-sulfuric acid mixtures. However, soil treated with rock-sulfuric preblends exhibited cured strengths substantially higher than those stabilized with phosphoric acid or pure phosphate-sulfuric acid mixtures. The explanation for the latter result lies, it is believed, in the previously established catalytic activity of fluoride ion in the phosphoric acid-alumina reaction (4). Included in Table 2 are data determined earlier for phosphoric acid-stabilized MCS containing a small amount of sodium fluosilicate; compressive strengths for these formulations are equal

TABLE 2  
STABILIZATION OF MCS WITH PHOSPHATE ROCK AND SULFURIC ACID

Formulation and Manner of Addition to Soil	1-Day Humid Cure			7-Day Humid Cure			14-Day Humid Cure		
	Comp. Str. (psi)	% Volatile at Test	Dry Solids Density at Test (pcf)	Comp. Str. (psi)	% Volatile at Test	Dry Solids Density at Test (pcf)	Comp. Str. (psi)	% Volatile at Test	Dry Solids Density at Test (pcf)
<b>(Concentration Equivalent to 2% <math>\text{H}_3\text{PO}_4</math>)</b>									
$\text{H}_3\text{PO}_4$	184 <sup>±</sup> 4	11.4	128.3	—	—	—	271 <sup>±</sup> 15	11.3	128.5
$\text{H}_2\text{PO}_4$	242 <sup>±</sup> 9	11.0	131.0	—	—	—	300	11.0	131.0
$\text{H}_2\text{PO}_4 + 0.5\% \text{Na}_2\text{SiF}_6$	305 <sup>±</sup> 17	11.3	129.3	—	—	—	357	11.3	129.0
Phosphate rock-sulfuric acid, dry mixed	158 <sup>±</sup> 8	9.8 <sup>±</sup> 0.2	128.3 <sup>±</sup> 0.3	191 <sup>±</sup> 10	9.9 <sup>±</sup> 0.1	127.5 <sup>±</sup> 0.7	204	9.9	128.1
Phosphate rock-sulfuric acid, dry mixed	222 <sup>±</sup> 12	10.3 <sup>±</sup> 0	128.0 <sup>±</sup> 0	171 <sup>±</sup> 14	10.7 <sup>±</sup> 0.1	128.3 <sup>±</sup> 0.8	145 <sup>±</sup> 35	11.3 <sup>±</sup> 0.6	128.1 <sup>±</sup> 0.1
Phosphate rock-sulfuric acid, dry mixed	282 <sup>±</sup> 6	11.4 <sup>±</sup> 0.2	131.8 <sup>±</sup> 0.2	378 <sup>±</sup> 6	11.5 <sup>±</sup> 0.1	131.4 <sup>±</sup> 0.1	—	—	—
Phosphate rock-sulfuric acid, preblended	305 <sup>±</sup> 25	9.8 <sup>±</sup> 0.5	129.8 <sup>±</sup> 0.3	387 <sup>±</sup> 45	8.9 <sup>±</sup> 0.1	130.3 <sup>±</sup> 0.6	415 <sup>±</sup> 27	8.7 <sup>±</sup> 0.1	130.0 <sup>±</sup> 1.5
<b>(Concentration Equivalent to 5% <math>\text{H}_3\text{PO}_4</math>)</b>									
$\text{H}_3\text{PO}_4$	225 <sup>±</sup> 10	10.7	132.5	—	—	—	843	10.8	132.5
$\text{H}_2\text{PO}_4 + 0.5\% \text{Na}_2\text{SiF}_6$	495 <sup>±</sup> 20	11.0	131.0	—	—	—	1,015	11.0	131.0
Phosphate rock-sulfuric acid, dry mixed	242 <sup>±</sup> 23	10.1 <sup>±</sup> 0	128.3 <sup>±</sup> 0.2	—	—	—	307 <sup>±</sup> 4	11.6 <sup>±</sup> 0.1	127.4 <sup>±</sup> 0.2
Phosphate rock-sulfuric acid, preblended	464 <sup>±</sup> 8	10.0 <sup>±</sup> 0.1	127.8 <sup>±</sup> 0.3	620 <sup>±</sup> 25	9.6 <sup>±</sup> 0.2	128.4 <sup>±</sup> 0.2	9.0 <sup>±</sup> 5	10.0	128.7
Phosphate rock-sulfuric acid, preblended	529 <sup>±</sup> 34	9.4 <sup>±</sup> 0.1	127.0 <sup>±</sup> 0.2	740 <sup>±</sup> 75	9.4 <sup>±</sup> 0.1	126.5 <sup>±</sup> 0.7	870 <sup>±</sup> 55	9.2 <sup>±</sup> 0.2	—

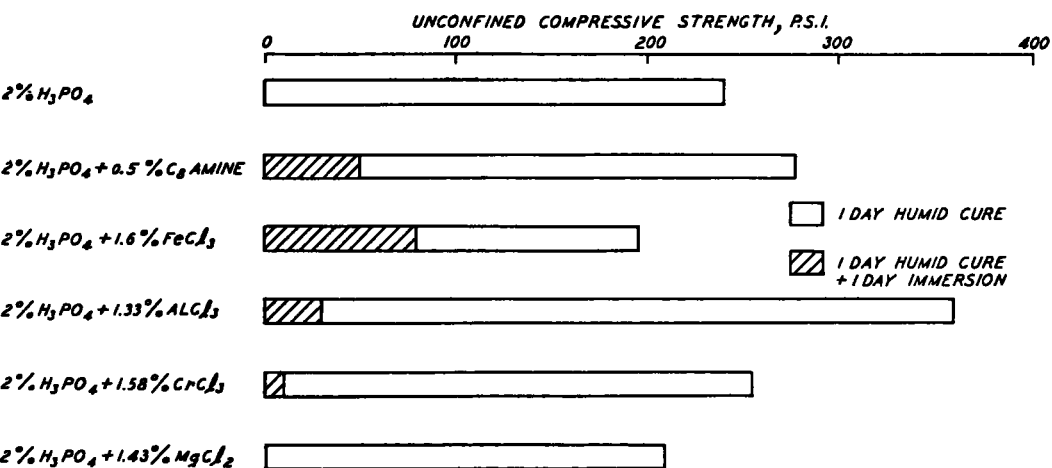


Figure 2. Comparison of various waterproofing additives on phosphoric acid-stabilized Vicksburg buckshot clay.

to or higher than those for rock-sulfuric acid treated specimens. It is thus apparent that a phosphate rock-sulfuric acid premix is quite suitable as a stabilizer for MCS; although all the phosphoric acid potentially available from such a mixture is evidently not used, the presence of fluoride more than compensates as regards rate of strength development.

Another cause of poor strength development in soils blended initially with rock arises from small amounts of calcium carbonate in the rock; this salt evolves carbon dioxide in contact with acid, building up pressure within the soil and breaking down the structure. Thus, another advantage of preblending rock and acid is elimination of gaseous reaction products prior to incorporation with the soil.

**Accelerating Effect of Sodium Fluosilicate.** — To determine whether the strength development of soil stabilized with pure calcium phosphate-sulfuric acid systems could be accelerated by means of fluoride, a series of tests was run with the various phosphates, at both 2 percent and 5 percent equivalent  $H_3PO_4$ , and both with and without 0.5 percent sodium fluosilicate. These results are presented and compared with other fluoride-containing systems in Table 3.

Table 3 shows that, at the equivalent of 2 percent  $H_3PO_4$ , fluosilicate results in only about 50 percent increase in 1-day strength with either pure phosphoric acid or pure calcium phosphate. This relatively slight accelerating effect, in addition to the even smaller increase in ultimate (7-day) strength, are in agreement with previous results. At the equivalent of 5 percent  $H_3PO_4$ , however, the accelerating effect of sodium fluosilicate is much more apparent, although ultimate strength is again only slightly increased. There is a significant and consistent decrease in the effectiveness of fluosilicate as the acidity of the phosphate decreases; that is, the accelerating effect is 150 percent with  $H_3PO_4$ , 135 percent with  $Ca(H_2PO_4)_2$ , and 95 percent with  $Ca_3(PO_4)_2$ . The fact that fluosilicate results in only 18 percent increase in 1-day strength with phosphate rock is undoubtedly due to the already high fluorine content of this material, making additional fluosilicate relatively ineffective.

#### Waterproofing of VBC with Polyvalent Metal Ions

Previous work (4) has indicated that the mechanism of waterproofing VBC by amines involves reducing swelling of the montmorillonoid minerals by replacing interlayer metallic cations with amine molecules. Because some polyvalent metallic cations, such as ferric ion, are known to have a similar effect in reducing lattice swelling, an investigation was carried out to evaluate a number of soluble salts of metallic ions as waterproofers for VBC stabilized with phosphoric acid. Salts studied included  $FeCl_3$ ,



TABLE 3  
COMPARISON OF THE EFFECT OF  $\text{Na}_2\text{SiF}_6$  ON THE STRENGTH OF  
MCS STABILIZED WITH VARIOUS ACIDIC PHOSPHORUS SYSTEMS

Phosphate	% Equivalent $\text{H}_3\text{PO}_4$	Compressive Strength (psi)					
		1-Day Humid Cure			7-Day Humid Cure		
		% $\text{Na}_2\text{SiF}_6$		% Increase Caused by $\text{Na}_2\text{SiF}_6$	% $\text{Na}_2\text{SiF}_6$		% Increase Caused by $\text{Na}_2\text{SiF}_6$
		0	0.5		0	0.5	
$\text{H}_3\text{PO}_4$	2	210	305	45	285	345	21
$\text{Ca}_3(\text{PO}_4)_2$ (dry mixed)	2	235	350	50	333	414	25
$\text{H}_3\text{PO}_4$	5	165	420	150	840	1,100	35
$\text{Ca}(\text{H}_2\text{PO}_4)_2$ (dry mixed)	5	269	655	135	890	1,155	30
$\text{Ca}_3(\text{PO}_4)_2$ (dry mixed)	5	252	487	95	418	510	22
Rock (preblended)	5	464	529	18	620	740	19

$\text{AlCl}_3$ ,  $\text{CrCl}_3$ , and  $\text{MgCl}_2$ . Ferric chloride and two percent phosphoric acid were used in most cases and salt concentration was usually maintained at 30 milliequivalents per 100 g of soil, which is the cation exchange capacity of VBC.

Although none of the samples waterproofed with polyvalent metal ions resisted immediate immersion, Table 4 shows that the addition of salts noticeably improved the water resistance of the soil after 24-hr humid cure. Here strengths of up to 80 psi were obtained, and 1.62 percent  $\text{FeCl}_3$  seems to be as effective as 0.5 percent octylamine.

Table 4 also indicates that none of the salts have any appreciable effect on 24-hr humid cure strength, if used at a concentration equivalent to the cation exchange capacity of the soil. When double this amount of  $\text{FeCl}_3$  was used, however, humid cure strength was decreased. In these samples a reddish exudation was observed; because

TABLE 4

INVESTIGATION OF POLYVALENT METALLIC SALTS AS WATERPROOFERS FOR VBC STABILIZED WITH  $\text{H}_3\text{PO}_4$

$\text{H}_3\text{PO}_4$ (%)	$\text{FeCl}_3$ (%)	Pre-treatment (hr)	24-Hr Immersion Comp. Str. (psi)	24-Hr Humid Cure			24-Hr Humid Cure and 24-Hr Immersion		
				Comp. Strength (psi)	Dry Density (pcf)	Volatiles (%)	Comp. Strength (psi)	Dry Density (pcf)	Volatiles (%)
2	—	—	0	240 ± 5	112.0 ± 0.2	16.9 ± 0.4	0	—	—
2	0.81	48 <sup>a</sup>	0	242 ± 20	113.7 ± 0.1	15.4 ± 0	TWTT <sup>b</sup>	—	—
2	0.81	— <sup>a</sup>	0	333 ± 15	115.2 ± 0.4	15.25 ± 0.1	70 ± 5	108.5 ± 0.6	21.1 ± 0.1
2	1.62	0	0	220 ± 11	112.4 ± 0.4	15.7 ± 0.1	TWTT <sup>b</sup>	—	—
2	1.62	2	0	174 ± 7	112.5 ± 0.3	15.75 ± 0.1	TWTT <sup>b</sup>	—	—
2	1.62	2	TWTT <sup>b</sup>	280 ± 35	112.9 ± 0.2	17.9 ± 0.2	60 ± 5	106.3 ± 0.5	22.5 ± 0.4
2	1.62	2	TWTT <sup>b</sup>	196 ± 5	110.9 ± 0.2	19.0 ± 0.1	80 ± 15	105.4 ± 0.8	21.7 ± 0.3
2	3.24	24	0	162 ± 16	115.2 ± 0.2	15.1 ± 0.1	TWTT <sup>b</sup>	—	—
3	1.62	0	0	380 ± 20	115.1 ± 0.1	15.2 ± 0.2	48 ± 7	108.0	20.5
5	1.62	0	0	255 ± 15	115.1 ± 0.1	16.0 ± 0.1	79 ± 4	108.3 ± 0.2	20.6 ± 0.1
Other Salts									
2	1.33% $\text{AlCl}_3$	0	TWTT <sup>b</sup>	360 ± 10	114.0 ± 0.4	17.4 ± 0.2	32 ± 1	—	—
2	1.58% $\text{CrCl}_3$	0	0	258 ± 8	113.9 ± 0.4	17.3 ± 0.2	7 ± 2	—	—
2	1.43% $\text{MgCl}_2$	0	0	210 ± 25	114.4 ± 0.7	17.1 ± 0.3	TWTT <sup>b</sup>	—	—

<sup>a</sup>For these samples, the soil was pretreated for 48 hr with  $\text{FeCl}_3$ , then dried at 110 deg for 24 hr, and finally re-equilibrated with water for 24 hr.

<sup>b</sup>TWTT means that the samples retained their shape but were too weak to test.

TABLE 5  
STABILIZATION OF MCS WITH 1.25 PERCENT ORTHORHOMBIC  $P_2O_5$   
(Equivalent to 2 Percent Phosphoric Acid)

Formulation and Manner of Addition to Soil	1-Day Humid Cure			7-Day Humid Cure			14-Day Humid Cure		
	Comp. Str. (psi)	% Volatile at Test	Dry Solids Density at Test (pcf)	Comp. Str. (psi)	% Volatile at Test	Dry Solids Density at Test (pcf)	Comp. Str. (psi)	% Volatile at Test	Dry Solids Density at Test (pcf)
2% $H_3PO_4$	184 ± 4	11.4	128.3	—	—	—	271 ± 15	11.3	128.5
2% $H_3PO_4$	242 ± 9	11.0	131.0	—	—	—	300	11.0	131.0
1.25% $P_2O_5$ dry mixed with soil	215 ± 4	11.1 ± 0.1	133.1 ± 0.1	364	9.6	133.4	—	—	—
1.25% $P_2O_5$ dry mixed with soil	230 ± 5	9.2 ± 0.1	129.4 ± 0.1	—	—	—	300 ± 87	8.7 ± 0.3	127.8 ± 0.6
1.25% $P_2O_5$ solution, prepared by letting $P_2O_5$ stand undisturbed in water overnight	268 ± 8	10.2 ± 0.2	134.8 ± 0.2	386 ± 22	10.6 ± 0.1	136.1 ± 0.1	490 ± 5	9.2 ± 0.1	136.5 ± 1.1
1.25% $P_2O_5$ solution, prepared by heating the $P_2O_5$ - $H_2O$ mixture until clear solution was obtained (½ hr)	246 ± 4	10.6 ± 0.1	133.3 ± 0.4	571 ± 69	9.6 ± 0.4	133.1 ± 0.4	680	8.1	132.7
3.75% $P_2O_5$ solution, prepared by heating the $P_2O_5$ - $H_2O$ mixture until clear solution was obtained (½ hr)	231 ± 29	10.6 ± 0	131.0 ± 0	527 ± 18	10.3	132.0	585 ± 10	10.2 ± 0.1	131.1 ± 0.3

Note: All percentages based on dry soil weight.

this effect is not produced at lower concentrations it is concluded that a certain amount of salt combines with the soil, presumably in a cation exchange reaction. The excess  $FeCl_3$  apparently is detrimental to humid cure strength. It is also likely that reaction of excess ferric ion with phosphoric acid to produce insoluble ferric phosphate reduces the quantity of acid available for cementation of the clay.

It was believed that pretreatment of the soil with the ferric chloride before adding the phosphoric acid might allow the salt to diffuse into and adsorb on the clay. The effect of the pretreatment in practice (Table 4) is not entirely clear, but it does not seem to increase even the resistance of the soil to immediate immersion, the condition under which it would be expected to be most noticeable.

In one case, however, soil was pretreated with  $FeCl_3$ , allowed to equilibrate, dried at 110 C for 24 hr, then re-equilibrated with water for 24 hr and stabilized with  $H_3PO_4$ . In this case the soil proved to be much more water resistant than other  $FeCl_3$ -treated samples. This supports the postulate that ferric ion exchanges with univalent ions in

TABLE 6  
EFFECT OF TRACE QUANTITIES OF  $\alpha$ - $P_2O_5$  ON MCS STABILIZED WITH THE EQUIVALENT OF 2%  $H_3PO_4$

Formulation and Manner of Addition to Soil	1-Day Humid Cure			7-Day Humid Cure			14-Day Humid Cure		
	Comp. Str. (psi)	% Volatile at Test	Dry Solids Density at Test (pcf)	Comp. Str. (psi)	% Volatile at Test	Dry Solids Density at Test (pcf)	Comp. Str. (psi)	% Volatile at Test	Dry Solids Density at Test (pcf)
2% $H_3PO_4$ alone	242 ± 9	11.0	131.0	—	—	—	300	11.0	131.0
2% $H_3PO_4$ + 0.1% $P_2O_5$ added to acid	315 ± 3	9.8 ± 0.1	135.2 ± 0.3	333 ± 54	9.6 ± 0	134.7 ± 0.2	538 ± 2	9.0 ± 0	135.1 ± 0.2
Phosphate rock-sulfuric acid mixture + 0.1% $P_2O_5$ added to soil dry (phosphate rock dry mixed)	188 ± 15	9.5 ± 0	126.3 ± 0.3	200 ± 28	8.8 ± 0.4	127.0 ± 1.0	—	—	—
Phosphate rock-sulfuric acid mixture alone (phosphate rock dry mixed)	265 ± 35	10.5 ± 0.1	128.7 ± 0.7	315 ± 65	10.5 ± 0.1	128.7 ± 0.7	—	—	—
Phosphate rock-sulfuric acid mixture + 0.1% $P_2O_5$ added to acid (phosphate rock dry mixed)	341 ± 23	9.8 ± 0.1	134.6 ± 0.6	422 ± 19	9.8 ± 0.1	134.1 ± 0.2	437 ± 27	10.9 ± 0.2	134.3 ± 0.1

the montmorillonoid lattice. The lattice shrinkage occurring on drying would then be irreversible due to the binding effect of the ferric ions. In fact, soil treated in this way behaves more like a clayey silt than a heavy clay.

Of the other polyvalent metal salts tested, only  $AlCl_3$  gave results comparable to those obtained with  $FeCl_3$  (Table 4).  $CrCl_3$  and  $MgCl_2$  appear to be much less effective as waterproofers.

### Soil Stabilization with Orthorhombic Phosphoric Anhydride

**Characteristics of Orthorhombic Phosphoric Anhydride ( $\alpha$ - $P_2O_5$ ).**— Previous work (2) showed that  $P_2O_5$  was effective as a soil stabilizer and possessed several important advantages over phosphoric acid, including lower material requirements and higher permissible soil water content. The irritating properties of the powdered anhydride, however, and its violent reactivity with water, make its use in the field both troublesome and hazardous.

Recently a new crystalline form of phosphoric anhydride (orthorhombic  $P_2O_5$ ) has become available which, because of its lower reactivity, seemed promising as a soil stabilizer. It is the anhydride of metaphosphoric acid, rather than orthophosphoric acid, and differs from normal  $P_2O_5$  in that it is only slowly soluble in cold water, yielding a solution of the following composition (6):

Orthophosphoric acid	6.3 %
Pyrophosphoric acid	0.6 %
Tetraphosphoric acid	0.4 %
Tri-metaphosphoric acid	8.6 %
Tetra-metaphosphoric acid	9.1 %
Penta-metaphosphoric acid	6.4 %
Higher ( $P_6$ - $P_9$ ) metaphosphoric acids	12.0 %
Poly ( $P_{10}$ and up) metaphosphoric acid	56.8 %

Although the reaction with water is quite exothermic, the slow rate of hydration prevents extreme local heat generation. At room temperature, solution of an amount sufficient to produce the equivalent of 50 percent  $H_3PO_4$  solution occurs only after about 24 hr. By keeping the mixture near its boiling point, however, the process can be speeded and a clear solution obtained in 30 min. It is presumed that in the soil these various condensed phosphoric acids are converted progressively into orthophosphoric acid, the stable form in aqueous medium, which reacts with the soil alumina in the usual manner.

**Stabilization of MCS with  $\alpha$ - $P_2O_5$**  (Tables 5 and 6). — Initial tests were made by hand mixing the soil at approximately 8 percent volatiles with dry  $\alpha$ - $P_2O_5$  and then adding the rest of the required water during the mechanical mixing process (dry mixing). Strengths comparable to those obtained with  $H_3PO_4$  were developed in this manner (Table 5).

To clarify the effect of the slow dissolution rate of the anhydride on strength development, soil samples were prepared using the poly-acid solution produced by dissolving the anhydride in water. Results (Table 5) show that, although the 1-day humid cure strength is not significantly improved by this technique, the 7- and 14-day strengths are markedly increased, greatly surpassing even those obtained with orthophosphoric acid. From these observations, it is concluded that (a) polyphosphoric acid is superior to orthophosphoric acid as a soil stabilizer (at least for moderately fine-grained soils), and (b) orthorhombic  $P_2O_5$  is less effective as a stabilizer than its hydration products. The latter result probably is explained by the fact that admixture of granular anhydride with the soil leads to non-uniformity of acid distribution, and hence non-uniform cementation on cure.

The excellent stabilizing action of polyphosphoric acids appears primarily to result from the pronounced dispersive action of these acids on MCS, allowing increases in compacted density of as much as 4 pcf. Because very small quantities of orthorhombic  $P_2O_5$  (or its hydration products) were found to exert this dispersing effect, the influence of trace quantities (0.1 percent by weight of soil) of the anhydride on strength of other acidic-phosphorus-compound stabilized soils was studied. Results are given in Table 6.

**TABLE 7**  
**COST COMPARISONS FOR EQUIVALENT STABILIZATION OF MCS**

Method	Cost	
	¢/lb	\$/cu yd, soil
<b>Orthophosphoric acid</b>		
H <sub>3</sub> PO <sub>4</sub> : 54 lb/cu yd	6.65	3.59
Na <sub>2</sub> SiF <sub>6</sub> : 13 lb/cu yd	6.50	0.85
<b>Total</b>		<b>4.44</b>
<b>Phosphate rock-sulfuric acid (pre-blended)</b>		
Phosphate rock: 105 lb/cu yd	0.30	0.32
Sulfuric acid: 70 lb/cu yd	1.05	0.74
<b>Total</b>		<b>1.06</b>
<b>Phosphate rock-sulfuric acid (no pre-blending)</b>		
Phosphate rock: 105 lb/cu yd	0.30	0.32
Sulfuric acid: 70 lb/cu yd	1.05	0.74
Orthorhombic P <sub>2</sub> O <sub>5</sub> : 3 lb/cu yd	30.0 <sup>a</sup>	0.90
<b>Total</b>		<b>1.96</b>
<b>Orthophosphoric acid-o-P<sub>2</sub>O<sub>5</sub></b>		
H <sub>3</sub> PO <sub>4</sub> : 30 lb/cu yd (est.)	6.65	2.00
0-P <sub>2</sub> O <sub>5</sub> : 3 lb/cu yd	30.0 <sup>a</sup>	0.90
<b>Total</b>		<b>2.90</b>

<sup>a</sup>Experimental product; estimated tonnage price.

When added to orthophosphoric acid, traces of o-P<sub>2</sub>O<sub>5</sub> very greatly improve humid-cure strength, measured values closely matching those obtained using polyphosphoric acids alone, as do compacted densities. This essentially confirms the hypothesis that the dispersive action of polyphosphoric acid is responsible for its superior stabilizing ability. Of even greater practical interest (Table 6) is the fact that addition of a trace of o-P<sub>2</sub>O<sub>5</sub> to sulfuric acid, followed by mixing with soil blended with phosphate rock (that is, without preblending rock and acid), yields humid-cure strength values significantly higher than obtained with orthophosphoric acid alone. Because the incremental cost of 0.1 percent o-P<sub>2</sub>O<sub>5</sub> is rather small, this modified formulation appears to offer excellent promise, on both economic and technical grounds, as a very low cost soil stabilizer. Whether such a stabilizing system will be operative in heavy clay soils (where phosphoric acid-amine mixtures appear to be uniquely effective) remains to be established; studies with this objective in mind are currently under way.

#### PRACTICAL AND ECONOMIC ASPECTS OF PHOSPHORIC SOIL STABILIZATION

The results of this study indicate that "marginal" soils, such as MCS, can be successfully stabilized not only with phosphoric acid, but also with (a) phosphate rock-sulfuric acid mixtures, provided rock and acid are preblended before compaction in soil; (b) phosphate rock-sulfuric acid mixtures without preblending if 0.1 percent orthorhombic P<sub>2</sub>O<sub>5</sub> is added; and (c) orthophosphoric acid-o-P<sub>2</sub>O<sub>5</sub> mixtures. Comparative costs of these four systems are given in Table 7. It is evident from these calculations that rock-sulfuric mixtures are substantially lower in cost than treatments based on orthophosphoric acid alone, or with additives. Indeed, the cost of treatment with a rock-sulfuric preblend is lower than that of a conventional stabilization with lime, cement, or asphalt which would produce the equivalent mechanical strength. From the stand-

point of convenience in the field, use of a plant-mixed preblend of rock and acid would appear to be preferable to the two-step operation of mixing first rock, and then acid with the soil. Should the two-stage treatment for any reason prove to be desirable, however, this could be carried out successfully, but at an increment of cost (for orthorhombic  $P_2O_5$ ) of about 90 cents per cubic yard of soil.

For stabilization of heavy clay soils, such as VBC, where significant amounts of waterproofing additives are required in addition to phosphoric acid, materials costs are substantially higher than for "marginal" soils such as MCS. The discovery that ferric chloride can be used as a replacement for octylamine as a waterproofer in VBC, however, results in a significant cost reduction: treatment with 2 percent phosphoric acid and 0.5 percent octylamine (at 30 cents per lb) costs about \$7.60/cu yd, while 2 percent phosphoric acid and 1.6 percent ferric chloride (at 5 cents per lb) costs only \$5.74/cu yd. Despite this improvement, the expense of stabilization of heavy clays remains much higher than that of soils of moderate plasticity. Continuation of the search for a low-cost stabilizing system for heavy clays thus remains a major research objective.

#### ACKNOWLEDGMENTS

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# Use of Phosphoric Acid in Soil Stabilization

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The purpose of this investigation was to determine the effect of phosphoric acid treatments on the stability of compacted calcareous and non-calcareous clay soils. Unimmersed and immersed strengths and freeze-thaw resistance of phosphoric acid-treated soil specimens were used as criteria for stability.

The unimmersed and immersed strengths of phosphoric acid-treated soils were studied relative to the curing conditions, carbonate contents of soils, and use of secondary additives. Moist curing was found beneficial to strength. Soils containing  $\text{CaCO}_3$  required more phosphoric acid than  $\text{CaCO}_3$  free soils because  $\text{CaCO}_3$  reacted with the acid available for stabilization. No worthwhile benefit was found from the use of secondary additives.

X-ray diffraction analysis and quantitative chemical analyses were used to trace the chemical reactions after treatment. These analyses lead to the conclusion that phosphoric acid and clay minerals react to produce an amorphous aluminum phosphate gel which serves as a cementing agent.

● PHOSPHORIC acid stabilization of clay soils is a relatively new method, first introduced in 1957 by Lyons (5) who found that compacted specimens of clayey soils treated with as little as 2 percent phosphoric acid withstood exposure to water and to wintering.

An evaluation study by the Soil Research Laboratory, Iowa Engineering Experiment Station, Iowa State University, showed that the water resistance of a non-calcareous plastic loess was improved by treatments with phosphoric acid, or phosphoric acid and salts of calcium, magnesium or aluminum (3). It was hypothesized that the immersed strength of the compacted, moist-cured mixtures was due to the formation of amorphous phosphates in a gel form, and that the metallic cations needed for the insoluble gel formation could be provided by the soil or by adding salts of the desired cations.

Previous work indicated that phosphoric acid, used in amounts between 1 and 10 percent of the dry soil weight, was a promising stabilizer for fine-grained soils (6). Strengths were dependent on moisture content and density, highest strengths were developed when specimens were cured under humid conditions, and certain additives accelerated the curing process and improved the strength retention after immersion. On the basis of more recent studies it was concluded that phosphoric acid with added fluorine compounds and/or amines promised low-cost stabilization of fine-grained, carbonate-free soils under field conditions (7).

The present investigation was undertaken to evaluate further the use of phosphoric acid for stabilizing clay soils. The principal objectives of the investigation were:

1. Determine the effect of phosphoric acid treatments on the strength of compacted clay soils.
2. Determine the resistance of phosphoric acid stabilized clay soil to alternate cycles of freeze and thaw.
3. Determine the factors and soil compositional variables, if any, which may limit the use of phosphoric acid stabilization.

To accomplish these objectives it was first necessary to determine:

1. The relationship between the molding moisture content for maximum dry density and the molding moisture content for maximum immersed strength for the phosphoric acid-treated soils.
2. The best method of curing phosphoric acid-treated soil specimens.

A preliminary study was made to determine whether combination treatments of a clay soil with phosphoric acid and other chemicals would give better immersed strength than obtainable from phosphoric acid treatment alone.

## MATERIALS USED

### Soils

Descriptions and properties of the seven soils used in the investigation are given in Table 1. The first five soils were used in the strength studies; the leached and unleached Kansan till samples were used in supplementary studies.

The five main study soils were chosen so that the effects of different clay minerals and of varying calcium carbonate content could be evaluated. Three of these soils had montmorillonite as the dominant clay mineral: the non-calcareous plastic loess, the non-calcareous Kansan gumbotil and the calcareous alluvial clay. The dominant clay minerals in the calcareous Detroit clay were illite and chlorite; kaolinite (halloysite) and vermiculite were the dominant clay minerals in the non-calcareous Virginia clay.

### Phosphoric Acid

The phosphoric acid used was reagent grade, 86 percent orthophosphoric acid,  $H_3PO_4$ . All additives are expressed as percentages of pure (100 percent) phosphoric acid, based on the oven-dry weight of the soil.

### Other Chemicals

The following chemicals were used with phosphoric acid. Additives of these chemicals are expressed as percentages of the oven-dry weight of the soil, with no correction made for crystal water or solvents.

Aluminum Sulfate. — Reagent grade aluminum sulfate with 18 moles of crystal water,  $Al_2(SO_4)_3 \cdot 18H_2O$ , was dissolved in distilled water before adding to the soil.

Arquad 2HT. — Suspensions of Arquad 2HT in distilled water were prepared from chemical supplied by Armour and Company, Chicago, Ill. This chemical contained 75 percent active Arquad 2HT.

Sodium Fluoride. — Reagent grade sodium fluoride, NaF, was added to the soil in powdered form.

Aluminum Hydroxide. — Reagent grade aluminum hydroxide,  $Al_2O_3 \cdot 3H_2O$ , was used in powdered form.

Calcium Chloride. — Reagent grade anhydrous calcium chloride,  $CaCl_2$ , was dissolved in distilled water before being added to the soil.

Sodium Metasilicate. — Reagent grade sodium metasilicate with 9 moles of crystal water,  $Na_2SiO_3 \cdot 9H_2O$ , was dissolved in distilled water before being added to the soil.

Calcium Carbonate. — Reagent grade calcium carbonate,  $CaCO_3$ , was added to the soil in powdered form.

## METHODS OF PREPARING AND TESTING SPECIMENS

### Preparation of Mixtures

Air dried soil passing the No. 10 sieve was weighed and placed in an aluminum mixing bowl. Distilled water for the specified moisture content was added slowly to the soil while it was being mixed with a Hobart Model C-100 mixer. (Allowance was made for the water in the 86 percent phosphoric acid and the hygroscopic water of the soil in calculating the mix water needed for a specified moisture content.) After adding the water, phosphoric acid in the required amount was added slowly as mixing continued

TABLE 1  
DESCRIPTION AND PROPERTIES OF SOIL SAMPLES

Sample	Plastic Loess (Lab. No. 528-4)	Kansan Gumbotl (Lab. No. 528-8)	Alluvial Clay (Lab. No. 827-1)	Detroit Clay (Lab. No. AR-4)	Virginia Clay (Lab. No. AR-5)	Leached Kansan Till (Lab. No. 418-4)	Unleached Kansan Till (Lab. No. 409-12C)
Geological description	Wisconsin-age loess, non-calcareous	Kansan-age gumbotl, highly weathered, non-calcareous	Missouri River floodplain alluvium, calcareous	Probably Wisconsin-age glacial till, calcareous	Residual soil over diorite, non-calcareous	Kansan-age till, non-calcareous	Kansan-age glacial till, calcareous
Location	Keokuk Co., Ia.	Keokuk Co., Ia.	Harrison Co., Iowa	Monroe Co., Mich.	Virginia	Warren Co., Iowa	Ringgold Co., Iowa
Soil Series	Mahaska	Mahaska	None	Unknown	Davidson	Shelby	Shelby (Burchard)
Horizon	C	Fossil B <sup>†</sup>	None	C	B	C	C
Sampling depth, ft	3 to 6 1/2	7 1/2 to 8 1/2	0 to 4	Unknown	Unknown	7 to 9	4 1/2 to 10 1/2
Textural composition, %							
Gravel (>2 mm)	0	0	0.0	0.0	0	1.3	0
Sand (2-0.074 mm)	0.2	16.0	0.5	7.0	21.0	32.6	31.5
Silt (74-5 μ)	60.8	13.5	27.5	36.0	37.0	29.1	30.0
Clay (<5 μ)	39.0	70.5	72.0	57.0	42.0	37.0	38.5
Colloids (<1 μ)	31.0	66.0	50.0	43.0	29.0	27.0	31.0
Predominant clay mineral <sup>‡</sup>	Montmorillonite	Montmorillonite	Montmorillonite	Illite-Chlorite	Kaolinite (Halloysite) - Vermiculite	Montmorillonite	Montmorillonite
Chemical properties:							
Cat. ex. cap, m.e./gm <sup>b</sup>	23.5	41.0	36.4	14.5	12.4	14.8	29.5
Carbonates, %	0.0	0.8	5.1	7.2	0.7	0.0	4.9
pH <sup>c</sup>	5.6	6.5	6.5	8.4	5.9	7.4	6.3
Organic matter, %	0.2	0.2	1.3	0.6	2.6	0.75	0.2
Physical properties:							
Liquid limit, %	52.1	75.6	65.8	44.0	43.5	38.2	42.4
Plastic limit, %	20.0	25.8	26.4	21.1	27.0	15.1	20.5
Plasticity index, %	32.1	50.0	39.4	22.9	16.5	23.1	21.9
Classification							
Textural <sup>§</sup>	Silty clay	Clay	Clay	Clay	Clay	Clay	Clay
Engineering (AASHTO)	A-7-6(18)	A-7-6(20)	A-7-6(20)	A-7-6(14)	A-7-6(12)	A-6(11)	A-7-6(12)

<sup>a</sup>By x-ray diffraction analysis.

<sup>b</sup>For fraction passing No. 10 sieve.

<sup>c</sup>By versenate method, includes all acid extractable calcium as calcium carbonate.

<sup>d</sup>For fraction passing No. 10 sieve.

<sup>e</sup>From triangular chart developed by U.S. Bureau of Public Roads, but 0.074 mm was used as the lower limit of the sand fraction.

<sup>†</sup>Underlies C horizon loess of Mahaska series.

This mixing of water and phosphoric acid took about 1 min. The mixture was then machine mixed for an additional 1 min; hand mixed for 30 sec to insure that no unmixed soil adhered to the bottom of the bowl, machine mixed for another minute, and finally hand mixed for approximately 30 sec.

Chemicals used in the dry form were dry mixed with the soil for 1 min prior to the addition of water. The other chemicals were added to the soil in the mix water before adding the phosphoric acid.

A 2-min period of machine mixing following the addition of phosphoric acid was chosen because longer mixing produced a hard and lumpy mix. A shorter period of machine mixing did not properly mix all the ingredients with the soil.

## Molding

From each mix, six 2-in. diameter by  $2 \pm 0.05$ -in. high test specimens were prepared by use of the molding apparatus shown in Figure 1. The 5-lb hammer was dropped a distance of 12 in. The specimen was compacted by five blows of the hammer on each end of the specimen for a total of ten blows. This gave a compactive effort approximately equal to the standard ASTM (Proctor) compactive effort (ASTM Method D 558-57) when the molding is done on a wooden table (9). Immediately before molding the first specimen and immediately after molding the last specimen, a sample was taken from the mix for moisture content determination, and the two moisture contents were averaged. A moisture content of  $\pm 1$  percent of the specified moisture content was maintained. All specimens were weighed to the nearest 0.1 gram and measured for height to the nearest 0.001 in.

Mixtures of plastic loess and varying percentages of phosphoric acid were used to determine the optimum molding moisture content for maximum immersed strength. Specimens of each mixture were molded at different moisture contents to determine the dry density versus moisture content relationship. After the specimens had been weighed and measured for density calculations, they were moist cured for 7 days and then were immersed for 24 hr in distilled water before being tested for unconfined compressive



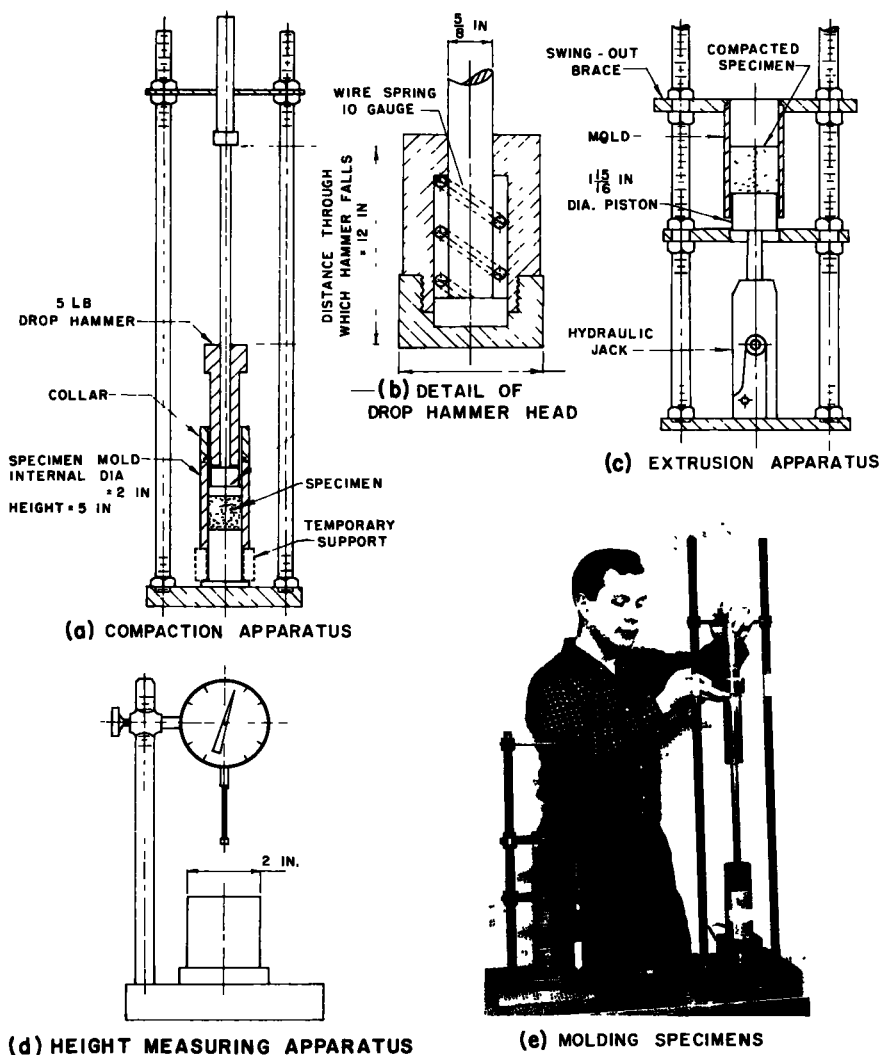


Figure 1. Apparatus for molding 2-in. diameter by 2-in. high test specimens.

strength. Optimum moisture contents for maximum dry density and for maximum immersed strength of each mixture were interpolated from graphs of density versus molding moisture content, and of immersed strength versus molding moisture content. These optimum moisture contents were then plotted against phosphoric acid content as shown in Figure 2. It is evident that the two optimum moisture contents are nearly identical. Hence, optimum molding moisture contents for mixtures of phosphoric acid and each of the other soils used in the main study were interpolated from dry density versus moisture content curves and specimens for strength studies were molded at these optimum moistures. The effect of phosphoric acid content on the optimum molding moisture contents so selected are shown in Figure 3 for each of the five soils. The data indicate that there is an inverse relationship between optimum moisture content and phosphoric acid content, with optimum moisture decreasing about 3 to 5 percent, based on the dry soil weight, as the phosphoric acid content is increased to 10 percent. The untreated soil generally had a slightly lower optimum moisture content than after treatment with 1 to 2 percent acid.

Curing

Two methods of curing phosphoric acid-treated soil were compared before selecting a curing method for the main study. The test specimens for the comparative study were prepared from a mixture of plastic loess and 7.5 percent phosphoric acid. In both curing

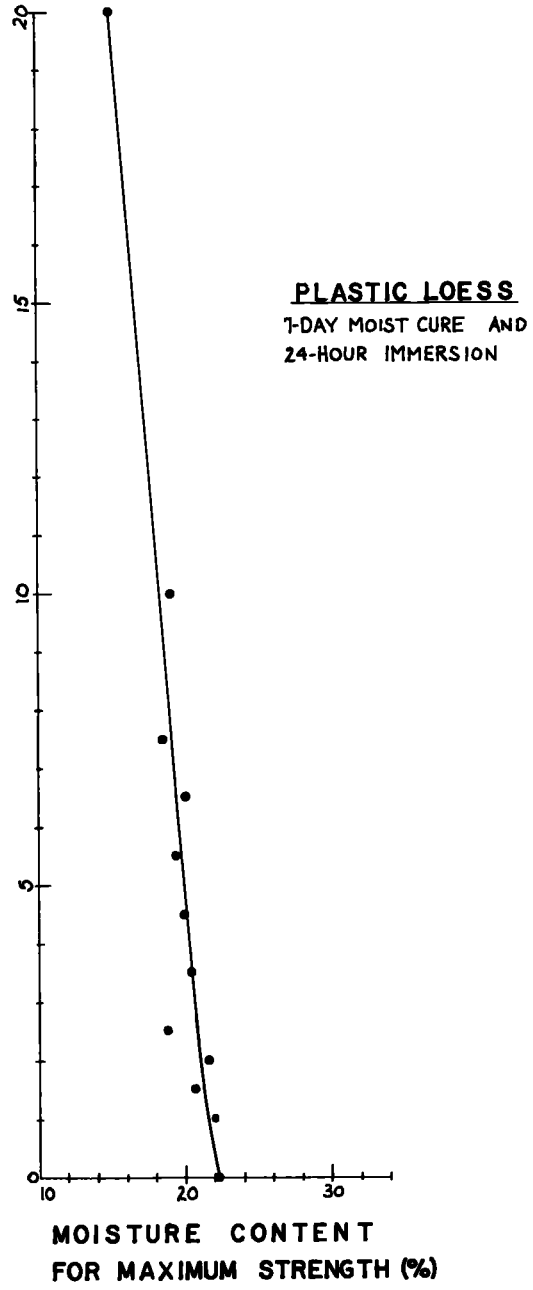
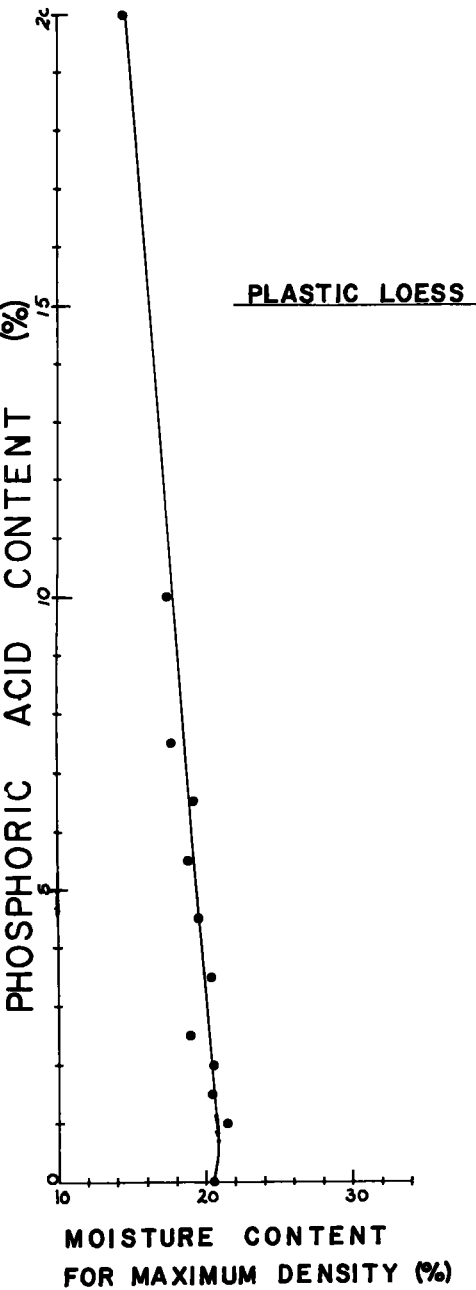


Figure 2. Effect of phosphoric acid content of mixtures with plastic loess on the optimum moisture contents for maximum standard Proctor density and for maximum immersed strength.

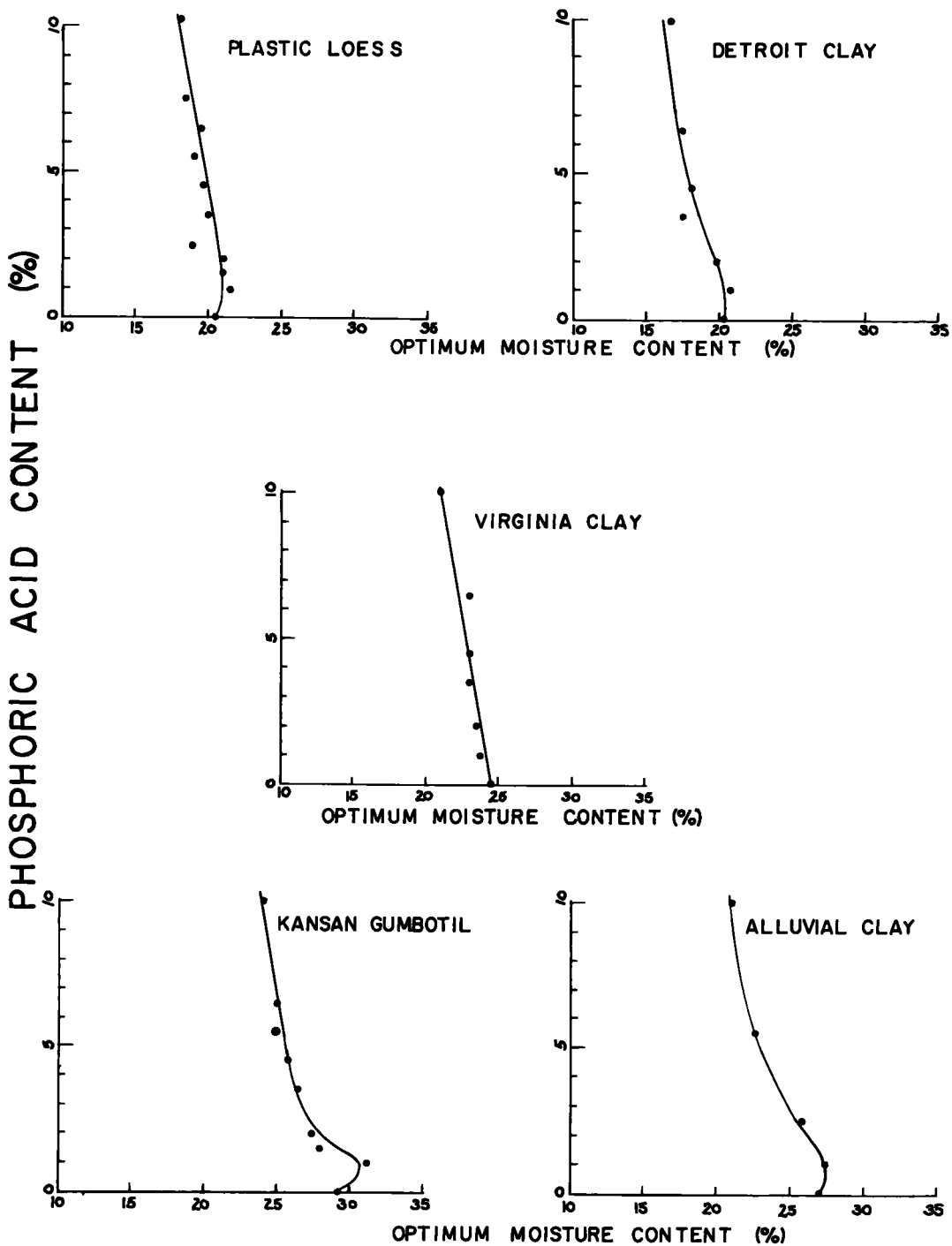


Figure 3. Effect of phosphoric acid content of mixtures with plastic loess, Detroit clay, Virginia clay, Kansan gumbotil and alluvial clay on optimum moisture content for maximum standard Proctor density.

methods the molded specimens, wrapped in wax paper and sealed with cellophane tape, were first placed in a curing room at  $70 \pm 4$  F and  $95 \pm 5$  percent relative humidity for 7, 14 or 28 days. In one method the specimens were unwrapped after each moist curing period and immersed in distilled water for 24 hr before testing for strength. In the other method, after moist curing 7, 14 or 28 days, the specimens were unwrapped and placed in open air for 7 days and then immersed for 24 hr in distilled water before testing for strength.

The specimens cured by the first method were molded at optimum moisture for maximum dry density because maximum density gives near maximum immersed strength as previously discussed. In the second curing method the specimens were molded at various moisture contents for 7 days moist curing and 7 days air curing; all specimens cured in this manner slaked on immersion. The last specimens to slake were those molded at optimum moisture for maximum dry density. For this reason all the specimens prepared for 14 or 28 days moist curing and 7 days air curing were molded at optimum moisture content; however, these specimens also slaked on immersion.

On the basis of these results it was decided, for the major portion of the investigation, to cure all specimens in the humid room and immediately after each curing period to either test them for unimmersed strength or immerse them in distilled water for 24 hr before testing for unconfined compressive strength. Immersion in water before testing was to simulate the detrimental effect of water on inadequately stabilized soil in the field, although complete immersion for 24 hr is a more severe treatment than usually encountered in roads.

#### Unconfined Compressive Strength Testing

The specimens were tested for unconfined compressive strength by a testing machine having a load travel rate of 0.1 in. per minute. The compressive strength was taken as the maximum test load in pounds per square inch sustained by a specimen. Unless otherwise indicated, strengths reported in this paper are averages for three specimens. In only a few did the difference between the individual values and the average value exceed 10 percent of the average value.

#### Freeze-Thaw Testing

The resistance of phosphoric acid stabilized soil to alternate cycles of freeze and

### ADDITIVES

0.6%  $H_3PO_4$

0.6%  $H_3PO_4 + 3\% Al_2(SO_4)_3 \cdot 18H_2O$

2%  $H_3PO_4$

2%  $H_3PO_4 + 3\% Al_2(SO_4)_3 \cdot 18H_2O$

2%  $H_3PO_4 + 10\% Al_2(SO_4)_3 \cdot 18H_2O$

4%  $H_3PO_4$

4%  $H_3PO_4 + 1\% Al_2(SO_4)_3 \cdot 18H_2O$

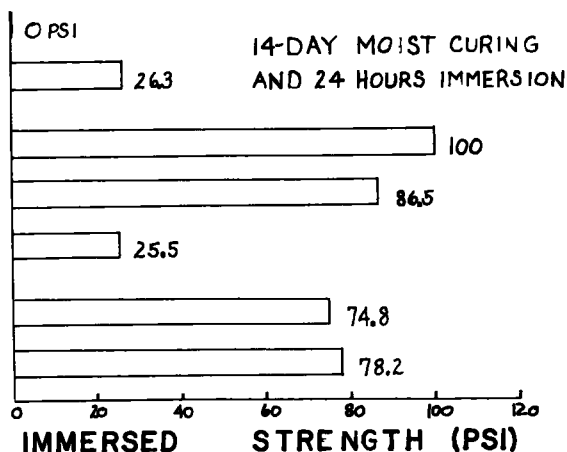


Figure 4. Immersed strengths of 14-day cured specimens of non-calcareous Kansan till treated with phosphoric acid and with phosphoric acid plus aluminum sulfate.

thaw was evaluated by the modified British standard freeze-thaw test (2). Only one soil, the plastic loess, treated with 2, 4.5 and 7.5 percent phosphoric acid was used in this study. For each curing period, two identical 2-in. diameter by  $2 \pm 0.05$ -in. high specimens were molded from each mixture at optimum moisture content; the curing periods were 7 and 14 days. One specimen was designated the control specimen and the other the freeze-thaw specimen (1, 2, 4). After moist curing, the top flat surface of both specimens was spray painted with a resin-base paint to a thickness of approximately 1 mm. The control specimen was immersed for 15 days in distilled water at a temperature of  $77 \pm 4$  F and then tested for unconfined compressive strength. The freeze-thaw specimen was immersed in distilled water at a temperature of  $77 \pm 4$  F for 24 hr, then was exposed alternately to temperatures of  $23 \pm 2$  F and  $77 \pm 4$  F for 14 cycles, each cycle lasting 24 hr, and finally tested for unconfined compressive strength. (A vacuum flask specimen container was used to cause freezing to occur from the top down and to supply unfrozen water to the bottom of the specimen throughout the test.)

### PRELIMINARY STUDY

Previous work had indicated the possibility that metallic salts or amines with phosphoric acid might give better results than phosphoric acid alone. A preliminary study was made to check this. The soil used was the leached (non-calcareous) kansan till (Table 1).

Figure 4 shows the 14-day cured immersed strengths produced by soil-phosphoric acid mixtures with and without aluminum sulfate. At 0.6 percent phosphoric acid, 3 percent aluminum sulfate increased immersed strength; at 2 percent phosphoric acid, 3 percent aluminum sulfate decreased immersed strength, and 10 percent caused a further decrease; at 4 percent phosphoric acid, 1 percent aluminum sulfate only slightly increased immersed strength.

### ADDITIVES

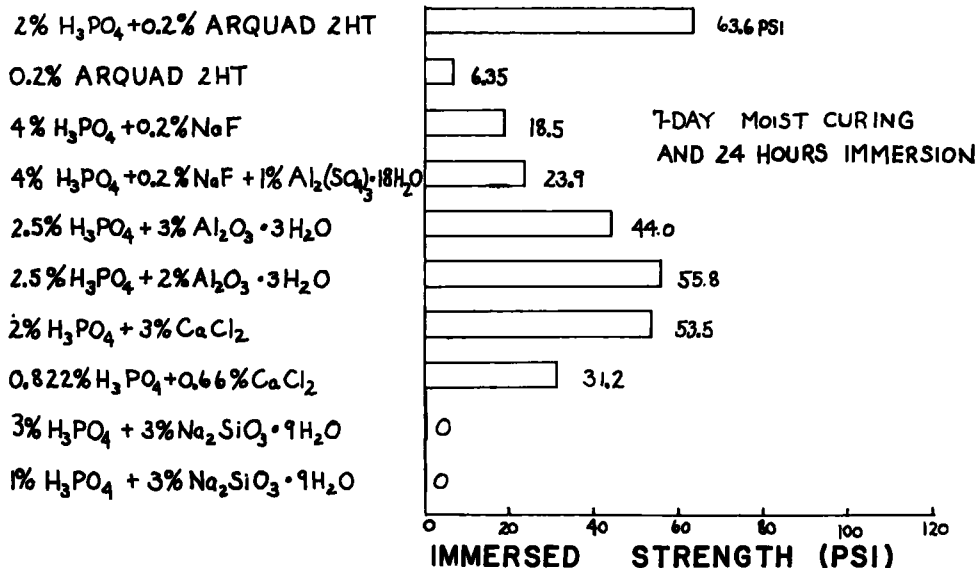


Figure 5. Immersed strengths of 7-day cured specimens of non-calcareous Kansan till treated with phosphoric acid plus various other chemicals.

The 7-day cured immersed strength results obtained with treatments of the other chemicals show that the highest immersed strength was obtained with 2 percent phosphoric acid and 0.2 percent Arquad 2HT, but it is probably only slightly better than obtainable with 2 percent phosphoric acid alone (Fig. 5). The other additives also did not produce promising results.

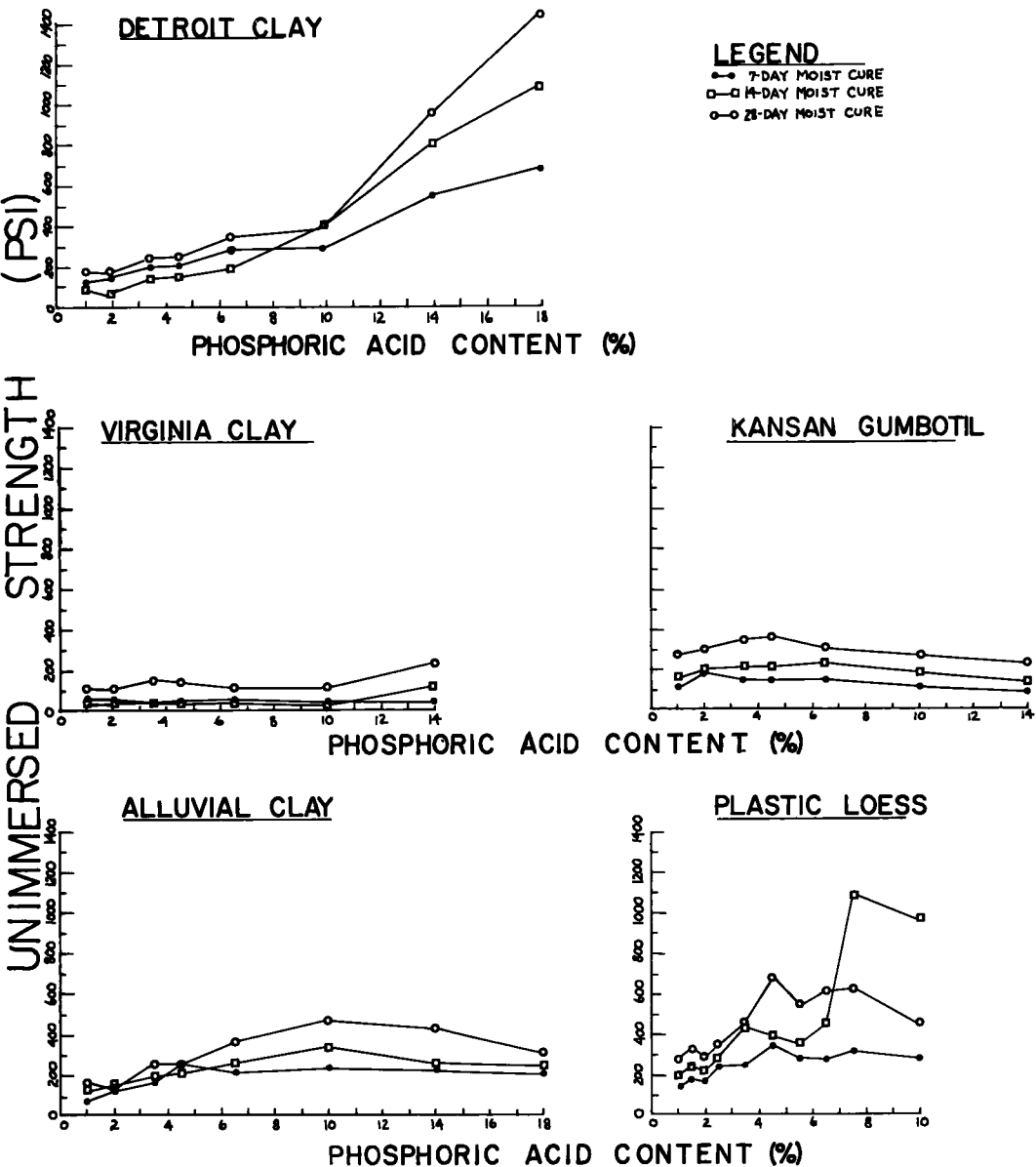


Figure 6. Effect of phosphoric acid content of clay soils on unimmersed strengths of 7-, 14- and 28-day cured test specimens.

**STRENGTH STUDIES**

**Unimmersed Strength**

The effects of amount of phosphoric acid treatment on the unimmersed specimen strengths of the five soils after 7, 14 and 28 days moist curing are shown in Figure 6. The montmorillonitic clay soils (Kansan gumbotil, alluvial clay, and plastic loess) generally reached a maximum strength with from 2 to 10 percent phosphoric acid. More chemical generally caused a decrease in strength.

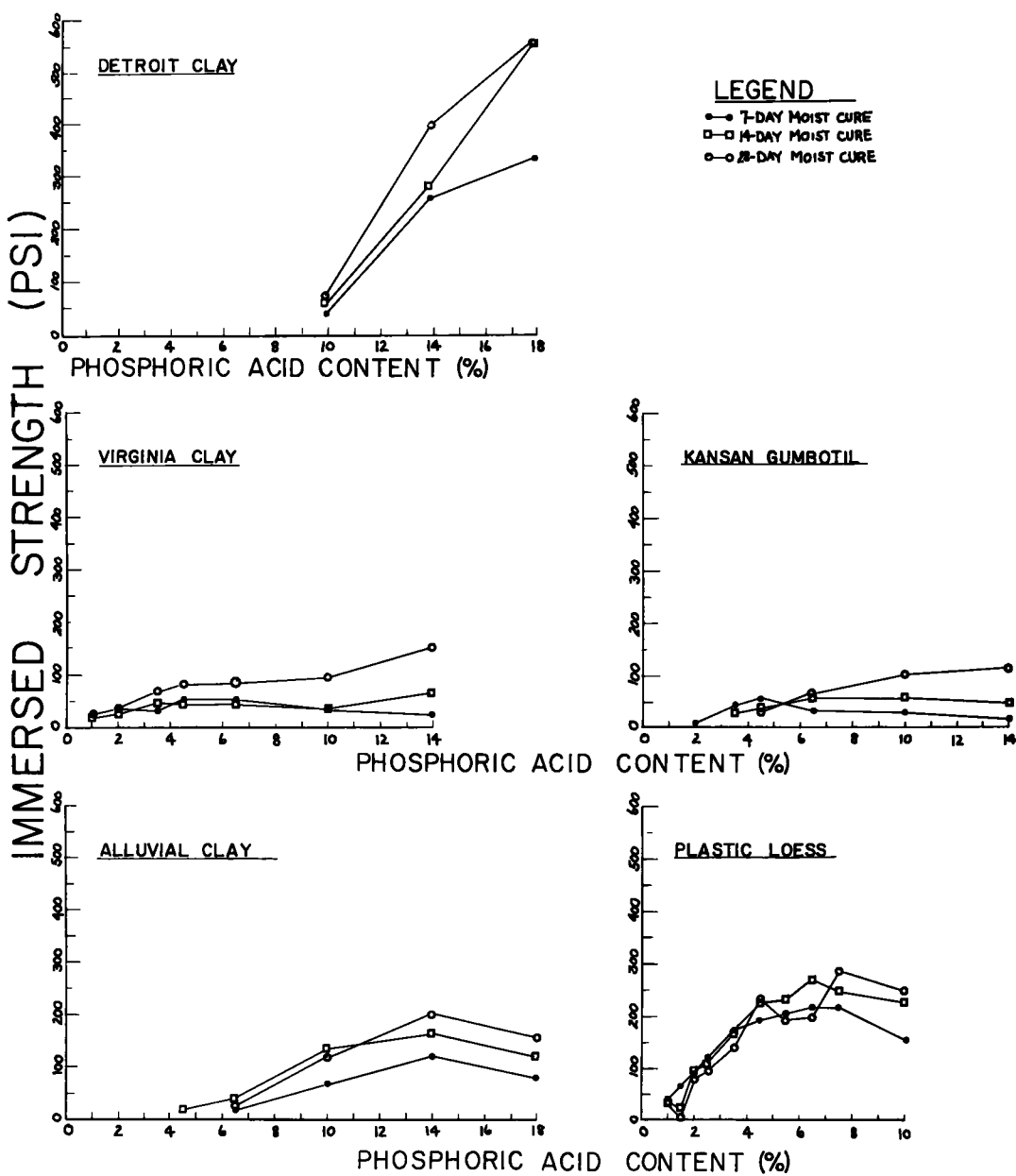


Figure 7. Effect of phosphoric acid content of clay soils on immersed strengths of 7-, 14- and 28-day cured test specimens.

No optimum phosphoric acid content was observed with the illitic-chloritic clay soil (Detroit clay). Up to 10 percent phosphoric acid, strength increased slowly at a uniform rate with increased acid content; but at 10 percent or above, the rate of strength gain increased sharply, and at all curing times it continued at the increased rate up to 18 percent acid, the largest amount used. This change in the rate of strength increase, discussed later, was due to the calcium carbonate content of the soil.

The strengths of the phosphoric acid-treated kaolinite-vermiculite clay soil (Virginia clay) specimens were low, and did not reach a maximum with from 1 to 14 percent acid. At 14 percent phosphoric acid, the 14- and 28-day strengths were still increasing, indicating that use of more acid might further increase strength.

Because the unimmersed strength of a stabilized soil is an indicator of cementing action derived from the stabilizer, phosphoric acid treatment of soil can be said to produce a cementing agent.

### Immersed Strength

The effect of the phosphoric acid treatments on the immersed strength of the five soils after 7, 14 and 28 days moist curing are shown in Figure 7. The montmorillonitic clay soils (Kansan gumbotil, alluvial clay, and plastic loess) tended to reach a maximum or near maximum immersed strength at each curing time with from 4 to 14 percent phosphoric acid. More than the optimum amount of acid generally caused a decrease in strength.

Specimens of the illitic-chloritic clay soil (Detroit clay) containing less than 10 percent phosphoric acid failed during immersion. However strength gain was rapid as acid content was increased above 10 percent, and at 18 percent the strength probably had not reached a maximum.

The 14- and 28-day strengths of the kaolinite-vermiculite clay soil (Virginia clay) specimens generally showed gradual increase with from 1 to 10 percent phosphoric acid without reaching to a maximum, and above 10 percent the rate of strength gain was somewhat greater. The 7-day strength reached a maximum at 4 to 5 percent acid, and strength slowly decreased with use of more acid.

The immersed strengths obtained in these tests indicate that the cementing product produced by the interaction of phosphoric acid and soil is water insoluble.

It will be noted in Figure 7 that the use of small percentages of phosphoric acid with

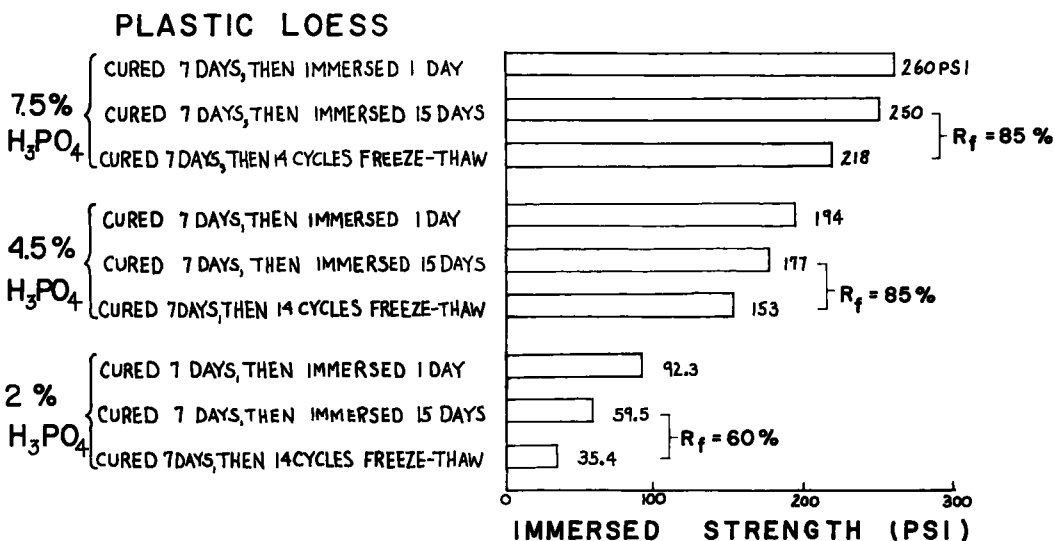


Figure 8. Results of modified British freeze-thaw tests of 7-day cured specimens of phosphoric acid-treated plastic loess.



Detroit clay and alluvial clay did not produce immersed strength. This is due to the calcium carbonate content of these soils, as explained later. Here it is enough to say that phosphoric acid reacts with calcium carbonate and this increases the amount of phosphoric acid necessary to obtain immersed strength.

### Analysis of the Strength Data

The different responses of different soils to phosphoric acid treatments (Figs. 6 and 7) may be hypothesized as being due to differences in the rapidity and the extent of the chemical reactions which furnish the reaction products that cement soil grains together and thus increase strength. Any remaining unreacted phosphoric acid lubricates the soil particles and tends to decrease the strength. Therefore, when the ratio of unreacted phosphoric acid to the reaction product is excessive, a net decrease in specimen strength occurs. The relative amounts of unreacted phosphoric acid and the reaction product depend on how far the reaction has progressed towards equilibrium, which for any given time depends on the rapidity of the reaction.

Generally both a maximum unimmersed and immersed strength were obtained with montmorillonitic clay soils; but with the range of treatment, maxima were not reached with the illite-chlorite and the kaolinite-vermiculite rich clay soils. The highest strengths were obtained with the illite-chlorite rich soil and the lowest strengths with the kaolinite-vermiculite rich soil. The three montmorillonitic soils gave mid-range strengths.

The rapid attainment of high strengths with the illitic-chloritic clay soil (Detroit clay) (Figs. 6 and 7) may be due to the rapid and complete reaction of phosphoric acid with the chlorite in the soil. As discussed later, x-ray diffraction analysis (Fig. 11) showed that chlorite is much more highly reactive with phosphoric acid than the other clay minerals; therefore it is believed that the chlorite in the Detroit clay reacted completely with phosphoric acid, which was largely responsible for the high strengths obtained.

Phosphoric acid-treated montmorillonitic clay soils attained moderate strengths rapidly (Figs. 6 and 7). This indicates that the rate of reaction may be as rapid in the montmorillonitic clay soils as it is in the illitic-chloritic clay soil, but that the extent of the reaction is not as great in the montmorillonitic clay soils. Consequently with the montmorillonitic clay soils an appreciable amount of unreacted phosphoric acid

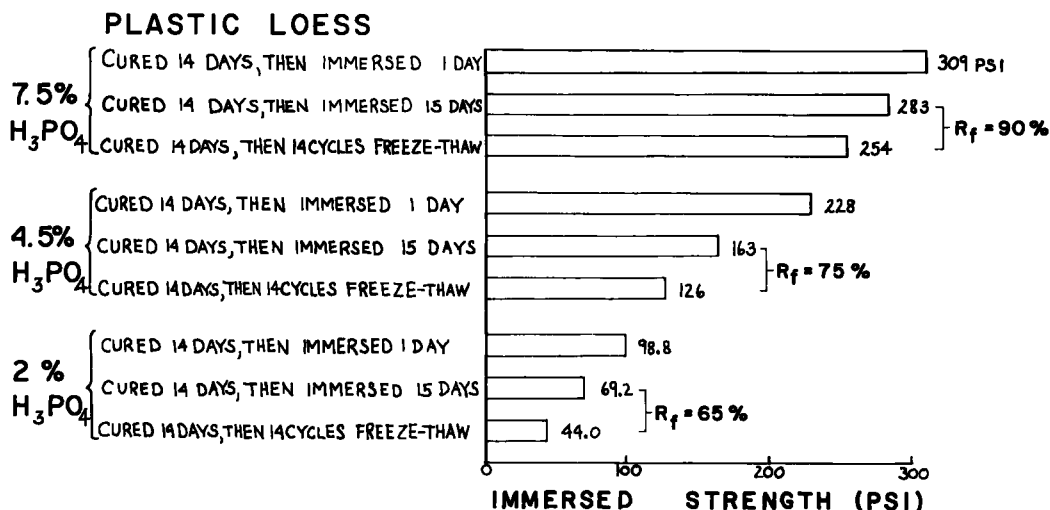


Figure 9. Results of modified British freeze-thaw tests of 14-day cured specimens of phosphoric acid-treated plastic loess.

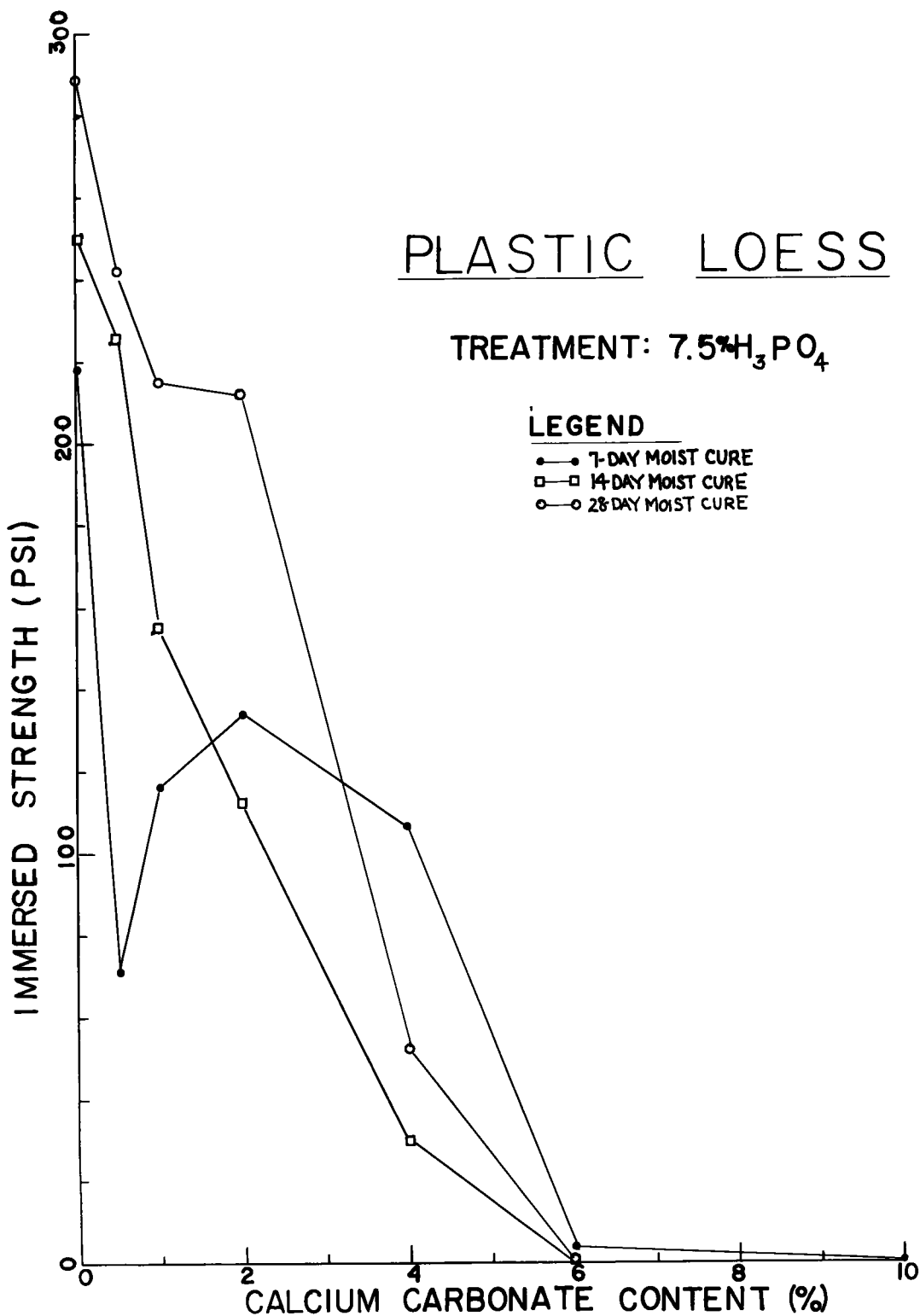


Figure 10. Effect of calcium carbonate additives to plastic loess on the immersed strengths of 7-, 14- and 28-day cured specimens containing 7.5 percent phosphoric acid, based on the dry weight of the loess- $CaCO_3$  mixture.

was always present when equilibrium was reached and its lubricating action tended to detract from the cementation effects of the reaction product. When the amount of unreacted phosphoric acid at higher levels of treatment became excessive, a net decrease in strength resulted, giving rise to the occurrence of the strength maxima in Figures 6 and 7. It also appears that both the amount of the optimum phosphoric acid treatment

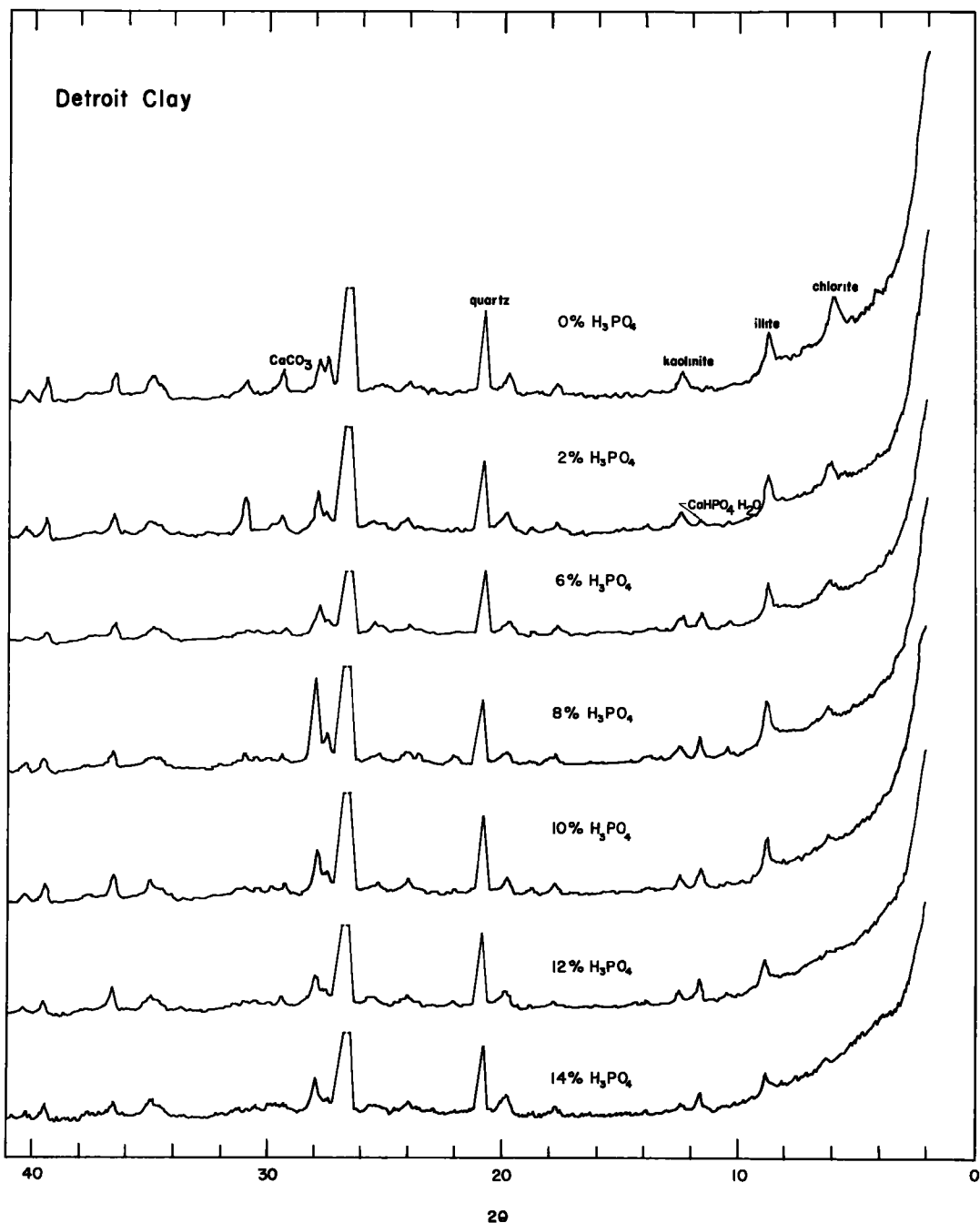


Figure 11. X-ray diffractometer chart showing the effect of various percentages of phosphoric acid on the clay minerals of the Detroit clay. (Filtered copper K $\alpha$  radiation was used.)

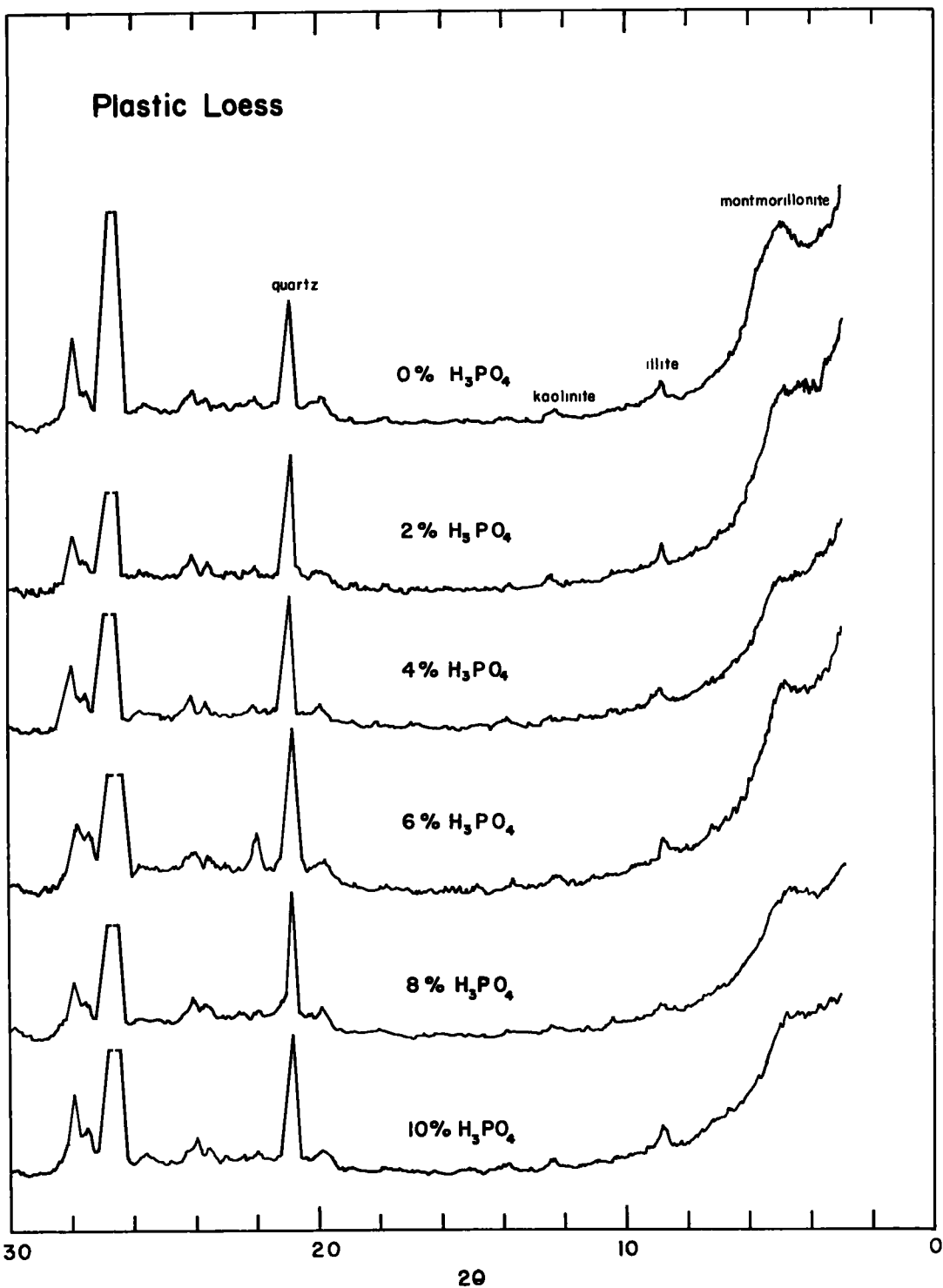


Figure 12. X-ray diffractometer chart showing the effect of various percentages of phosphoric acid on the clay minerals of the plastic loess. (Filtered copper K $\alpha$  radiation was used.)

and the magnitude of the maximum strength depend on the amount of clay-size material present in the montmorillonitic clay soils. Generally, the higher the clay content the more phosphoric acid was required for optimum results and the lower the maximum strength value. Reduction of the maximum strength with increasing clay content may be due to the resulting decrease in coarser grained fractions which contribute to internal friction in the stabilized soil and oppose the lubricating action of untreated phosphoric acid.

Phosphoric acid-treated kaolinite-vermiculite clay soil (Virginia clay) specimens generally showed a gradual increase in strength with longer curing time and with increased phosphoric acid contents. This may be attributed to a slow rate of reaction and the equilibrium conditions in favor of the products. The continued progress of the reaction more than compensates for the lubricating effect of the unreacted phosphoric acid as well as reducing the amount of unreacted material. Thus the slow but continuous strength gain results.

It is believed that the reaction product which furnishes cementation is essentially the same for all clay minerals and is a complex amorphous aluminum phosphate in the form of a gel—a conclusion also reached more or less by others (3, 6, 7). This point is further discussed in the mechanism section of this paper.

Summarizing the results of the foregoing discussion, chlorite clay minerals seem to be the most reactive of clay mineral groups investigated in this study. Montmorillonitic minerals react rapidly with phosphoric acid but the reaction does not proceed very far. Kaolinite and vermiculite react slowly with phosphoric acid, and the reaction may proceed to an appreciable extent. An accelerator or a catalyst may aid the stabilization of kaolinite or vermiculite rich soils by speeding the reaction but can hardly benefit montmorillonite rich soils, inasmuch as the reaction is already rapid. This conclusion is substantiated by the results reported by Michaels and Tausch (7), which show that fluorides and fluosilicates do not affect the phosphoric acid-clay reaction in montmorillonitic soils; however, they do accelerate the reaction in illitic soils. Thus, in all probability, illite reacts similarly to kaolinite or vermiculite. The general behavior of the illite group minerals cannot be predicted from the data of the present study, because the reaction was obscured by the presence of chlorite.

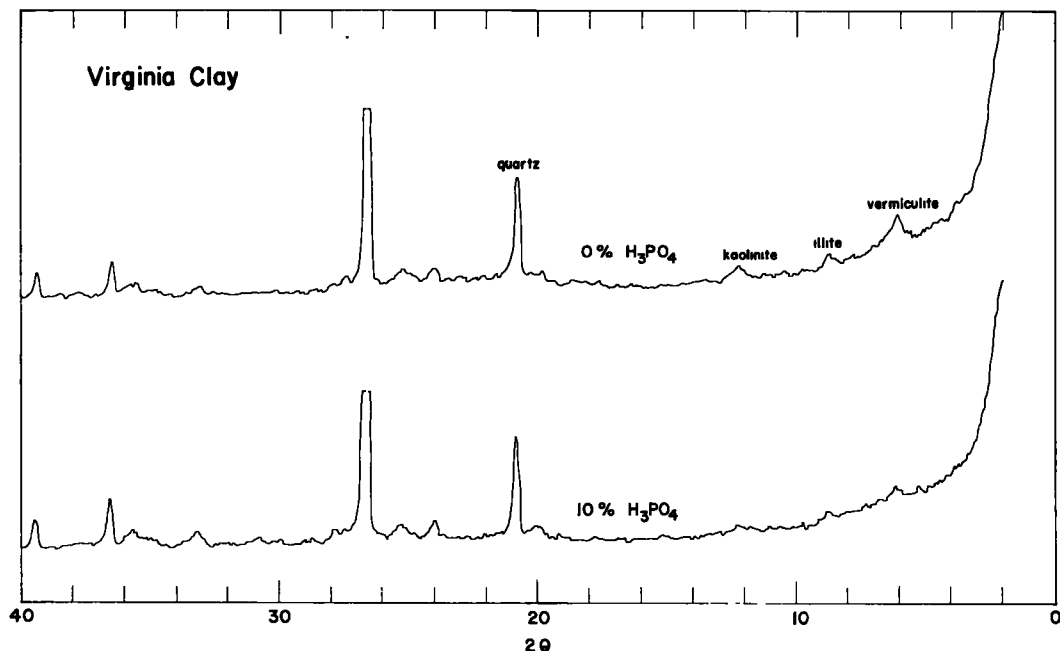


Figure 13. X-ray diffractometer chart showing the effect of 10 percent phosphoric acid on the clay minerals of the Virginia clay. (Filtered copper K $\alpha$  radiation was used.)

## FREEZE-THAW STUDY

The modified British freeze-thaw test (2) was used to evaluate the durability of 7- and 14-day cured specimens of selected mixtures of plastic loess and phosphoric acid. Bar graphs in Figures 8 and 9 show unconfined compressive strengths after curing and 1-day immersion; after curing and 15 days immersion (control specimen,  $p_c$ ); and after curing, 1-day immersion, and 14 cycles of freeze-thaw (freeze-thaw specimen,  $p_f$ ).

### Criteria of Durability

Two criteria have been used for the evaluation of the freeze-thaw test results (2). The first criterion is the value of the index of resistance to the effect of freezing ( $R_f$ ), defined by the following equation and expressed as a percentage of the nearest 5 percent:

$$R_f = \frac{100 p_f}{p_c}$$

The minimum  $R_f$  value considered indicative of satisfactory field performance of stabilized soils in road base courses in Great Britain is 75 percent. (Private communication, 1958, from D. J. Maclean, Harmondsworth, Middlesex, England — information on the British Standard Test No. 1924.) This same minimum value was adopted for the present study.

The second criterion requires a minimum value for the unconfined compressive strengths of the control specimen ( $p_c$ ) and the freeze-thaw specimen ( $p_f$ ). The tentative minimum requirement for both  $p_c$  and  $p_f$ , for stabilized base course mix design in Iowa, is 250 psi, the value used in the present study. The selection of this value is discussed by Davidson and Bruns (2).

### Discussion of Results

**Specimens Cured 7 Days.** — Resistance to the damaging effects of immersion and of freezing and thawing showed marked increases with increased phosphoric acid content in the treatment range 2 percent to 7.5 percent (Fig. 8), but judged on the basis of 7

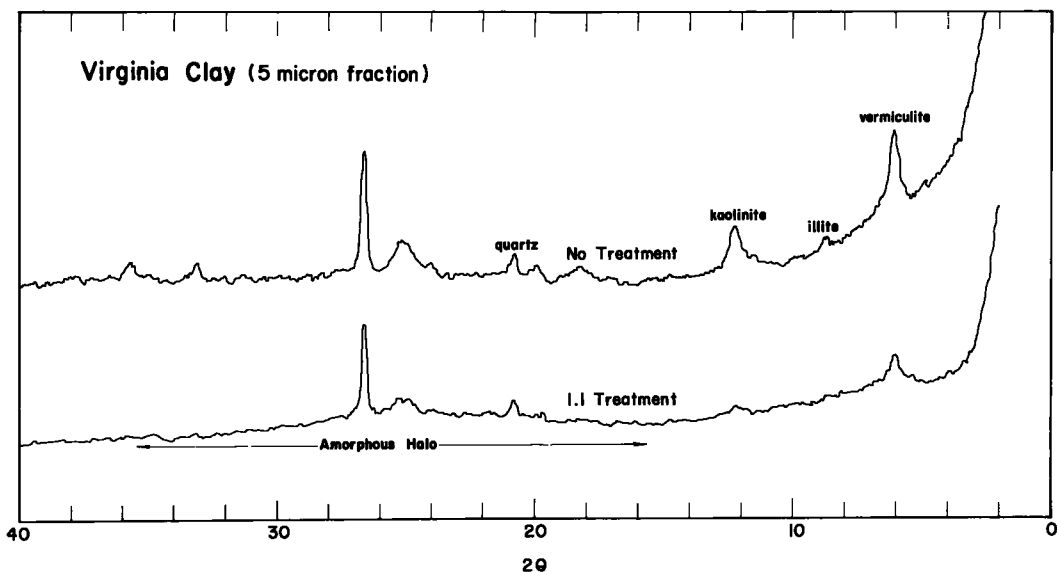


Figure 14. X-ray diffractometer chart showing the effect of phosphoric acid on the clay minerals of the minus 5 micron fraction of the Virginia clay. (Filtered K $\alpha$  radiation was used.)

days curing, none of the mixtures would be considered adequate for road bases in Iowa. However, the mixture with 7.5 percent phosphoric acid came close to being satisfactory in that  $R_f = 85$  percent,  $p_c = 250$  psi and  $p_f = 218$  psi, the latter value being the only one that did not meet the minimum requirements for durability.

**Specimens Cured 14 Days.** — If mix design was based on 14-day curing of test specimens, Figure 9 indicates that 7.5 percent phosphoric acid would satisfactorily stabilize plastic loess for road bases in Iowa. Lesser amounts of phosphoric acid probably would not provide adequate durability.

**Time of Immersion.** — A comparison of the strengths of the specimens immersed for one day with those of similar specimens immersed for 15 days (Figs. 8 and 9) shows that as phosphoric acid content was increased the amount of strength reduction due to the longer time of immersion was generally reduced.

### LIMITATION STUDY

The presence of calcium carbonate in many soils can be one limitation to the use of phosphoric acid stabilization. The cost of phosphoric acid stabilization can be another.

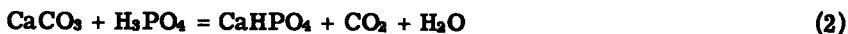
#### Calcium Carbonate

In a preliminary study of phosphoric acid stabilization using the calcareous (unleached) Kansan till (Table 1) no immersed strengths were obtained, that is, the specimens failed by slaking. Because of this it was decided to study the effect of calcium carbonate content on the immersed strength of phosphoric acid stabilized soil. Two experimental approaches were used. In one, natural soils that contained known amounts of calcium carbonate were used to evaluate the effect of calcium carbonate on immersed strength. The five soils used contained amounts of calcium carbonate varying from 0 to 7.2 percent (Table 1). The second approach was to use the non-calcareous plastic loess and add specific amounts of calcium carbonate and study the effect on immersed strength.

**Natural Soils.** — The effect of calcium carbonate content on immersed strength is shown by the curves for the five soils in Figure 7. Non-calcareous plastic loess showed an immediate increase of immersed strength with the addition of 1 percent phosphoric acid. Alluvial clay with 5.1 percent  $\text{CaCO}_3$  did not develop immersed strength until about 6.5 percent phosphoric acid was used. Detroit clay which contained 7.2 percent  $\text{CaCO}_3$  did not develop immersed strength until the phosphoric acid content was at least 10 percent. Therefore it seems that the amount of phosphoric acid necessary to obtain immersed strength is increased as the calcium carbonate content is increased.

**Plastic Loess Plus  $\text{CaCO}_3$  Additives.** — The effects on 7-, 14- and 28-day cured immersed strengths of adding different amounts of calcium carbonate to plastic loess are shown in Figure 10. (The phosphoric acid treatment was held constant at 7.5 percent based on the dry weight of the soil- $\text{CaCO}_3$  mixtures.) Increasing the amount of calcium carbonate decreased the immersed strengths, until at about 6 percent  $\text{CaCO}_3$  the strengths were zero or close to it.

Phosphoric acid ( $\text{H}_3\text{PO}_4$ ) gives three principal calcium salts: tricalcium diphosphate,  $\text{Ca}_3(\text{PO}_4)_2$ ; calcium hydrogen phosphate,  $\text{CaHPO}_4$ ; monocalcium tetra hydrogen diphosphate,  $\text{CaH}_4(\text{PO}_4)_2$ . Formation of these salts from  $\text{CaCO}_3$  and  $\text{H}_3\text{PO}_4$  may be represented stoichiometrically, by the following equations:



The actual occurrence of these reactions is governed by the acid and base strengths of the reactants and the products. Because  $\text{Ca}_3(\text{PO}_4)_2$  is a much stronger base than  $\text{CaCO}_3$ , the first reaction can hardly take place. However, if the pH of the medium in which  $\text{CaCO}_3$  and  $\text{H}_3\text{PO}_4$  are reacting is controlled, any of these reactions may be made possible. Constituents of a soil may furnish such a control and make any one of these reactions possible.

An indirect approach was used to decide which one of these reactions predominantly takes place when a calcareous soil is treated with phosphoric acid. The ratio of the weight of phosphoric acid to calcium carbonate based on the aforementioned three stoichiometric equations is, respectively, 0.987, 1.98 and 2.96. The ratio of phosphoric acid to calcium carbonate at which immersed strengths were obtained with the calcareous soils studied was as follows: alluvial clay, 1.18; Detroit clay, 1.25. For the plastic loess with different amounts of calcium carbonate added the ratio of phosphoric acid to calcium carbonate content at the point where the immersed strength was zero was 1.25. These three ratios are very close to one another and to the ratio of phosphoric acid to calcium carbonate, 0.987, in Eq. 1. The slight difference between the first three values and the latter may be because phosphoric acid reacts with the calcium carbonate to a small extent according to the second reaction in Eq. 2, and also because small amounts of phosphoric acid may react with other soil constituents.

The presence of calcium carbonate in soil increases the amount of phosphoric acid necessary to stabilize the soil. This appears to be due to the phosphoric acid acting to neutralize the calcium carbonate before reacting with the other soil constituents. The amount of phosphoric acid necessary to neutralize the calcium carbonate in the soils studied was about 1.25 times the amount of the calcium carbonate present. The neutralization of the calcium carbonate in a soil apparently contributes nothing to strength but would of course add to the cost of the stabilization.

**Cost.** — Cost is another factor that may limit the use of phosphoric acid for soil stabilization. Phosphoric acid currently costs about 7.5 cents a pound (8). For comparison, portland cement costs about 1 cent a pound, and is used in soil stabilization of fine-grained soils in amounts of from 12 to 20 percent. To compete cost-wise with cement, the amount of phosphoric acid that can be used must be in the range 1.6 to 2.7 percent. This estimate is based only on the cost of the additive, and does not take into account methods of construction. Phosphoric acid could be added in the water for compaction, which might be an advantage over cement. Also phosphoric acid might be cheaper to handle than portland cement, due to the smaller amount of additive involved. These are matters to be investigated, and experience in the field is necessary for a final decision on them.

### MECHANISM

The mechanism of phosphoric acid stabilization is embodied in chemical changes taking place on treating soils. In the present study X-ray diffraction analysis and quantitative chemical analysis were used for tracing the chemical changes. Figures 11, 12 and 13 show the X-ray diffraction charts for the three soils — Detroit clay, plastic loess, and Virginia clay — all treated with various percentages of phosphoric acid and cured 7 days in a humid atmosphere. Figure 14 shows the X-ray diffractometer charts for the minus 5 micron fraction of the kaolinite-vermiculite rich soil (Virginia clay) before and after treatment with an equal amount of phosphoric acid by weight, diluted approximately to liquid limit consistency with water, and cured in a humid atmosphere for 7 days. The purpose of this 1:1 treatment was to augment the changes taking place on phosphoric acid treatment.

Examination of the X-ray charts indicates an amorphous halo and no newly formed crystalline substances resulting from phosphoric acid treatment, except that the chart for calcareous Detroit clay gives calcium phosphate peaks resulting from the reaction between calcium carbonate and phosphoric acid. All three treated soils showed a decrease in the intensities of clay mineral peaks depending on the level of the treatment, suggesting that phosphoric acid had reacted with clay minerals. The amorphous halo is attributed to the reaction products.

Quantitative chemical analyses made on Detroit clay treated with phosphoric acid showed an appreciable increase in HCl-soluble aluminum compounds depending on the level of the phosphoric acid treatment (Table 2). The cation exchange capacity of Detroit clay and plastic loess treated with various percentages of phosphoric acid increased, depending on the level of the treatment (Table 2).

Based on these data it is hypothesized that phosphoric acid releases aluminum ions



TABLE 2

EFFECT OF PHOSPHORIC ACID ON THE CATION EXCHANGE CAPACITIES OF DETROIT CLAY AND PLASTIC LOESS, AND ON THE AMOUNTS OF HYDROCHLORIC ACID EXTRACTABLE ALUMINUM COMPOUNDS OF THE DETROIT CLAY

Soil	H <sub>3</sub> PO <sub>4</sub> <sup>a</sup> (%)	Cation Exchange Capacity <sup>b</sup> (me/100 g)		N. HCl Extractable Aluminum <sup>b</sup> (%)	
		On the Basis of the Total Weight of the Mixture	On the Basis of the Soil Fraction of the Mixture	On the Basis of the Total Weight of the Mixture	On the Basis of the Soil Fraction of the Mixture
Detroit Clay	0	14.5	14.5	1.1	1.1
	2	18.9	19.3		
	4	21.6	22.5		
	6	27.9	29.7		
	8	28.8	31.3	1.7	1.9
	10	29.4	32.7		
	12	38.0	43.2		
	14	45.9	53.4		
Plastic Loess	0	23.5	23.5		
	1	40.4	40.8		
	2	43.4	44.3		
	4	45.0	46.9		
	6	52.5	55.9		
	8	52.3	56.8		
	10	50.3	55.9		

<sup>a</sup>Expressed as percentage of pure (100 percent) phosphoric acid based on the oven-dry weight of the soil.

<sup>b</sup>Determined after moist curing for 7 days, and expressed as a percentage of the oven-dry weight of the sample.

from clay mineral lattices by destroying the clay mineral structure. Aluminum ions released combine with the phosphate anions to form a irreversible gel having a high cation exchange capacity. This gel acts as a cementing agent by bonding to the surfaces of other mineral constituents of the soil.

### CONCLUSIONS

1. Phosphoric acid treatment improves the strength and durability characteristics of compacted, moist cured, clayey soils. The degree of the improvement depends on the amount of phosphoric acid used and on the types and amounts of clay minerals in the soil. Chlorite appears to be much more reactive with phosphoric acid than montmorillonite-, illite-, kaolinite-, or vermiculite-type clay minerals.

2. There is an optimum amount of phosphoric acid which produces the highest unimmersed and immersed strengths in stabilized soils.

3. Moist curing phosphoric acid stabilized soils gives higher immersed strengths than curing in open air.

4. The molding moisture content for maximum dry density of soil-phosphoric acid mixtures correlates closely with molding moisture content for maximum immersed strength.

5. For each combination of phosphoric acid and soil there appears to be a curing time beyond which there will be no further increase in immersed strength. This time is a function of the amount and the type of clay minerals reacting, and the amount of phosphoric acid available for the reactions.

6. The resistance of phosphoric acid stabilized soil to alternate cycles of freeze

and thaw increases with increasing percentages of phosphoric acid.

7. Phosphoric acid acts to neutralize the calcium carbonate in a soil before reacting with the other soil constituents; this increases the amount of phosphoric acid necessary to stabilize the soil. Each percentage of calcium carbonate consumes about 1.25 percent phosphoric acid.

8. Based on the (1959) relative costs of portland cement and phosphoric acid, the maximum percentage of phosphoric acid that can be used economically in soil stabilization is about 3 percent, based on the dry soil weight. Soils containing more than 2.5 percent calcium carbonate may consume more than 3 percent phosphoric acid in the neutralization of calcium carbonate without gaining in stability. Therefore in regions like Iowa, where calcareous soils are abundant, phosphoric acid stabilization may not be an economical method of stabilizing soils for roads. A reduction in the price of phosphoric acid, however, could make it economical to use for stabilizing calcareous clayey soils.

9. In reacting with clay minerals phosphoric acid produces a water insoluble cementing compound. Based on chemical and X-ray diffraction analyses it is believed that this cementing compound is an irreversible gel composed of amorphous aluminum phosphates.

#### ACKNOWLEDGMENT

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# Sodium Chloride Stabilized Roads in Iowa

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Sodium chloride has been used as a secondary road stabilizer for a number of years and the performance of salt-stabilized roads has been reported periodically. These reports have differed widely as to success, costs, maintenance methods, riding qualities, and other items. The Iowa Engineering Experiment Station has undertaken a project to clarify and better understand the use and mechanism of sodium chloride stabilization.

The counties in Iowa which have existing salt-stabilized roads were contacted and each county engineer was interviewed. This paper reports a summation of these interviews and gives the composition and dimensions of the salt-stabilized roads, as well as maintenance practices, present physical conditions of surfaces, effects of winter, and comments of county engineers.

● SODIUM CHLORIDE has been used in varying degrees as secondary road wearing surface stabilizers for a number of years. The successes or failures of this stabilizing agent have been reported periodically but these reports differ widely as to degree of success, maintenance requirements, costs, riding qualities, and other items. County engineers in Iowa have discussed sodium chloride stabilization pro and con without reaching any general agreement as to effectiveness or economic value. However, each county engineer has proceeded according to his own convictions which are generally based on experience and opinion. As a consequence, some counties use large amounts of NaCl and others use none.

The Iowa Engineering Experiment Station has undertaken a project designed to clarify and better understand the use and mechanism of chemical stabilization in this state. The purpose of the project is to study the physical and chemical characteristics of chemically treated roadway surfaces, particularly those which have been treated either with sodium chloride or calcium chloride, or both, in the original construction of the roadway. The main objectives are (1) the determination of the reasons for differences in results from a given method of treatment for a series of aggregate and soil mixtures and for different aggregates and soils, and (2) the evaluation of the benefits derived from chemical treatment of any given mixture of aggregate and soil and for different aggregates and soils.

This paper presents the initial phase of the study in which counties using salt treatment of secondary roads were located and the respective county engineer was interviewed. The results of the interviews follow and are presented under the various headings. Counties using salt stabilization are shown in Figure 1.

## SODIUM CHLORIDE

### Composition and Dimensions

Fourteen counties using salt for chemical stabilization were found to have a total of 524 mi of salt-stabilized roads. Jones county has the least mileage with only 4 mi of salt-stabilized roads, and Franklin and Linn counties are high with 106 mi. Some salt-stabilized roads have been blacktopped and are included in total mileage.

Table 1 shows counties using salt and gives information pertinent to the material

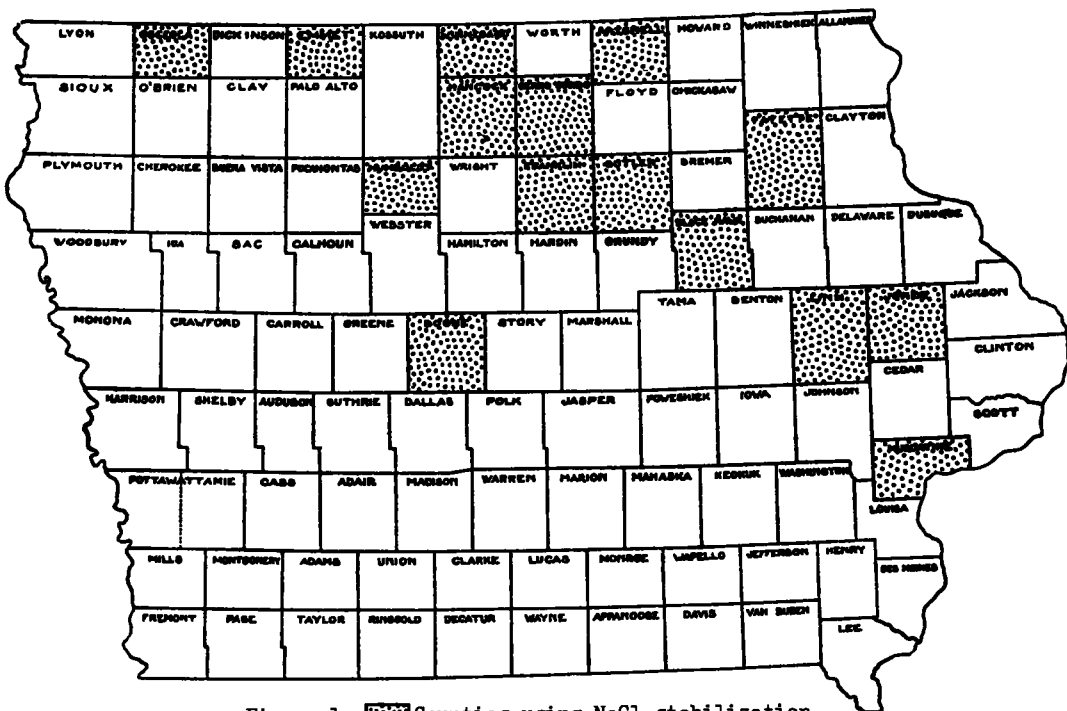


Figure 1. Counties using NaCl stabilization.

content of stabilized roads. Nearly all stabilized wearing courses are deep in the middle and feathered to the edge. The average depth of stabilized material varies from 2 to 4 in., the width varies from 22 ft to 28 ft. Nine counties use soil-aggregate-salt mix, and four counties use salt and aggregate only. The amount of salt used is generally expressed on a ton per mile basis and varies from 5 to 20 tons per mile. Some engineers express salt contents as pounds of salt per ton of soil material, the amounts quoted on this basis varied from 8 to 12 lb per ton. The quotations in Table 1 are expressed on a ton per mile basis by estimating the soil material used per mile and the density of the in-place material. Estimates comparable with 8 to 12 lb per ton are 12 to 19 tons per mile. Aggregates used were either pit run gravel or crushed stone. Glacial clay was used in all soil-aggregate roads. Seven counties include a surface application of about 5 tons of calcium chloride per mile per season to aid in surface moisture retention and dust palliation.

Mitchell County uses a mixture of 75 percent salt and 25 percent calcium chloride, initially applied at a rate of  $4\frac{1}{2}$  tons per mile. Subsequent applications bring the total chemical application up as high as 17 tons per mile. The primary purpose of chemical stabilization is to hold the material in place prior to blacktopping.

#### Construction Procedure and Costs

Most roads are constructed from materials mixed in the field either by blade or by a Seaman Pulvimixer. In general, the gravel and soil materials are windrowed and bladed several times, salt is distributed and bladed in, water is added, the material is uniformly spread and compacted to 90 or 95 percent standard Proctor density. The clay is either pulverized before spreading or pulverized on the road bed before blading. A few counties use plant mixed materials exclusively and some use plant mixed material occasionally. The plant mix method is preferred by many although it is considered somewhat more expensive. One man thought the plant mix was cheaper than a road mix. The general belief is that increased road quality compensates for any added expense.

Cost data proved to be rather scarce. Reports varied from \$2,500 to \$4,000 per

TABLE 1  
COUNTIES USING SODIUM CHLORIDE STABILIZATION AND DATA RELATIVE TO SURFACE  
COURSE COMPOSITION

County	Miles of NaCl Stabilized Roads	Amount of Original Chemical Treatment (T/mi)	Annual Additional Treatment with CaCl <sub>2</sub>	Average Thickness (in.)	Width (ft)	Aggregate	Binder
Black Hawk	38	6-10	—	2	22	Class A crushed limestone	Glacial till
Boone	4	20	4 T/mi	4	24	3/4 in. pit run	Glacial till
Butler	88	10	Some	3	24	1, 320 T 3/8 in. gravel 600 T 3/8 in. rock	300 yd glacial clay
Cerro Gordo	25	5-9	—	3	24	Rock and gravel	Glacial clay
Fayette	16	16-20 <sup>b</sup>	Some	3-5	26	2, 200 T 3/4 in.	About 1 in. from old surface
Franklin	106	9-10	Some	4	26	Gravel	Glacial till
Hancock	50	16 <sup>b</sup>	2 lb/sq yd	—	27	Pit run	18-20 percent glacial till
Humboldt	25	13 <sup>b</sup>	—	4	26	3/4 in. pit run	20 percent glacial clay
Jones	4	20 <sup>b</sup>	5 T/mi	—	28	3/4 in. crushed rock 40 percent dust	None
Linn	106	—	—	—	—	3/4 in. crushed rock	None
Muscatine <sup>a</sup>	1	—	—	—	—	—	—
Mitchell	32	4 1/2	—	—	—	3/4 in. crushed rock	None
Osceola	11	9	—	3	26	Pit run	None
Winnebago	19	15	2-3 times a year	4	27	Pit run	12-16 percent glacial clay

<sup>a</sup>Experimental.

<sup>b</sup>Estimated from width, depth and assumed density. Originally expressed in pounds salt per ton of material.

mile of completed road. One engineer reported a cost of \$2.00 per ton of laid material, and another reported a cost of \$450 per mile for materials alone (gravel, clay and chemical).

### Maintenance Practice

Blade maintenance following a rain was the common denominator in all counties, with most engineers expressing the opinion that a shallow cut is essential to good performance. A deep cut destroys the crust that has formed on the immediate wearing surface and allows undue traffic abrasion until a new crust is formed. The formation of a new crust may come too late or not at all, and in either case the road is destined to short life.

All county engineers seem to have a problem in educating the blade men. Most maintenance men have developed the idea that the purpose has not been accomplished unless a large roll of material is carried in front of the blade. Local citizens are also guilty of this misconception and voice such an opinion in no uncertain terms to the blademen. Such harassing can cause improper blading, because maintenance men are prone to slide back into old ways under pressure.

Many of the counties use supplemental applications of calcium chloride for dust palliation and retention of the binding surficial clay and dust. Potholes are hand patched in some counties with mixtures of clay, gravel and calcium chloride. This seems to produce better results than filling the potholes with bladed material and depending on traffic compaction for stability. The latter method is evidently only a temporary measure at best because the bladed material usually does not contain a sufficient amount of binder.

Most engineers using this type of road agree that maintenance attention is needed

to insure a good riding surface. Surprisingly, attention to maintaining crowns was mentioned very little. Only one engineer said that it is necessary to rebuild the crown every spring.

### Present Physical Condition of Surfaces

County engineers reported everything from smooth to rough when asked to describe the present riding qualities of their salt-stabilized roads. These descriptions have little meaning because they depend on the type of surface with which the salt road is compared. However, every county reports that salt roads exhibit a decided tendency to pit. The amount of pitting appears to depend on the traffic volume, maintenance methods, and the amount of moisture in the road. The moisture content depends mostly on rainfall inasmuch as salt is not deliquescent.

Engineers were asked to describe the dust conditions on the salt roads as either dust free, slightly dusty, dusty, or very dusty. None of the roads were described as dust free or very dusty, and in most cases they were called slightly dusty. Sodium chloride thus appears to have some value as a dust palliative.

Every engineer agreed that salt-stabilized roads exhibit excellent aggregate retention. This is especially a favorable recommendation because traffic on these roads ranges up to 450 cars per day, with an average of about 200 cars per day.

### Effects of Winter

Comments on the effects of winter freezing were favorable to salt roads. Most engineers said that treated roads have less tendency to form frost boils than untreated roads, and several county engineers have noticed a few adverse effects due to freezing. Butler and Franklin County engineers have noticed that treated roads do not soften during the usual spring break-up period.

A very interesting and unusual phenomenon was reported in Fayette County where both salt stabilization and calcium chloride stabilization are used. Calcium chloride roads were observed to ice over during winter, but salt roads either do not or have less tendency to ice over.

As previously mentioned, Mitchell County uses a mixture of salt and calcium chloride for stabilization of soil aggregate material. The county engineer has found that the calcium chloride content must be held down to minimize the effects of freezing and thawing.

### Comments

Many interesting comments concerning salt-stabilized roads resulted during casual conversations and are listed by county, as follows:

**Black Hawk.** — Most everyone concerned with salt-stabilized roads is well satisfied with the performance of these roads. This stabilization method conserves gravel.

**Boone.** — The salt road gets very hard during dry weather and was slippery during wet weather because of too much clay. The slipperiness was corrected by the addition of pea gravel. Some pits developed but were easily repaired. The road was top dressed and rolled with a steel roller before blacktopping.

**Butler.** — Three-quarters of the clay mixed with the gravel is effective and one-quarter is lost during construction. We formerly added lime dust as a maintenance measure but found that this created a layer of different texture. This layer tended to scale so the practice has been discontinued. Some of the road beds have been tested during the spring and were found to be hard and relatively dry. However, the material was easily broken up with a pick. Salt stabilization saves about 250 tons of aggregate per mile per year on a heavily traveled road. A new road is constructed after an old road is worn out, usually every five years.

**Cerro Gordo.** — Salt-stabilized roads were constructed in 1954 and 1957. These roads have performed satisfactorily although no additional maintenance material, either salt or calcium chloride, has been applied since construction.

**Fayette.** — Most of our trouble develops on heavily traveled roads. Surface applica-

tions of calcium chloride to the mid 16 ft helps to hold the gravel. This type of stabilization works best on roads carrying about 125 cars per day. Salt stabilization is responsible for large savings of both maintenance and gravel.

Franklin. — Salt roads hold moisture longer than untreated roads. Salt roads also develop a surface crust and stay hard through the spring. Roads do not hold up too well when traffic exceeds 150 cars per day, they tend to be ground up. Stabilized roads are in better all-around condition than untreated roads and therefore save gravel on adjacent roads by drawing traffic.

Hancock. — Salt stabilized roads are used in a 2 to 3 stage blacktop construction. Bad spots appear and are corrected before blacktopping. Roads are in service two or four years before the blacktop is applied. We find a plant mix cheaper than a road mix.

Humboldt. — A sheepsfoot is not as satisfactory for compaction of salt treated roads as a rubber tire roller. We tried to apply the chemical in solution but had spraybar trouble. Salt stabilization costs less to keep aggregate over a period of time and conserves aggregate. Gravel requirements on all roads appear to be reduced due to better surfacing of blacktop over chemically stabilized material.

Jones. — (Trial road) An old road was covered by 1,500 tons of aggregate per mile prior to chemical stabilization. Moisture was added at the quarry and supplemented by a water wagon on the road. The road was constructed in one lift and seems to be performing quite satisfactorily.

Linn. — Road work is contracted and salt is pug mill mixed at quarry. Costs vary from 2.19 to 2.98 dollars per ton of in-place material.

### Economics

O.W. Zack maintains records of roads in Butler County which indicate that an untreated gravel road initially requires 1,250 tons of soil-aggregate per mile, followed by 250 tons of aggregate per mile per year for maintenance and that such a road lasts about 4 years. Little or no gravel remains after the 4-yr life of the road so the road must be entirely rebuilt with new material.

Zack's records also indicate that salt-treated roads constructed with 2,200 tons of soil-aggregate-salt per mile last 5 to 7 years before they require rebuilding. Layers of 1 to 2 in. of material remain at this time but the material is rather loose and lacking in binder. The road must then be rebuilt but only a fraction of the original material requirement is needed.

These data have been used to compute cumulative road investments of money and aggregate for an economic comparison of both types of road construction in Butler County. The cumulative cost of each type of road is a stepwise function; yearly for the untreated road and every 5 to 7 years for the treated roads. The average cost per year has been used in both cases to give a continuous rather than a stepwise function. Costs for the treated road have been computed for a 5-year life and for both 1 and 2 in. of remaining material.

Figure 2a shows a comparison of the cumulative investment for both types of road. The initial cost of a salt-stabilized road is almost twice that of an untreated road but the cost rate of an untreated road is higher than that for a treated road. The net result is that after a period of  $3\frac{1}{2}$  to  $7\frac{1}{4}$  years (depending on material retention in the treated roads) the same amount of money has been spent to have either type of road. Henceforth the treated road becomes considerably cheaper to maintain. The same trend in the amount of aggregate used is shown in Figure 2B.

Figures 2c and 2d show the amount of saving in dollars and ton of aggregate, respectively, as a function of time. The negative values indicate that an untreated road is cheaper and requires less aggregate than a treated road during the first few years. A little arithmetic shows that if 500 miles of county roads were constructed with salt-stabilized soil-aggregate ( $1\frac{1}{2}$ -in. retention) rather than untreated soil-aggregate the county would save about \$2,400,000 and a little more than 2 million tons of aggregate over a period of about 20 years.

The savings in dollars over this 20-yr period is of considerable importance to a county treasury and to each individual taxpayer. Perhaps of more importance is the

savings in natural gravel resources which is equivalent to a block of solid stone which would cover a 160-acre farm to a depth of 4 ft.

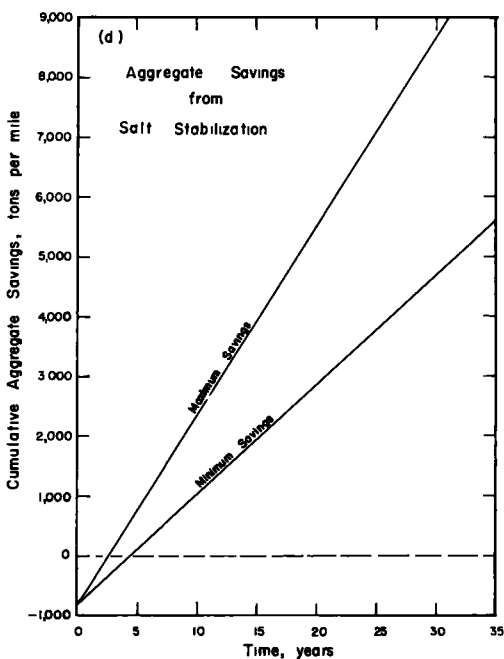
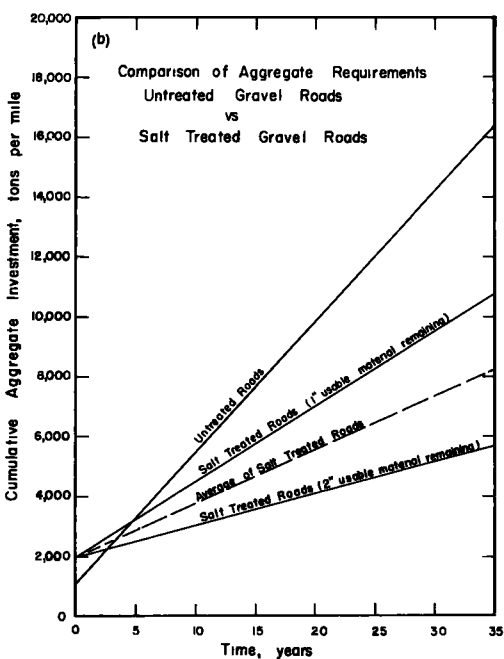
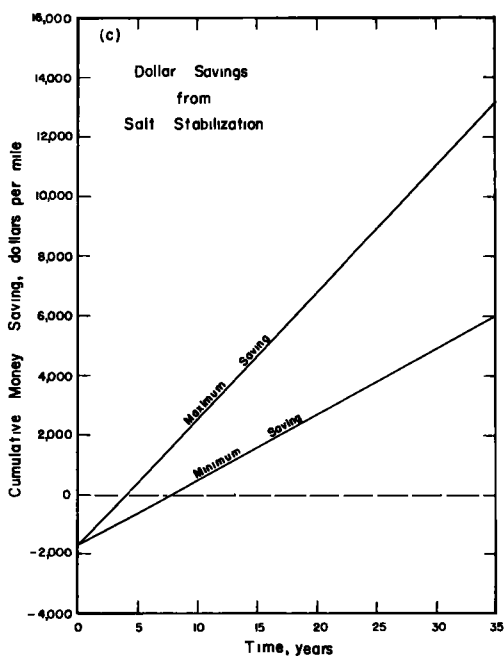
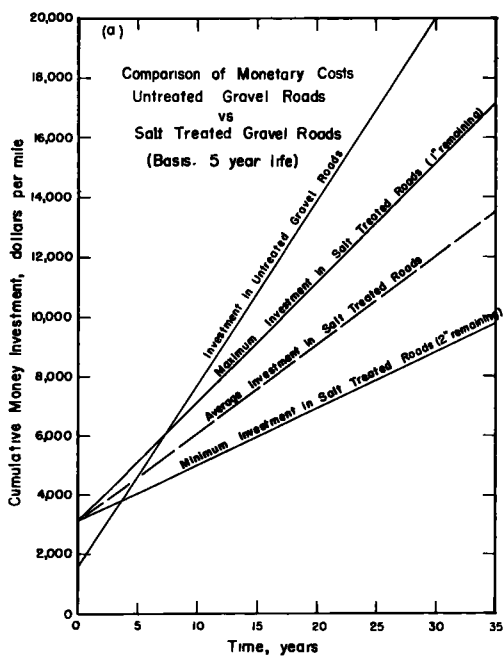


Figure 2. Comparison of cumulative monetary and aggregate investments required to maintain untreated and salt-treated secondary soil-aggregate roads in Butler County, Iowa.



## Discussion

The summarized testimony of the county engineers interviewed shows general agreement that salt treatment improves the performance of soil-aggregate roads in several ways. The most important being the long-range conservation of natural resources and reduction in road investments. Iowa's deposits of gravel and stone are being rapidly depleted and are entirely gone in some areas. Should aggregate consumption continue at the present rate the cost of secondary roads will rise still higher because extinction of gravel pits and stone quarries will necessitate long hauls in addition to high prices at the sources of supply. Taylor County in southern Iowa is now faced with this situation. Gravel must be trucked in from outside the county and costs are over \$2.00 a ton at the pit.

Salt roads further improve the over-all secondary road picture by presenting a smoother and less dusty riding surface than non-treated roads. Such surfaces attract traffic from adjacent roads and are thus responsible for still more savings in gravel and money by reducing the wear on these adjacent roads. Salt roads also retain a good riding surface longer than untreated roads and therefore require less blade work. Salt roads also show better resistance to winter and spring break-up.

Why do salt roads have all these advantages over non-treated soil-aggregate roads? The improvement has been partially and sometimes completely attributed to a number of reasons, none of which have been completely proved or disproved. A listing of reasons includes high density, low permeability due to clay expansion following leaching, moisture retention, lowered freezing point of water, recrystallization of salt, increased solubility of calcium carbonate, gel formation, flocculation and increased cohesiveness of clay due to sodium ions.

The Iowa Engineering Experiment Station now has a project underway which plans investigation of the physico-chemical phenomena of salt-treated soil-aggregate road material and the effects of these phenomena on the performance of roads constructed of such material. The project is sponsored by the Iowa Highway Research Board and supported by funds from the Iowa State Highway Commission.

# Relative Effects of Chlorides, Lignosulfonates And Molasses on Properties of a Soil-Aggregate Mix

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Selection of the most economical stabilizing agent for a desired effect on the properties of a soil should be based on knowledge of relative effects of the agents rather than on personal prejudice or the publicity given to an agent. The objective of this paper is to provide a starting point for cost-effect comparisons by showing the effects of four agents on certain properties of a single soil-aggregate surface course mix.

Properties compared are (a) moisture-density relations as determined by standard Proctor density tests; (b) strength as determined from California Bearing Ratio studies; (c) moisture retention as determined by a non-standard test; and (d) the plastic properties of the mix as determined by the Atterberg limits tests. Additives, used in treatments of 0, 0.5, 1, 1.5 and 2 percent, are (a) Peladow calcium chloride pellets; (b) Sterling FC and CC salts; (c) Toranil A and Lignin Liquor spent sulphite liquors, and (d) a 79.5 Brix molasses.

The mechanism of stabilization with each agent is discussed to permit extrapolation of effectiveness to other soils. No selection of the "best" agent is made because this is a function of the desired effect on properties and available funds.

● SCARCITY AND COSTS of better grade road aggregates combined with ever increasing demands for low-cost secondary roads have focused the attention of highway engineers on the use of chemical additives to conserve available aggregate and to improve the performance of inferior materials. Despite the extensive research conducted relative to chemical stabilization, no record is available reflecting a direct comparison of the effects of the additives selected for this study on the properties of a specific soil.

Because logical selection of the most economical additive for a desired effect cannot be made without a basis for comparison, this study was initiated to provide a starting point for cost-effect comparisons. The objective of the paper is to compare the effects of calcium chloride, sodium chloride, lignosulfonates, and molasses on the moisture-density relationships, strength, moisture retention and plasticity characteristics of a soil-aggregate surface course mix.

## MECHANISM OF STABILIZATION

The two main factors contributing to the stability (resistance to lateral flow) of a granular road are internal friction and cohesion (10). Internal friction in a soil-aggregate mix is attributed to the granular fraction, and is a partial function of density. Cohesion, on the other hand is a function of soil fines, moisture films, and cementing agents (8).

### Chloride Stabilization

Davidson and Handy (5) attribute increased stability of granular road bases with the use of chlorides to the following seven effects:

Lubrication. — Based on the unctuous nature of a chloride solution, increased densities of 1 to 7 percent are cited in comparison with an untreated soil. This increase in density may logically be expected to increase internal friction and reduce the rate of moisture loss. Although this effect is less pronounced with sodium chloride than with calcium chloride, the deficiency may be negligible because sodium chloride contributes to dispersion of the binder preventing localized concentrations and thus eliminating soft spots.

Flocculation of Clays. — Presence of certain amounts of sodium or calcium ions in the mix will flocculate the binder into silt-sized particles. This amounts to changing the grain-size distribution and may have the effect of reducing the plasticity index. Other concentrations of ions (more or less) will cause electrical imbalances which will tend to disperse the clay. Crystallization of the salt or leaching by rainfall will change the concentration of ions, permitting dispersion of the clay, with consequent plugging of voids contributing to watertightness.

Moisture Retention. — Because of the deliquescence of calcium chloride and the hygroscopicity of sodium chloride, the rate of evaporation of soil moisture will be reduced. As a result, cohesion will be retained and the binder fraction will function as desired rather than being lost as dust.

Solubility of Road Aggregate. — This effect is mentioned as a possible means of cementation within the mix and is based on the solubility of limestone and dolomite in chloride solutions, resulting in precipitation of cementitious carbonates. The effectiveness of this reaction is unevaluated.

Freezing Point Depression. — By lowering the freezing point of the mixture, resistance to temperature effects is obtained with the use of chlorides. For complete freezing, road temperature must be -59.8 F with calcium chloride treatment or -6 F with sodium chloride treatment; thus the creation of ice lenses is inhibited.

Surface Tension. — The increase of this property due to the presence of chlorides results in strengthening water film bonds between soil grains, adding to apparent cohesion. With evaporation, the chloride concentration increases, causing a further increase in surface tension. The degree of increase may be such that increased densities will result.

Crystallization. — This effect is present only with sodium chloride and may compensate for the lesser effectiveness of salt altering other properties. Advantages which result from the formation of salt crystals are (a) the crystals plug voids thus retarding evaporation and shrinkage, (b) they act as a cement which prevents aggregate losses, and (c) they provide a hard, tightly knit traffic surface.

### Lignosulfonate Stabilization

According to Davidson and Handy (5), the principal effects of lignosulfonates on a soil-aggregate mixture are those of cementation and dispersion of clay.

Cementation is derived from the fact that the lignosulfonates, being water soluble polymers, act in the soil similarly to a glue. Best results are obtained with mixes rich in binder soil, because open-type mixes permit rapid leaching of the lignosulfonate. Hygroscopicity of the wood sugars present may also contribute to strength by retarding evaporation, thus benefiting cohesion.

Dispersion of the clay fraction benefits stability of the soil-aggregate mix by: (a) plugging voids and consequently improving watertightness and reducing frost susceptibility, (b) eliminating soft spots caused by local concentrations of binder soil, (c) filling voids with fines thus increasing density, and (d) increasing the effective surface area of the binder fraction which results in a greater contribution to strength from cohesion.

### Molasses Stabilization

Phillips (7) implies that the principal effects of molasses in a soil-aggregate mix

**TABLE 1**  
**MECHANICAL ANALYSES OF DRY MATERIALS USED IN STUDY**

Sieve Size	Weight Percentage Retained on Sieve			
	Soil-Aggregate	CaCl <sub>2</sub> (Peladow)	NaCl (CC)	NaCl (FC)
¾ in.	0	- <sup>a</sup>	-	-
⅝ in.	-	Nil	0	-
No. 4	18.55	0.2	35.0	-
No. 8	-	37.0	48.0	0
No. 10	18.0	-	-	-
No. 12	-	-	-	5.0
No. 16	-	-	15.9	-
No. 20	-	62.8	-	42.0
No. 40	29.15	-	-	-
No. 80	-	-	-	43.0
No. 200	11.20	-	-	-
Pan	23.20	0.1	1.1	10.0

<sup>a</sup>Sieve not used in analysis.

are increased moisture retention and cementation. Moisture retention effects may be attributed to the hygroscopicity of the molasses and to reduced vapor pressure. From his description of the cementation effect, it would appear that the actual effect is not cementation, which implies a rigid bond between particles, but rather an increase in surface tension with consequent increase in apparent cohesion. As with lignosulfonates, best results may be anticipated with a high percentage of binder soil owing to the ease with which molasses will enter solution and be leached from the mix.

### INVESTIGATION

#### Materials Used

**Soil-Aggregate Mix.** — Three natural soils containing about 6 percent moisture, were blended in a 2-cu ft cement mixer to give a soil-aggregate mix conforming to Iowa State Highway Commission Specification 4111 (6) for stabilized surface course materials. The three soils used were (a) a pit-run gravel from a glacio-fluvial deposit north of Ames, Iowa; (b) a silty clay loam (Monona Series, C horizon) from Shelby County, Iowa; and (c) a C horizon oxidized Wisconsin-age (Cary) glacial till from near Ames, Iowa.

The gradation of the resultant mix is given in Table 1, other physical properties of the soil-aggregate mix are as follows:

Liquid limit — 25.5 percent  
Plastic limit — 16.8 percent  
Plasticity index — 8.7  
Optimum moisture — 9 percent  
Std. Proc. dens. — 129.2 pcf  
Specific gravity — 2.69  
Carbon content — 0.1 percent  
Carbonate content — 21.2 percent  
Cation exchange capacity — 7.5 mc/100 gm  
Exchangeable cation — Ca<sup>++</sup>  
pH — 8

X-ray analysis of the fraction passing the No. 200 sieve revealed the presence of quartz, calcite, dolomite, and feldspars, as well as the clay minerals illite, kaolinite, montmorillonite and vermiculite, although recent work indicates that the latter may be

TABLE 2

PETROGRAPHIC ANALYSIS OF PORTION OF THE SOIL-AGGREGATE MIX COARSER THAN THE NO. 200 SIEVE

Sieve Retaining Material	Weight Percentage				
	Quartz <sup>a</sup>	Feldspar	Heavy <sup>b</sup>	Carbonates <sup>c</sup>	Aggregate <sup>d</sup>
No. 4	0	2	0	51	47
No. 10	21	1	0	38	40
No. 40	~ 78	trace	0	~ 6	~ 16
No. 200	73	6	7	7	7

<sup>a</sup>Includes quartz and quartzite.

<sup>b</sup>Includes turmaline, augite, magnetite, and opaque minerals.

<sup>c</sup>Includes limestone and calcite.

<sup>d</sup>Includes all rock fragments; for example, granite, etc.

a non-expanding 14 angstrom montmorillonite (11). No quantitative analysis of these minerals was attempted but the predominant clay mineral was montmorillonite.

Results of a petrographic analysis (12) of the remaining portion of the soil-aggregate mix are given in Table 2.

**Calcium Chloride.** — Calcium chloride for this study was provided in the form of Peladow pellets by the Dow Chemical Company. In this form the chloride is anhydrous, contains about 95 percent  $\text{CaCl}_2$ , and costs approximately \$30 per ton undelivered. Mechanical and chemical analyses of the material as given by the manufacturer (13) are given in Tables 1 and 3.

**Sodium Chloride.** — Rock salt was provided by the International Salt Company in two gradations. The larger size salt is designated by the manufacturer as Sterling CC, and the smaller as Sterling FC. Identification in this paper will be by the letter designations CC and FC, respectively. Costs at the mine for these salts vary from about \$8 to \$13 per ton depending on the size shipment and the packaging requirements. Typical mechanical and chemical analyses of the salts as provided by the producer (14) are given in Tables 1 and 3.

TABLE 3

TYPICAL CHEMICAL ANALYSES OF CHLORIDES

Constituents	Weight Percentage		
	$\text{CaCl}_2$ (Peladow)	$\text{NaCl}$ (CC)	$\text{NaCl}$ (FC)
$\text{CaCl}_2$	95.2	0.053	0.071
$\text{NaCl}$	1.15	98.220	98.236
KCl	2.96	- <sup>a</sup>	-
$\text{Ca}(\text{OH})_2$	0.068	-	-
$\text{CaCO}_3$	0.024	-	-
$\text{CaSO}_4$	-	0.643	0.651
$\text{MgCl}_2$	133 <sup>b</sup>	0.052	0.085
Heavy metals as Pb	0.5	-	-
S as $\text{SO}_4$	0.009	-	-
Fe	16 <sup>b</sup>	-	-
Water insolubles	-	1.041	0.957

<sup>a</sup>Not determined.

<sup>b</sup>Parts per million.

**Lignosulfonates.** — Two types of spent sulphite liquor were used in the study. The first, Toranil A, is a desugared calcium lignosulfonate which was provided in the concentrated form (50 percent solids) by the Lake States Yeast Corporation. The other, Lignin Liquor, is also a concentrate rich in calcium lignosulfonate, but contains a higher percentage of sugars. It was provided by the Appleton Division of the Consolidated Water Power and Paper Company. Table 4 gives a summary of the property analyses of the liquors as provided by the manufacturers (15, 16), costs are about 4.5 cents per gallon undelivered.

TABLE 4

## PROPERTY ANALYSES OF SPENT SULPHITE LIQUORS AS GIVEN BY PRODUCERS

Property	Toranil A	Lignin Liquor
Total solids, %	50	52.4
Total sugar, %	0.6	11.5
Total sulfur, %	- <sup>a</sup>	3.21
Ash, %	8.65	8.58
Calcium oxide, %	3.75	3.40
Specific gravity	1.24	1.267
pH	4.55	4.0
Manufacturer	Lake States Yeast Corp., Rhinelander, Wis.	Consolidated Water Power & Paper Co., Wisconsin Rapids, Wis.

<sup>a</sup>Not determined.

**Molasses.** — The molasses used in the study was provided by the Inland Molasses Company. Although the Brix reading was 79.5, representing a sugar content of 79.5 percent, evaporation to constant weight at 70 C revealed a solids content of only 50 percent by weight. A typical analysis (17) of the molasses is given in Table 5. Current cost is \$35 per ton in eastern Iowa.

TABLE 5

## TYPICAL ANALYSIS OF 79.5 BRX MOLASSES

Constituent	Weight Percentage
Minimum invert sugar	48
Reducing sugar	16-20
Sucrose	28-35
Nitrogen as crude protein	2-4
Ash	7-12

Method and Procedure

To permit direct comparison of effects, test specimens were prepared containing 0, 0.5, 1.0, 1.5, and 2.0 percent of each of the previously described additives. The percentages were based on the weight of solids contained in the additive, and the oven dry weight of the soil-aggregate mix. Duplicate specimens were prepared for each phase of testing.

Daily readings of temperature and relative humidity from a psychrometer located in the laboratory were recorded throughout the test periods to permit correlation with effects.

Mixing was accomplished in a model CC20 Blakeslee mixer in the sequence soil, additive, water. Materials were mixed mechanically for about 3 min, then transferred to a shallow pan where final mixing was done by hand. Next the mix was compacted to a depth of about 2 in. using a 5.5-lb rammer. Curing for about 5 min while covered with a damp cloth (to retard evaporation), permitted more uniform absorption of the moisture.

Standard Proctor Moisture-Density Relationships. — These tests were performed

in accordance with method D, ASTM tentative method of test D 698-57 T (2), except that the height of the molded specimens was 5 in. instead of the specified 4.584 in.

**California Bearing Ratio.** — These tests were performed in accordance with the procedures of the U. S. Corps of Engineers (9). For each percentage of additive, four specimens were prepared at optimum moisture content and maximum density as determined by moisture-density tests. Two specimens were tested immediately to determine the CBR values. Then, after four days immersion using a 5-lb surcharge, the soaked CBR values were obtained from the opposite ends of the specimens. The second pair of specimens was permitted to air cure under room conditions for a week prior to being subjected to the same tests. Expansion of the specimens during the periods of immersion was also determined.

**Moisture Retention.** — This property was determined through a non-standard method of testing. Two specimens, identical to those for the CBR tests, were prepared for each percentage of the various additives. Specimens were extruded and stored on open shelves under room conditions. The weight of the specimens was recorded after extrusion and after lapses of 1, 2, 7, 15, 30, 60, and 90 days.

**Atterberg Limits Tests.** — These tests were performed in accordance with ASTM tentative methods for test D 423-54 T (1) and D 424-54 T (3). Material for testing was obtained from specimens used in the moisture retention tests.

## RESULTS

### Moisture-Density Relationships

**Effect of Calcium Chloride.** — Marked increases in density were realized with the addition of calcium chloride to the mix. The dry densities obtained are shown in Figure 1. Of particular interest is the fact that maximum dry density was obtained at 1 percent calcium chloride, and that addition of more chloride tended to decrease the density.

This phenomenon may be explained by reference to the diffuse double layer concept, which considers that the clay particles have negative surface charges caused either by isomorphous substitution of lower valence cations within the lattice or by adsorbed hydroxyl ions due to broken valence bonds. These negative surface charges cause repulsive forces between clay particles thus increasing the compactive effort required for a given density, or conversely lessening the density which might be attained with a given effort. Introduction of  $\text{Ca}^{++}$  ions will give two beneficial effects. First, the negative surface charges may be neutralized, thus eliminating the repulsive forces, and second, because the calcium ion carries a double positive charge, valence bonds may be set up between neighboring clay micelles. Both of these effects will mean greater densities with the given compactive effort.

Continued addition of calcium ions will give increased benefits only until the negative surface charges are satisfied. Beyond this optimum point additional ions will be detrimental because the media will then assume a positive charge and repulsive forces will be reinstated. This, apparently, is the explanation of the lower densities obtained in this study with the higher percentages of calcium chloride.

An additional point of interest in this phase of the investigation is the apparent inverse relationship between optimum moisture content and maximum dry density, indicating the need for greater lubrication to assist in overcoming the repulsive forces.

**Sodium Chloride.** — As with calcium chloride, densities obtained with the addition of sodium chloride exceeded the maximum obtained with the untreated soil-aggregate mix. Unlike the specimens treated with calcium chloride, those treated with sodium chloride were not restricted to a single optimum chloride content. Instead, as may be seen in Figure 1, whether the salt used was CC or FC, a minimum dry density (treated material) was observed at 1 percent salt.

Because the hydrated sodium ion is larger and more reactive than the calcium ion, and because a gram of sodium chloride contains almost twice as many cations as does a gram of calcium chloride, it is reasonable to expect an effective neutralization of the negative surface charge of clay by a lower weight percentage of sodium chloride. At 0.5 percent salt, a maximum density was attained probably because of an effective

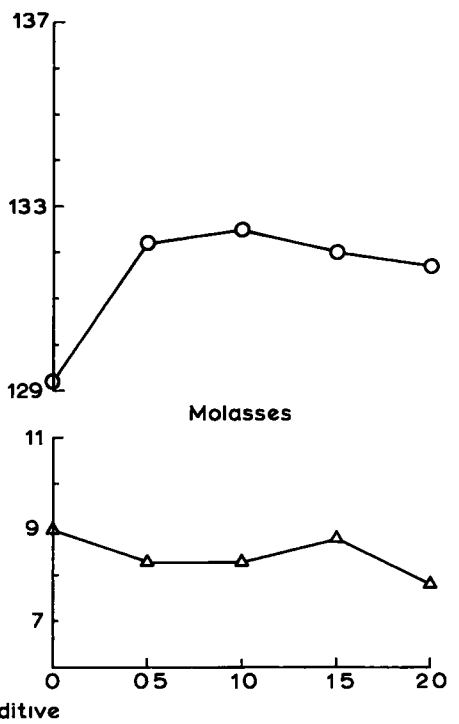
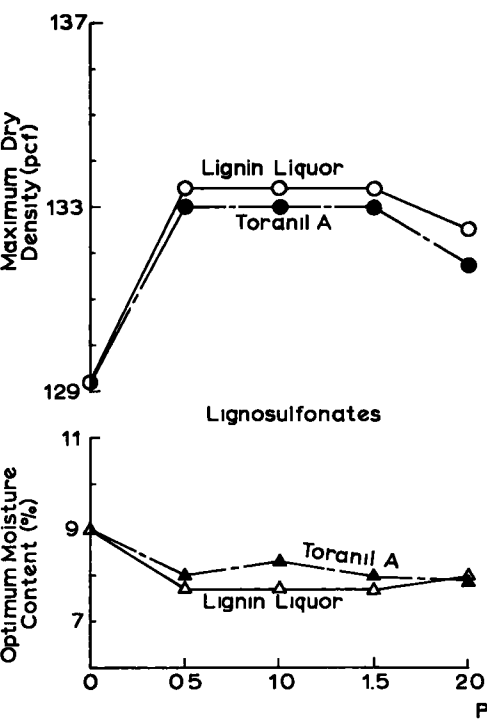
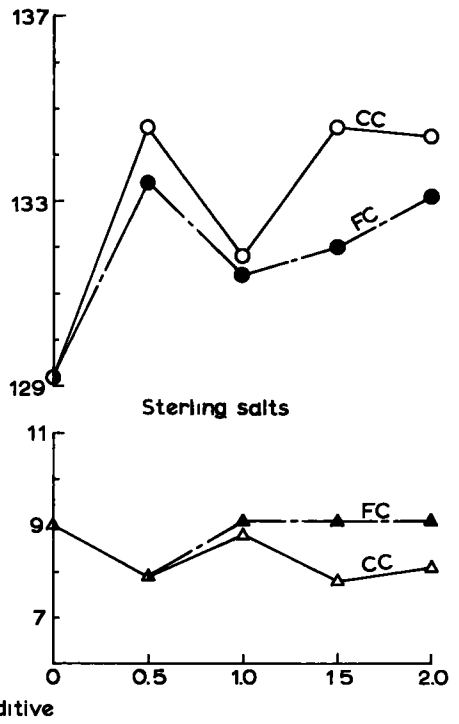
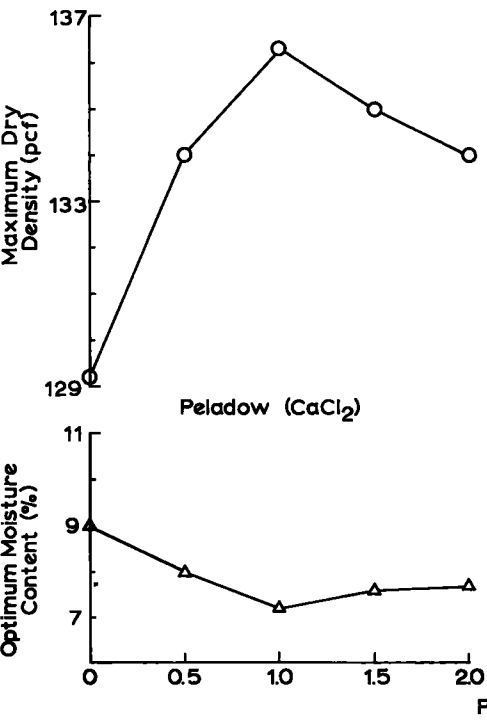


Figure 1. Variation of optimum moisture content and maximum dry density with percent treatment of indicated additives.



neutralization of negative surface charges on clay particles. This maximum density was not as great as that for 1 percent calcium chloride treatment because valence bonding of clay micelles is less probable, and because the size of the hydrated sodium ion precludes as close an approach of the neutral clay particles. At 1 percent sodium chloride content, the repulsive forces again were active because of the excess sodium cations, and lower densities resulted. Because the magnitudes of the repulsive forces are related inversely to the square of the distance between charges, and because of the imposition of aggregates and voids, a maximum effective value of repulsive forces is attained which is relatively unaffected by the addition of mere salt to the mixture. Thus, in effect, the increased densities which were observed at 1.5 and 2 percent sodium chloride may be partially attributed to the additional weight of salt contained within the specimens.

The fact that all densities resulting from FC salt treatment are less than those resulting from corresponding percentages of CC salt may be explained by the gradation differences. The FC salt, being finer grained, went into solution more rapidly than the coarser grained CC salt. Instances were noted where the CC salt crystals were not completely dissolved at the time of molding. Thus the number of active sodium ions was greater in the specimens prepared with FC salt.

The inverse relationship between moisture content and maximum dry density was again evident in the case of sodium chloride treatment, with the exception of samples containing 1.5 and 2 percent FC salt. This may have resulted from incomplete dissolution or from experimental error.

TABLE 6  
RESULTS OF CALIFORNIA BEARING RATIO TESTS

Additive	California Bearing Ratio, %						
		Uncured Specimens		Cured Specimens		Expansion, %	
Kind	%	Immediate	Immersed	Dry	Immersed	Uncured	Cured
Untreated	-	15.3	9.8	40.5	22.5	0.19	0.51
CaCl <sub>2</sub> (Peladow)	0.5	42.0	35.0	129.0	67.0	0.20	0.31
	1.0	103.0	48.0	160.0	48.0	0.26	0.40
	1.5	51.0	43.0	82.0	53.0	0.21	0.19
	2.0	38.0	27.5	44.0	30.0	0.25	0.19
NaCl (CC)	0.5	56.5	45.0	90.0	52.5	0.33	0.11
	1.0	12.5	6.5	31.5	21.0	0.09	0.03
	1.5	51.0	36.0	120.0	57.0	0.05-	0.09
	2.0	34.0	25.0	54.5	43.5	0.10	0.09
NaCl (FC)	0.5	51.7	42.0	61.0	44.0	0.29	0.20
	1.0	16.8	10.3	21.0	17.7	0.22	0.16
	1.5	12.0	8.3	20.0	18.0	0.06	0.15
	2.0	8.6	6.4	14.5	10.0	0.05	0.08
Lignin Liquor	0.5	71.0	49.5	97.5	36.5	0.23	0.18
	1.0	53.0	35.5	76.0	42.0	0.21	0.26
	1.5	47.0	33.0	56.5	33.8	0.24	0.26
	2.0	16.0	12.0	24.0	13.5	0.04	0.32
Toranil A	0.5	64.0	52.0	97.0	46.0	0.17	0.29
	1.0	39.0	32.5	64.0	30.0	0.10	0.13
	1.5	42.0	29.0	56.0	28.0	0.14	0.24
	2.0	26.5	20.0	35.5	21.3	0.13	0.54
Molasses	0.5	32.0	24.5	48.0	30.0	1.07	0.39
	1.0	28.0	20.5	50.0	23.5	0.70	0.57
	1.5	12.0	11.5	25.0	19.0	0.52	0.61
	2.0	23.0	26.0	46.5	32.0	0.13	0.33

**Lignosulfonates.** — Densities obtained with the use of lignosulfonates as additives were higher than those obtained for the untreated soil-aggregate mix. The Lignin Liquor showed a slight advantage over Toranil A in all percentages. The maximum dry densities are shown in Figure 1.

With the lignosulfonates, constant maximum densities were obtained at 0.5, 1, and 1.5 percent additive, and a decrease in density was obtained with 2 percent additive.

Speculation as to the cause of density variations has led to the conclusion that the initial density increase at 0.5 percent lignosulfonate treatment is caused by better lubrication derived from the decrease of surface tension. Because the lignosulfonates are assumed to coat the soil particles, the addition of further lignosulfonates should lead to thicker films and consequent separation of the soil grains. The specific gravity of the liquor is lower than that of the soil so that displacement of soil by lignin should lead to lower densities. Presumably, at 1 and 1.5 percent liquor, the beneficial effect of lubrication compensates for the thicker films and the over-all density remains constant. At 2 percent lignosulfonate, the effect of thicker films predominates and density decreases.

The difference in densities obtained with Lignin Liquor and with Toranil A is believed to be caused by the different specific gravities of the liquors. Lignin Liquor, having a slightly higher specific gravity, gave greater maximum dry densities. Other possible reasons for this difference might be found in the difference in lubricating properties because of the differing sugar contents, or in experimental error.

The inverse relationship between maximum dry density and optimum moisture content was again apparent with Lignin Liquor, but was not shown by the Toranil A treated specimens. A possible explanation is that the clay lumps may have been coated in aggregated masses with the higher percentages of Toranil A, preventing water intrusion, thus changing the effective gradation of the mix and lessening the moisture requirement for optimum lubrication.

**Molasses.** — Although density increases were appreciable with the addition of molasses to the soil-aggregate mix (Fig. 1), they were not as great as with any of the other additives. As with lignosulfonates, density increases are attributed to the lubricating effects accompanying reduced surface tension.

Optimum molasses content for maximum dry density occurred at 1 percent additive, with only slight deviation at contents of 0.5 and 1.5 percent. The same mechanism, as was proposed for the variation of density with lignosulfonates, is believed applicable to the molasses treated specimens; benefit from lubrication being overridden at higher percentages of additive by dispersion caused by thicker films.

The erratic nature of variation of optimum moisture with percent molasses is believed to be due to causes discussed for Toranil A.

### California Bearing Ratio Study

Although CBR tests (as performed) included obtaining values for specimens permitted to air dry for seven days, direct comparison of these results would not be realistic inasmuch as all specimens were not subjected to similar conditions of relative humidity and temperature. For this reason the discussion of results will center on test values for specimens tested immediately, and after four days immersion, with only occasional mention of curing effects.

**Effects of Calcium Chloride.** — CBR values were increased in all cases by the addition of calcium chloride, the trend both in immediate and soaked values apparently correlating with the density of the specimen. As given in Table 6, the highest immediate CBR value was that with 1 percent calcium chloride. This value (103.0 percent) represents an increase in strength of more than 600 percent when compared to the value obtained for the untreated soil-aggregate mix. After four days immersion, the strength is 490 percent that of the untreated mix tested under the same circumstances.

After seven days curing, all of the calcium chloride treated specimens reflected an increase in unimmersed strength with only the 1 percent specimens failing to give higher immersed strengths than the corresponding uncured specimens. Expansion of the specimens during soaking was negligible (Table 6).

**Effects of Sodium Chloride.** — Marked increases in immediate CBR values were obtained with 0.5, 1.5, and 2 percent CC salt treatments, whereas only the 0.5 percent FC salt treatment gave values exceeding the untreated value. At 1 percent CC salt, and at both 1.5 and 2 percent FC salt, despite the fact that densities exceeded that of the untreated soil-aggregate mix, CBR values were lower than the untreated CBR. If one considers the 1.5 and 2 percent CC salt treated specimens to be not truly representative (because of incomplete dissolution of the salt), then the loss of strength may be attributed to loss of internal friction due to the lubricating properties of the chloride solutions, and to loss of cohesion due to repulsive forces caused by excess sodium ions. Loss of strength on immersion was less in the case of 0.5 percent sodium chloride treatment than in the case of 1 percent calcium chloride treatment, probably because of the lower percentage and the lesser hygroscopicity of the salt. Considering both the accuracy of the test, and the CBR curve corrections required, the best four day immersed CRB values (no curing) of the three chlorides tested (FC, CC, and Peladow) may be considered to be essentially equal.

As in the case of calcium chloride, the CBR values of rock salt treated specimens reflected strength gains after curing, both dry and immersed. Of note were the increases associated with 1.5 and 2 percent CC salt specimens, indicating rapid evaporation, and again supporting the assumption of incomplete dissolution of the salt. As given in Table 6, expansion of the salt treated specimens on immersion was negligible.

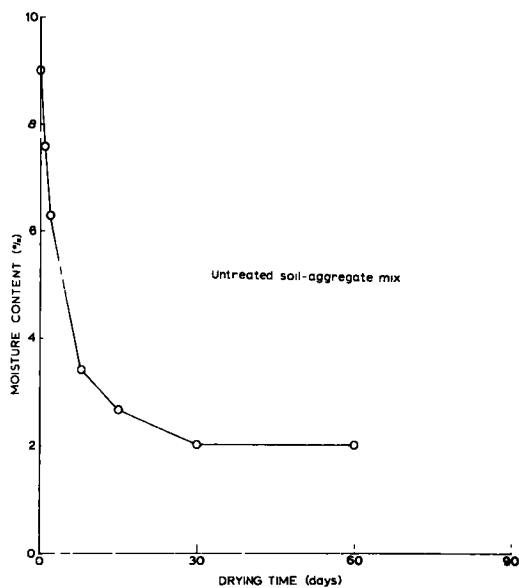


Figure 2. Variation of moisture content with drying time, untreated soil-aggregate specimens.

**Effects of Lignosulfonates.** — All percentages of lignosulfonates used in this study increased the strength of the specimens over the strength obtained with the untreated soil-aggregate mix. The greatest immediate gain was at 0.5 percent Lignin Liquor, with the greatest immersed strength (uncured and considering all additives) associated with 0.5 percent Toranil A. Immediate CBR values generally decreased with increase in lignosulfonate content, indicating the detrimental effects of increased lubrication. The exception in this trend was found at 1.0 percent Toranil A treatment. This may be attributed to the higher moisture content at molding and the consequent increase of lubrication.

Strength loss on immersion of the Toranil A treated specimens was generally less than that of the specimens treated with Lignin Liquor possibly due to the lesser hygroscopicity of Toranil A. Another factor which may account for the strength loss associated with immersion of Lignin Liquor treated specimens was the fact that fungal consumption of the lignin was indicated by the presence of a mold on the surface of the specimens.

Curing either benefited immersed strength or left it unchanged except in the case of 0.5 percent Lignin Liquor. This loss may have been caused by lower waterproofing potential at this content or may have been caused by loss of lignin due to fungal consumption. Expansion on immersion was negligible.

**Effects of Molasses.** — With CBR strength as a criterion, molasses was the least beneficial of the additives tested. Maximum immediate strength gain was with 0.5 percent molasses treatment, which, as given in Table 6, gave approximately a twofold increase over the untreated soil-aggregate mix strength. At 1.5 percent molasses content, a loss of strength was apparent.

Results of this phase of the investigation are not considered to be very reliable because the treated specimens were subject to rapid fungal action, becoming moldy both during the curing and immersion periods. This factor, combined with the degree of

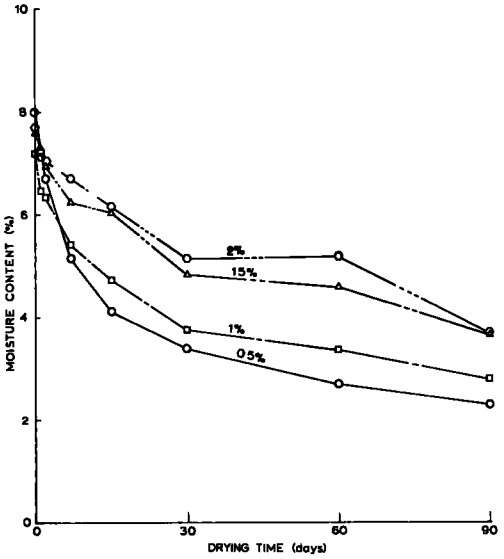


Figure 3. Variation of moisture content with drying time, calcium chloride treated specimens.

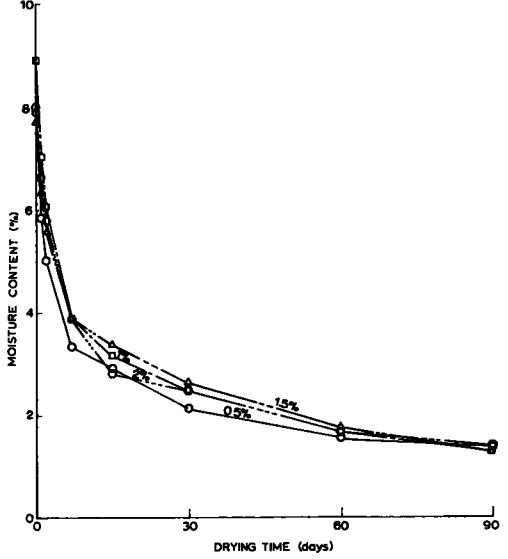


Figure 4. Variation of moisture content with drying time, CC salt treated specimens.

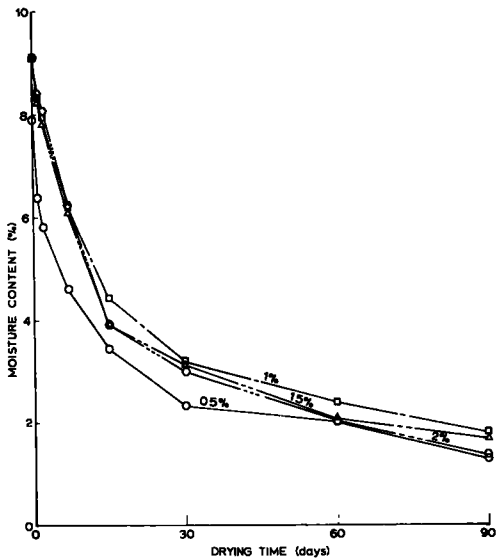


Figure 5. Variation of moisture content with drying time, FC salt treated specimens.

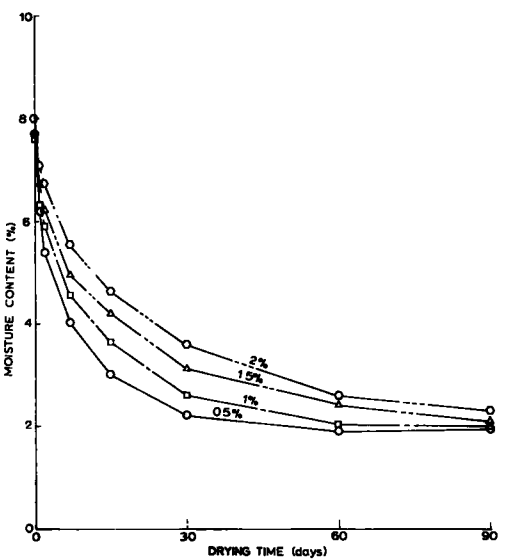


Figure 6. Variation of moisture content with drying time, Lignin Liquor treated specimens.

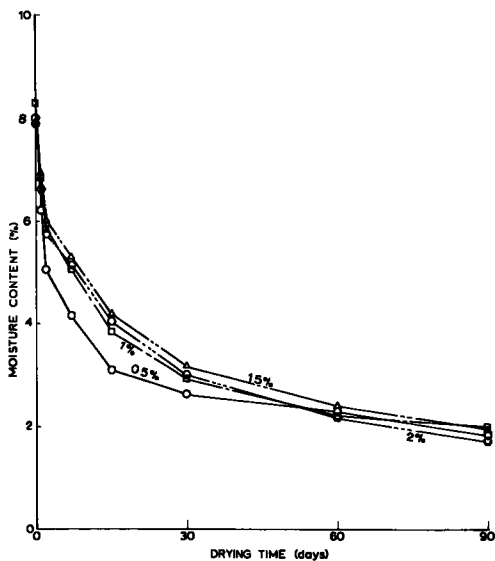


Figure 7. Variation of moisture content with drying time, Toranil A treated specimens.

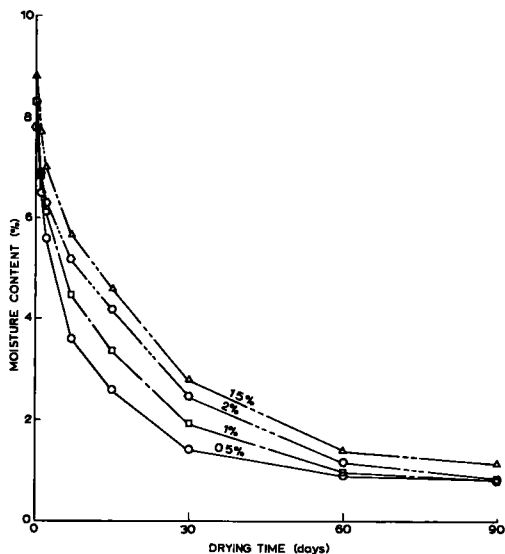


Figure 8. Variation of moisture content with drying time, molasses treated specimens.

expansion (particularly at 0.5 percent molasses content), indicates that the use of molasses as a stabilizing agent should be limited to areas with extremely dry climates.

### Moisture Retention

A direct comparison of the moisture contents of the various specimens as a function of time of curing proved inadequate as a means of determining the relative effectiveness of the additives. Figures 2 through 8 reflect the inadequacy of direct comparison because they show inconsistencies even among various percentages of the same additive, as well as showing higher equilibrium moisture contents for the untreated soil-aggregate mix than for specimens containing hygroscopic additives. The reason for this apparent discrepancy is that the conditions under which samples dried varied; therefore a more complete analysis is required to compare the effectiveness of the additives.

The process of drying may be divided into four phases during which the rates of drying will differ because of changes in the governing properties. The phases of drying, and the governing properties during each phase are given by Brown et al (4) as:

**Phase 1.**— During this phase of drying, the water content of the specimen is at a maximum (near saturation) and thus evaporation may be considered as essentially that from a free water surface. The area of escape for water vapor can be considered to be the exposed surface area of the specimen. This period is one of instability because the specimen is adjusting to the temperature of the drying medium.

**Phase 2.**— This is a period of steady rate drying which, owing to the lower moisture content, is governed by the rate of capillary movement of moisture and the rate of flow of air into the soil voids. During this phase the area of escape is some portion of the exposed area of voids.

**Phase 3.**— This period of drying is characterized by a constantly decreasing rate of evaporation more or less proportional to the moisture content of the specimen. Because of the low water availability, the rate of evaporation is governed by the rate of diffusion within the specimen. The area of escape is essentially the exposed surface area of the voids.

**Phase 4.**— The final phase of drying again is characterized by a decreasing rate of moisture loss. During this period, the rate of loss is governed by the water retention

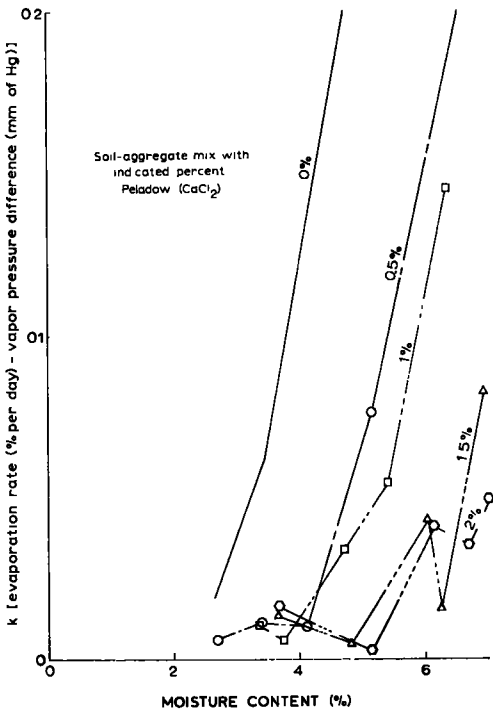


Figure 9. Index of moisture retention effectiveness for calcium chloride treated specimens.

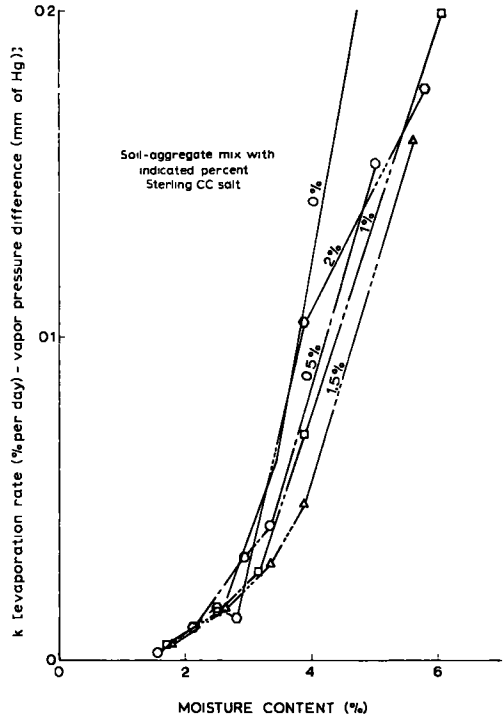


Figure 10. Index of moisture retention effectiveness for CC salt treated specimens.

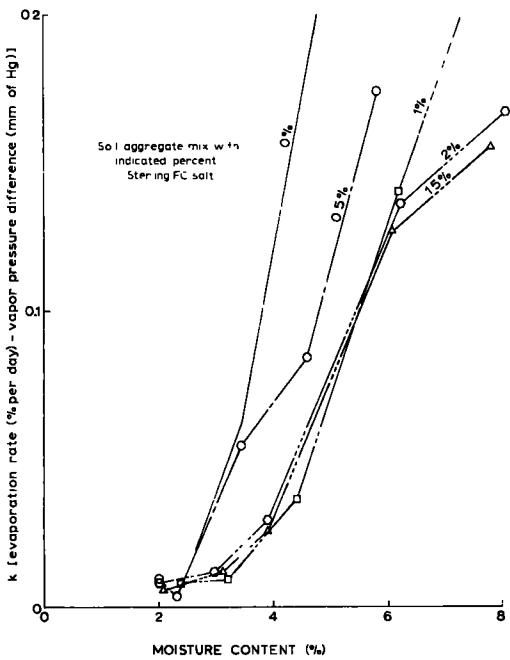


Figure 11. Index of moisture retention effectiveness for FC salt treated specimens.

forces of the soil, vapor pressure of water on clay, with the area of escape remaining unchanged.

From comparison of natural evaporation formulas and constant condition drying formulas, it is apparent that the rate of evaporation is proportional to the vapor pressure difference between the vapor pressure of the moisture in the soil and the partial pressure exerted by the air. This is expressed:  $d\lambda/dt = -k \Delta p$  in which

- $\lambda$  = moisture content
- $t$  = time
- $\Delta p$  = vapor pressure differential
- $k$  = a constant of proportionality for any given moisture content

Based on the foregoing equation Figures 9 through 14 were prepared to permit evaluation of the effectiveness of the additives. The figures show a plot of  $k$  versus moisture content. In preparing the figures, several approximations were required. First, data for the early periods were discarded because of the unstable nature of this phase of drying and because the temperature of the

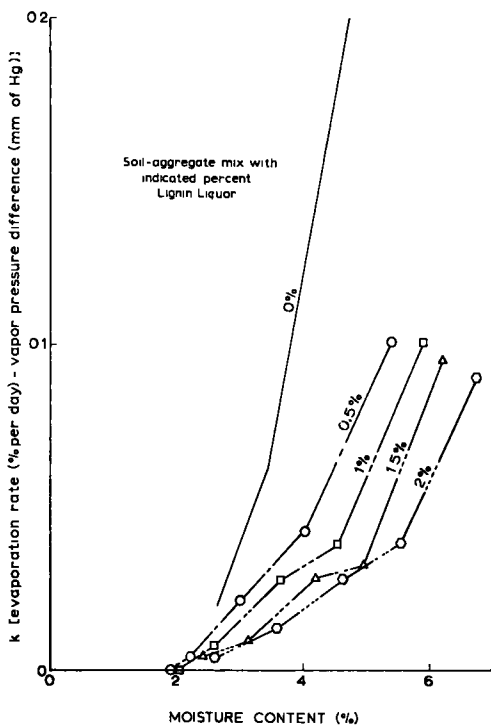


Figure 12. Index of moisture retention effectiveness for Lignin Liquor treated specimens.

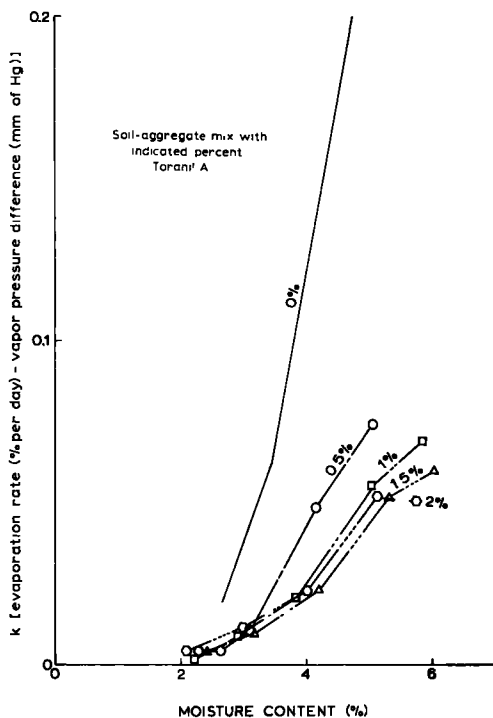


Figure 13. Index of moisture retention effectiveness for Toranil A treated specimens.

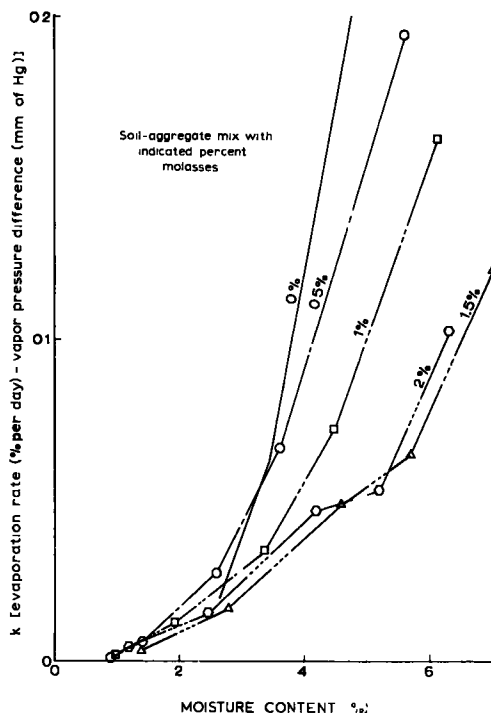
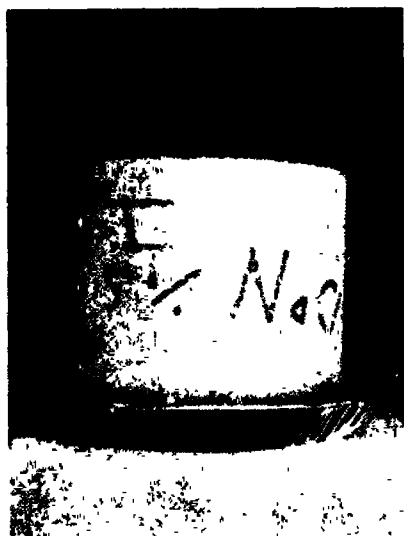


Figure 14. Index of moisture retention effectiveness for molasses treated specimens.

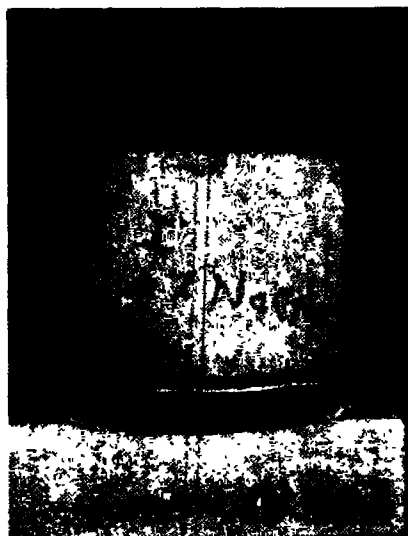
specimens could not logically be estimated as air temperature. Next, inasmuch as the vapor pressure differential was not constant throughout the drying periods, the assumption was made that the average of differences for the day of weighing and the two previous days would be representative of the conditions for the entire period. In this respect, the vapor pressure of moisture in the specimen was based on pure water because there was no knowledge of the concentration of chemicals in the solution. By doing this no serious error is introduced, because the net result is that of reflecting the vapor pressure effects of the chemicals in the calculated values of "k".

Based on these assumption, the formula was solved for k, and the resultant values were plotted against the moisture content of the specimen at the start of the time period associated with the incremental loss of moisture. Although certain irregularities appear in the curves as a result of the approximations, sufficient consistency exists to permit comparison.

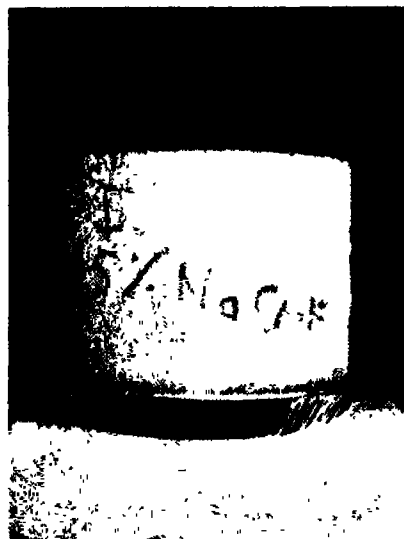
Relative moisture retention capacity of the additives may be determined from the curves by comparing the value of  $k$  for the various additives at a given moisture content; the greater the value, the less effective the additive. Less credence should be given



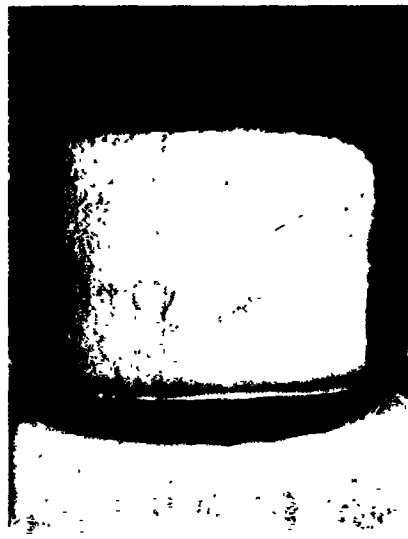
0.5% FC salt



1.0% FC salt



1.5% FC salt



2.0% FC salt

Figure 15. Crystal growth on Sterling FC salt treated specimens.



to the lower ends of the curves because weighing errors would have greater effects in this range, and because specimens may have reached equilibrium moisture content before lapse of the time increment used.

No discussion of the actual results obtained is deemed necessary because the relative effectiveness may be readily observed from the curves. However, the poor showing of the salt treated specimens does merit some discussion. Factors which should be considered because they are not reflected in the curves are (a) salt crystals grew out from the surface of the specimens (Fig. 15), (b) under traffic these crystals would be packed into void spaces, and (c) the exposed surface area per unit volume of the specimens was greater than for comparable material in a road; thus the crystallization was less dense at the surface and moisture retention was therefore less.

### Atterberg Limits Tests

The Atterberg limits give an indication of the plastic and cohesive properties of soils. Specifically, these tests give: (a) the liquid limit of the soil, or the minimum moisture content at which the soil-water mixture reacts as a viscous liquid; (b) the plastic limit, or the minimum moisture content at which the mixture acts as a plastic solid; and (c) the plasticity index, which is the range of moisture content through which the mixture exhibits plastic behavior (8). From this it is possible to assume that improved lubrication will tend to decrease both the plastic and liquid limit. Increased viscosity would have the opposite effect.

Effect of Chlorides. — Because of the amount of individual judgment involved in these tests, the degree of change in limits caused by incorporation of chlorides in the mix is

TABLE 7  
RESULTS OF ATTERBERG LIMITS TESTS

Additive				
Kind	%	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index
Untreated	-	16.8	25.5	8.7
CaCl <sub>2</sub>	0.5	15.6	26.1	10.5
	1.0	16.0	26.3	10.3
	1.5	16.5	24.9	8.4
	2.0	15.0	25.5	10.5
NaCl (CC)	0.5	16.2	24.7	8.5
	1.0	16.5	26.1	9.6
	1.5	17.0	25.3	8.3
	2.0	16.3	26.1	9.8
NaCl (FC)	0.5	17.2	26.2	9.0
	1.0	17.3	25.4	8.1
	1.5	17.5	24.9	7.4
	2.0	16.7	24.7	8.0
Lignin Liquor	0.5	16.4	26.5	10.0
	1.0	15.7	29.0	13.3
	1.5	14.6	28.4	13.8
	2.0	14.2	27.5	13.3
Toranil A	0.5	15.8	28.9	13.1
	1.0	16.2	30.1	13.9
	1.5	15.3	27.2	11.9
	2.0	14.9	29.0	14.1
Molasses	0.5	17.0	32.5	15.5
	1.0	16.4	31.7	15.3
	1.5	17.5	31.6	14.1
	2.0	17.9	31.2	13.3

not significant. Results, (Table 7) are erratic and establish no definite trend. One contributing factor in the case of sodium chloride treated specimens evolves from the fact that the salt migrated to the surface of the specimens (Fig. 15) thus causing changes in salt concentration in the material tested.

**Effect of Lignosulfonates.** — A consistent decrease of plastic limit coupled with consistent increase in liquid limit was observed with increased lignosulfonate treatment. This trend lead to the natural consequence of increasing the plasticity index, irrespective of the spent sulphite liquor involved. Table 7 gives the results of the Atterberg limits tests, and inasmuch as it can be observed that no consistent significant difference exists between decrease in plastic limit and increase in liquid limit, the increased plasticity index cannot be attributed to either lubrication or viscosity changes alone, but must be considered as a combination of both effects.

**Effect of Molasses.** — Results given in Table 7 indicate that, of the additives tested, molasses had the greatest effect on the plasticity index of the mixture. Because significant changes were not noted in the plastic limits, but marked increases were noted in the liquid limits, the increased range of plasticity is attributed to higher viscosity because of the presence of molasses.

### CONCLUSIONS

Conclusions are based on the use of additives of Peladow calcium chloride pellets, Sterling FC and CC salts, Toranil A and Lignin Liquor spent sulphite liquors, and a 79.5 Brix molasses with a single soil-aggregate surface course mix.

Using standard Proctor compactive effort the value of maximum density is increased by use of any of the additives. An inverse relationship exists between maximum dry density and optimum moisture content when the additives are used. Density is sensitive to the amount of additive in the case of chloride stabilization.

Calcium chloride is the most effective agent for densification. In order of decreasing effectiveness, the other additives evaluate as Sterling CC salt, Sterling FC salt, Lignin Liquor, Toranil A, and molasses.

Strength of the soil-aggregate mix can be improved by any of the additives. Immersed strengths with optimum amount of additives are approximately equal for all additives except molasses. Molasses is subject to fungal action and only improves strength about one-half as much as the other additives.

Calcium chloride is the most effective of the additives for moisture retention. Lignosulfonates and molasses have moderate effects, whereas rock salt is relatively ineffective.

Chlorides have little effect on the plastic properties of the mix; lignosulfonates and molasses raise the plasticity index.

### RECOMMENDATIONS

Recommendations based on the results of this study are that: (a) additional studies of the effect of these additives be initiated to provide a basis for comparison applicable to other soils; (b) investigation of the effects of the additives on moisture retention be carried out in a controlled humidity room to permit immediate comparison of results; and (c) curing of specimens for CBR tests be conducted under controlled conditions of relative humidity and temperature to permit equitable comparison of results.

### ACKNOWLEDGMENTS

The subject matter of this report was obtained as part of the research under Project 283-S of the Engineering Experiment Station of Iowa State University. This project ("The Loess and Glacial Till Materials of Iowa; an Investigation of Their Physical and Chemical Properties and Techniques for Processing Them to Increase Their All-Weather Stability for Road Construction") is under contract with the Iowa Highway Research Board as Project HR-1 and is supported by funds supplied by the Iowa State Highway Commission. Certain equipment used in the research is on loan from the Office of Naval Research under Contract Nonr-2625(00).

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# Bituminous Stabilization of Wyoming Heat-Altered Shale

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Wyoming has future highway projects that will pass through areas lacking first-class aggregates, but containing a quantity of unproven heat-altered shale aggregate. Considering the possible transportation savings if the shale were to be used, research was undertaken to explore the possibility of improving the utility of asphalt as a stabilizer for heat-altered shale by the use of trace quantities of chemical additives known to react with asphalt, or with soil-mineral surfaces. Resulting Marshall design control methods (ASTM D 1559-58T) and effective weathering tests were used to evaluate the materials used.

Various asphalts and/or additives were well above the minimum design criteria for roads receiving heavy traffic. Thus, constructing roads of heat-altered shale would be justified.

● **KNOWN SOURCES** of local quality aggregates for road building purposes are being rapidly depleted throughout the nation. Areas in the northeastern section of Wyoming are deficient in aggregate materials of proven quality. Future highway construction projects in these areas may require transporting sizable quantities of gravel aggregates for distances as great as 70 miles. However, large quantities of a heat-altered shale are available in the vicinity near highway locations.

Field problem investigation indicated these shales may provide satisfactory road building materials. Heat-altered shale has been used as a patching material, but has not been incorporated as a pavement aggregate. The performance of shale as a patching material shows that it compares favorably with gravel aggregates as a surface course.

Laboratory research, supported by a grant from the Wyoming Highway Department, was conducted to determine the possibilities of improving the utility of asphalt as a stabilizer for Wyoming heat-altered shale. Trace quantities of chemical additives known to react with asphalt and/or with soil mineral surfaces were incorporated into the investigation.

If these shales could be shown to produce satisfactory stability for the subbase, base, and pavement, future construction with this material would result in a tremendous savings.

## DESCRIPTION OF MATERIALS

To determine the stability of heat-altered shale, laboratory work was conducted in the bituminous-soils laboratory at the University of Wyoming. The materials used in the testing procedure included shale aggregate; asphalt cements of various penetrations; non-organic and organic additives.

### Aggregate

**Mineralogical Classification and Properties.** — The baked shale originally was a sedimentary bed. Stratigraphically below these sea beds was a large vein of coal. At one time, portions of this coal bed burned; the heat and escaping gases affected the sedimentary beds above to produce heat-altered shale. Evidence of the coal seam, about 100 ft thick, is seen at the WyoDak coal mine (Fig. 1).

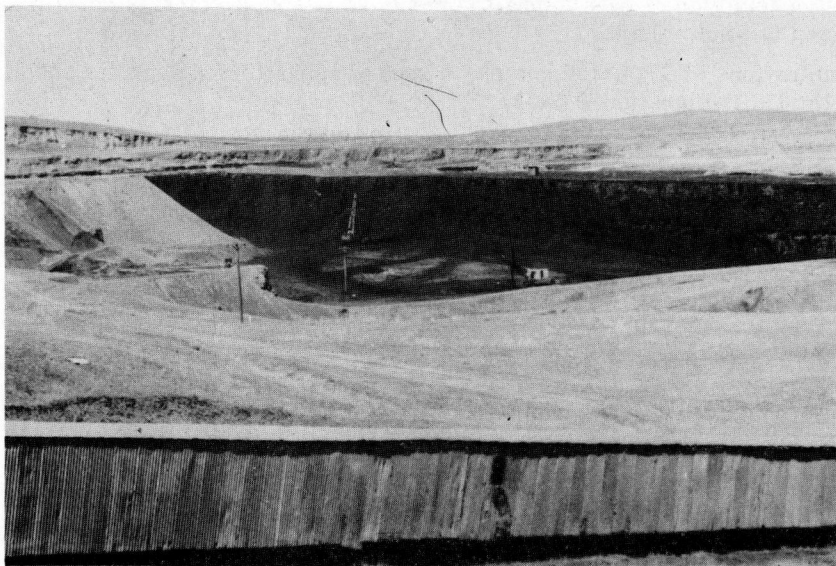


Figure 1. Coal seam at Wyo Dak.

Baked shale is variable in quality because of the manner in which it was formed. Some of the formation received extremely high temperatures, especially the rock centered near an escaping gas vent. Harder clinkers in the formation were formed by the higher temperatures. Heat-altered shale has a natural affinity for water and, as a result, has poor resistance to prolonged atmospheric exposure. Figure 2 shows the variability in the weathering of the heat-altered shale. The center of the picture illustrates slaking of a soft baked shale after prolonged exposure to the elements, but the shales shown on the sides of the picture which have been in the area for the same length of time have not weathered very much.

Gradation. — The aggregate used in this investigation was representative heat-altered shale. Samples were obtained from a pit located 1 mi south of WyoDak coal mine and 6 mi east of Gillette, Wyoming. The shale, taken from the pit deposit in irregular sized slab-like clinkers, was laminated in texture. After crushing, the aggregate particles were flat, elongated chips.

Wyoming Highway Specifications (1956) and the Asphalt Institute Specifications (1956) were followed as gradation analysis standards (Fig. 3). The percentage passing the No. 4 sieve somewhat exceeds the specifications. Wyoming specifications allow a 6 percent deviation on the gradation uniformity passing the No. 4 sieve during a day of crushing.

Engineering Classification. — Figure 4 is a micro-photograph of a thin slice of baked shale. The white or light-textured material is quartz or  $\text{SiO}_2$ , showing the high percentage of  $\text{SiO}_2$  as was found by a chemical analysis of shale (Table 1). Table 2 contains the engineering classification of the aggregate.

### Asphalts and Additives

The asphalt cements used in the testing program were Husky AC 150 pen., and Husky Rubberized AC 160 pen., and Socony Mobil 85-100 pen. Laboratory tests on the asphalt cements are compiled in Table 3.

Non-organic additives used in the combination of the asphalt-aggregate system were anhydrous lime and concentrated phosphoric acid. The organic additives incorporated in the testing of heat-altered shale for bituminous stabilization (added to Husky AC 150 or to the aggregate) were diamine No. 21 (N-coco trimethylene diamine) and Alamine No. 26 (primary tallow amine). Characteristics of the amines are given in Table 4.

Further description of base asphalt cements and asphalt cements containing additives (summarized in Table 5) are:

1. Penetrations at 77 F (100 grams, 5 sec) of asphalts and asphalts containing additives (ASTM Designation D 5-52).

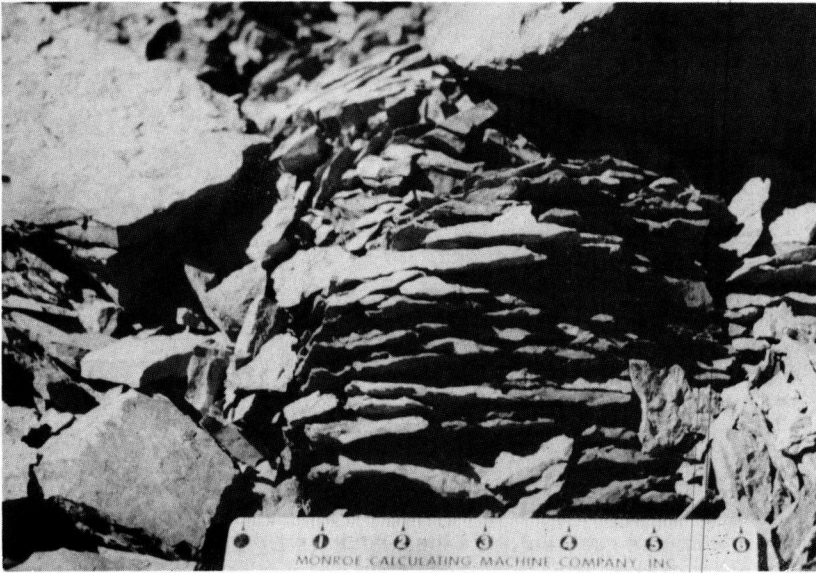


Figure 2. Slaked shale after prolonged exposure to the atmosphere.

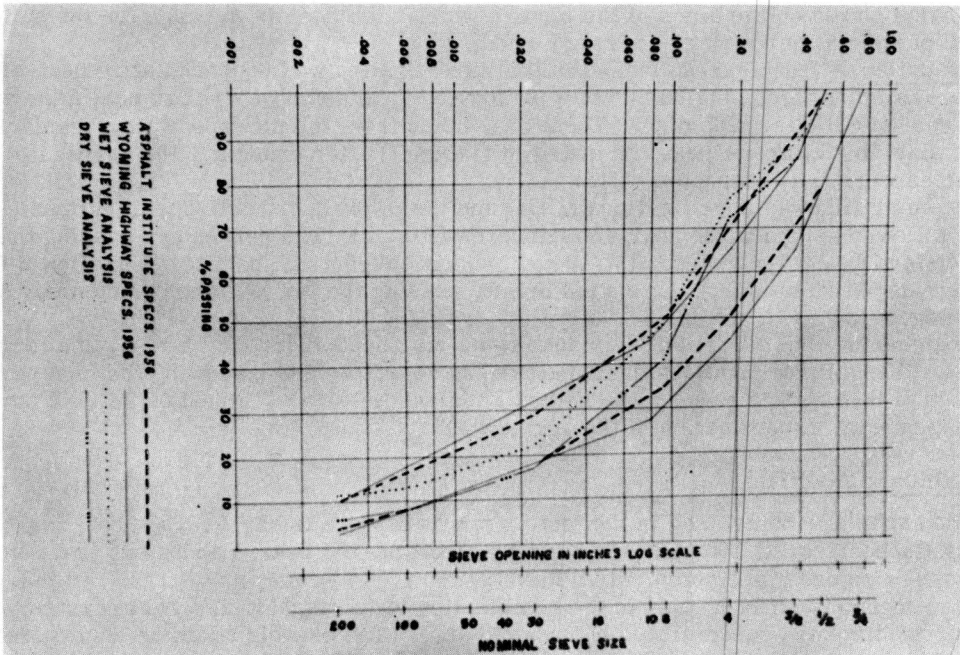


Figure 3. Gradation analysis.

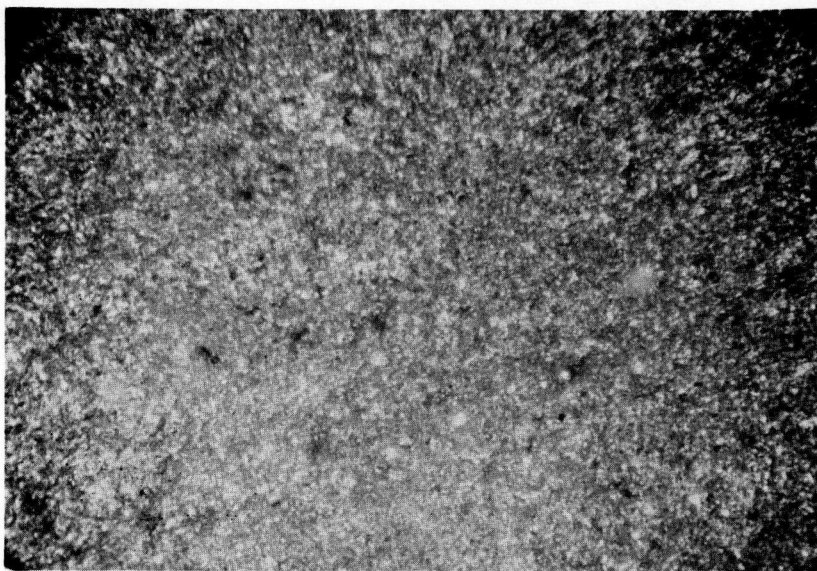


Figure 4. Photograph of a thin slice of heat-altered shale taken through a microscope. The light textured material is quartz.

2. Specific gravity at 77/77 F of asphalts and asphalts containing additives (ASTM Designation D 70-52).

3. Contact angles formed between drops of various mediums and a glass plate surface.

The penetration and specific gravity are standard qualitative tests; the measurement of contact angles has been used by others "...as a measure of the suitability of materials for bituminous stabilization" and as an evaluation of the effect of pretreatment of soil and (or) asphalt.

TABLE 1  
CHEMICAL ANALYSIS OF WYOMING  
HEAT-ALTERED SHALE

Constituents	Percent by Weight
L. O. I.	0.78
SiO <sub>2</sub>	68.0
Al <sub>2</sub> O <sub>3</sub>	15.3
Fe <sub>2</sub> O <sub>3</sub>	5.88
TiO <sub>2</sub>	0.62
P <sub>2</sub> O <sub>5</sub>	0.20
MnO	0.06
CaO	3.23
MgO	2.09
Na <sub>2</sub> O	0.26
K <sub>2</sub> O	3.01
Li <sub>2</sub> O	none
Summation	99.40 %

The contact angles were determined from dimensions obtained from an enlarged profile photograph of small uniform liquid drops resting upon a horizontal glass plate (Fig. 5). These figures, applied to calculations and tables by Mack and Lee, gave the resulting contact angles summarized in Table 5.

The smaller contact angles indicate increased spreading or dispersion ability of the liquid. Thus better dispersion of asphalt should result in improved waterproofness and consequent retention of stability when wet. However, the ease of coating is but one factor that need be considered in evaluating the effect of admixes.

The admixes have more effect on the physical properties and stability of compression specimens after immersion than is demonstrated by their effect on the contact angle.

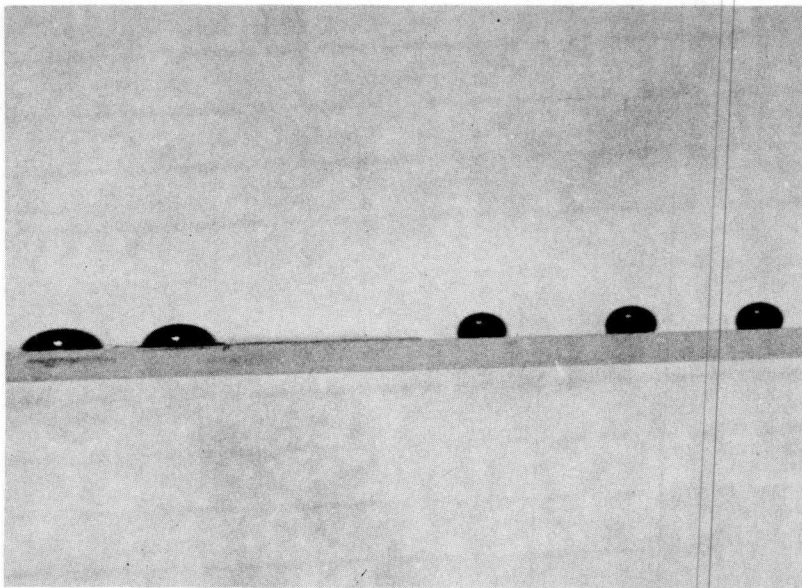


Figure 5. Silhouette photograph showing shape of asphaltic droplet for contact angle measurements (left: rubber AC 160, angle of contact  $52.5^{\circ}$ ; right: AC containing phosphoric acid, angle of contact  $98^{\circ}$ ).

TABLE 2  
ENGINEERING CLASSIFICATIONS OF HEAT-ALTERED SHALE

Determination	Value	ASTM Desig.
Liquid limit, -40 (%)	30.3	D423-54T
Plastic limit, -80 (%)	none	D424-54T
Plasticity index (%)	none	D424-54T
Spec. gr., (evacuation)	2.50	C128-42
Unit wt., dry rodded (pcf)	88.3	C29-55T
Voids vol., dry rodded (%)	0.425	C30-37
pH	7.0	

### INVESTIGATION

The stability of heat-altered shale in combination with asphalt and chemical additives was investigated by the following methods:

1. Modified Marshall method— for design and control of paving mixtures (ASTM Designation: D 1559-58T).
2. Relative test— for effect of water on cohesion of compacted bituminous mixtures.
3. Evaluation of mixtures— made by subjecting specimens to accelerated weathering tests and comparing respective stabilities.

Previously, others had achieved successful use of 2 and 4 percent of the additives used in this investigation for improving asphalt as a stabilizer. These percentages served as a guide for this study.



**TABLE 3**  
**LABORATORY TESTS ON ASPHALT CEMENTS FOR THE BITUMINOUS**  
**STABILIZATION OF HEAT-ALTERED SHALE**

Material	Husky 160 A. C.		
	Husky 150 A. C.	Rubberized	Socony 85-100 A. C.
Soft. point, F	105	—	114
Pen. at 77 F (100 g, 5 sec)	150	160	95
Pen. at 32 F (200 g, 60 sec)	31	—	—
Pen. at 115 F (50 g, 5 sec)	T. S.	T. S.	—
Duct. at 77 F (5 cm/min)	100+	150+	196+
Duct. at 39.2 F (5 cm/min)	40	150+	—
Flash, deg F, C. O. C.	535	535	540
Vis. at 325 F, s. f. s.	41	112	95
Loss on heat, 325 F, 5 hr, %	0.03	0.03	0.06
Pen. after L. O. H. % of orig.	84	84	85
Spot test	Neg	Neg	Neg
Sol in CS <sub>2</sub> , %	99.8+	99.0+	—
Sol in CCl <sub>4</sub> , %	99.8	99.0	99.89
Water, %	none	none	—
Specific gravity at 60 F/60 F	1.029	1.029	1.0105 at 77
Float at 122 F, sec.	—	—	—
Res of 100 pen.	—	—	—

**TABLE 4**  
**AMINE SPECIFICATIONS AND PHYSICAL PROPERTIES (6)**

Product		Melting Point (°C)	Sp. Gr.	Amine Number	Water Content (%)	Cost (\$/lb)
Name	Formula					
Diamine No. 21	(N-Coco trimethylene diamine) RCH <sub>2</sub> NH(CH <sub>2</sub> ) <sub>3</sub> NH <sub>2</sub>	20-25	—	375 min	2	0.63
Alamine No. 26	(primary tallow amine) RCH <sub>2</sub> NH <sub>2</sub> or (RCH <sub>2</sub> ) <sub>2</sub> NH	35	0.85	192 min	—	0.33

#### Modified Marshall Test and Relative Test

A total of 355 Marshall test specimens with an asphaltic range from 9 to 15 percent, and an additive range from 0 to 4 percent were prepared. The specimens were compacted in the specified manner (that is, proper temperature, mix time and proportions; 10-lb hammer drop of 18 in. and 50 blows to each side of the sample).

However, the Marshall method was modified in the following manner: Four representative specimens of each mixture were compacted for the modified Marshall stability tests. Following mass and dimension measurements, the samples were placed in a constant temperature oven (140 F) for four days. Then, two of these specimens were completely immersed in a water bath (140 F) for 24 hr, while the other two samples remained in the dry oven. Immediately preceding the stability tests, the dry cured specimens were immersed in this water bath for 20 min.

Marshall test property curves (unit weight; Marshall stability; flow; percentage of

voids — total mix; percentage of aggregate voids — filled) were plotted following measurements and testing. The optimum percentages of asphalt — 11 and 12 percent — were determined from these curves and are summarized in Figures 6 and 7. Large variations were observed between the strength of an untreated asphalt cured "dry" at 140 F for

TABLE 5  
PENETRATIONS, SPECIFIC GRAVITIES, AND CONTACT ANGLES OF  
ADDITIVES IN ASPHALT MEDIUMS

Medium	Penetration <sup>a</sup>	Specific Gravity <sup>b</sup>	Average Contact Angle <sup>c</sup>
Husky Rubber AC160	160	1.028	52.5
Mobil AC85-100	95	1.010	56.0
Husky AC150	150	1.025	52.1
2% phosphoric acid in AC	112	1.028	98.0
4% phosphoric acid in AC	75	1.035	
2% diamine No. 21 in AC	Too soft	1.017	53.4
4% diamine No. 21 in AC	Too soft	1.014	
2% alamine No. 26 in AC	259	1.017	56.2
4% alamine No. 26 in AC	Too soft	1.009	
AC 150 on detergent	—	—	49.1
AC 150 on phosphoric acid	—	—	neg
AC 150 on alamine No. 26	—	—	48.6
AC 150 on diamine No. 21	—	—	46.8
Water on alamine No. 26	—	—	18.0
Water on detergent	—	—	neg

<sup>a</sup>Penetration at 77 F (100 grams, 5 sec) of asphalts and asphalts containing additives. (ASTM Designation D 5-52).

<sup>b</sup>Specific gravity at 77/77 F of asphalts and asphalts containing additives. (ASTM Designation D 70-520).

<sup>c</sup>Contact angles formed between drops of various mediums and a glass plate surface.

TABLE 6  
ADDITIVES IN ASPHALT MEDIUMS AND INDICES OF RETAINED STRENGTH  
FROM EFFECT OF WATER

Additive and Medium	Index of Retained Strength
Mobil AC 85-100	68
Husky rubber AC 160	92
Husky AC 150	71
2% phosphoric acid on aggregate	61
2% phosphoric acid in AC	82
4% phosphoric acid in AC	0
2% diamine No. 21 on aggregate	96
2% diamine No. 21 in AC	101
4% diamine No. 21 in AC	100
2% alamine No. 26 on aggregate	68
2% alamine No. 26 in AC	101
4% alamine No. 26 in AC	81
2% lime on aggregate	98
4% lime on aggregate	65

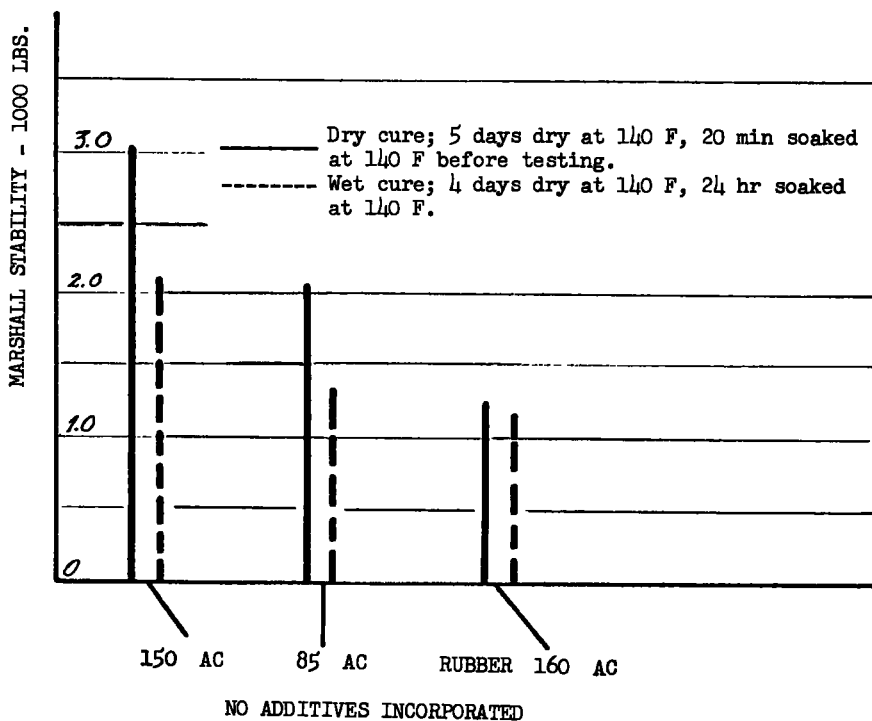


Figure 6a. Optimum Marshall stability for asphalts and for additives.

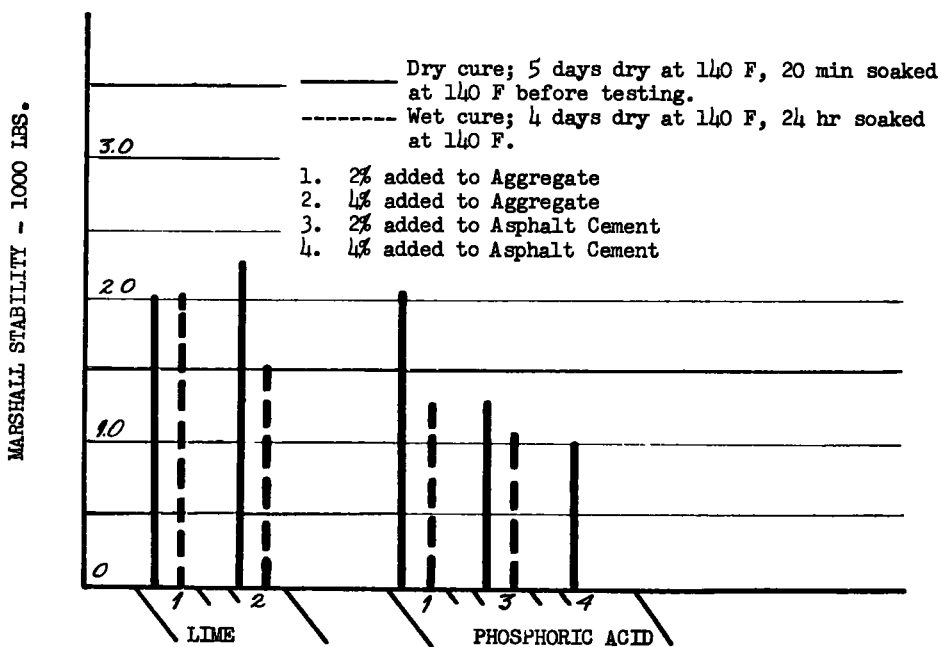


Figure 6b. Optimum Marshall stability for asphalts and for additives.

MARSHALL STABILITY - 1000 LBS.

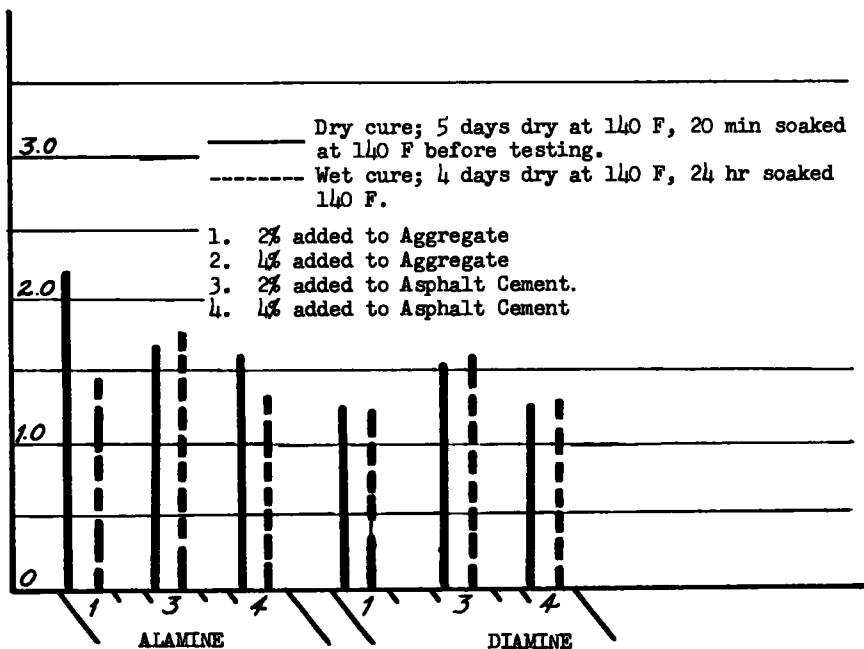


Figure 6c. Optimum Marshall stability for asphalts and for additives.

MARSHALL FLOW - 0.01 INCH

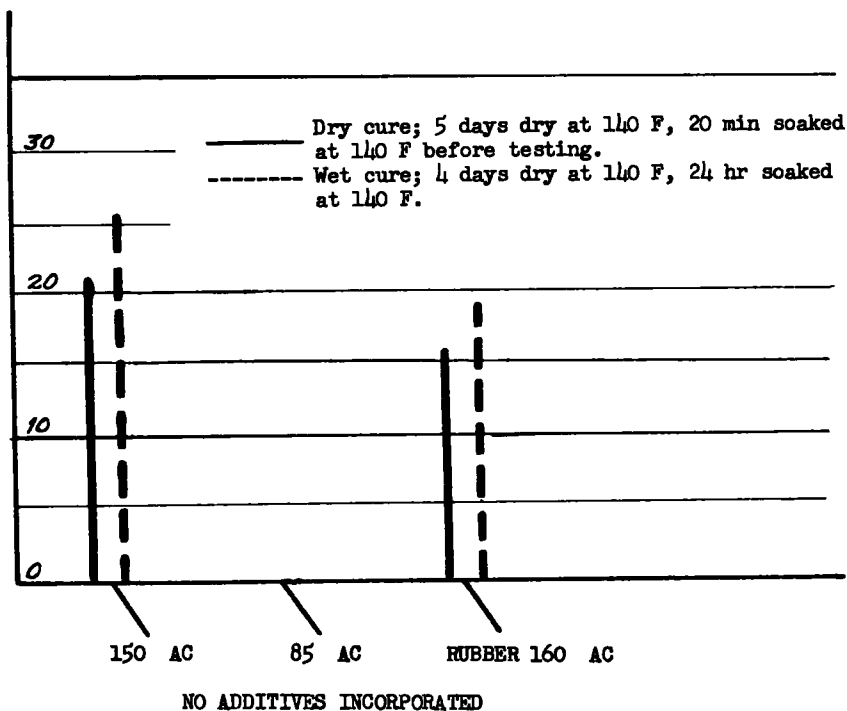


Figure 7a. Flow values for optimum Marshall stabilities shown in Figure 6.

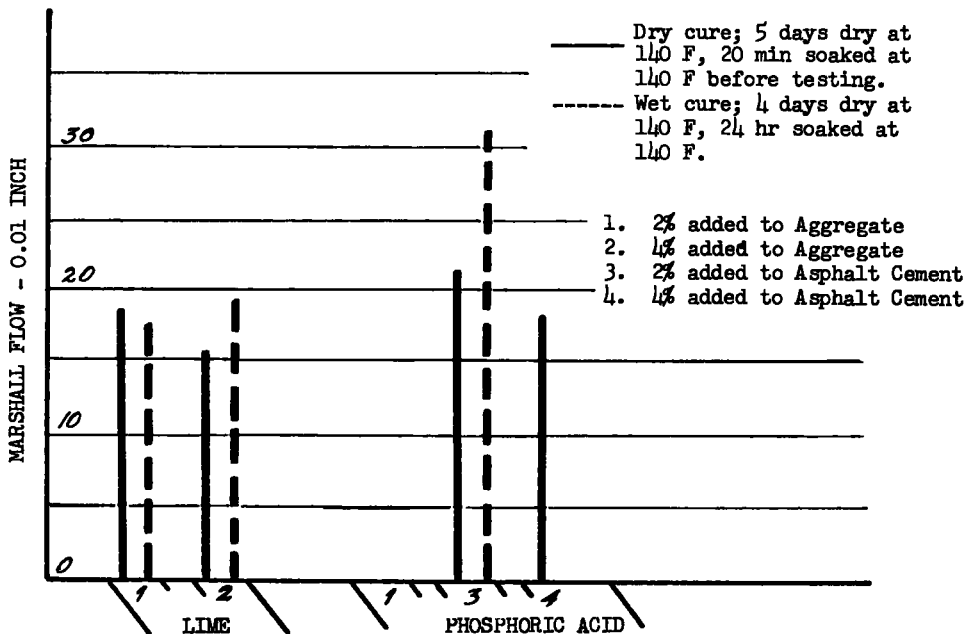


Figure 7b. Flow values for optimum Marshall stabilities shown in Figure 6.

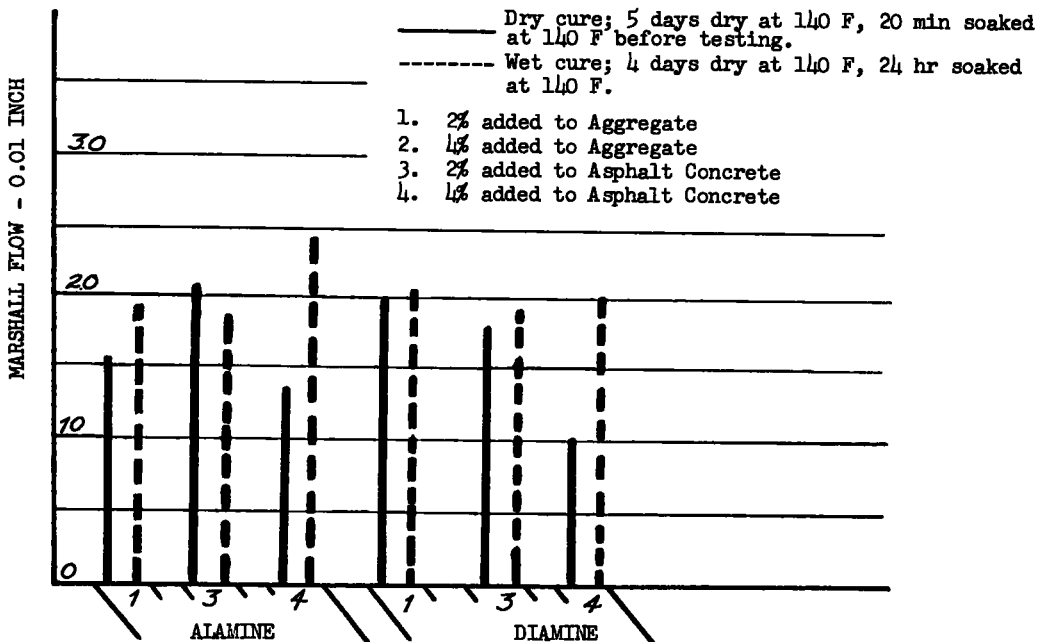


Figure 7c. Flow values for optimum Marshall stabilities shown in Figure 6.

five days and one cured "wet" at the same temperature. The Marshall stability values were lowered when the sample was soaked in water at a temperature of 140 F which tended to soften the asphalt. The indices in Table 6 were computed from the resulting values in Figure 6. This was considered the relative test for the effect of water on cohesion of compacted bituminous mixtures.

### Weathering Test

A desirable characteristic of major importance to an asphalt stabilized aggregate is its ability to withstand exposure to weather. Marshall test specimens of asphalts with or without additives presenting favorable results in earlier investigations were subjected to duplicable accelerated weathering conditions.

A weathering test device designed for the testing of coal briquettes was used for the weathering of asphalt compression specimens. "The device itself supplies dry air, heat, ultraviolet radiation and wetting under any cycle desired. The effect of freezing was included by removing specimens, at a specified time each day, and placing them in a freezer at -10 F. Ultraviolet radiation was supplied from a type S-1 mercury lamp at a distance of 15 in. This is approximately nine times the intensity of a normal mid-day sun. The infrared radiation from the lamp, combined with the heat from a 500-watt heating coil, maintains a temperature of 150 F during the heating cycle. A blower circulates air at the rate of 42 cu ft per min, giving a complete change of air every 9 sec. The three atomizing sprayheads in the top of the chamber provide a fine mist of water which wets all specimens on the shelves."

The daily cycle for 28 days of simulated weathering, as followed in the study, was:

1. Twenty-hour heating period, employing ultraviolet radiation (S-1 mercury lamp at 15 in.) and hot air (150 F) circulated on specimens.

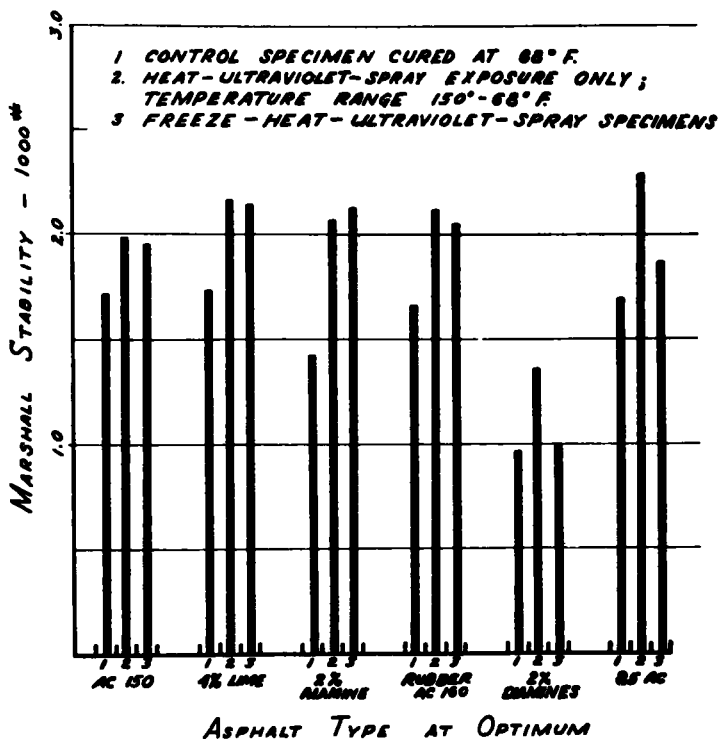


Figure 8. Weathering investigation, comparison of stabilities following 28 days of exposure.

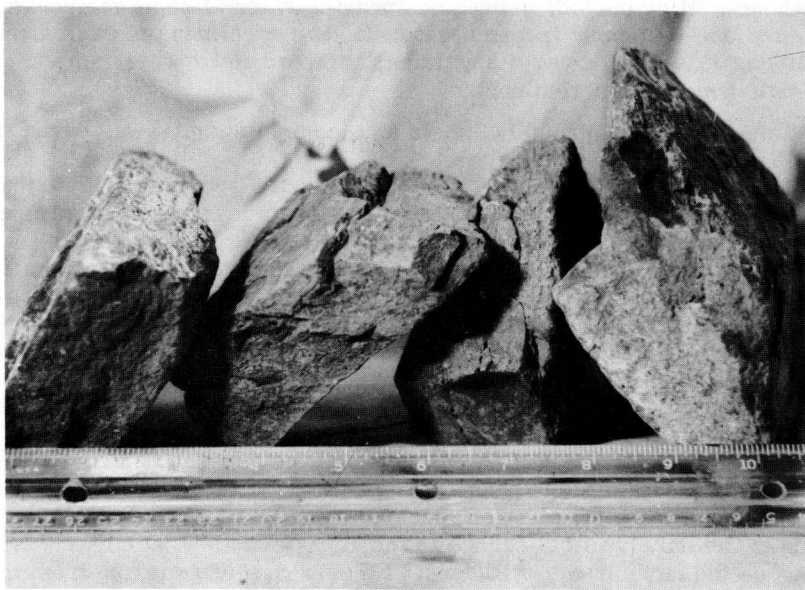


Figure 9. Heat-altered shale clinkers prior to subjection to the accelerated weathering exposure.



Figure 10. Clinkers following exposure to accelerated weathering test. The two pieces on the left were exposed for 28 days of alternating heat-spray-rest cycles. The samples on the right were subjected to spray-freeze-rest-heat cycles.

2. Two-hour wetting period, using fine spray of water at room temperature.
3. One- and one-half-hour freeze period: (a) two specimens of each sample subjected to -10 deg F, and (b) control specimen, retained under water spray.
4. One-half-hour acclimation period.

One specimen from each group (withheld as a control sample) was cured for 28 days at room temperature.

Figure 8 represents the Marshall stabilities when the specimens of various asphalts and admixes had been subjected to 28 days of accelerated weathering and soaked in the water bath (140 F) prior to compression.

A distinguishable difference is noted in each case between the control "air cured" specimen and specimens that actually received weathering exposure. The fact that some of the exposure specimens were also frozen as part of the test seemed to have little significant effect. The reader in comparing Figures 6a, 6b, 6c and Figure 8 will also note a distinct effect of the curing process on the stabilities. (Note especially Rubber AC 160 pen.)

It was interesting to compare the damage that occurred to large pieces of aggregate shale clinkers subjected to the same weathering as the asphaltic specimens (Figs. 9 and 10). The clinkers shown in Figure 10 had actually cleaved or broken apart, and thus for the extent of the test, the test appeared complete for the purposes of this investigation. The only physical change to the asphalt-shale specimens was a slight discoloration on the faces of the cylinders. No breakage or cracking from the weathering test cycles, either heat or freezing, was observed. Visual inspection after the weathering test was completed thus indicated that the asphalt-aggregate specimens held up better than the shale clinkers.

### CONCLUSIONS

The modified Marshall curing period used, simulates conditions in the field. Immediately following compaction and a minimum curing time, traffic loads are allowed on a finished roadway. Thus, the short-time cured specimens closely represent the newly finished roadway. In contrast to this, the stabilities received from the long-time weather cured specimens indicate the expected strengths of a roadway over a period of months. The effective weathering test gives a qualified answer to the question: "What will the condition of the roadway be in a few years?"

The following conclusions have been derived from the data:

1. Considering the minimum design criteria for main streets and roads receiving heavy traffic (1,800 lb Marshall stability), highway construction would be justified by using asphalt cements 85-100 and AC 150 penetration without additives.

The 85-100 penetration asphalt cement was used as a control asphalt. Resulting test values were for 12 percent AC: 2,110 lb dry, 1,310 lb wet, for the short cure and 2,560 lb weathered.

The resistance to the accelerated weathering and the high stabilities for AC 150 penetration warrants the use of this asphalt with heat-altered shale as an aggregate in highway construction (12 percent 150 AC pen. — 3,000 lb dry, 2,125 lb wet, and 1,900 lb weathered).

2. The addition of lime to the baked shale in small percentages gave a high index of retained strength. Under the short cure time used for the Marshall test, the stabilities did not exceed the compressive strength of the heat-altered shale with normal 150 penetration asphalt cement; however, the values were above the 1,800 lb — designated as the minimum. Examining the weathering results, it is noted that the lime additives produce stabilities higher than the asphalt-aggregate system using 150 penetration asphalt cement. Stabilities received were: 2 percent lime — 12 percent AC produced 2,050 lb dry, 2,025 lb wet, and 2,300 lb weathered. Lime added in small amounts for increased stability is justified.

3. Rubberized asphalt 160 penetration gave the following stabilities: 12 percent R AC 160 — 1,280 lb dry, 1,180 lb wet and 2,100 lb weathered. The rubberized asphalt had a high index of retained strength and had stabilities comparable to normal asphalt.



Rubberized asphalt has a quality of high temperature stability coupled with low temperature flexibility.

4. The Marshall stabilities and weathering investigation demonstrated that diamines and phosphoric acid additives would not be usable in a baked shale aggregate system.

5. The retained strength from the effect of water on compacted specimens containing the amine group of additives signified that the additives definitely aid in the resistance to the detrimental effect of water. However, the stability values are lower than those of heat-altered shale with normal asphalt of 150 penetration, under the short cure. The weathered specimens, of the alamine amine however are above those given by normal 150 penetration asphalt. The stabilities produced would make questionable the use of the alamine additive. The proper way in which to evaluate the effectiveness of the additive would be its incorporation into a highway test strip.

### RECOMMENDATIONS

Recommendations for highway test road construction using Wyoming heat-altered shale and additives are as follows:

1. Highway test strips of heat-altered shale should be constructed and better additives and asphalts should be evaluated. Test strips should be compared to parallel sections of equivalent normal asphalt cement using aggregate of proven quality.

2. The test strips should be constructed in accordance with strict specifications, presently in effect, to produce a successful roadway.

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# Preliminary Findings and Future Programming Of a Basic Research Project Involving Calcium Chloride with Pure Clays

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The ultimate purpose of this project is to devise a method for predicting the effect of calcium chloride on stabilization of natural soils. In past and current practice, empirical testing is done for each soil in question; this procedure is most inefficient, and gives little or no basic knowledge of the principles involved.

It is proposed to study the effect of calcium chloride on each of several pure ion types of each of three pure clay minerals. After effects are known (both physico-chemical changes occurring and their related changes in physical and mechanical properties) for each pure sample, these results will be used to predict effects on mixtures of pure samples, and these predictions will be checked by experiment. Finally, the same procedure will be used for naturally occurring soils that have been carefully analyzed.

Thus, it may be possible ultimately to analyze a natural soil, and predict with reasonable accuracy the effect of calcium chloride on that soil, by referring to results on the pure constituents making up that soil. Perhaps more important, such a study would add to the store of knowledge of the principles governing soil stabilization.

Preliminary results showed that treating a sodium montmorillonite clay with 4 percent calcium chloride gave an increase in maximum dry density of about 11 percent.

● PREVIOUS WORK in this field has been almost entirely empirical. Laboratory studies have used natural soil samples of unknown composition; field studies have done the same thing. On some soils, the chemical has shown marked beneficial effects; on others, it has had little effect; and in a few cases, detrimental effects have been suspected. This empirical approach leads to the individual testing of every sample of soil proposed to be used with the chemical — a most inefficient approach. Every sample, not just every engineering or agricultural type of soil, must be tested because the engineering and agricultural classifications are not based on the physico-chemical properties involved in stabilization with calcium chloride.

## OVER-ALL PLAN

It is proposed that the effects of calcium chloride be studied on pure ion types of pure clay minerals, one-by-one. This would involve obtaining relatively pure samples (in large quantity) of each of the major types of clay minerals. Each mineral type would have to be purified in the laboratory, then converted to each of several pure ion types, by base exchange procedures. It is expected that the ion types to be prepared and studied would include the following:  $H^+$ ,  $Na^+$ ,  $K^+$ ,  $Ca^{++}$ ,  $Mg^{++}$ ,  $Al^{+++}$ , and  $Fe^{+++}$ . Preparation of pure ion types of pure clays is time-consuming and expensive. In some cases, the  $H^+$  ion types may be unstable after short periods of time.

The effects of calcium chloride would then be studied on each ion type of each clay mineral; the samples being studied would represent the colloidal fraction of naturally occurring soils. Presumably, some samples would be reactive (good or bad) and others would not. The extent of reaction, and the amount of chemical required for certain desired effects could be determined. Analysis by base exchange procedures would tell how much of the  $\text{Ca}^{++}$  (from the calcium chloride) reacted, with what it reacted, and how much of another base was released. Electrical, physical, and mechanical properties of each sample would be determined, both before and after treatment with calcium chloride, and correlated with the base exchange observed.

It is, of course, being assumed that the primary stabilization effects of the chemical are almost entirely centered in the clay fraction of the soils, except for moisture control which is being considered as a separate and distinct effect. The evidence justifying this assumption is very strong.

The information accumulated by the studies described would reasonably be expected to provide a key whereby the action of calcium chloride could be predicted for any mixture of the pure materials studied. Later stages of the study would have to include tests on mixtures of the pure samples to determine if any side effects occur because of the simultaneous presence of two or more materials. Mixtures of the pure samples would be reacted with the chemical to test the accuracy of the predictions.

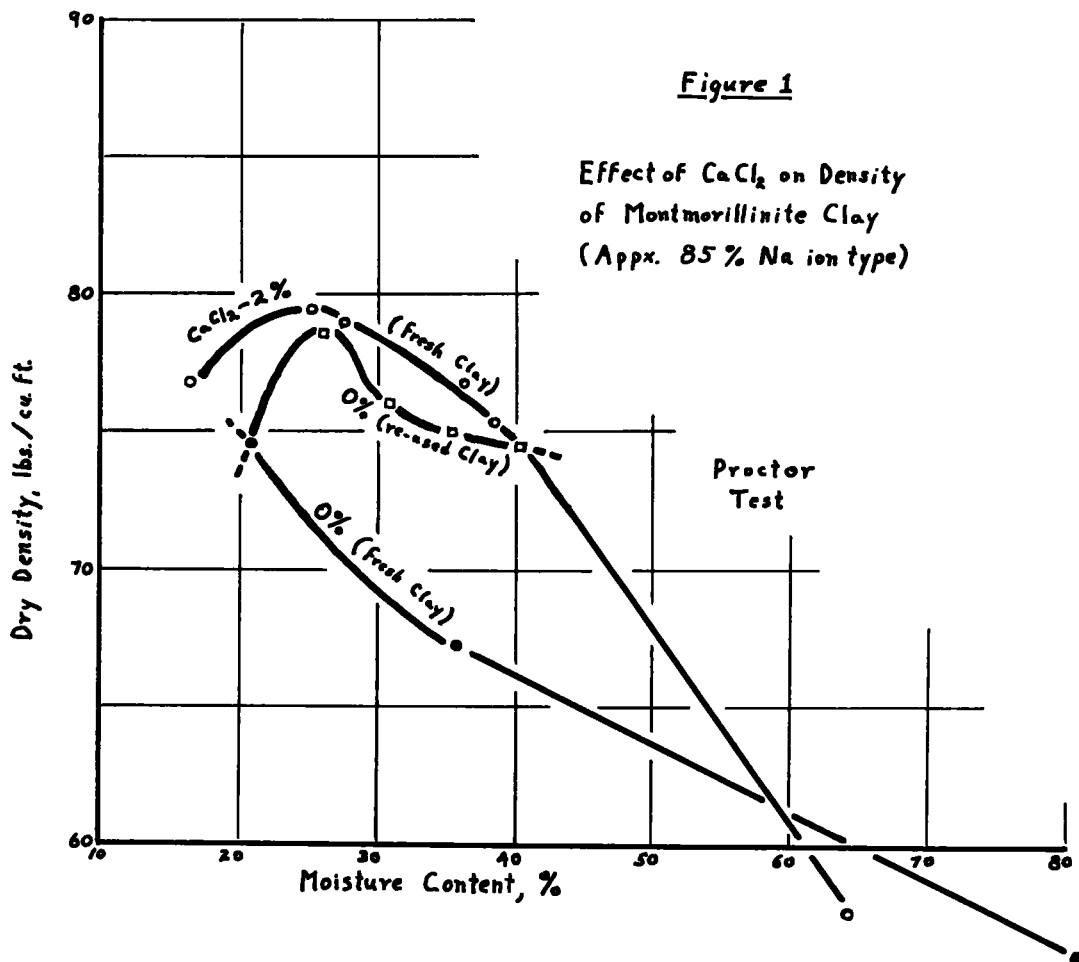


Figure 1. Effect of  $\text{CaCl}_2$  on density of montmorillonite clay (approximately 85 percent Na ion type).

If the predictions on mixed pure samples were reasonably accurate, the next step would be to obtain naturally occurring "impure" clays, analyze them for type of clay minerals present, and for the ion types present, then predict the action of calcium chloride on the naturally occurring clay, on the basis of the known action of the chemical on the various pure components that make up the clay being studied. The prediction would then be tested for the naturally occurring clay.

Finally, naturally occurring soils of various kinds, with various clay contents, would be obtained and analyzed for amount of each clay mineral, and for the amount of each ion type of each clay mineral. Then a prediction would be made as to changes in physical and mechanical properties that would be caused by various amounts of calcium chloride, based upon the keys established from the original studies. The predictions would be checked.

Assuming that this scheme was found to work, a recommendation could be made that calcium chloride be used with a certain soil; that a specific amount be used for certain predictable effects; or that the chemical not be used at all if the composition of the soil were known, in terms of the pure components that had been studied. Since the chemical action is primarily confined to the colloidal fraction of the soil, the composition analysis would have to include: percentage of clay present, type of clay minerals present in approximate percentages, and the ion types of the clay (or the ratio of ion types).

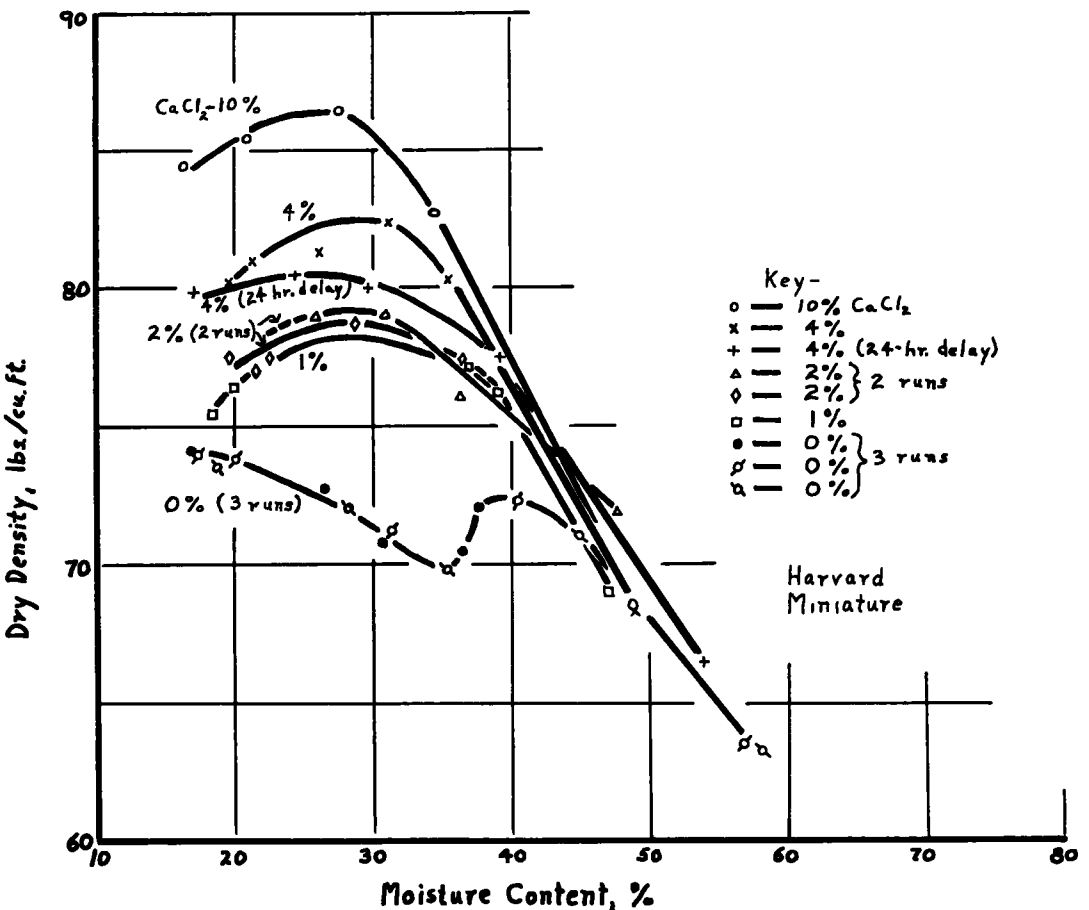


Figure 2. Effect of  $\text{CaCl}_2$  on density of montmorillonite clay (approximately 85 percent Na ion type).

This entire argument is based on the assumption that it is much easier to determine the composition of a soil than to run a series of empirical stabilization tests on that soil. Much simpler methods of determining soil composition are continuously forthcoming, and in the not far-distant future, analyses of composition may become routine as part of the general classification of soil types.

The proposed scheme is difficult to execute. The attempt to find keys for classification of soils to be treated or not to be treated, and in what amounts, might fail. However, it is almost certain that valuable information would be provided about how the chemical works — a type of information not provided by the popular empirical tests. If that information were available, any number of approaches to more intelligent use of the chemical might become evident.

Work on the project has been started, and some preliminary results have been obtained.

#### PRELIMINARY EXPERIMENTAL WORK

The objective of the preliminary experimental work was to determine whether or not it is feasible to carry out the large-scale program of study of effects of calcium chloride on pure-ion types of pure clays. If no appreciable effects are noted, it might be fruitless to proceed; if appreciable effects are noted, an extended study seems

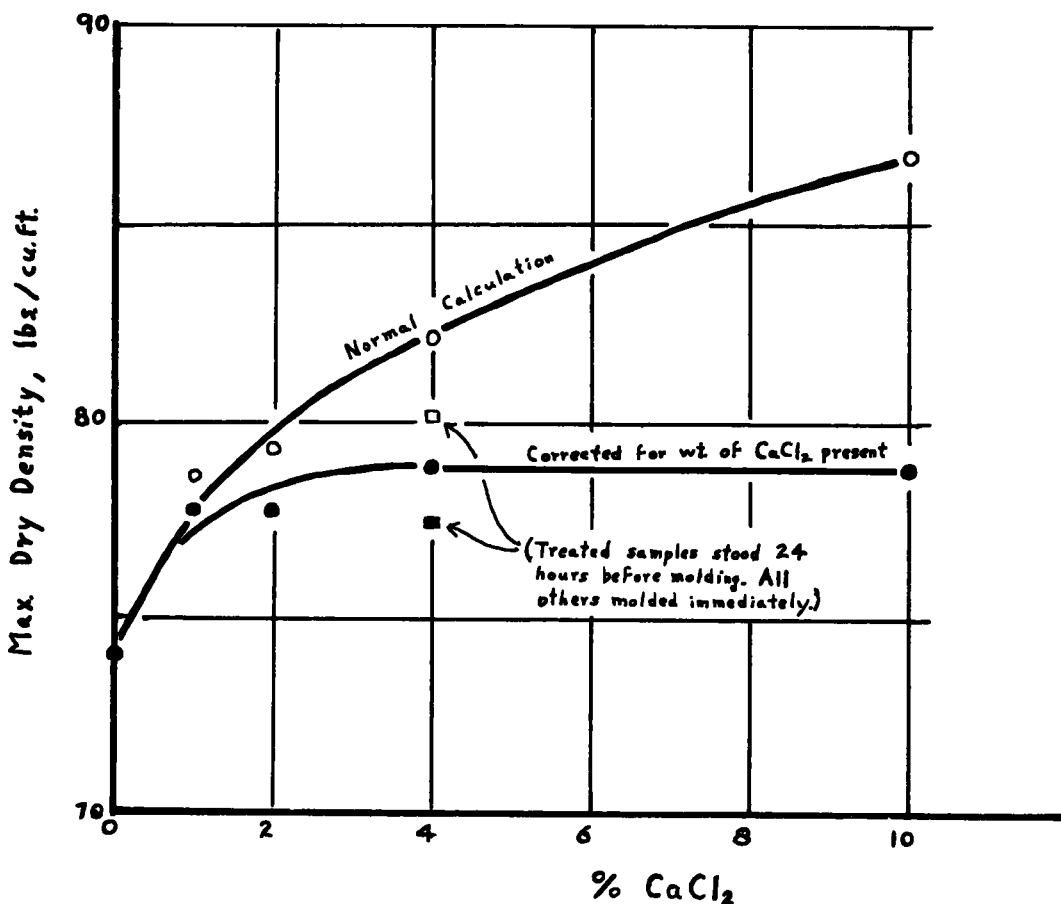


Figure 3. Effect of CaCl<sub>2</sub> on density of montmorillonite clay (approximately 85 percent Na ion type).

justified. Some people still contend that base exchange reactions are not involved appreciably in stabilization with calcium chloride, that moisture control only is involved, that an increase in density is caused merely by the weight of the chemical added, or that other mechanisms account for most of any changes in physical and mechanical properties.

The specific purpose of this preliminary experimental work was twofold: (a) To determine definitely whether or not the addition of calcium chloride to a predominately sodium-type montmorillonite clay will change its maximum dry density in a Proctor-type test, and if so, by how much; and (b) To determine whether or not the Harvard miniature compaction apparatus will give highly reproducible results with pure clay minerals. (It is desired to use as small a sample as possible, for the expensive, pure-ion types of pure clays to be used later.)

### Materials

The soil used was Volclay, consisting of about 90 percent montmorillonite, having exchangeable bases approximately as follows: calcium 22, and sodium 85 milliequivalents per 100 (these two bases constituted almost all the exchangeable bases, and represented a clay that was primarily, but far from completely, a sodium clay). The moisture content of the air-dry clay was about 4.5 percent.

Distilled water and chemically pure calcium chloride were used.

### Procedure

**Proctor Compaction Test.** — The calcium chloride was dissolved in the water to be added to the clay. The water was added to the clay as a spray, to aid in mixing. The clay and water were thoroughly mixed by spatula and by kneading by hand. The wet clay was compacted as follows: A hammer weighing 5.5 lb was dropped 12 in. to compact  $\frac{1}{30}$  cu ft of clay in 3 layers, with 25 blows per layer. About 4 lb clay was required per test. The materials were used and tested at room temperature. It was noted that when water was added to the Volclay, the temperature of the mix rose about 10 F above room temperature. Exact moisture contents were determined by running moisture contents on portions of the compacted samples, rather than by trusting the weights of water added to a given weight of soil, and assuming zero evaporation.

A comparison was made between using fresh material for each moisture content, and re-using the initial compacted sample for the higher moisture contents, all with no calcium chloride. The startling difference in results is shown in Figure 1. For all other tests, fresh samples were used for each moisture content.

Use of 2 percent calcium chloride increased the maximum dry density (for fresh samples, and for the driest clay — without the chemical — which could be readily compacted) from 74.5 pcf to 79.2 pcf, an increase of 6.3 percent — Fig. 1).

**Harvard Miniature Compaction Test.** — The general procedure followed that described for the Proctor testing. Compaction was done in three layers, 25 blows per layer, with the spring set to give a force of 30 lb for each pushing blow. The amount of clay required for each test was only 100 gr, compared to 4 lb for the Proctor test. Results checked reasonably closely with those for the Proctor test (curves for 2 percent calcium chloride and for 0 percent calcium chloride (fresh material) in Fig. 1 and 2).

Apparently the montmorillonite continues to swell, with added water, as expected, when no chemical was added. A "hump" was observed (Fig. 2) at about 40 percent water with the Harvard miniature test; the presence of this hump was confirmed by three closely checking runs, at 0 percent calcium chloride. It is probable that this hump was not observed in the Proctor test merely because no data points were taken in the pertinent range.

It was noted that the presence of calcium chloride greatly increased the ease and speed of wetting of the dry montmorillonite.

Figure 2 shows that when calcium chloride is present the montmorillonite gives "normal" Proctor-type curves, although the tops of the curves are fairly flat. Increase in the percentage of the chemical seems to cause the curves to have a sharper peak.

The reproducibility of the results from the Harvard miniature was excellent, although

it should be stated that considerable care was used in the experiments. Three check runs were made at 0 percent, and two check runs at 2 percent chemical, with extremely close checking. Of a total of 45 data points, all points were well within 1 percent of the expected position (with respect to a curve), except for two points, which varied about 1.3 percent from the expected values. Variations inherent in the test procedure are apparently very small in comparison with variations caused by deliberate change of variable.

Only one run was made to check the effect of delay of time between addition of the water and chemical, and molding of the specimens. The results are shown in Figure 2 for 4 percent calcium chloride. This cursory check indicated that the effect may be appreciable. The chemical reactions involved in base exchange are not necessarily first-order reactions, and a time effect may be expected. This could be of vital importance in field applications, and should be studied intensively. In addition, previous work has indicated that certain favorable "curing" actions may follow appreciably after application of calcium chloride. Study of these time effects will increase the number of variables to be studied, and greatly increase the magnitude of a complete study, but they must not be ignored.

The effect of calcium chloride on maximum dry density of montmorillonite is summarized in Figure 3. An appreciable increase in density is caused by the chemical. Calcium chloride in amount of 4 percent by weight increased the maximum dry density by about 11 percent (by normal calculation), or by about 7 percent (after subtracting out the weight of the calcium chloride added). Apparently additions of the chemical in amounts above 4 percent made no further increases in density of the soil, except for that caused by the weight of the chemical itself.

It can be conclusively stated that the use of calcium chloride results in appreciable increases in the maximum dry density of sodium montmorillonite, and that the Harvard miniature compaction apparatus can be used for careful work with the pure clays.

#### ACKNOWLEDGMENT

The author wishes to thank Mohsen Tewfik for his help in the experimental work reported in this paper. In lieu of numerous references pertinent to this paper, the author refers the reader to HRB Bibliography 24, "Stabilization of Soil with Calcium Chloride," 1958, by Floyd O. Slate and A.W. Johnson.

# Current Interpretation of Stability Measurements On Two Experimental Projects in Maryland

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WILLIAM B. GREENE, Soils Engineer, Maryland State Roads Commission

The roads being evaluated are (a) a stabilized granular soil base surfaced with a double seal and (b) a soil-lime-fly ash stabilized base.

Generally, when using the Benkelman deflectometer with or without a Helmer deflection profile recorder, measurements of the vertically downward deflections of a road surface are valuable measures of the adequacy of a flexible pavement design. In addition, measurements are also made of the recoveries from deflections, as well as residual deflections. In the past, the last two measures were generally scrutinized at least to the extent that their values were not considered to be unusual.

Using the deflectometer with a profile recorder makes possible a determination of the longitudinal bending or flexing of the road surface in the general area of the test point being evaluated.

In this research, the recorded deflection profiles are being used to determine all of these measures. In addition, they are examined for evidence of potential extrusion, evidence of any traveling rolling wave ahead of the dual wheels (sometimes also found to exist behind the wheels), and evidence that the supports of the apparatus (the datum) are or are not in the zone of significant influence.

It is admitted that the recorded deflection profile leaves much to be desired in the line of accuracy. The greatest inaccuracies are considered to be caused by the zone of influence which introduces a varying bias to the profile and extrusion of the road surface materials upward between the tires. When the zone of influence is negligible, and the surface materials are stable the recorded profile is considered to be a virtual image of the deflection of the road surface along a line between the tires of the dual wheels of a moving truck.

The importance and meaning of the various data obtainable from the profiles are carefully analyzed. In the case of the soil-lime-fly ash stabilization project, the use of calcium chloride was effective in improving stability, particularly at early ages. In the base of the granular soil stabilization project the use of calcium chloride was effective in improving and maintaining stability.

● THE COMMITTEE on "Soils-Calcium-Chloride Roads" initiated certain field research investigations to evaluate the effect of calcium chloride in soil base stabilization. The State Roads Commission of Maryland has cooperated with this committee on the continuing study of these experimental projects. An interim report was published in HRB Bulletin 241.

The method adopted to evaluate these test sections was principally through use of the Benkelman deflectometer with a Helmer deflection profile recorder to measure and record the deflection characteristics of a road under comparatively natural condi-



tions of loading. Experience in evaluation of these two projects suggest that there is a definite value in using this non-destructive method of test.

It is the hope of the committee that this report will stimulate interest in similar studies by other agencies and that they might benefit from such interpretations of stability measurements as advanced herein.

The roads selected in this study represent what might be considered to be near extremes in stabilization. The first test project, approximately 800 ft in length on Maryland Route 2, consisted of stabilizing a granular soil base course 24 ft wide. Calcium chloride was applied to six test sections at varying rates of  $\frac{1}{2}$ , 1 and  $1\frac{1}{2}$  lb per sq yd per 4-in. compacted course. These quantities of calcium chloride were approximately 0.15, 0.30 and 0.45 percent, respectively, by unit weight of the granular base. Eight untreated control sections were alternated in the test area (Fig. 1). Test points for evaluation of the base were located in the outer wheel path in each lane and were spaced at  $12\frac{1}{2}$  - ft intervals.

The project was opened to traffic in October 1957 and was cured for one month prior to double sealing the base with cut-back asphalt and stone chips. The road has since been maintained by resealing areas where failures in the form of alligator cracks would not heal by themselves. Areas that developed holes were patched with a cold mix so as to restore satisfactory riding qualities to the road.

Visual observation is used to evaluate the performance of the road. The performance is described in terms of the type and severity of the failure. These types and severities of failures are the properties against which the recorded profiles are being compared. The profiles are also being examined for any peculiarities that recur and speculation is being made on their significance.

Deflection profiles are being made at the test points at least twice a year. On Feb. 25, 1958, a special effort was made to obtain profiles on the effects of frost action. On this occasion, there was time enough to get deflection profiles at only every other test point.

### OPERATING PROCEDURE

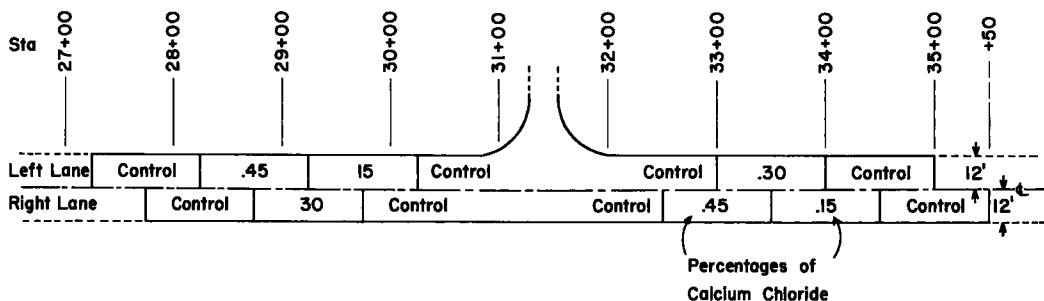
In the process of using the Benkelman beam, with or without the Helmer recorder, it is the practice to locate the probe point of the beam 4.5 ft ahead of the dual wheels so that the front datum supports are 4.5 ft behind the dual wheels (see schematic in Fig. 9). It is still the practice to get an elevation reading of the probe point or test point as nearly the instant as possible that the truck begins to move forward. Next, the elevation reading indicating the maximum deflection is obtained, and finally, an elevation reading when the truck rear duals are estimated to be at least as far ahead of the probe point as they originally were behind the probe point. On one project where the zone of influence is extensive and is considered significant, the probe to wheel distance at which the final elevation reading is taken is 9 ft (see Fig. 9).

If a Helmer recorder is used in conjunction with the beam to trace out the changes in elevation of the test point, the elevation readings can be determined from the recorded profile as well as or better than from a dial indicating micrometer.

### PROFILE USE

From experience it seems best to select a "zero" elevation reference point on the profile that is not in the zone of significant influence, and to refer the elevation of any other points on the profile to this zero elevation. The zero elevation reference point on one project was taken when the dual wheels of the truck were 4.5 ft beyond the test point and on another project when they were 9 ft beyond. When using these distances in computing for rebound deflection or residual deflection, the data are annotated by (DR 4.5 ft) or (DR 9 ft) which is intended to mean "to where on the profile the rebound deflection, etc., is referred".

When the front or rear beam datum supports are in the zone of influence, the recorded profile is variably biased. It is believed that measurement of the area and magnitude of the zone of significant influence will, along with the other measures, be a valuable measure in evaluating stability. At a time when the full length of the profiles



Plan  
12" Granular Soil Base

Md 2

Figure 1.

can be corrected for the effect of the zone of influence, all other measurements taken from the true profile should then be more realistic.

Essentially by convention, it has been suggested that signs be assigned to deflection data. Rebound deflection measurements have always been found to indicate that there always is a deflection under a load, and since the elevation of the point of maximum deflection is always lower than the rebound point with which it is compared, they are considered to be negative. The conventional measure of deflection has for the most part been found to be negative but under certain circumstances was found to be positive. Likewise, residual deflections are generally found to be negative, primarily because of the zone of influence, but, under certain circumstances they could be positive. Flection values are given a negative sign because it is intended to assign a positive sign to measure of reverse flections almost invariably found both immediately ahead and behind the part of the profile used for the regular determination of flection.

TABLE 1  
ACTUAL TRUCK WEIGHTS IN POUNDS

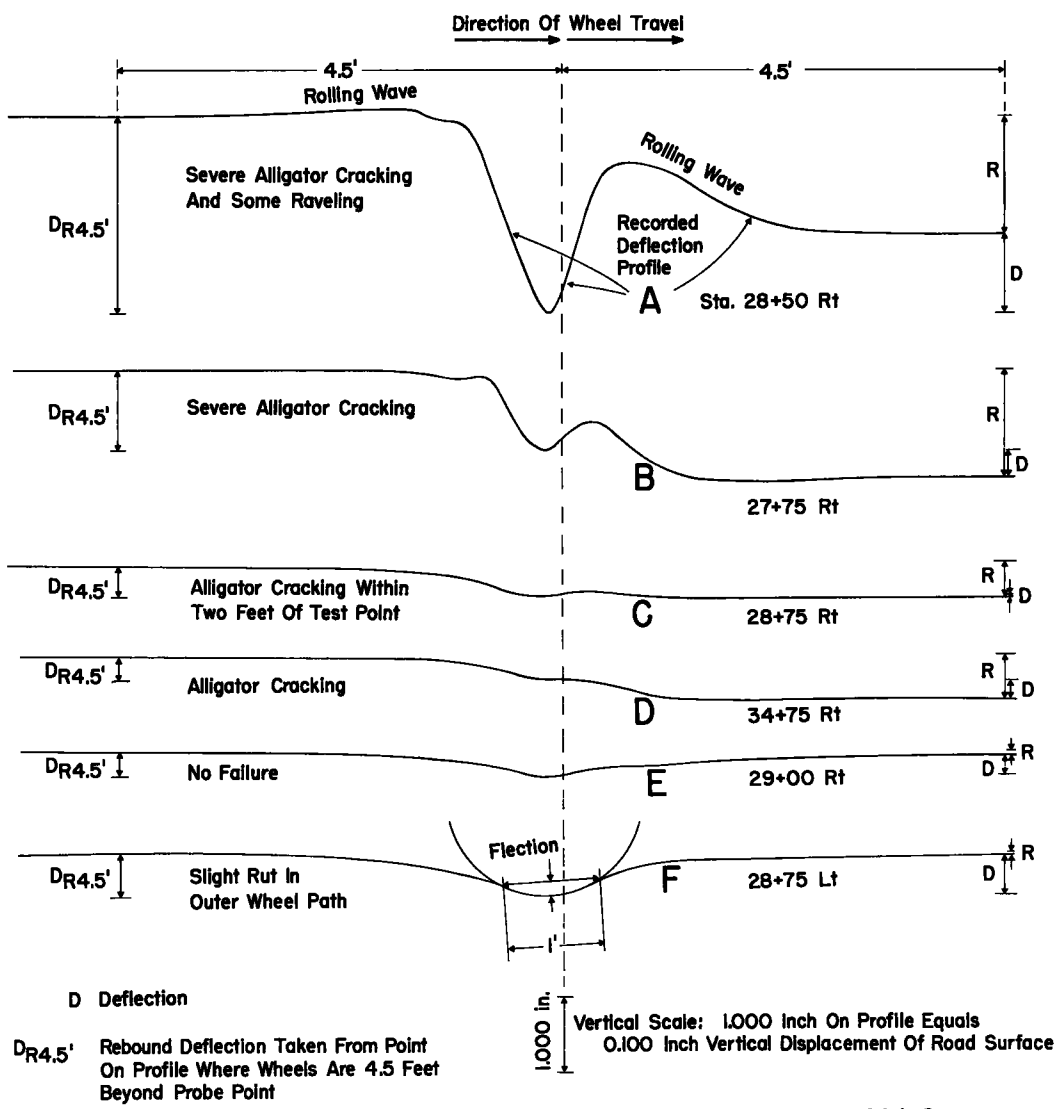
Date	Gross	Front Axle	Rear Axle	Rear Dual Wheel
Maryland Route 2				
2-25-58	20,760	4,760	16,000	8,000
4-9-58	18,520	4,480	14,040	7,020
10-2-58	18,840	5,760	13,080	6,540
3-4-59	19,580	5,340	14,240	7,120
10-17-59	19,480	5,620	13,860	6,930
Maryland Route 33				
8-26-58	33,000	10,280	22,720	11,360
9-8-58	32,790	10,470	22,320	11,160
11-13-58	32,380	10,730	21,650	10,825
3-3-59	30,240	9,270	20,970	10,485
5-19-59	31,630	10,500	21,130	10,565
9-1-59	29,820	9,350	20,470	10,235

Because the rear axle load of the trucks used in the project could not conveniently be duplicated or be equal in weight of trucks used by other researchers, all of the data used in this report are reduced to inches per 1,000 lb per dual wheel. Actual wheel loads are given in Table 1.

In the future, other characteristics of the profiles (corrected for the effect of the zone of influence) that are intended to be given numerical values are: (a) the traveling rolling wave ahead of (and behind) the wheels; (b) inflection points of the profile curves; and, (c) possible estimate of transverse flection from tire tread spacing and residual deflection.

### SHAPE OF PROFILES

Figure 2 shows accurate reproductions of several actual deflection profiles, not



Md 2  
2-25-58

Figure 2.

corrected for the effect of the zone of influence, illustrating the variety of profiles obtained on a day during the late winter thaw (Feb. 25, 1958). Visual inspection is all that is necessary to arrive at some conclusions concerning profiles such as these.

It should be emphasized here that profiles A, C, and E cover a linear distance of only 50 ft of road and that all three of these profiles were obtained within a time interval of approximately 4 min, and further, that profile F was obtained in the other lane opposite profile C approximately 18 ft distant an hour later.

Profile A suggests a serious unstable condition existing to a considerable depth. A traveling rolling wave is ahead of the wheel. Also, a similar but much smaller wave is following the wheel. Excessive positive residual deflection (R) suggests a high potential for fine alligator cracking as is caused by what is referred to as "extrusion".

Extrusion as used in this report is defined as the movement of the double seal and base materials upward between the tires of the duals. Extrusion can also occur on warm days on bituminous pavements that are particularly rich in bitumen that has remained alive. Visual inspection of the road surface at the test point represented by profile A showed severe alligating and some raveling.

Profile B is similar to A with the exception that the depth of severe instability is less and the traveling rolling wave is smaller. The potential for fine alligator cracking should be nearly equal to that of profile A as might be inferred from the comparable magnitude and sign of the residual deflection (R).

Instability indicated by profiles C and D is apparently limited largely to the granular base or at least within the top stratum of the subgrade. There is a difference in the shapes of these two profiles; also, one test point showed failure and the other did not. Speculation on the profile characteristics with respect to performance of the road at these test points would be questionable, because profile D was obtained 600 ft from profile C and the profile C was obtained only 25 ft from profile A. Because of this proximity of profiles A and C, the conditions that exist at the test point for profile A may have influenced the shape of profile C; also, alligating extends to within 2 ft of the test point for profile C.

Profile E shows what might be a rolling wave ahead of the wheel; however, what appears to be a rolling wave here could actually be a slight extrusion of the road surface as might be inferred from the residual deflection. On the day that the profiles were obtained, generally the profiles had the shape of profile E with about 25 percent showing no visible evidence of what may be either a traveling rolling wave ahead of the wheel or extrusion of the road surface materials upward between the wheels.

Profile F is similar to profile E except that it appears that the subbase may not be furnishing the stabilized granular base the same support as at the test point for profile E. There may also be a natural stability deficiency in the granular base itself. Longitudinal flection or bending of the road surface can be determined as is illustrated on profile F. Actually, flection is determined as described in the Appendix.

Measures of flection from profiles such as profiles A through D are considered highly biased because of extrusion which in essence can be pictured as a transverse flection. Profile E may also be significantly biased. The magnitude of this transverse flection is actually the magnitude of the positive signed residual deflection plus approximately +0.001 in. for 1,000 lb per dual wheel or more as caused by the effect of the zone of influence. This transverse flection or extrusion is serious because it is confined to the space between the edges of the tire treads of the dual wheels. The tread spacing between the tires used in these investigations was approximately five inches.

It is admitted that the Benkelman beam is always in the zone of influence during the process of getting any deflection profile at any of the test points. It is most probable that the beam, when used in the conventional manner, is always in the zone of influence. Measurable or not, however, the magnitude under certain conditions appears to be insignificant with respect to the magnitude of some of the other measurements taken from the profiles.

For practical purposes, on this project it is assumed that the zone of significant influence does not extend more than 4.5 ft from the center of the rear dual wheels of the truck. It is thereby assumed that the elevations of the test points at or outside of

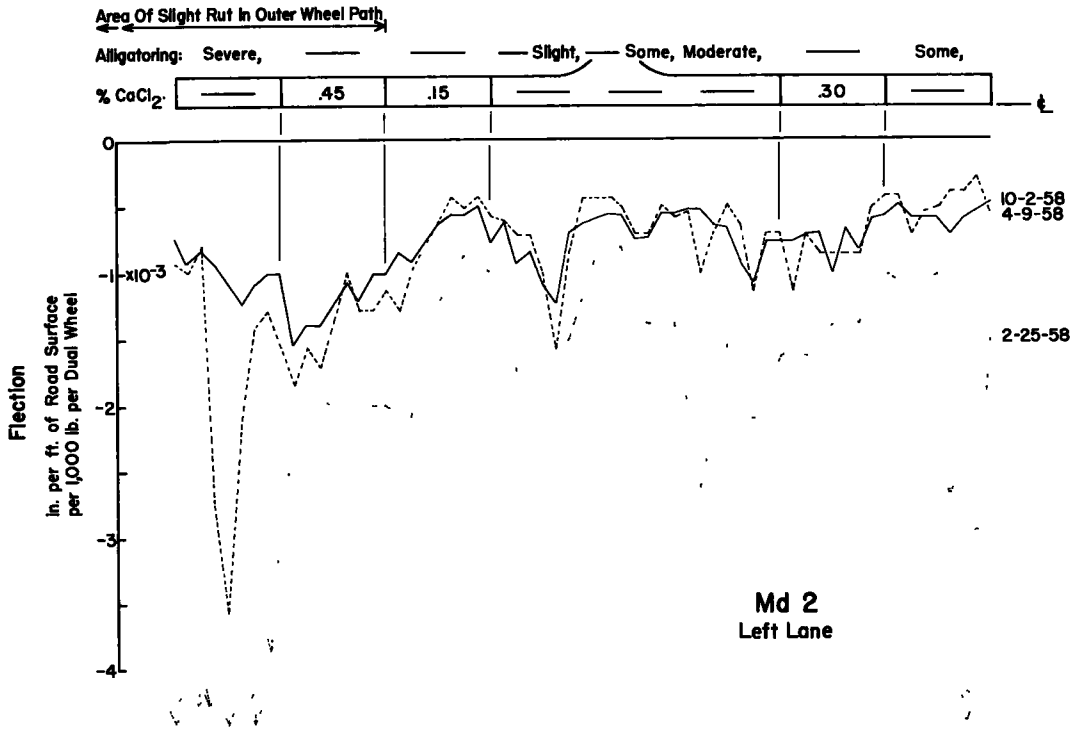


Figure 3a.

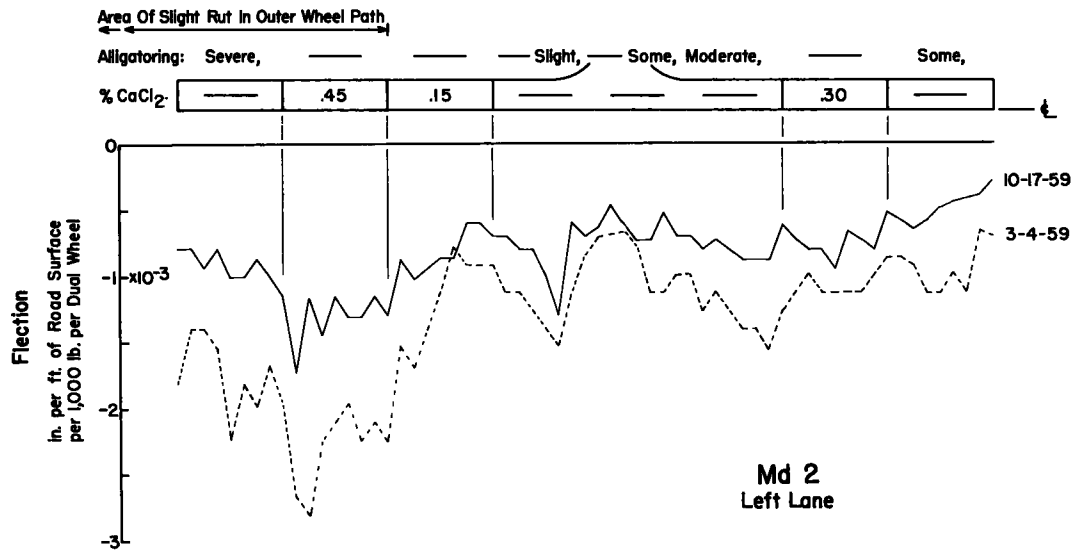


Figure 3c.

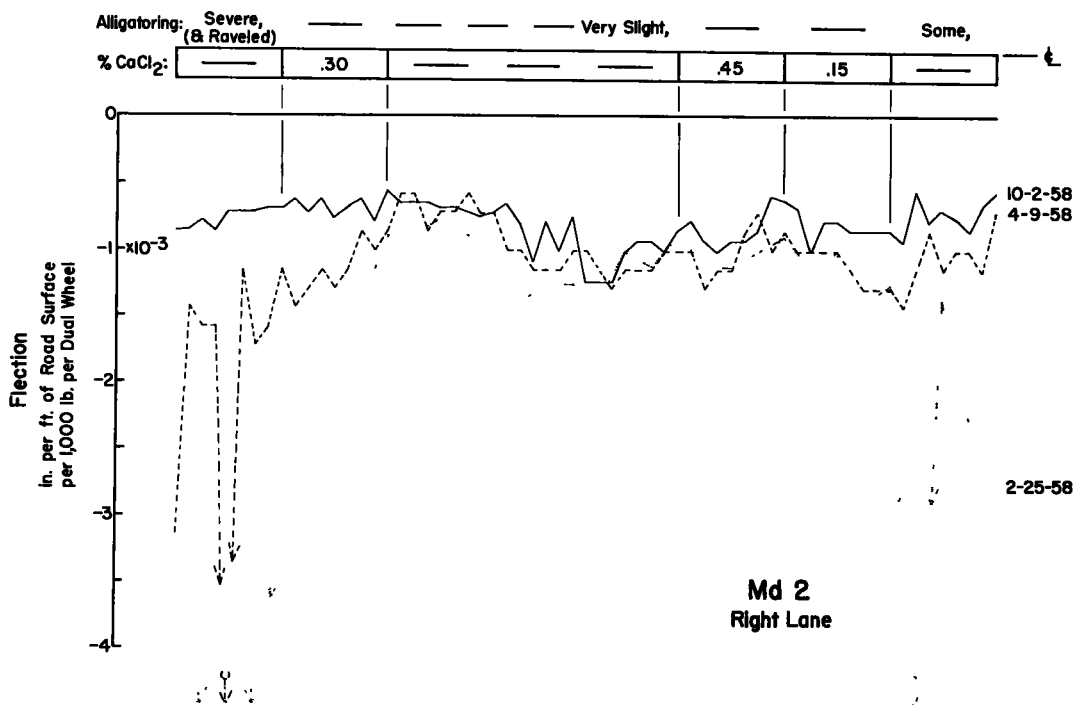


Figure 3b.

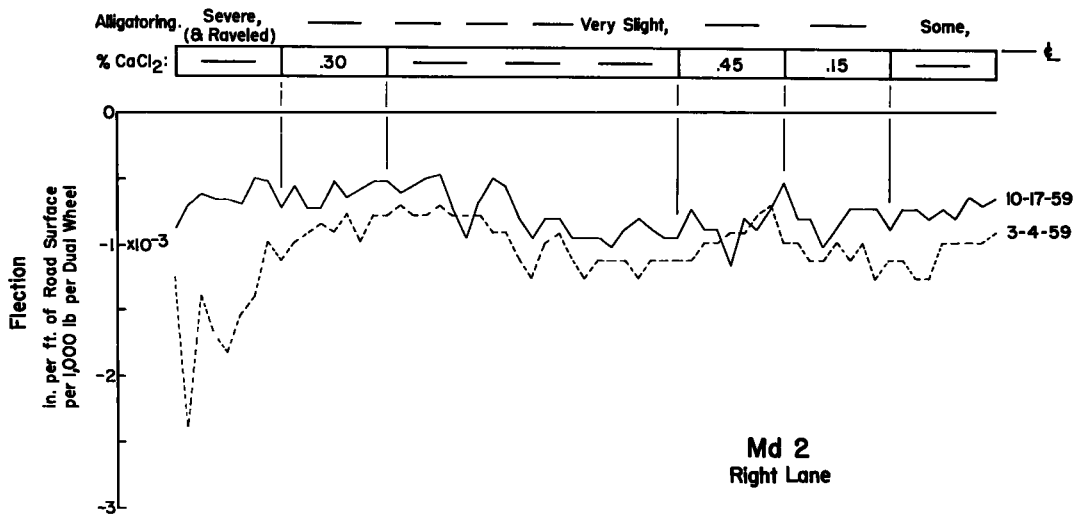
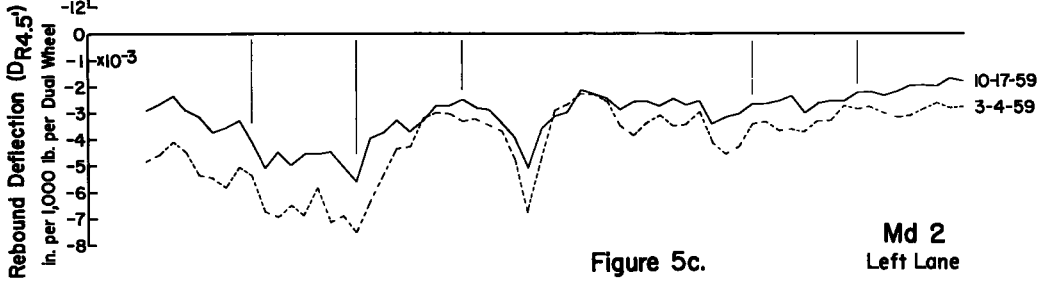
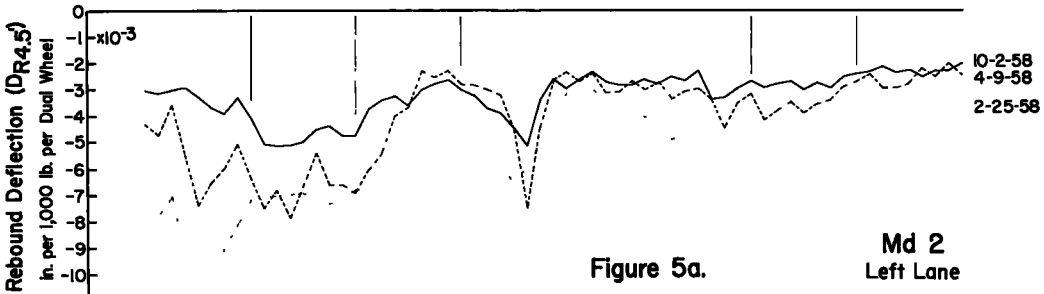
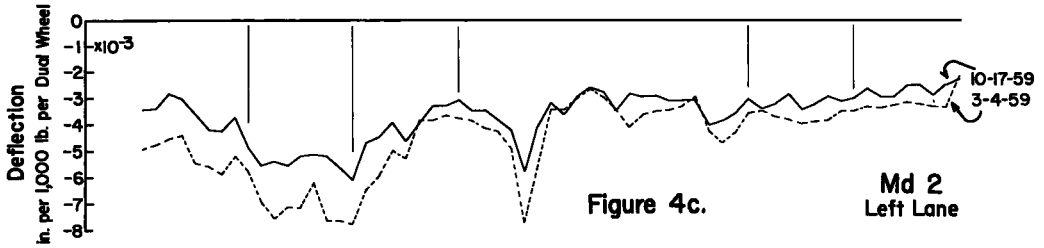
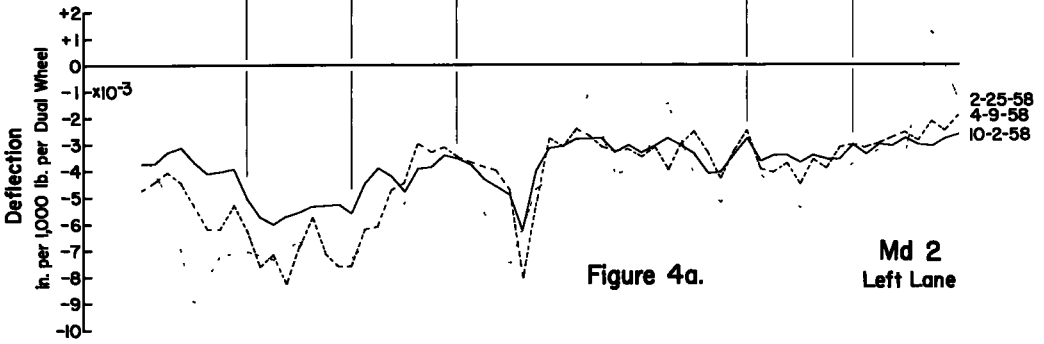
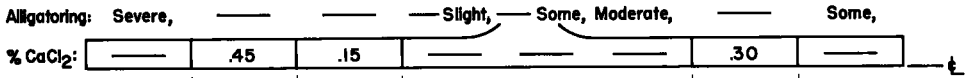
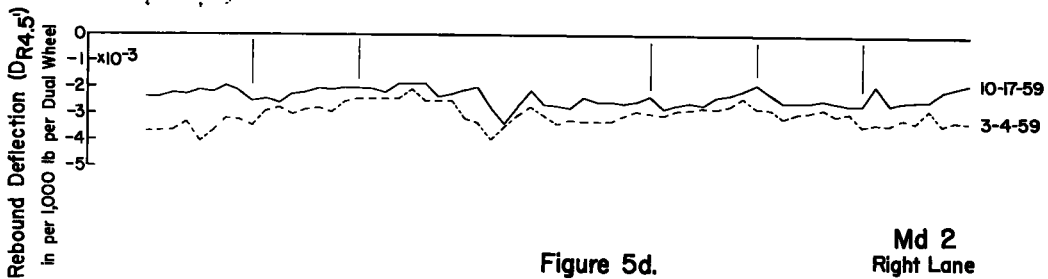
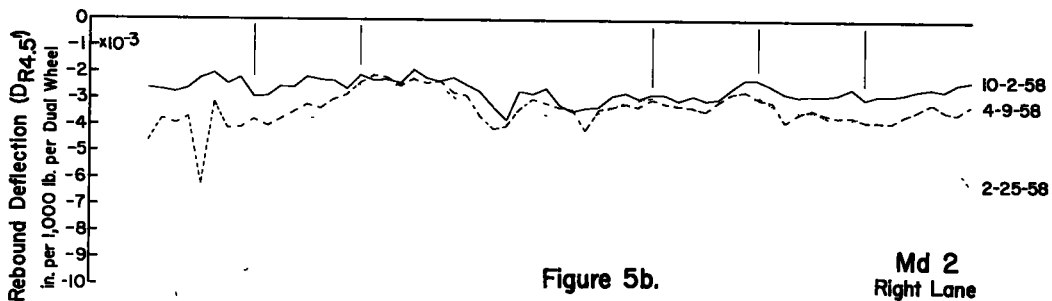
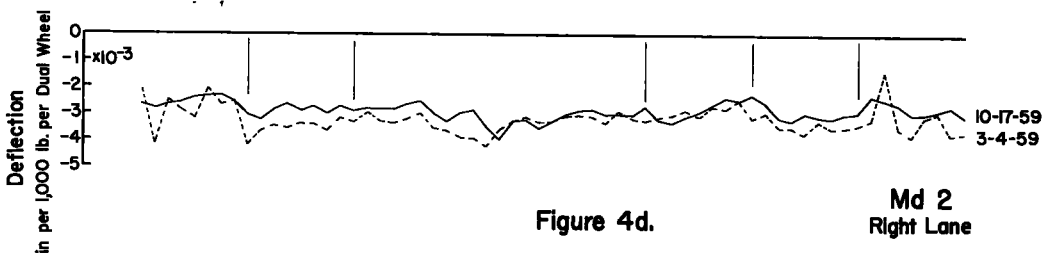
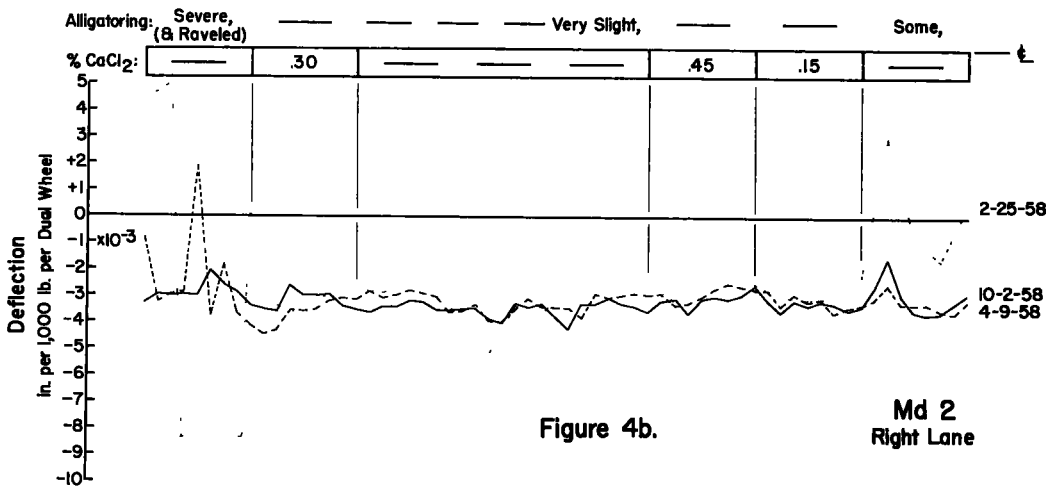


Figure 3d.

Area Of Slight Rut In Outer Wheel Path







that radius are not significantly affected by the vehicle. Such points are illustrated in Figure 2 by the extreme right hand limits of the profiles labeled with the D's and R's and near the left hand limits at the vertical arrows identified as measures of rebound deflections ( $D_R 4.5$  ft). It is assumed that as the profile is being recorded the recorded elevation of the test point on the profile at ( $D_R 4.5$  ft) would not change even if the test vehicle were moved much further from the test point.

Extrusion seems to bias the conventional measure of deflection to a greater extent than any other phenomenon occurring during the determination. It is believed that extrusion is not a linear continuing process continuing throughout the time that the tire treads are in close proximity and straddling the probe point. Rather, it is believed that more than 90 percent of the extrusion has taken place before the dual wheel axle is directly over the test point. On this premise, rebound deflection ( $D_R 4.5$  ft) is relied upon to be a better estimate of what ordinarily conventional deflection would satisfactorily indicate. In Figure 2, comparisons of deflections (D) with respect to rebound deflections ( $D_R 4.5$  ft) for the several profiles would indicate that the rebound deflection ( $D_R 4.5$  ft) is more realistic.

It should be emphasized that ordinarily deflection profiles will not appear as shown in Figure 2 but rather like profiles E and F and very often even flatter than profile E. Furthermore, ordinarily the conventional measured residual deflection will be negative whereas every profile in Figure 2 has a positive residual deflection.

#### CHARACTERISTICS OF PROFILES VS OBSERVED PERFORMANCE

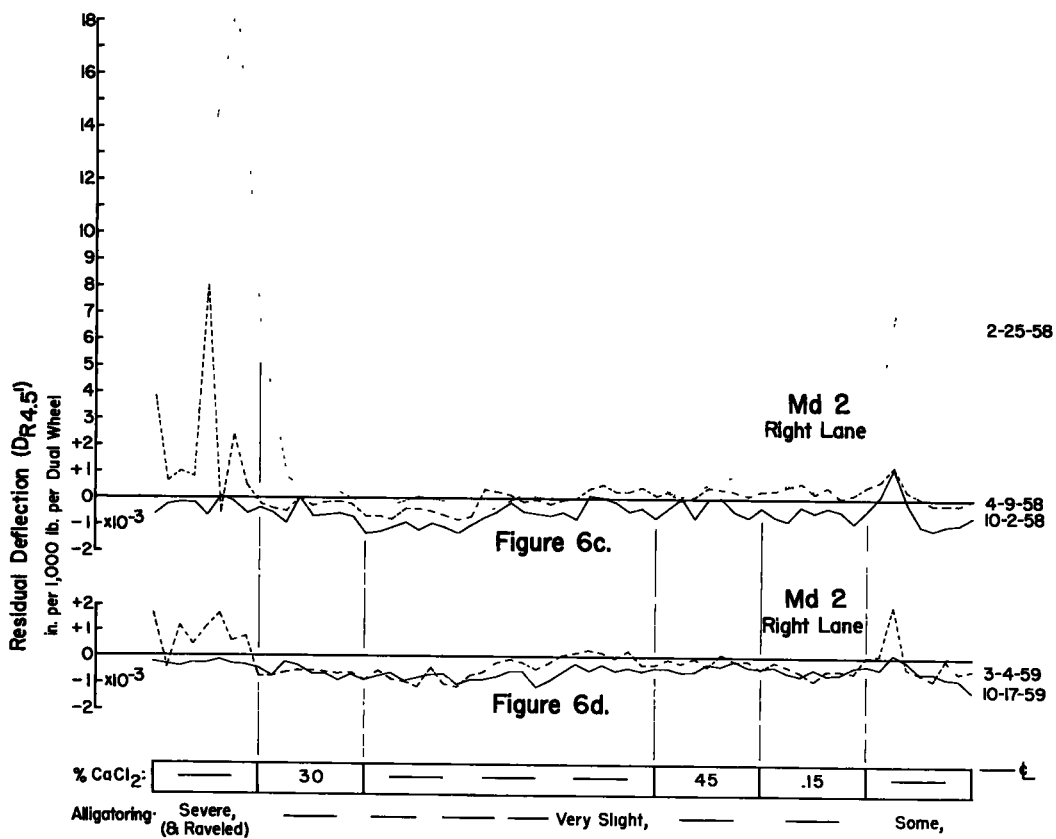
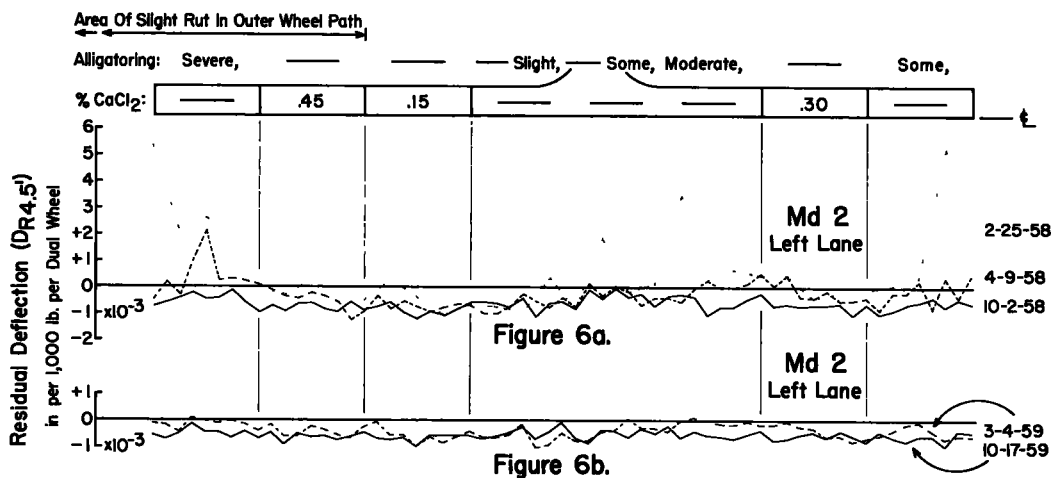
Characteristics of the profiles collected over a 2-yr period following construction have been plotted against the observed performance of the road. Figure 3a and b indicate a fair association of failures with excessive magnitudes of flection for data collected Feb 25, 1958. Practically all of the data collected April 9, 1958 and Oct. 2, 1958 in Figure 3a and all of the data in the central and right (not extreme right) portions of Figure 3b indicate that there is much to be desired to be able to claim that there is a distinct effect of frost action or seasonal effect on the flection properties of this road. The data collected in 1959, Figures 3c and d, do point to a seasonal effect on the flections.

In Figures 4a, b, c and d, the conventional measures of deflection indicate a questionable correlation with the fine alligator cracking. Except for extreme values, most of the values of deflection for both years bracket each other to the extent that the seasonal effect on deflections can hardly be distinguished. At the extreme left in Figure 4b, the first two point of the Feb. 25 data are highly positive, the next two points are very negative, and, the last point is nearly 0. The erratic values are results of getting profiles such as profiles A and B in Figure 2.

Comparing Figures 4a, b, c and d with their corresponding Figures 5a, b, c and d, the measures of rebound deflection indicate a better expected general behavior for deflections of a road as a result of frost action. The correlation with the failures however is apparently not improved. The erratic values in Figures 4a and b are displaced to their proper place; that is, they are all negative in Figures 5a and b. Figures 5c and d represent the road behavior after the project had gone through its second winter of seasoning; therefore, it is believed that the rebound deflections represent what might be expected as a result of frost action in the future.

In a general comparison of Figures 3a, b, c and d, 5a, b, c and d and in part Figures 4a, b, c and d, there seems to be a remarkable repeatability of measurements on most individual test points, groups of test points and also in a consideration of all the test points in individual lanes from year to year. This repeatability is best for data collected in the autumn of both 1958 and 1959 where, for example in the middle of Figures 5a and c, there is a localized area of highly negative rebound deflections, to the left of which is an area of much lower negative rebound deflections and so on. These similarities do not appear to be accidental.

Since neither of the measures of flection, deflection nor rebound deflection ( $D_R 4.5$  ft) contributed enough to good correlation with the observed failures, there remains the evaluation of any other unusual characteristics of the profile shapes such as shown



in Figure 2. Only a very few profiles show evidences of a distinct traveling rolling wave as is indicated by the extreme profiles A and B in Figure 2; however, those that have the distinct rolling wave seem to be associated particularly with severe alligatoring and raveling. The profiles were also examined for evidence of a significant bias that may have been caused by the zone of influence. It was concluded that the zone of influence was negligible and would not account for the failures either.

The several comparisons already made have left only fair to questionable correlations with the observed failures. The remaining measure to be evaluated is residual deflection ( $D_R 4.5 \text{ ft}$ ). These measures are plotted in Figures 6a and b for the left lane and Figures 6c and d for the right lane. These data would indicate almost perfect correlation would exist with all of the alligator cracking if all positive values of residual deflections ( $D_R 4.5 \text{ ft}$ ) of +0.001 in. or greater per 1,000 lb per dual wheel were considered as the cause of failure. In the middle of Figure 6a, the alligatoring classified as "slight" does not have a corresponding positive residual deflection ( $D_R 4.5 \text{ ft}$ ) of 0.001 or greater. This is because no test was made at this point on this date.

Though the residual deflection ( $D_R 4.5 \text{ ft}$ ) correlated very well with the observed failures, it appears that the weaker correlations of measures of excess flexion, excess rebound deflection, and very likely the existence of the traveling rolling wave, may have contributed to the severity of the alligator cracking type of failure.

In all figures representing the left lane there is a notation of a type of failure "Area of Slight Rut in Outer Wheel Path." The rut is hardly perceptible in the 0.45 percent calcium chloride section; however, it is one-half inch or more deep in parts of the adjoining control section in the same lane. From the preceding figures, the 0.45 percent calcium chloride section consistently showed high flexions and high rebound deflections, and, the adjoining control section showed a similar behavior except for the Feb. 25 data. It is believed that the ordinary fine alligator cracking in some of the control sections permitted moisture to enter the base materials so as to aggravate an already unsatisfactory condition to the extent that the condition developed into a more severe one.

Because of the high year round flexions and rebound deflections found generally in the left lane and the sensitivity of the road to frost action in some areas, it is speculated that when the existing double seal is covered with a higher type of bituminous surface some failures are expected to occur.

Figures 7a and b show frequency distribution plots of the seasonal effect on the flexions over the 2-yr period following construction. The distribution in Figure 7b indicates a 30 to 50 percent shift in the distribution. Similar plots could be made of the rebound deflection data showing a similar behavior but perhaps with only about a 30 percent shift.

### SOIL-LIME-FLY ASH PROJECT

Up to this point in the report, the measures of stability were made on what may be considered the minimum in bituminous pavement design and rigidity of a base. The remainder of the report will be devoted to the soil-lime-fly ash project on Maryland Route 33, constructed August 12, 1958, where the type of stability of the base should be regarded as semi-rigid. This project is very small but is considered valuable.

Figure 8 shows the layout of the test sections. Due to heavy rains, a high water table, poor drainage, an elevation of 18 ft above sea level, and, primarily to the uncertainty of the effects that calcium chloride would have on the stability of the base, the size of the treated sections was held to a minimum. Limiting the size was considered mandatory so that if the treated sections should fail and have to be replaced, the cost would not be prohibitive.

The only quantity of calcium chloride intended to be used was 0.5 percent by total weight of the soil-lime-fly ash mixture. It would be placed in a 100-ft section in one lane, and at the end of that, in a similar 100-ft section in the other lane. Inadvertently, the quantities of calcium chloride added were increased shortly after the treated section was being placed. This induced discrepancy later proved to be an asset.

All of the materials — soil, lime, fly ash, water and calcium chloride (if added) —

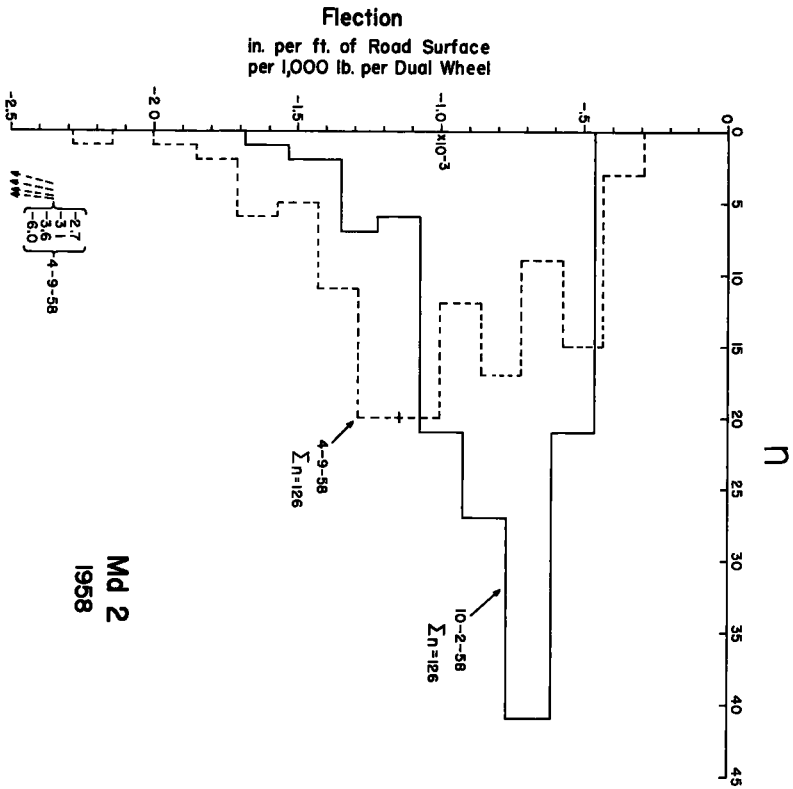


Figure 7a.

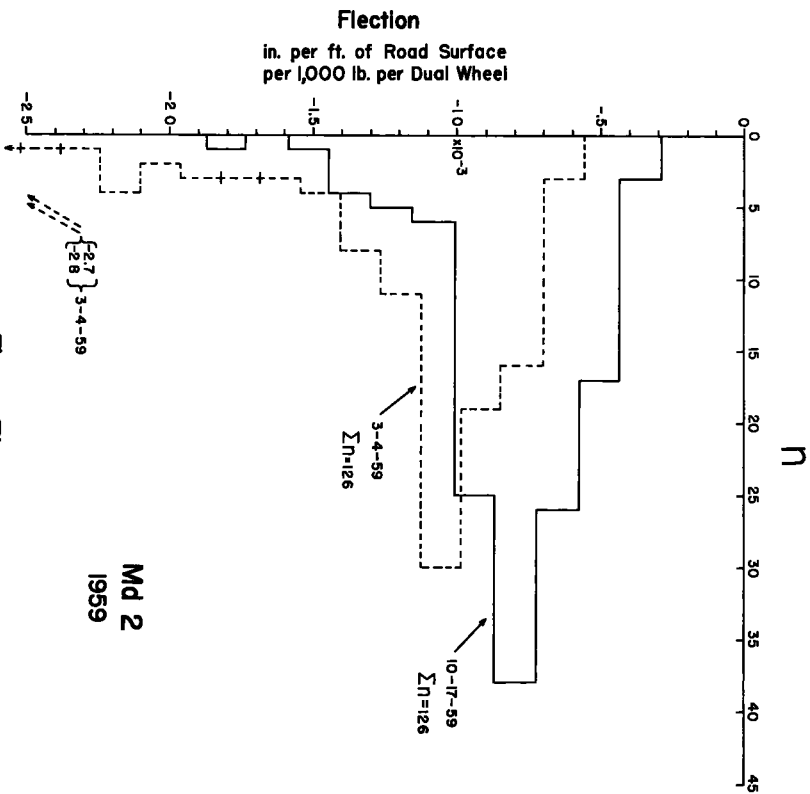
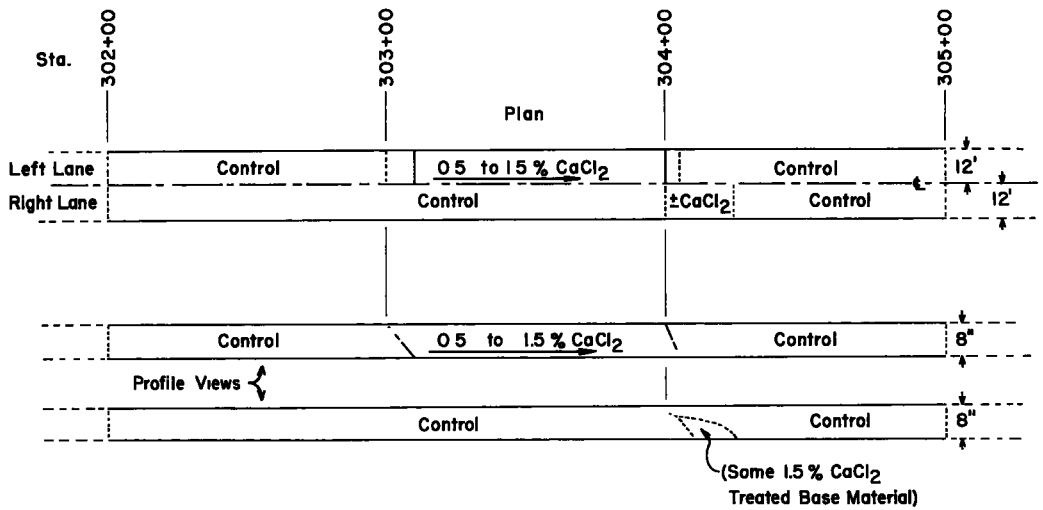


Figure 7b.



Soil-Lime-Fly Ash

Md 33

Figure 8.

were pug mill plant mixed. Spreading and primary compaction was by a bulldozer, and final surface compaction was by a rubber-tired roller. At the end of the day of construction, an estimated 2-in. rain fell. The surface of the stabilized base was shaped seven days after construction with first seal applied on the eighth day. The second seal was applied forty days after construction. The test section was opened to traffic immediately after application of each seal coat.

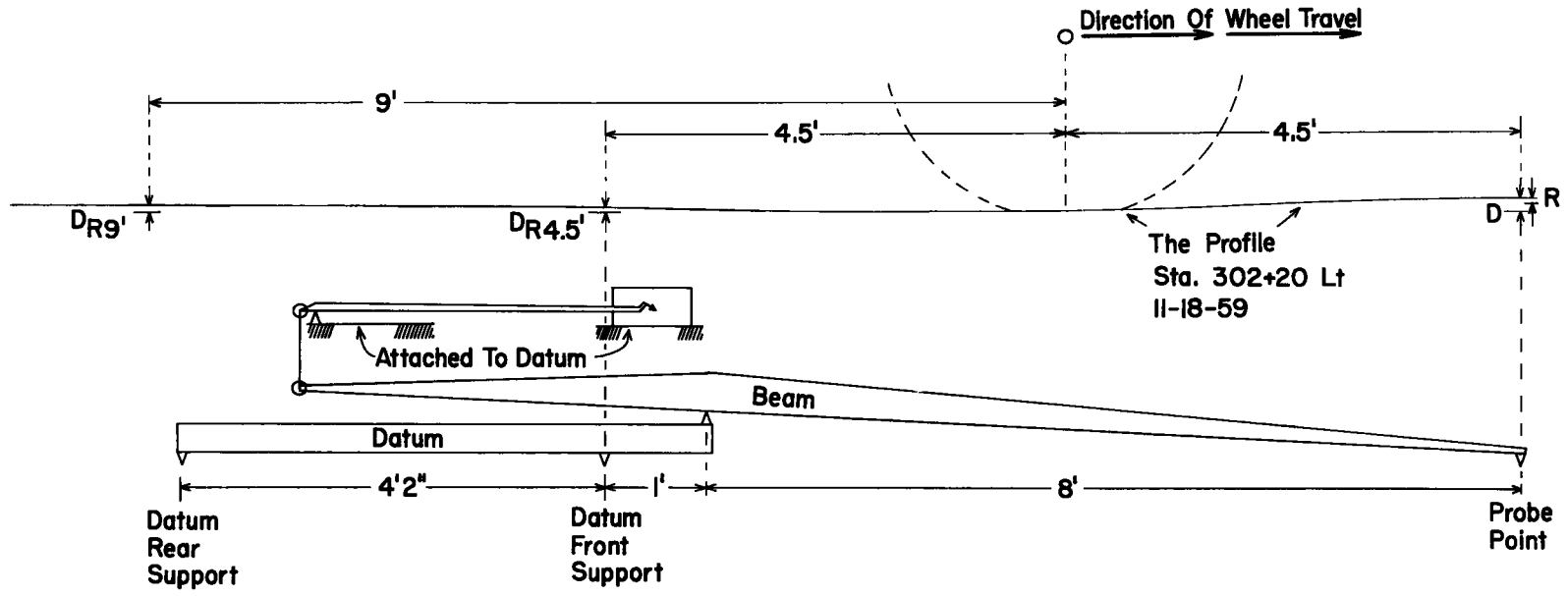
As a result of the unfavorable conditions that existed, including the rains that followed, there was practically no hope that the project would produce any information; however, the project was not abandoned.

In the course of collecting deflection profiles, it was found that shortly after construction (2 weeks), some of the test points and areas showed appreciable deflections as well as flections, while others showed considerable rigidity. With the passage of time sufficient pozzalanic reaction had taken place to stabilize the soil to an extent that the base in most areas exhibited a high degree of rigidity.

#### PROFILE SHAPE AND ZONE OF INFLUENCE

In the upper part of Figure 9 is a tracing of a recently obtained typical deflection profile. An outline of a rear dual wheel is superimposed on the profile 4.5 ft from the probe end. Below is a schematic of a Benkelman beam proportioned lengthwise to match the length of the road surface represented by the profile and positioned so as to indicate its relationship to the truck rear dual wheel and the virtual image of the profile. The shape of this profile is very much flatter than any of the profiles in Figure 2. Also, the beam is in the zone of significant influence.

The significance of the zone of influence can be demonstrated: referring to Figure 9, assuming that the road is uniform in the test area, with a base of rigidity such as this, the profile is variably biased because parts of the beam datum as well as the probe (the beam as customarily used in the past) are within the zone of significant influence. The influence as measured from this profile is 0.00225 in. ( $\pm 0.00025$  at best) on the road surface. (It is not claimed that all measurements concerning deflection profiles are accurate to  $\pm 0.00025$  in. of the road surface. The end accuracy is governed by the quality of the recorded profile, and, as yet is not determinable to better than  $\pm 0.00025$  in.).



D Deflection

$D_{R4.5'}$  } Rebound Deflections Taken From Points On Profile Where  
 $D_{R9'}$  } Wheels Are 4.5 Or 9 Feet Respectively Beyond Probe Point

R Residual Deflection (From  $D_{R9'}$  Point)

Md 33

Figure 9.

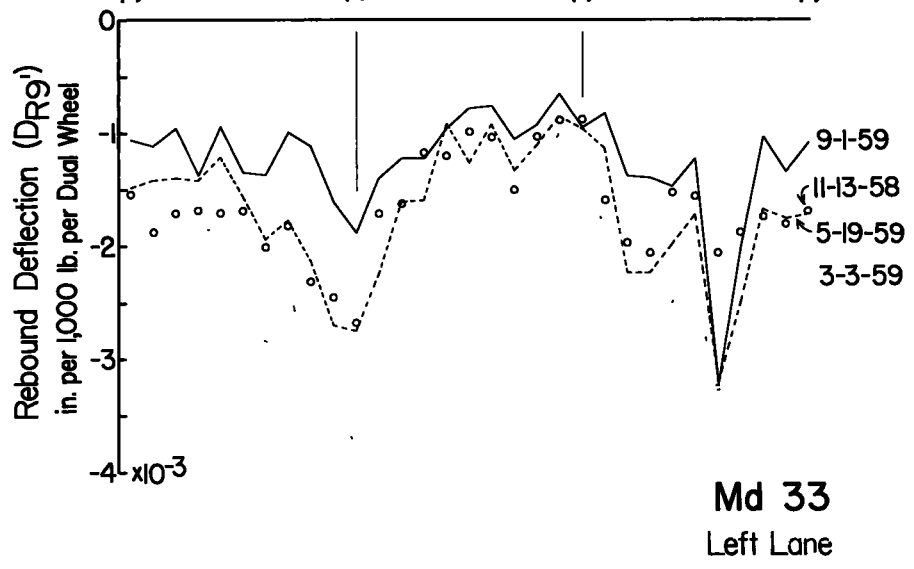
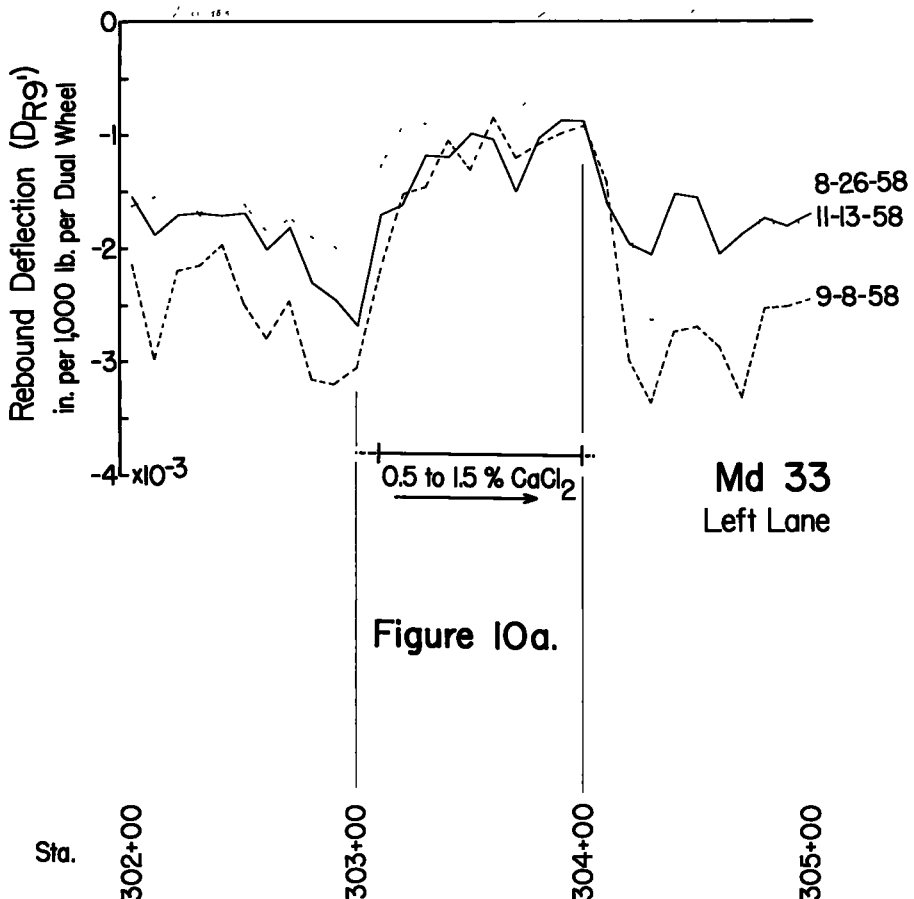
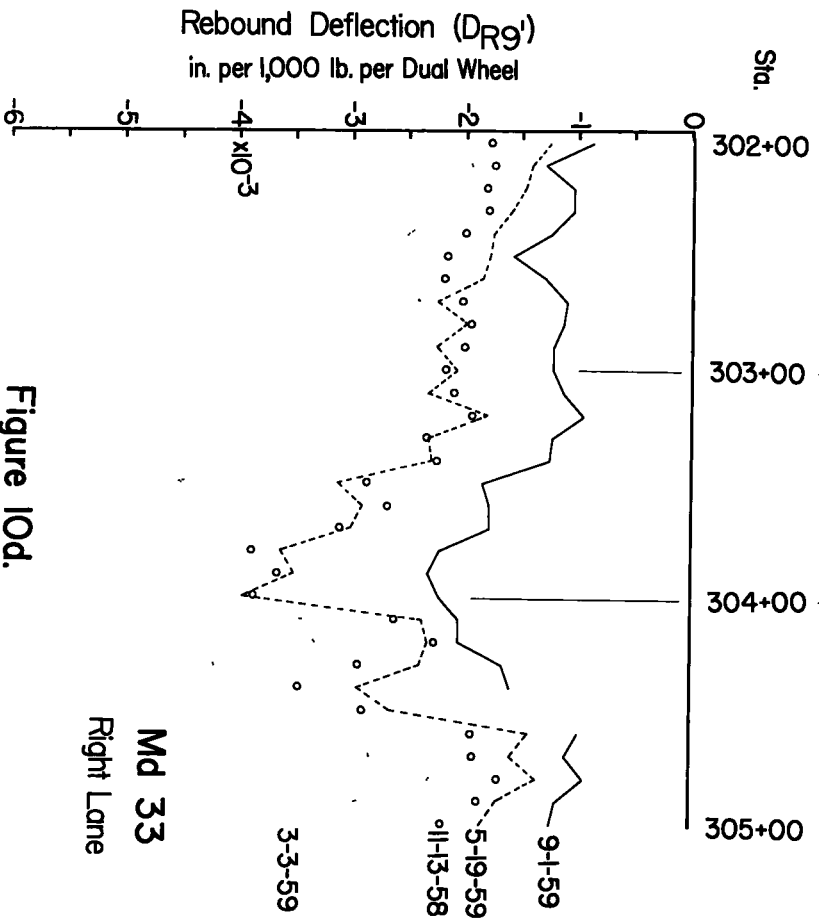
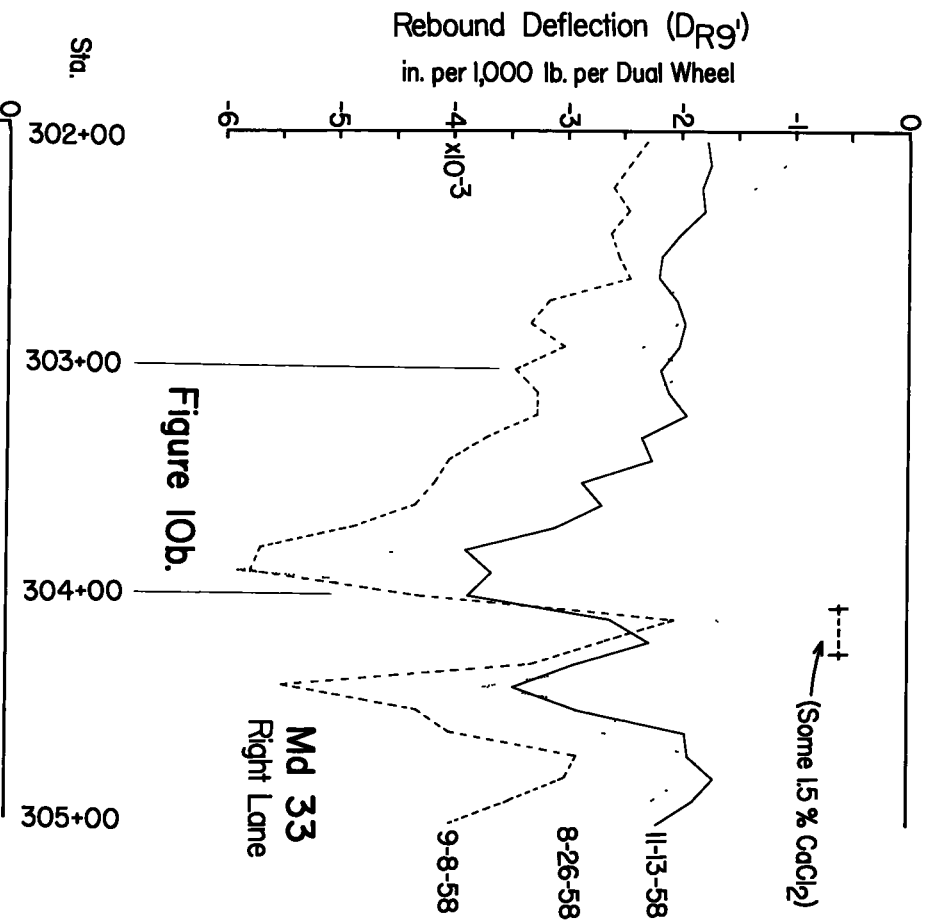
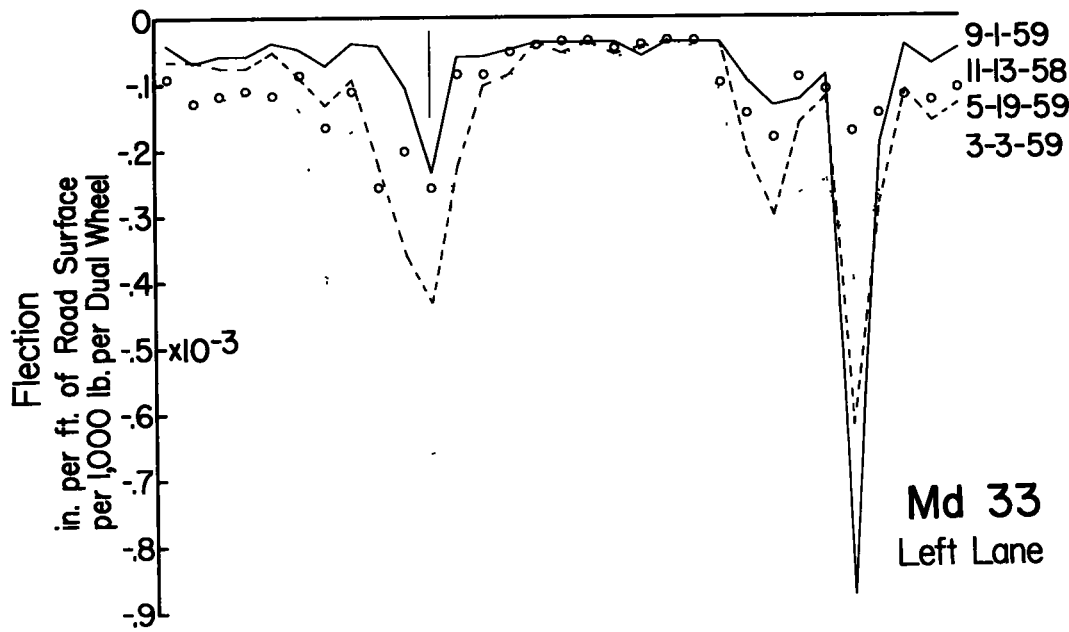
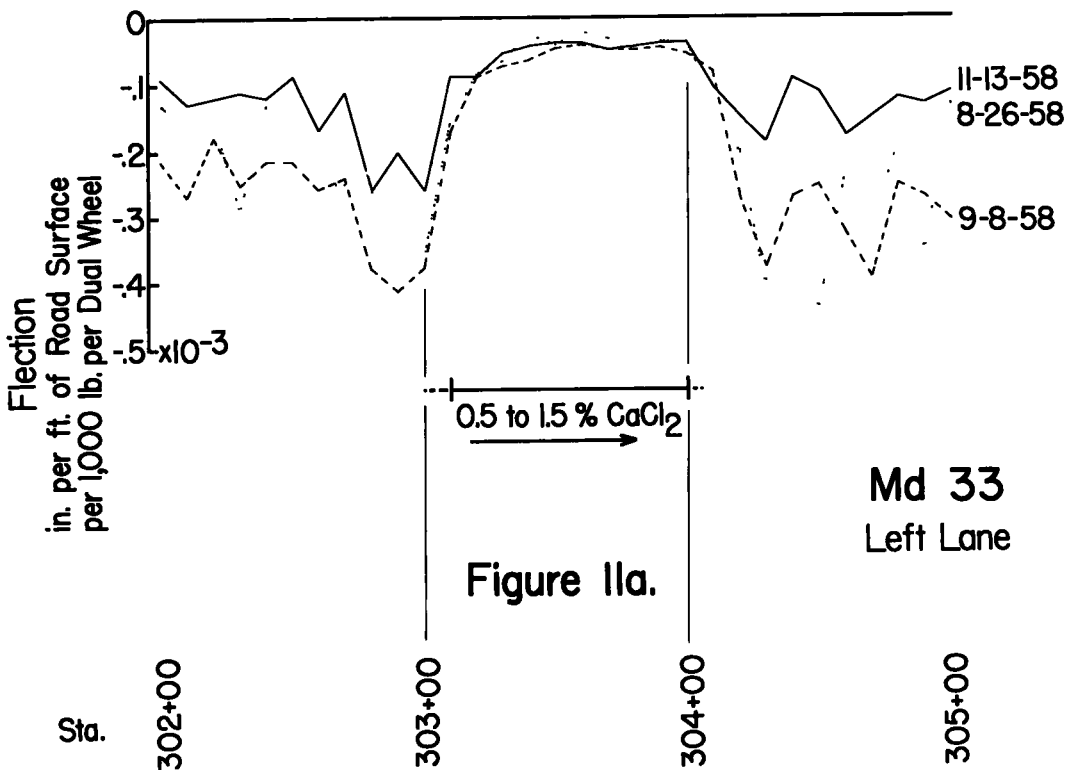


Figure 10c.







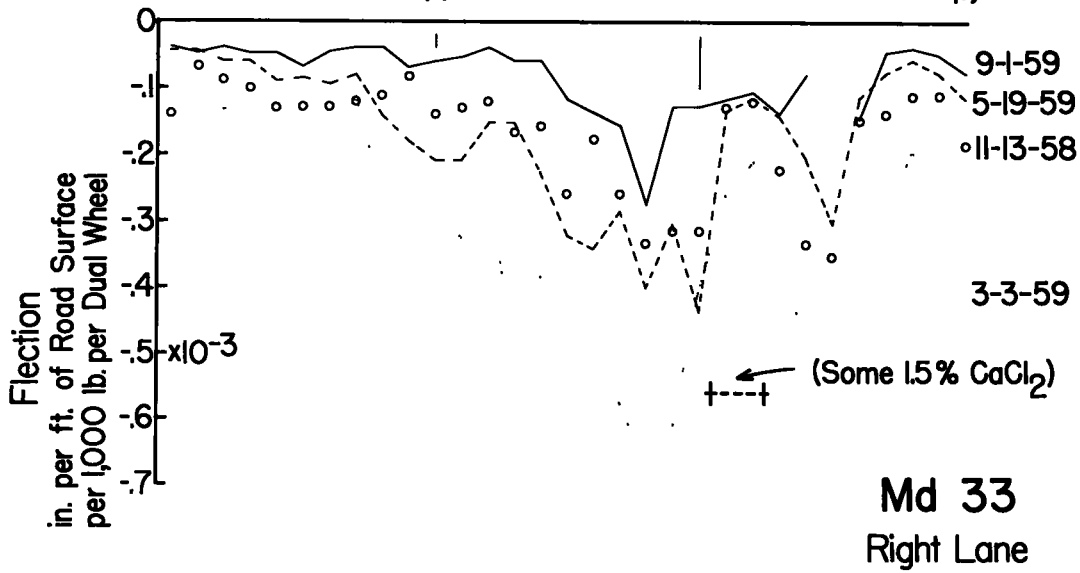
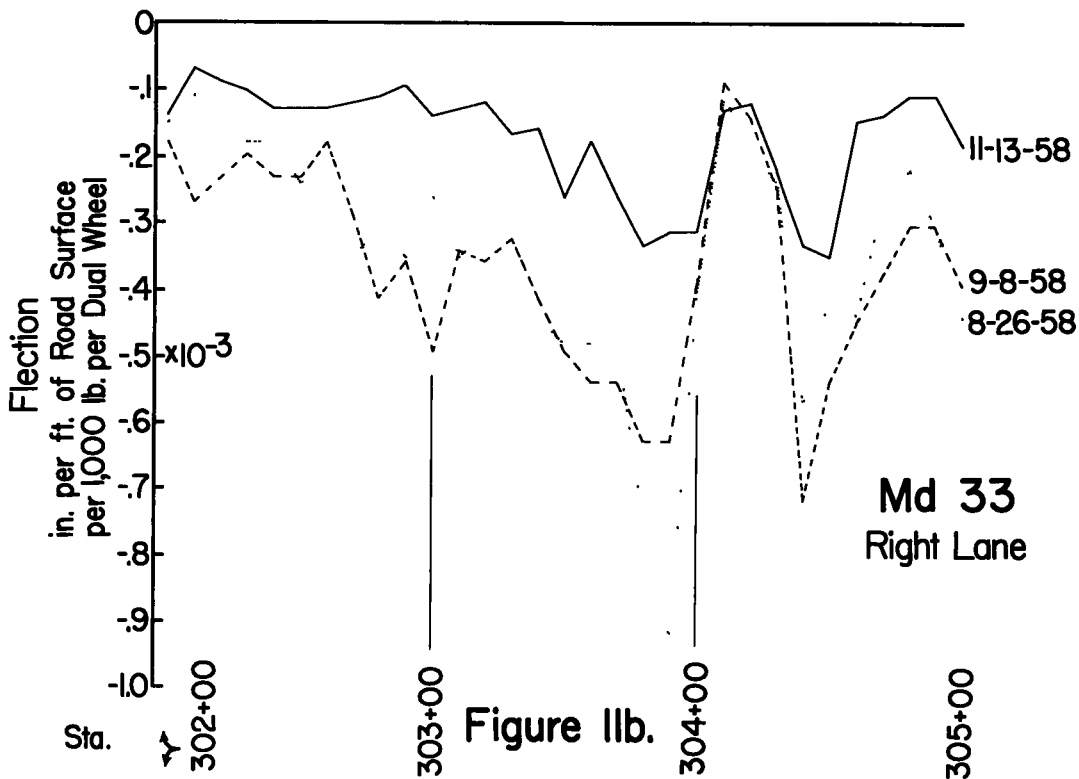


Figure 11d.

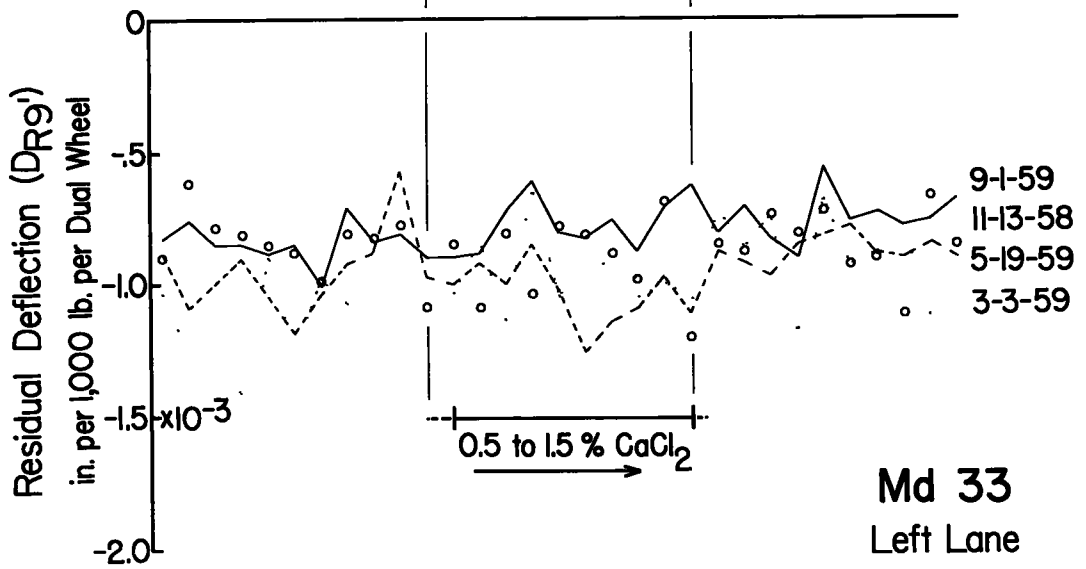
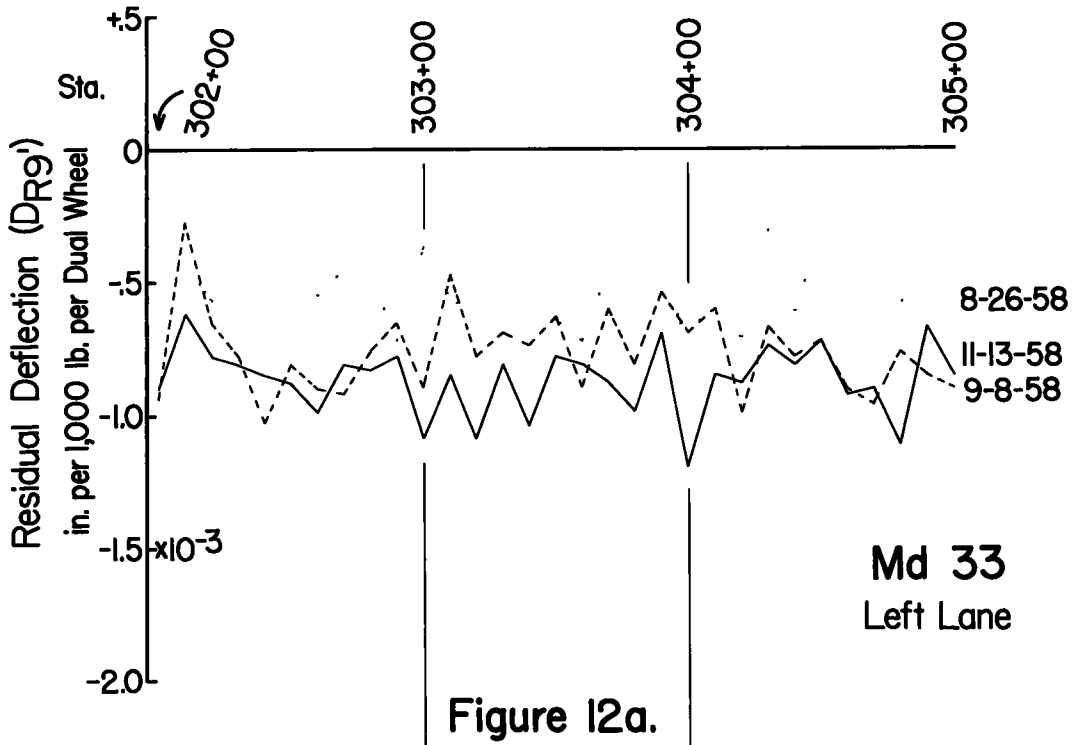


Figure 12c.

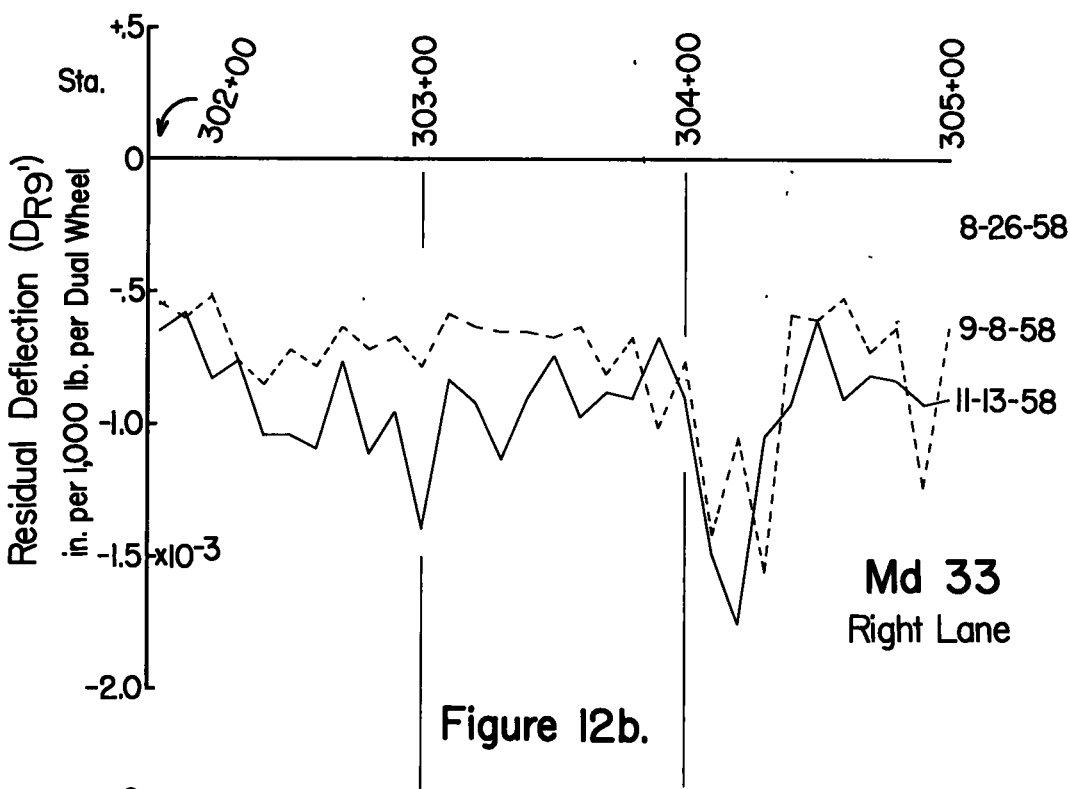


Figure 12b.

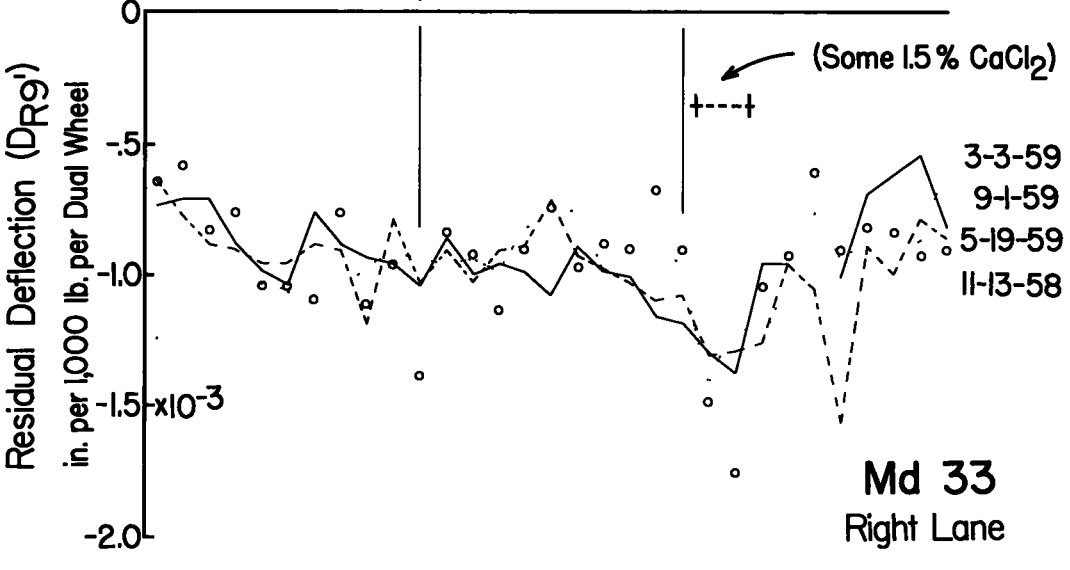
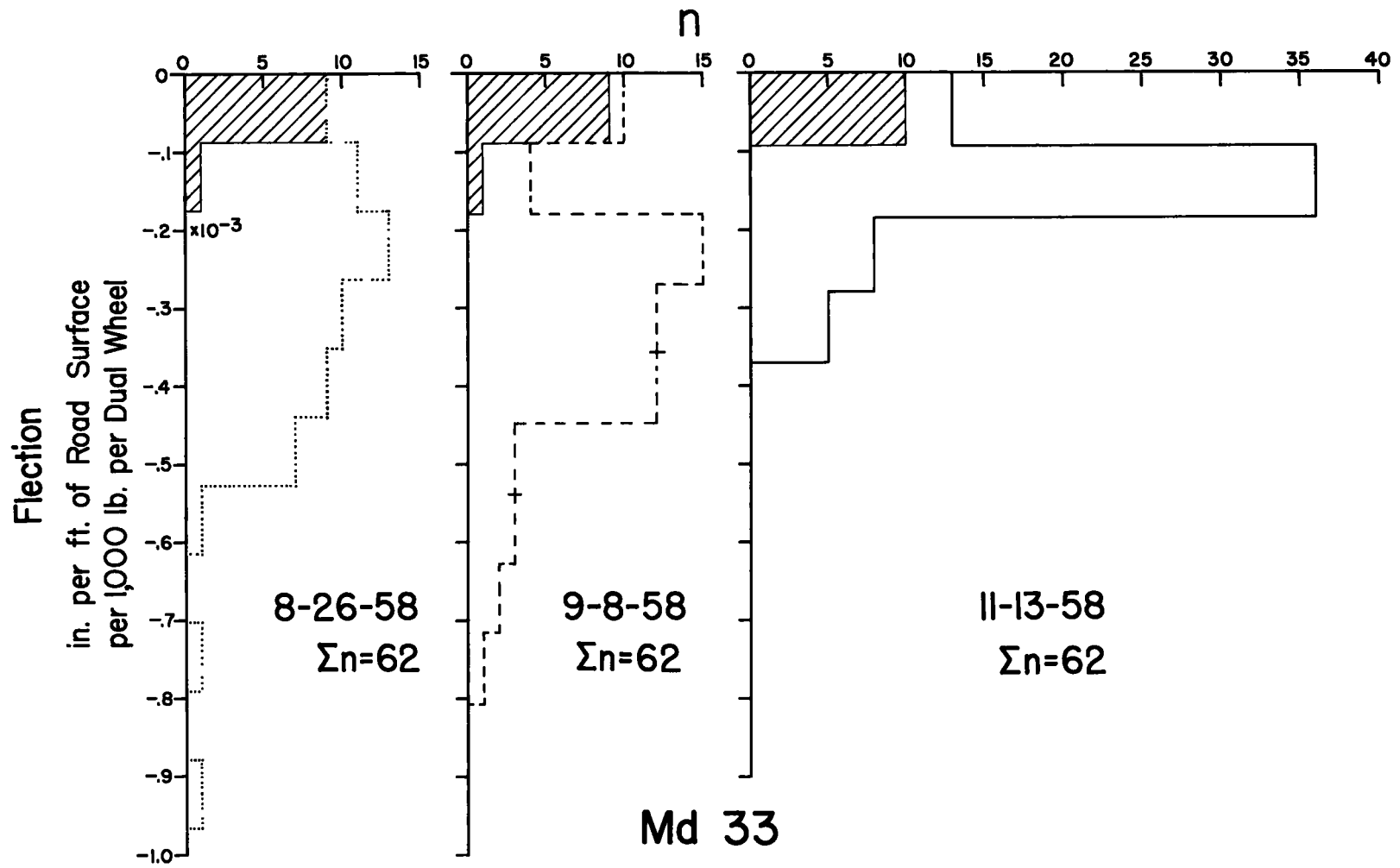
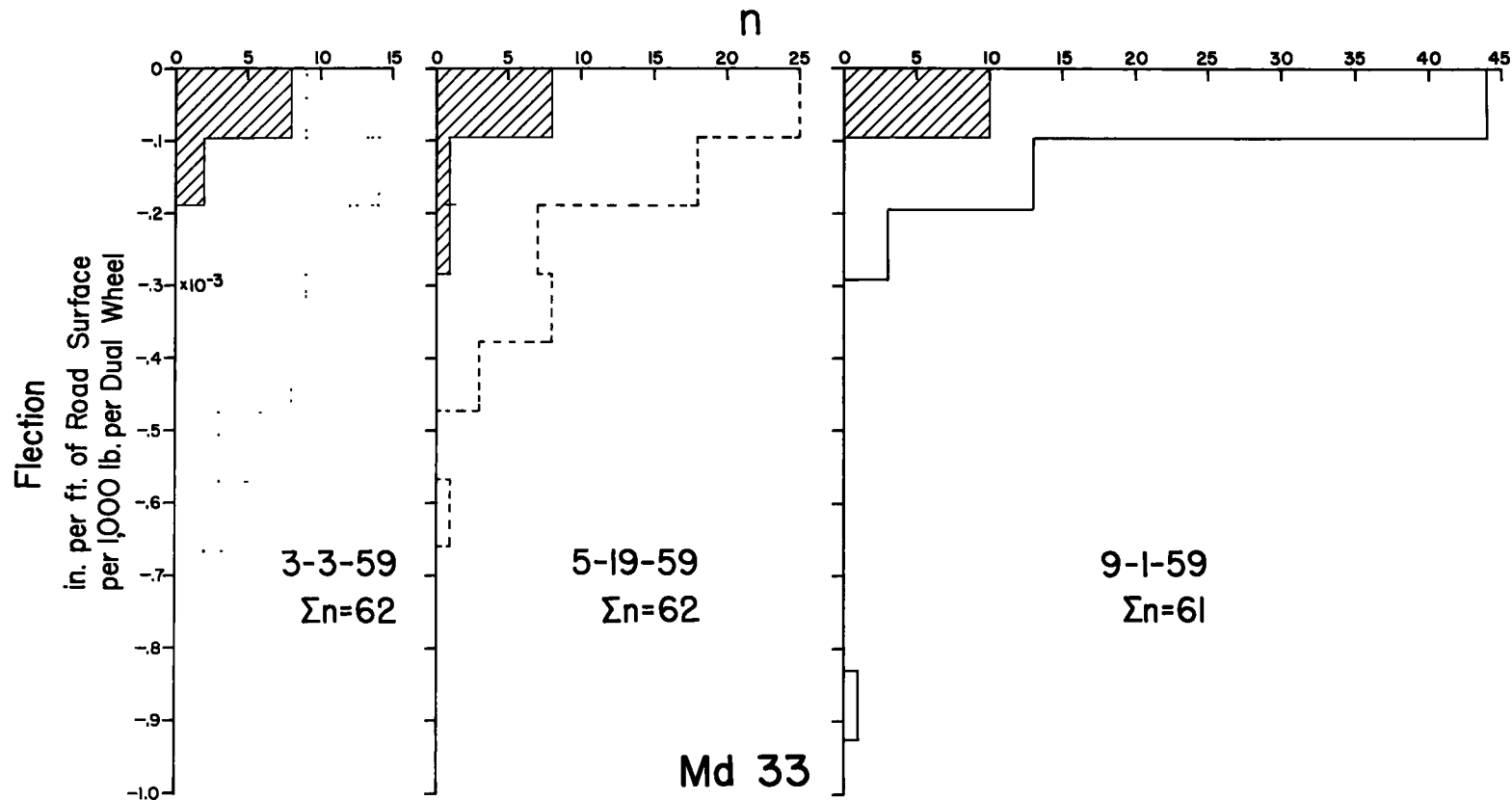


Figure 12d.



Md 33  
Figure 13a.



Because of the distances between the datum supports and the probe point (for this particular beam), the zone of influence amplification factor is 3.160. The bias added to the profile at the probe end then becomes  $-0.00711 \pm 0.0008$  in. on the road surface. Conventionally the total maximum deflection (D) would be  $-0.01750$  in. The rebound deflection at ( $D_R$  4.5 ft) would be  $-0.00650$  in. and at ( $D_R$  9 ft) would be  $-0.00875$  in. These last rebound deflection values are obtained by assuming the elevation of the probe point is zero and outside of the zone of influence when the rear dual wheels are 4.5 or 9 ft, respectively, ahead of the probe point. If deflection (D) is taken to be equal to  $-0.01750$  in. and the zone of influence bias is taken to be  $+0.00711$  in., a more accurate deflection (D) would be on the order of  $-0.0104$  in. which is not too different from the rebound deflection ( $D_R$  9 ft) value of  $-0.00875$  in. (Similar data shown in figures are reduced to inches per 1,000 lb per dual wheel.)

CHARACTERISTICS OF THE PROFILES

Figure 10a and b are plots of rebound deflections ( $D_R$  9 ft) the first of which were obtained Aug. 26, 1958 or 14 days following construction, the second group Sept. 8, 1958 or 27 days after construction, the third group just before cold weather set in. A particular pattern of rebound deflections exists and is repeatable with respect to time. The reversal of the magnitude of these rebound deflections appears to be significant; however, for the present it remains without an explanation.

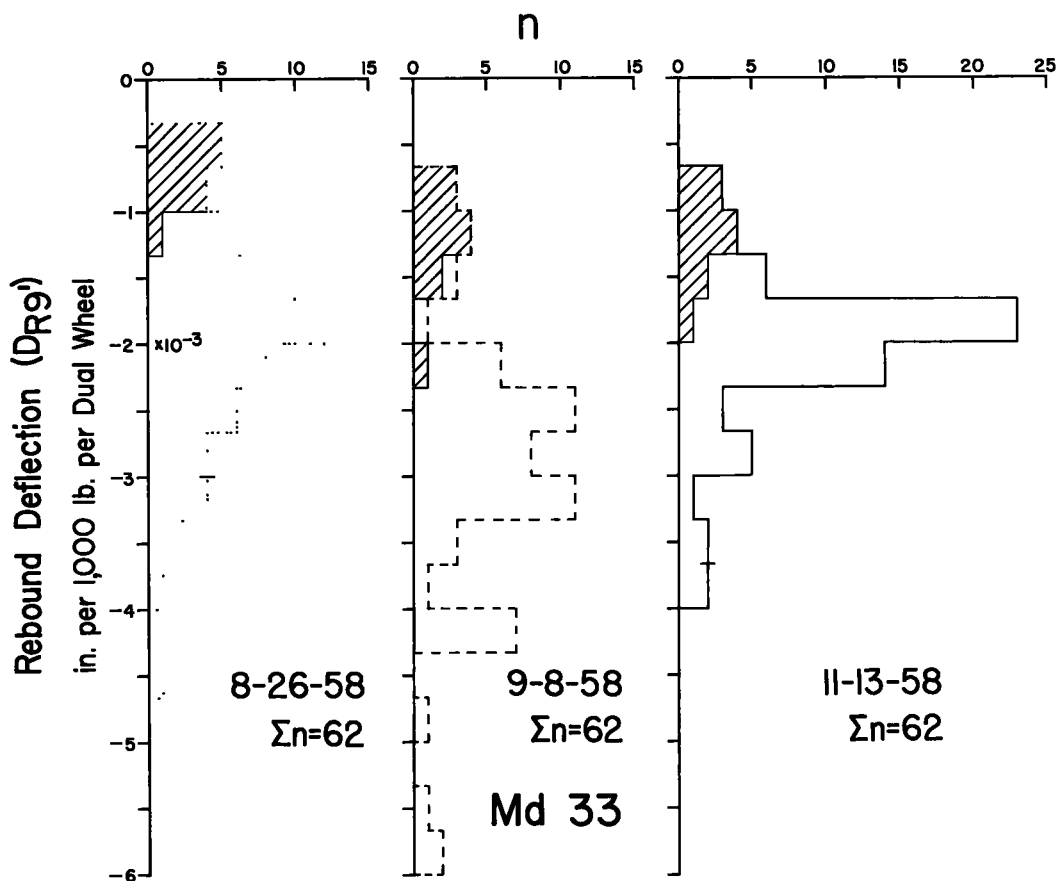


Figure 14a.

Figures 10c and d are plots of the rebound deflections as found in 1959. For comparison, the autumn 1958 values are plotted as circles. It is evident that frost action caused appreciable increases in the rebound deflections but recovered around the middle of May. By September 1 one of the test points seem to be reaching a general ultimate value of about  $-0.001$  in. per 1,000 lb per dual wheel.

Just as measures of deflection of a relatively rigid base are biased by the zone of influence, it is assumed that the measures of flexing of the road base or pavement are likewise biased and believed to be biased in direction of exaggerating the flection. Until the device is available that will correct the profiles for the zone of influence, the values for flection will continue to be used as they have been in the past, that is, making the assumption that the profile is still a virtual image of the road surface deflection. Used as such, Figures 11a and b show some rather erratic behavior of the flections at early ages except for the section containing the calcium chloride. Values for flection of less than  $-0.04 \times 10^{-3}$  in. per ft per 1,000 lb are not generally determined, mainly because of the quality of some of the profiles and also because of the limits of the template used to determine the flection from the profiles. It is entirely possible that a number of values for flection plotted to equal  $-0.04 \times 10^{-3}$  could be much less. With respect to the measure of flection it appears that the approximately 1.5 percent calcium chloride treated material reached its ultimate values within the first 14 days of curing, whereas the 0.5 percent calcium chloride area seems to have lagged behind. A number of the

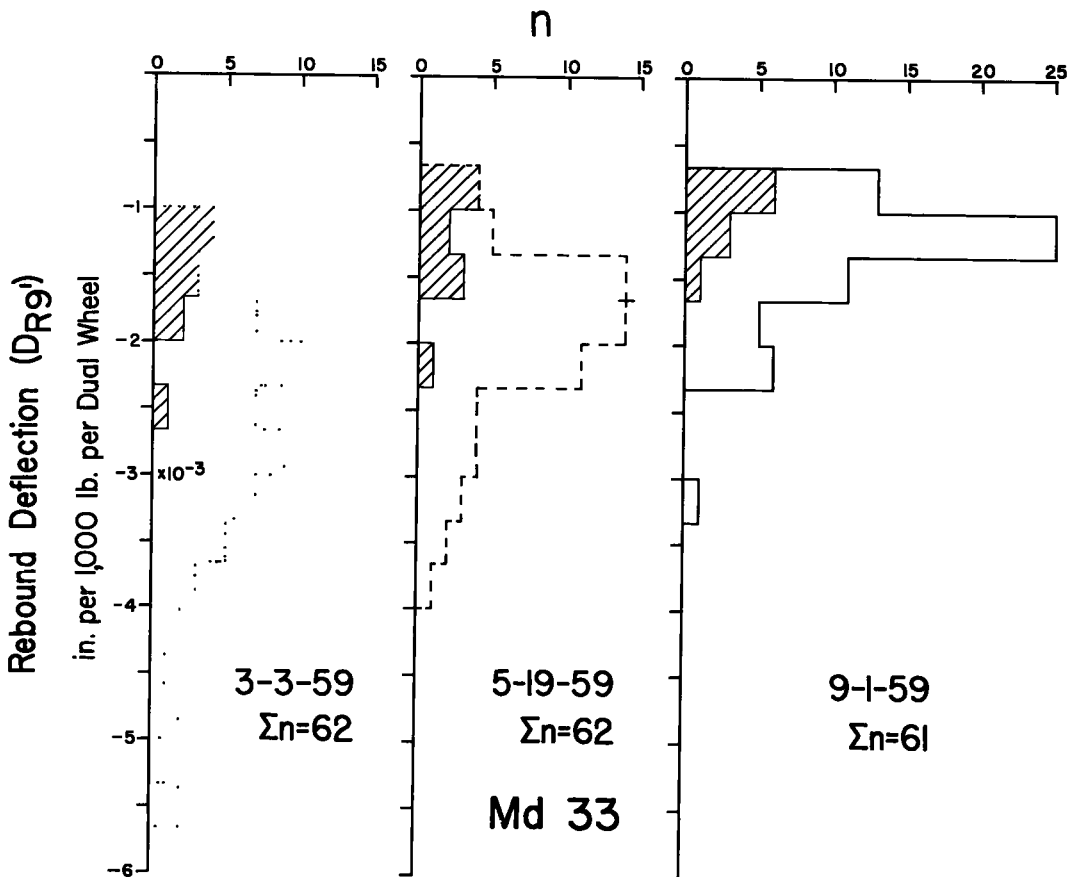


Figure 14b.



areas not containing calcium chloride still have a long way to go to get a flexion value of  $-0.04 \times 10^{-3}$  in. per ft per 1,000 lb (Fig. 11c and d). Figures 11c and d also show the effect of frost action on the flexions and the recoveries later in the spring and summer. Indications are that the calcium chloride section was not particularly affected by the frost action. Interesting also is the behavior of the test point at Station 304+60 Rt (Fig. 11c) for which there is yet no explanation. It is difficult to dismiss the reversal in behavior of the flexion measures as accidental.

In evaluating the effect of base rigidity on the measures of the residual deflection ( $D_R$  9 ft), Figures 12a, b, c and d and Table 2 suggest that eventually all values will be negative and will level off at a value on the order of  $-0.001$  in. or somewhat less per 1,000 lb per dual wheel.

In Figure 12b there is evidence of what probably is slight extrusion upward of the base materials between the dual wheels at Station 303+80 Rt and also probably at 304+40 Rt.

In Figure 12a, and more so in Figure 12b, indications are that the residual deflections ( $D_R$  9 ft) become more negative with respect to time. This is believed attributable essentially to the seasoning of the subbase and only in a very small part to rigidity. Rigidity is all but ruled out as being a major contributing factor to the increase in negativity of the residual deflections except where rigidity is practically nonexistent. This conclusion is based primarily on Figures 12a and c where if it were not known that calcium chloride has been incorporated in that particular lane, the plots of residual deflection certainly would not suggest where it might be incorporated, whereas, Figures 10a and c, and 11a and c would.

In Figures 12b and d at Stations 304+10 and 304+20 where some 1.5 percent calcium chloride treated base materials were placed as a part of the lower level of the 8-in. lift, the residual deflections do not conform to their general behaviors elsewhere on the project. Neither the exact cause nor the significance of this irregularity is known.

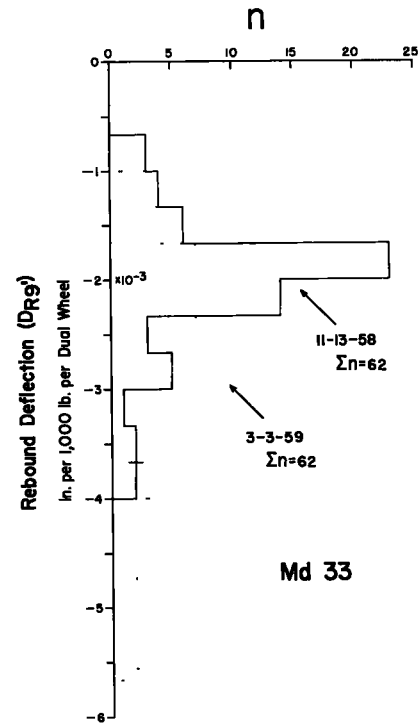


Figure 15a.

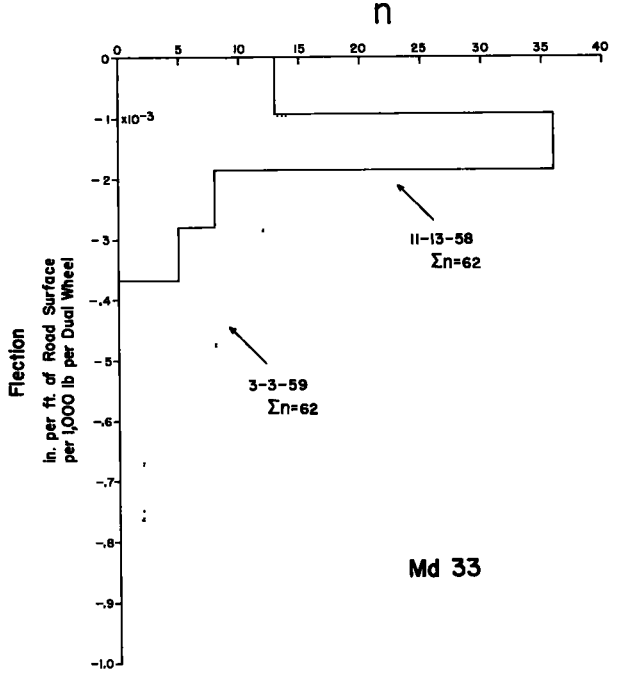


Figure 15b.

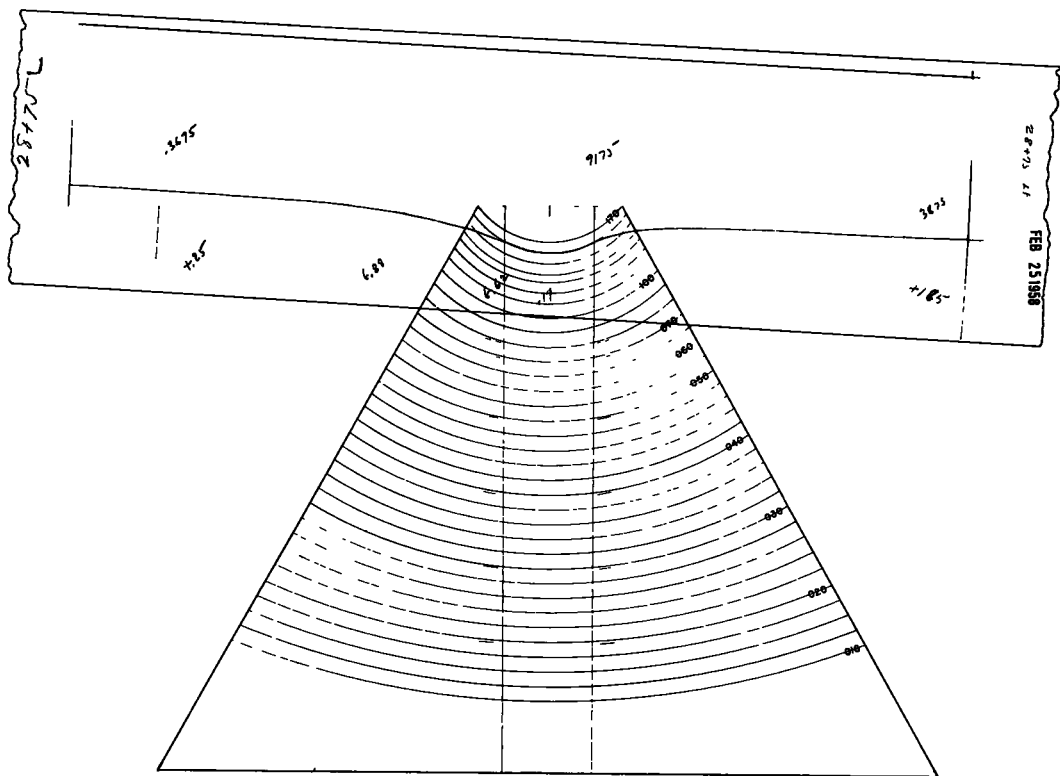
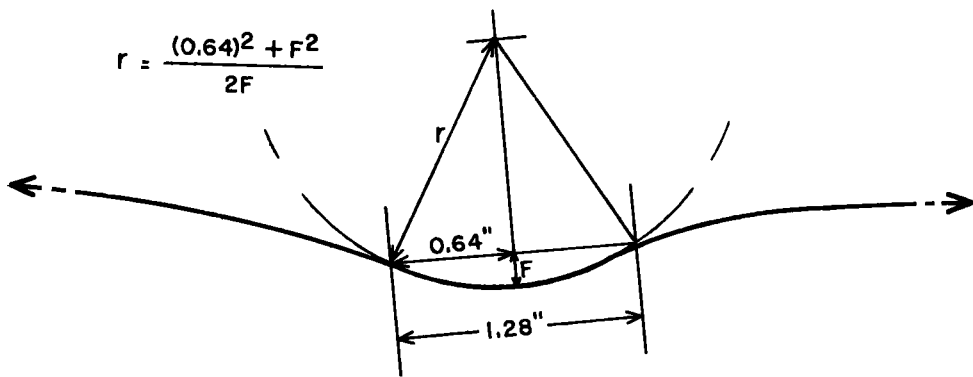


Figure 16a.



PROFILE TO ROAD DIMENSIONAL RELATIONSHIPS

LONGITUDINALLY: 1.28" ON PROFILE EQUALS 1' ON ROAD

VERTICALLY: 0.010" ON PROFILE EQUALS 0.001" ON ROAD

Figure 16b.

TABLE 2  
AVERAGES AND ESTIMATES OF THE STANDARD DEVIATION IN INCHES<sup>a</sup>

Date of Test	Left Lane		Right Lane	
	Average	Est. of Std. Dev.	Average	Est. of Std. Dev.
8-26-58	$-0.52 \times 10^{-3}$	$0.14 \times 10^{-3}$	$-0.44 \times 10^{-3}$	$0.16 \times 10^{-3}$
9-8-58	-0.76	0.16	-0.76	0.21
11-13-58	-0.88	0.13	-0.94	0.21
3-3-59	-0.99	0.17	-0.98	0.17
5-19-59	-0.95	0.13	-0.97	0.17
9-1-59	-0.79	0.10	-0.93	0.17

<sup>a</sup>As determined from average deviation  $\times 1.25$  for the residual deflections ( $D_{R 9 ft}$ ) for Figures 12a, 12b, 12c, and 12d.

The frequency distributions in Figures 13a and b show quite dramatically the great changes in the measures of flection that take place on gain in stability to where the base can be regarded as semi-rigid. The hatched portion in each histogram indicates the position with respect to measures of flections that the calcium chloride treated materials occupy. Similarly, frequency distribution plots of the rebound deflections ( $D_{R 9 ft}$ ) in Figures 14a and b show considerable changes with respect to time but perhaps not so spectacularly as the similar plots of flection. The hatched areas in the histograms in Figures 14a and b again represent the values for the calcium chloride treated materials.

The frequency distributions in Figures 15a and b need no more explanation other than that they were plotted to show the detrimental effects of frost action on measured rebound deflections and flections.

### Appendix

The longitudinal flexing of the road surface is actually determined by matching arcs of precomputed radii with the curvature of the deflection profile. The arcs of precomputed radii are inscribed on sheets of transparent plastic thereby making overlay type of scales and are referred to as "flection scales".

Figure 16a is a photograph of a flection scale with its 0.160-in. arc overlaying the highly flexed portion of the profile. This scale has arcs of 34 differing radii covering a range for total flections of 0.170 to 0.016 in. per ft of pavement. Another scale has 29 arcs extending the range to 0.004 in. per ft of pavement. As can be seen from the flatness of the profile in Figure 9, there is a need for a flection scale that will extend the range of arcs to a straight line.

Figure 16b is an actual-size tracing of the highly flexed portion of a deflection profile. Superimposed on the profile in the highly flexed zone is an arc of the radius  $r$  subtending a cord of a length equivalent to a linear foot of road surface. In the Figure 16b,

$F$  = total flection in inches as exaggerated by a ratio of 10:1, and

$r$  = radius for an arc that fits the curvature of the profile and subtends a cord 1.28 in. long (the 1.28-in. dimension is for the particular beam used in this study and not necessarily for any other beam).

As illustrated in Figure 16b,  $r$  is determined by solving for the length of the hypotenuse of a right triangle one leg of which is  $\frac{1}{2}$  of the cord length.

$$r^2 = (0.64)^2 + (r - F)^2 \quad \text{or} \quad r = \frac{(0.64)^2 + F^2}{2F}$$

**TABLE 3**  
**SUGGESTED F VALUES (in in.) FOR WHICH r SHOULD BE**  
**DETERMINED FOR FLECTION SCALE CONSTRUCTION**

Scale No. 1		Scale No. 2		Scale No. 3	
F	$\Delta F$	F	$\Delta F$	F	$\Delta F$
0.0100	—	0.100	—	0.70	—
—	0.0005	—	0.005	—	0.02
0.0050	—	0.050	—	0.30	—
—	0.0002	—	0.002	—	0.01
0.0000	—	0.020	—	0.10	—
—	—	0.010	0.001	—	—

It has been found preferable to keep F independent so that select values could be assigned to it. Values of F selected for an experimental model of a flection scale ranged from 0.02 to 0.50 in. in uniform increments of 0.01 in. This scale is no longer being used except for flections in excess of 0.170 in. per ft. The values of F as well as the increments of F selected for the two new flection scales used most often make it possible to get greater accuracy in flection determinations. The extended range of the new scales permits estimation of flection of much higher type bituminous pavements.

As a result of some experience in using the flection scales, the profiles and the quality of the profiles would suggest that three scales each with a different range should be constructed. Table 3 is given as a possible guide for the scale construction suggesting the ranges, increments and inferred best expected accuracy from the increments.

### *Discussion*

**W.H. CAMPEN and L.G. ERICKSON, Omaha Testing Laboratories** — There appears to be confusion in regard to the making of deflection tests on the surface of pavements.

In making the test, engineers are attempting to measure the amount of downward deformation by applying load on rigid steel plates or by applying load through inflated tires. By either method, the total downward movement or total deflection is measured when the load is applied. After the load is removed, the permanent deformation (residual deflection) is then measured. The difference between the total deflection and permanent deformation is denoted as elastic deflection.

If the test is made in this manner, the permanent deformation (residual deflection) cannot exceed the total deflection. Nevertheless, engineers making this type of test sometimes report, as in this paper, residual deflections which are greater than total deflection.

The question now is: under what conditions can residual deflection be greater than the total deflection? The only way such conditions can be visualized is to assume that the test is being made with a beam (as in this case) and that as the wheels approach the indicator point of the beam, the general area goes down while the surface is squeezed up between the spacing of the dual tire. After the load is removed, the general area rebounds and the surface which has been squeezed upward rises higher than its original position. When this condition prevails neither the total, elastic or residual deflection is correct because the squeezed portion of the surface has vitiated these readings.

The above reasoning leads to the conclusion that the beam test should not be used when the surface or the layered system beneath is so weak as to cause squeezing to take place between the dual tires. The plate method, however, could be used for determinations under these conditions.

It should be pointed out that the main purpose for making deflection tests is to determine the amount of elastic deflection which the surface can tolerate without the production of cracks. The maximum load which can be applied without the production

of cracks is then designated as the load carrying capacity of the pavement. It is evident, therefore, that deflection tests are not made for the purpose of detecting obviously weak flexible pavements. Rather they are made on good flexible pavements for the purpose of determining their load carrying capacities.

**CLOSURE, J. Eldridge Wood and William B. Greene**— It is extremely unfortunate that the complete paper could not be made available for distribution at or prior to the 39th Annual Meeting of the Highway Research Board. The preprint that was available was actually a brief on the paper in preparation.

The writers are entirely correct in recognizing that extrusion of the road surface can occur during the deflection determination. This extrusion does subtract from the conventional measure of deflection, however, it is felt very certain that the rebound deflections are only slightly affected by the extrusion except in extreme cases of extrusion. It was recognized that there were the biases and influences of the several movements, elastic and plastic, on the shapes of the deflection profiles and that more than one of the movements can be taking place at the same time and variably. These movements are sometimes additive on some measurements; sometimes they may cancel each other out or even show complete reversals of normal expected behaviors.

Because plate bearing determinations on either of these projects were not made it cannot be concluded that the plate bearing test would or would not have been more appropriate.

It seems that a misunderstanding developed concerning the purpose behind these research projects. In the case of the granular soil base project that is showing questionable performance, and in similar new construction it is preferred to delay the placing of a high type bituminous surface for approximately a year. On this project, deflection profiles were obtained throughout the placing of the granular soil base materials and frequently thereafter until stability was attained and winter had set in. The profiles obtained prior to and during attainment of stability provided clues as to how the profiles might be interpreted should stability be lost regardless of the cause. It was the intent to obtain deflection profiles from both these experimental projects under both the most favorable as well as the most unfavorable conditions during which the road would be open to unlimited natural traffic. Although it was attempted, there is no guarantee that these roads have been tested for deflections, etc., under either their most unstable or stable condition periods of the year. These projects will continue to be followed in the future though the evaluation may change.

# A Study of Occurrence of Potholes and Washboards on Soil-Aggregate Roads

EUGENE Y. HUANG, Associate Professor of Civil Engineering, University of Illinois

This report presents the results of a study aimed at determining some of the circumstances associated with the occurrence of potholes and washboards on soil-aggregate roads. The study consisted of a statistical analysis of the qualitative data obtained from a road condition survey involving road surfaces of the coarse-graded aggregate type composed of mineral aggregate such as gravel or crushed stone and some binder material. Results of the study indicate that the occurrence of potholes and washboards was definitely associated with the volume of traffic, the type of surface material, and the drainage condition of the road surface. Although the findings are admittedly limited to the types and conditions of the roads studied, it is hoped that the data may be of value in further understanding the formation of potholes and washboards.

●THE OCCURRENCE of potholes and washboards has long been a serious problem in soil-aggregate surfaces. These formations, however, have not been fully explained. Potholes on road surfaces are irregularly occurring, well-defined holes consisting of fairly deep cavities up to about 5 in. (Fig. 1). Washboards are transverse or nearly transverse waves on road surfaces, generally about 1 to 1½ in. in amplitude and spaced about 2 or 3 ft apart (Fig. 2), regardless of the nature of material in which they occur. (Although typical formations of potholes and washboards are readily distinguishable, there are, however, numerous possibilities of transitional forms between these two typical forms.) Both potholes and washboards are conducive to surface impact and vibration, which contribute in a great measure to the rapid deterioration of the road surface as well as the vehicle itself. Driving on such roads, if not hazardous, is extremely unpleasant.

The purpose of this study was to determine some of the circumstances that were closely associated with the development and condition of potholes and washboards on soil-aggregate roads. The road surfaces studied were of the coarse-graded aggregate type composed of mineral aggregates such as gravel or crushed stone and some binder material. No chemical additives had been introduced into these surface courses, nor had they been surface-treated with bituminous material.

## DEVELOPMENT AND SCOPE OF THE STUDY

The study was an outgrowth of a condition survey conducted during the summer of 1956 on soil-aggregate roads in Champaign County, Illinois, which was aimed at obtaining general information regarding road conditions in this area so that sections for special studies might be selected. This survey consisted of a field study of 749 separated road sections. A visual inspection was made of each road section noting the condition of the surface, type of the surface material, condition of the shoulders and side ditches, and other important features.

The methods and procedures employed in this area with respect to general maintenance of these roads were more or less uniform. Therefore, the effect on road conditions due to maintenance was considered insignificant and has not been included in this study.



Figure 1. Typical potholed road surface.



Figure 2. Road surface with washboard formations developed.

Traffic volume of these roads was obtained later from traffic volume maps supplied by the Illinois Division of Highways Bureau of Research and Planning. Information regarding subgrade soil conditions was obtained from engineering soil maps prepared by the Illinois Cooperative Project IHR-12, Soil Exploration and Mapping.

When the large amount of data was reviewed, it was noted that some of the recorded road characteristics appeared to be interrelated. In an effort to ascertain these relationships, the present investigation was instituted.

This study consists of a statistical analysis of the qualitative data obtained in the road condition survey. However, only those road sections which produced adequate data were used in the analysis. Consequently, the total number of road sections used in individual analyses may be less than 749.

This report is devoted mainly to the presentation and analysis of the data as well as the interpretation and discussion of the findings. To explain the method of analysis, the principle and procedure of chi-square test are briefly described in the early part of the report.

## METHOD OF ANALYSIS

### General

The association or independence of any two characteristics of road sections observed in the field was studied by a technique known as chi-square test. The procedure of this test is outlined as follows:

1. A frequency table is constructed classifying the observed data according to both characteristics.
2. Under the hypothesis that the two characteristics are independent, the frequencies which can be expected to occur in the various cells of the table are computed.
3. These expected cell frequencies are compared with the corresponding frequencies which are actually observed.
4. Decision is made on the basis of the criterion of chi-square distribution as to whether the discrepancies between the two sets of frequencies are large enough to permit a rejection of the hypothesis. If the hypothesis can be rejected, a significant relationship between these two characteristics is indicated.

### Observed Frequencies

The first step toward studying the relationship between two characteristics of road sections by the chi-square test consists of constructing a frequency table where the observed data are classified according to both characteristics and arranged in compartments or cells. Such a table is commonly called a contingency table. Let two characteristics A and B be considered, each of which permits a certain number of alternative descriptions. The characteristic A, for instance, may be subdivided into  $n$  categories, while the characteristic B may be subdivided into  $m$  categories. The contingency table, therefore, will consist of  $mn$  cells. The data can be set out in the form of  $m$  rows and  $n$  columns as shown in Table 1.

The observed cell frequencies are represented by the letters  $O_{11}, O_{12}, \dots$  and in general by the symbol  $O_{ij}$ , where the subscript  $i$  refers to the row and the subscript  $j$  refers to the column to which the respective cell belongs. For example, the frequency (or the number of observed road sections) which belongs to the cell which is contained in the second row and fourth column is written as  $O_{24}$ . The totals at the end of the rows and at the bottom of the columns give the total frequencies of subclasses. The total of the first row is denoted as  $O_{1.}$  and in general, the total of the  $i$ th row is given as  $O_{i.}$ . Similarly, the total of the  $j$ th column is written as  $O_{.j}$ . The whole number of observations,  $N$ , is given as the grand total in the bottom right-hand corner of Table 1.

### Expected Frequencies

The expected cell frequencies are calculated on the basis of the following assumptions:



TABLE 1  
TYPICAL CONTINGENCY TABLE

		Characteristic A							Row Totals	
		A <sub>1</sub>	A <sub>2</sub>	A <sub>3</sub>	A <sub>4</sub>	—	—	A <sub>n-1</sub>		A <sub>n</sub>
Characteristic B	B <sub>1</sub>	0 <sub>11</sub> (e <sub>11</sub> )	0 <sub>12</sub> (e <sub>12</sub> )	0 <sub>13</sub> (e <sub>13</sub> )	0 <sub>14</sub> (e <sub>14</sub> )	—	—	0 <sub>1(n-1)</sub> (e <sub>1(n-1)</sub> )	0 <sub>1n</sub> (e <sub>1n</sub> )	0 <sub>1.</sub>
	B <sub>2</sub>	0 <sub>21</sub> (e <sub>21</sub> )	0 <sub>22</sub> (e <sub>22</sub> )	0 <sub>23</sub> (e <sub>23</sub> )	0 <sub>24</sub> (e <sub>24</sub> )	—	—	0 <sub>2(n-1)</sub> (e <sub>2(n-1)</sub> )	0 <sub>2n</sub> (e <sub>2n</sub> )	0 <sub>2.</sub>
	B <sub>3</sub>	0 <sub>31</sub> (e <sub>31</sub> )	0 <sub>32</sub> (e <sub>32</sub> )	0 <sub>33</sub> (e <sub>33</sub> )	0 <sub>34</sub> (e <sub>34</sub> )	—	—	0 <sub>3(n-1)</sub> (e <sub>3(n-1)</sub> )	0 <sub>3n</sub> (e <sub>3n</sub> )	0 <sub>3.</sub>
	B <sub>4</sub>	0 <sub>41</sub> (e <sub>41</sub> )	0 <sub>42</sub> (e <sub>42</sub> )	0 <sub>43</sub> (e <sub>43</sub> )	0 <sub>44</sub> (e <sub>44</sub> )	—	—	0 <sub>4(n-1)</sub> (e <sub>4(n-1)</sub> )	0 <sub>4n</sub> (e <sub>4n</sub> )	0 <sub>4.</sub>
	—	—	—	—	—	—	—	—	—	—
	B <sub>m-1</sub>	0 <sub>(m-1)1</sub> (e <sub>(m-1)1</sub> )	0 <sub>(m-1)2</sub> (e <sub>(m-1)2</sub> )	0 <sub>(m-1)3</sub> (e <sub>(m-1)3</sub> )	0 <sub>(m-1)4</sub> (e <sub>(m-1)4</sub> )	—	—	0 <sub>(m-1)(n-1)</sub> (e <sub>(m-1)(n-1)</sub> )	0 <sub>(m-1)n</sub> (e <sub>(m-1)n</sub> )	0 <sub>(m-1).</sub>
	B <sub>m</sub>	0 <sub>m1</sub> (e <sub>m1</sub> )	0 <sub>m2</sub> (e <sub>m2</sub> )	0 <sub>m3</sub> (e <sub>m3</sub> )	0 <sub>m4</sub> (e <sub>m4</sub> )	—	—	0 <sub>m(n-1)</sub> (e <sub>m(n-1)</sub> )	0 <sub>mn</sub> (e <sub>mn</sub> )	0 <sub>m.</sub>
Column Totals	0 <sub>.1</sub>	0 <sub>.2</sub>	0 <sub>.3</sub>	0 <sub>.4</sub>	—	—	0 <sub>.(n-1)</sub>	0 <sub>.n</sub>	N	

1. The two characteristics A and B are independent, that is, the probability that an individual road section falls in any particular row in the table is unaffected by the particular column to which it belongs, and
2. The totals of rows and columns are fixed.

In accordance with the above assumptions, it is evident that the proportion (or fraction) of observations which fits description A<sub>j</sub> is given by the ratio 0<sub>.j</sub>/N, whereas the proportion which fits description B<sub>i</sub> is given by the ratio 0<sub>i.</sub>/N. Consequently the proportion of the total number of observations which, under the above assumptions, should fit both of these descriptions is given by the product

$$\left(\frac{0_{.j}}{N}\right)\left(\frac{0_{i.}}{N}\right) = \frac{(0_{.j})(0_{i.})}{N^2}$$

The above value is then multiplied by the whole number of observations N to obtain the expected frequency of the cell which belongs to the ith row and the jth column. The expected frequency e<sub>ij</sub> which corresponds to the observed frequency 0<sub>ij</sub>, is given by the formula

$$e_{ij} = \frac{(0_{.j})(0_{i.})}{N}$$

In other words, the expected frequency of a given cell is obtained by simply multiplying the totals of the respective row and column and dividing this product by the whole number of observations.

The expected cell frequencies, represented by the letters e<sub>11</sub>, e<sub>12</sub>, . . . and in general by the symbol e<sub>ij</sub>, are then entered in the cells of the frequency table to compare with the observed frequencies. The expected frequencies are written in parenthesis below the observed frequencies. It may be noted that the totals of the rows and columns of the expected frequencies must equal those of the original table.

## The Chi-Square

The comparison of the observed and the expected cell frequencies of the contingency table is made by the use of the statistic chi-square ( $\chi^2$ ) which was originated by Karl Pearson in 1900 as a criterion for testing hypotheses about frequency distributions (8). It is defined by the equation

$$\chi^2 = \sum_{i=1}^k \frac{(o_{ij} - e_{ij})^2}{e_{ij}}$$

where  $k$  is the number of pairs of frequencies to be compared ( $k=mn$ ),  $o_{ij}$  and  $e_{ij}$  the observed and the expected frequencies, respectively, and

$$\sum o_{ij} = \sum e_{ij}$$

Chi-square is a discrete variable. It is always a positive number, since each denominator is a positive number and each term in the numerator is a square. If the observed frequencies should agree completely with the expected, chi-square would be zero. Chi-square increases in size as the observed frequencies depart more and more from the expected. It is evident that the statistic chi-square affords a measure of the correspondence between fact and theory.

In using chi-square to test the compatibility of a set of observed and expected frequencies, there is always a question as to what extent the discrepancies between expectation and observation can be regarded as significant. Since chi-square is calculated on the basis of a set of data, it is subject to the chance variation which is displayed by its sampling distribution. If the use of chi-square is to be satisfactory, it is necessary to distinguish significant values from those which may have arisen by sample fluctuations. This distinction is based on the probability of getting a particular value of chi-square from a set of data chosen at random, or, in other words, on the sample distribution of chi-square given as follows:

$$f(\chi^2) = \frac{1}{2^{v/2} \Gamma \frac{v}{2}} (\chi^2)^{\frac{v}{2} - 1} e^{-\frac{\chi^2}{2}} \quad 0 \leq \chi^2 < \infty$$

in which the parameter  $v$  is called the number of degree of freedom and  $\Gamma$  represents the gamma function. (It may be noted that the equation is an approximation to the distribution function of chi-square obtained by using multinomial distribution and making certain approximations. The exact distribution of chi-square has not yet been established.)

Since  $f(\chi^2)$  depends only upon the parameter  $v$ , there will correspond a  $\chi^2$  curve to each value of  $v$ . When a random set of data is taken and chi-square is calculated, the probability of getting a value of chi-square as great as, or greater than, this particular value is as follows:

$$P = \int_{\chi^2}^{\infty} f(\chi^2) d(\chi^2).$$

Thus for any value of  $v$ , there corresponds a certain value of  $P$  to every value of chi-square. As chi-square is increased from zero to infinity,  $P$  diminishes from 1 to zero. Equally, there corresponds a certain value of chi-square to any value of  $P$  in this range. In applying the chi-square distribution to a test of significance, it is customary to select a critical  $P$  value to provide an approximate line of demarcation between acceptance and rejection of the significance of the observed deviations. If the computed chi-square is larger than the chi-square associated with the critical  $P$ , the

computed value is regarded significant, and the hypothesis that the two characteristics are unrelated must be rejected. Otherwise, the discrepancy between the observed and expected frequencies can be regarded as entirely due to chance, and no significant relationship can be proven to be of existence between the two characteristics. In this study the demarcation line has been drawn, as a matter of personal opinion, at  $P = 0.01$ . In order to evaluate critically the reliability of the conclusions drawn from the study, the actual values of  $P$ , expressed to three decimal places, are also presented in the individual analyses.

For the interpretation of the values of computed chi-square, use has been made of the tables prepared by Elderton (9). With the aid of these tables, the value of  $P$  for a computed chi-square and a given value of  $v$  has been determined.

#### Number of Degrees of Freedom

The number of degrees of freedom  $v$  in the equation of chi-square distribution is the number of cells for which expected frequency values can be assigned without restriction. In computing the expected frequencies the sum of the frequencies in each row is set up to equal the total observed cell frequencies in that row, and similarly for the columns. Since the frequencies in that row or column must add up to the total observed frequency in that row or column, one of the expected cell frequencies in a given row or column is defined by the knowledge of the total cell frequency and of the remaining cell frequencies in that row or column. Hence each row or column imposes a constraint, and for a contingency table of  $m$  rows and  $n$  columns the total number of constraints is  $m + n$ . However, as the sum of the frequencies in the rows equals the sum of those in the columns, and since these are not algebraically independent, one must be subtracted from  $m + n$  to give the actual total number of constraints. Thus the number of degrees of freedom for a contingency table of  $m$  rows and  $n$  columns is expressed by the equation

$$\begin{aligned} v &= mn - (m + n - 1) \\ &= (m - 1)(n - 1) \end{aligned}$$

#### Example — Condition of Crown vs Pothole Formation

To illustrate the method described above, the analysis of the relationship between the condition of crown and pothole formation is taken as an example. The numbers of

TABLE 2

#### CLASSIFICATION OF ROAD SECTIONS ACCORDING TO CONDITION OF CROWN AND POTHOLE FORMATION

Pothole Formation	Number of Sections Having Stated Pothole Formation for Crown Condition of:			Row Totals
	High Crown ( $\frac{1}{2}$ in. or more per ft)	Low Crown ( $\frac{1}{4}$ in. per ft)	No Crown	
No potholing	131 (94)	38 (57)	11 (29)	180
Slightly potholed	211 (220)	145 (132)	63 (67)	419
Severely potholed	51 (79)	54 (48)	45 (23)	150
Column totals	393	237	119	749

Note: Expected frequencies are shown in parentheses below the observed frequencies.

cases observed with regard to these two characteristics are represented by the figures not in parentheses in the cells in Table 2. The expected frequencies are calculated in the manner described in Section 7 and written in parentheses in the contingency table.

The chi-square is calculated on the basis of the observed and the expected frequencies by simply taking each cell in turn, squaring the difference between the observed and expected value, and dividing the result by the expected frequency. This is conveniently done by means of the following scheme:

The total of the last column in Table 3 gives the value of chi-square which is equal to 65.66. The contingency table has three rows and three columns, and hence the number of degrees of freedom

$$v = (3-1)(3-1) = 4$$

Entering these values in the chi-square table, a probability value less than 0.01 is indicated ( $P=0.000$ ). On this evidence, the hypothesis that the condition of crown and pothole formations are unrelated is rejected, and an inference that the two characteristics are associated is drawn.

To determine the sense of association the difference between observed and expected

TABLE 3  
COMPUTATION OF CHI-SQUARE  
(CONDITION OF CROWN VS POTHOLE FORMATION)

(1)	(2)	(3)	(4)	(5)
$O_{ij}$	$e_{ij}$	$O_{ij} - e_{ij}$	$(O_{ij} - e_{ij})^2$	$\frac{(O_{ij} - e_{ij})^2}{e_{ij}}$
131	94	37	1369	14.56
38	57	-19	361	6.33
11	29	-18	324	11.17
211	220	-9	81	0.37
145	132	13	169	1.28
63	67	-4	16	0.24
51	79	-28	784	9.92
54	48	6	36	0.75
45	23	22	484	21.04
				65.66

TABLE 4  
DIFFERENCE BETWEEN OBSERVED AND EXPECTED FREQUENCIES  
(CONDITION OF CROWN VS POTHOLE FORMATIONS)

Pothole Formation	Difference, for stated pothole formation, between number of sections observed and number of sections expected for crown condition of:		
	High Crown ( $\frac{1}{2}$ in. or more per ft)	Low Crown ( $\frac{1}{4}$ in. per ft)	No Crown
No potholing	37	-19	-18
Slightly potholed	-9	13	-4
Severely potholed	-28	6	22

frequencies ( $O_{ij} - e_{ij}$ ) in various cells are tabulated in Table 4. It is noted that on road sections with a high crown there were more cases of "no potholing" and less cases of "severely potholed" actually observed than expected. Contrarily, on road sections with no crown or a low crown there were more cases of severely potholed and less cases of no potholing actually observed than expected. It is evident that a soil-aggregate road surface with a high or an adequate crown would tend to have less pothole problems than those with no crown or a poor crown.

## POTHOLE FORMATION

### Road Characteristics Studied

In addition to the condition of crown, whose association with pothole formation is illustrated in the previous example, the road characteristics studied included type of surface material, width of road surface, condition of shoulders, condition of side ditches, and type of subgrade soil. The volume of traffic was also studied in connection with the formation of potholes. Not all these factors, however, were found significantly associated with pothole formations.

### Volume of Traffic vs Pothole Formation

The soil-aggregate roads investigated were located in rural areas providing adequate land service or access to every farm home not served by the primary or trunk line highways. The volume of traffic on these roads was rather light, most of which being below 100 vehicles per day. Table 5 shows observed and expected distributions of road sections according to their pothole condition and according to their average daily traffic.

From the data in Table 5 the chi-square computed is equal to 17.13. The contingency table has three rows and four columns, and hence the number of degrees of freedom is equal to 6. Entering these values in the chi-square table a probability value less than 0.01 is indicated ( $P=0.008$ ). It follows that the relationship between the volume of traffic and pothole formation is significant. The differences between observed and expected frequencies indicate that road sections with a smaller volume of traffic appear to have less pothole problems than those with a larger volume of traffic.

### Type of Surface Material vs Pothole Formation

Two major types of material, gravel and crushed stone, were used for road surfaces in the area investigated. All the gravel materials were pit-run, consisting of

TABLE 5  
CLASSIFICATION OF ROAD SECTIONS ACCORDING TO AVERAGE DAILY TRAFFIC  
AND POTHOLE FORMATION

Pothole Formation	Number of Sections Having Stated Pothole Formation for Average Daily Traffic of:				Row Totals
	0-24	25-49	50-74	75-99	
No potholing	64 (57)	50 (58)	18 (17)	7 (7)	139
Slightly potholed	131 (121)	115 (124)	34 (35)	16 (16)	296
Severely potholed	25 (42)	60 (43)	12 (12)	6 (6)	103
Column totals	220	225	64	29	538

Note: Expected frequencies are shown in parentheses below the observed frequencies.

natural mixtures of gravel, sand, silt, and clay, and hence with a sufficient amount of fine material. The crushed stone used in the area was obtained from various sources as crusher-run product. It in general lacked sufficient binder and fines and was highly segregated in its particle size because of an incorrect method of handling and placing the material on the road. The contingency table classifying investigated road sections with respect to surface material and pothole formation is shown in Table 6.

From the data in Table 6 the chi-square computed is 16.38; the number of degrees of freedom is 2. Entering these values in the chi-square table, a probability value less than 0.01 is obtained ( $P=0.000$ ). The result indicates that the relationship between the type of surface material and pothole formation is significant. The differences between observed and expected frequencies show that road sections surfaced with gravel appear to have less potholes than those surfaced with crushed stone.

**TABLE 6**  
**CLASSIFICATION OF ROAD SECTIONS ACCORDING TO TYPE OF SURFACE MATERIAL AND POTHOLE FORMATION**

Pothole Formation	Number of Sections Having Stated Pothole Formation for the Two Types of Surface Material:		Row Totals
	Gravel	Crushed Stone	
No potholing	81 (76)	62 (67)	143
Slightly potholed	169 (156)	127 (140)	296
Severely potholed	33 (51)	64 (46)	97
Column totals	283	253	536

Note: Expected frequencies are shown in parentheses below the observed frequencies.

**TABLE 7**  
**CLASSIFICATION OF ROAD SECTIONS ACCORDING TO WIDTH OF ROAD SURFACE AND POTHOLE FORMATION**

Pothole Formation	Number of Sections Having Stated Pothole Formation for Road Surface Width (ft) of:				Row Totals
	21-24	17-20	13-16	9-12	
No potholing	6 (14)	58 (61)	59 (52)	21 (17)	144
Slightly potholed	25 (28)	119 (119)	103 (101)	33 (32)	280
Severely potholed	21 (10)	47 (44)	28 (37)	6 (11)	102
Column totals	52	224	190	60	526

Note: Expected frequencies are shown in parentheses below the observed frequencies.

**Width of Road Surface vs Pothole Formation**

The road surfaces investigated were either two-lane or single-lane, with widths ranging from 9 to 24 ft. The contingency table giving observed and expected frequencies corresponding to width of road surface and pothole formation is shown in Table 7. The chi-square computed from the frequencies in Table 7 is 23.76. The number of degrees of freedom is 6. With 6 degrees of freedom, the computed chi-square shows a probability value less than 0.01 ( $P=0.001$ ). The result indicates a significant association between the width of road surface and pothole formation. Comparing the observed frequencies with the expected frequencies reveals that wider road surfaces were more severely potholed than narrower road surfaces.

TABLE 8

**CLASSIFICATION OF ROAD SECTIONS ACCORDING TO CONDITION OF SHOULDERS AND POTHOLE FORMATION**

Pothole Formation	Number of Sections Having Stated Pothole Formation for Shoulders in:			Row Totals
	Good Condition	Fair Condition	Poor Condition	
No potholing	44 (28)	22 (20)	96 (114)	162
Slightly potholed	48 (59)	39 (41)	252 (239)	339
Severely potholed	14 (19)	13 (13)	80 (75)	107
Column totals	106	74	428	608

Note: Expected frequencies are shown in parentheses below the observed frequencies.

TABLE 9

**CLASSIFICATION OF ROAD SECTIONS ACCORDING TO CONDITION OF SIDE DITCHES AND POTHOLE FORMATION**

Pothole Formation	Number of Sections Having Stated Pothole Formation for Side Ditches in:			Row Totals
	Good Condition	Fair Condition	Poor Condition	
No potholing	107 (95)	27 (30)	25 (34)	159
Slightly potholed	201 (206)	66 (66)	77 (72)	344
Severely potholed	68 (75)	27 (24)	30 (26)	125
Column totals	376	120	132	628

Note: Expected frequencies are shown in parentheses below the observed frequencies.

### Condition of Shoulders vs Pothole Formation

The condition of shoulders as to drainage facility is arbitrarily classified as good, fair, and poor. The observed and expected frequencies with regard to condition of shoulders and pothole formation are shown in Table 8. The calculated chi-square is 16.68. The number of degrees of freedom is equal to 4. With four degrees of freedom, the calculated chi-square 16.68 shows a probability value less than 0.01 ( $P = 0.002$ ). The result is thus significant. An inspection of the contingency table shows that road sections with good shoulders appear to have less pothole problems than those with poor shoulders.

### Condition of Side Ditches vs Pothole Formation

The condition of side ditches is classified into three categories in accordance with their ability of removing water away from the road bed, and designated arbitrarily as good, fair, and poor. The contingency table classifying the road sections according to condition of side ditches and according to pothole formation is shown in Table 9. The calculated chi-square is 6.31; and the number of degrees of freedom is 4. Entering these values in the chi-square table, a probability value more than 0.01 is obtained ( $P=0.177$ ). Inasmuch as the probability value associated with the computed chi-square is above the accepted level of significance, no ground is revealed by the test for supposing the hypothesis incorrect. In other words, the data do not suggest that the pothole formation varies with the condition of ditches, at least so far as this test is concerned.

### Type of Subgrade Soil vs Pothole Formation

Subgrade soils in the area are predominantly of the DH, DM, and ZS groups, in accordance with the soil classification system used in Illinois Engineering Soil Maps (11). The classification system is based on a two letter system with reference to a soil profile divided into several horizons. The first letter of the symbol expresses the plasticity of the Horizon A (surface and subsurface), and Horizon B (or subsoil). The second letter indicates the character of Horizons C and D (or substrata). The details of the classification system have been reported previously by T.H. Thornburn (10).

Soils in the DH group have from slightly to moderately plastic horizons A and B, and highly permeable horizons C and D. Soils in the DM group have from slightly to moderately plastic horizons A and B, but moderately permeable horizons C and D.

TABLE 10  
CLASSIFICATION OF ROAD SECTIONS ACCORDING TO SUBGRADE SOIL  
GROUP AND POTHOLE FORMATION

Pothole Formation	Number of Sections Having Stated Pothole Formation for Subgrade Soil Group:			Row Totals
	DH	DM	ZS	
No potholing	26 (25)	75 (77)	72 (71)	173
Slightly potholed	50 (56)	181 (172)	155 (158)	386
Severely potholed	25 (20)	56 (63)	60 (58)	141
Column totals	101	312	287	700

Note: Expected frequencies are shown in parentheses below the observed frequencies.



Soils in ZS group have highly plastic horizons A and B, and slowly permeable horizons C and D. The typical characteristics of these soils are given in the report accompanying the engineering soil map in detail in the form of soil data sheets.

The contingency table classifying road sections in accordance with the above subgrade soil groups and pothole formation is shown in Table 10. The data gives a chi-square of 3.37 and 4 degrees of freedom. Entering these values in the chi-square table, a probability value larger than 0.01 is obtained (P=0.498). This test gives no evidence to reject the hypothesis and does not prove that the two characteristics are related.

### General Discussion

Among the various road characteristics the condition of side ditches and the type of subgrade soil have not been ascertained to be associated with pothole formation by the chi-square test. Both these characteristics gave a probability value above the accepted 0.01 level of significance, implying that any possible correlation between either one of these characteristics and pothole formation might be attributed to chance. It must be noted, however, that while the chi-square test provided no ground to relate the pothole formation with the characteristics mentioned above, the test did not prove that these characteristics were unrelated to the pothole formation. In the chi-square test, it is the disagreement between the observed data and hypothesis which is important, because this disagreement gives reason for thinking the hypothesis incorrect. Agreement, on the other hand, does not mean that the hypothesis is correct. Regarding these two road characteristics, therefore, all that can be said is that the chi-square test revealed no grounds for supposing incorrect the hypothesis that the pothole formation is independent of these characteristics. As far as the evidence is concerned, it is appropriate to assume that the drainage condition of side ditches and the particular types of subgrade soil were homogeneous with respect to pothole formation.

The circumstances that have been ascertained to be associated with the formation of potholes embrace (a) volume of traffic, (b) width of road surface, (c) type of surface material, (d) condition of crown, and (e) condition of shoulders. Of these circumstances the volume of traffic is perhaps the most remarkable, representing the cyclic stresses imposed by the wheels of moving vehicles upon the surface of a road.

Under the wheels of vehicles a road surface is imposed upon by a complex system of stresses. The weight of a wheel causes stresses in the surface course and tends to produce settlement or deformation of the loaded part of the surface in a vertical direction. Likewise, horizontal shear stresses are set up over the contact area of the tire due to the resistance offered by the road surface to lateral displacement of the tread. When the vehicle is accelerated, the horizontal shear tends to push the surface material backward. When brakes are applied, a similar tendency is produced in the opposite direction. In addition, rolling wheels produce vibratory or dynamic stresses which tend to loosen the surface material. The intensity of these stresses is determined by the magnitude of the total load, the rigidity of and the inflation pressure in the tire, the condition of road surface, the driving force and the braking force of the vehicle, and other dynamic effects of moving loads.

Each time a wheel passes, the above system of transient stresses is imposed on the road surface. If the process is repeated, the stresses set up by moving traffic ultimately exceed those that can be safely tolerated by the road material, and surface failures result. The greater the volume of traffic, the greater is the disruptive action of moving wheels, and thus more failures occur. Even though the forms of developed failure may be different, the above effect of traffic is common on all types of road surfaces.

While the destructive forces are always present, the serviceability and economic life of the surface vary in general with the structural strength of the surface material. Basically all road materials are composed of mineral aggregates which are bound together by a cementing agent. The strength of a material is generally ascribed both to the friction between aggregate particles and to the cohesion introduced by the cementing agent. The capacity to resist wear and tear of traffic on a surface system, however, is to a large extent represented by the binding power of the cementing agent.

It is generally recognized that mineral aggregate for most types of road construction must consist of a properly graded mixture of hard, tough, and durable particles to provide frictional resistance. For high type pavements carrying large volumes of traffic, a cementing agent capable of supplying a strong and relatively permanent bond between these mineral aggregates is in addition a matter of virtual necessity. Thus among the high type surfaces, a portland cement concrete pavement must employ a hardened cement paste to completely surround and strongly hold together the mineral aggregates. Similarly, a bituminous concrete pavement has to resort to a suitable bituminous binder for properly cementing the otherwise loose aggregate mass.

In a soil-aggregate road surface, the cementing is provided by the clay in the binder soil which derives its cohesion from the adsorbed moisture films of individual soil particles. Since the cohesion varies with the moisture content, the binder soil can be depended upon for cementing action only when the moisture content is within a certain desirable range. In dry weather the soil moisture is not sufficient to develop the cohesion required to prevent surface abrasion which results in loss of material and causes dust nuisance. Conditions in wet weather may be just as unsatisfactory, due to excessive moisture. Under these conditions, the cohesion of the clay fraction may be completely vanished and the soil become a lubricant instead of a binder. In addition, the soil may expand the solid framework and cause a decrease in the mechanical contact between particles through excessive volume change. Thus the interlocking of granular particles and the mutual support so important to the frictional resistance of aggregate materials is readily, and in some cases, entirely eliminated.

Inasmuch as the binder soil, except under ideal conditions, does not act efficiently as a cementing medium, the capacity of the soil-aggregate mixture to resist wear and tear of traffic is essentially limited. Although a mixture of well compacted angular particles may withstand the action of a limited amount of traffic due to effective particle interlocking, a surface material that consists of rounded mineral particles in the absence of cohesion can hardly carry any appreciable amount of traffic without resulting in some kind of serious defect. As the vehicle crushes with its weight and tears and abrades with its moving wheels, the soil-aggregate material which is not firmly bound in the road surface will either be laterally displaced or be lifted and carried away, thus leaving small depressions or holes in the road surface. Following further actions of traffic, serious road failures will be developed. If the road material is not generally deficient in structural strength, the failure will be ordinarily localized at the points, in the form of potholes, where local weakness or injury takes place. Otherwise the surface failure will occur in a larger area, most commonly in the form of washboards where the weak surface material shapes itself into rhythmic undulations under the influence of traffic.

This depiction of the effect of volume of traffic and type of surface material on the development and condition of potholes on soil-aggregate road surfaces has been confirmed by the results of the present investigation. The chi-square test indicated convincingly that the formation of potholes was related to the volume of traffic. Although potholes were found in road sections in all classes of average daily traffic, it was noted that road sections with smaller volumes of traffic were less severely potholed than those with larger volumes of traffic. In addition, the effect of volume of traffic was evidenced by the correlation between the width of road surface and the pothole formation, in that wider road surfaces were more severely potholed than the narrower ones. Inasmuch as wider surfaces were also associated with larger volumes of traffic, the correlation between the width of road surface and pothole formation was, in effect, an indication of the relationship between the volume of traffic and pothole formation. Since larger volumes of traffic were carried by road sections with wider surfaces, the greater disruptive action of moving wheels resulted in more severe formations of potholes in these surfaces. (In the test of association between the width of road surface and the volume of traffic, the probability associated with the computed chi-square 91.70, for six degrees of freedom, is less than 0.01, ( $P=0.000$ ), indicating a significant correlation between the two.)

In regard to the two types of soil-aggregate material used for road surfacing, the chi-square test indicated that the pit-run gravel tended to be less associated with pot-

hole formation than the crushed stone. Due to lack of detailed data concerning the properties and characteristics of these two materials, a complete account of the occurrence was not possible. It was noted in the field investigation, however, that the crushed stone was not only deficient in binder content, but also highly segregated in its particle size because of incorrect method of handling and placing the material on the road. It was speculated, therefore, that due to the above circumstances the road surface was seriously weakened in many spots becoming particularly susceptible to the development of potholes. The pit-run gravel, on the other hand, generally contained enough binder soil which would hold the soil-aggregate material in place under normal service conditions. The presence of enough binder soil in the pit-run gravel would not necessarily exempt the road material from the development of potholes or other types of defect. The formation of washboards might well be promoted if a slight excess of binder soil were present in the surface, since the binder soil would soften and swell when wet and tend to unseat the coarser materials under traffic.

Among the road characteristics that have been ascertained to be associated with pothole formation, both the condition of crown and the condition of shoulders have perhaps no occasion for elaborate explanation. The strength of a soil-aggregate mixture is decreased by the penetration of moisture into the voids. Consequently, a soil-aggregate surface must be equipped with adequate crown and sloped shoulders to facilitate fast removal of water from the road surface. If the crown and shoulders fail to function properly, the surface material becomes softened by water standing on the surface, and under traffic the surface develops small pits. Water held in these depressions during subsequent wet periods further softens the hard-packed surface, and the finer soil material suspended in the water is lost as traffic throws the water aside. The coarser residual material, no longer held together by a binder, is also partly removed later when the road is dry. In this manner the small initial pits produce the typical potholes.

Although the data brought out by the present investigation indicate clearly the trend discussed above, it is interesting to note that similar results were disclosed in 1935 by Burggraf. In a study of soil-aggregate roads in Kansas, Illinois and Indiana, Burggraf showed a very definite relationship to exist between the service behavior of the road and the amount of crown, the serviceability of the road surface decreasing as the crown is lowered (4). Based on field data he concluded that the average crown where potholing had been developed was 0.26 in. per ft, and that where no surface defects appeared was 0.41 in. per ft. It is noted in the present investigation that, somewhat unconfirmable to Burggraf's conclusion, potholing was observed in road sections with a crown greater than 0.41 in. per ft. However, the effect of surface drainage on the development and general condition of potholes is essentially in agreement with the findings reported by Burggraf. It is realized that there were definitely other factors than condition of crown and shoulders affecting the formation of potholes. It would not be possible, therefore, in this study to ascribe the pothole formation only to any arbitrary rate of crown or slope of shoulders.

## WASHBOARD FORMATION

### Road Characteristics Studied

The road characteristics which were tested for association with washboard formation were the same ones studied with pothole formation, including type of surface material, width of road surface, condition of crown, condition of shoulders, condition of side ditches, and type of subgrade soil. In addition, the volume of traffic was studied in connection with the formation of washboards.

### Volume of Traffic vs Washboard Formation

The contingency table showing observed and expected distributions of road sections with respect to their washboard formation and volume of traffic is shown in Table 11. From the data, the chi-square computed is 30.04 and the number of degrees of freedom is 6. Entering these values in the chi-square table, a probability value less than 0.01 is obtained ( $P=0.000$ ). It follows that there was a significant correlation between the

volume of traffic and washboard formation. The differences between the expected and the observed frequencies reveal that the road sections with smaller volume of traffic appear to have less washboarding problems than those with larger volume of traffic.

#### Type of Surface Material vs Washboard Formation

The contingency table classifying road sections according to two types of surface material, gravel and crushed stone, and according to washboard formation is shown in Table 12. The chi-square computed is equal to 36.13. The number of degrees of freedom is 2. For these values, a probability value less than 0.01 is noted in the chi-square table ( $P=0.000$ ), indicating a significant relationship between the type of surface material and washboard formation. The differences between observed and expected frequencies show that road sections surfaced with crushed stone appear to have less washboards formed than those surfaced with gravel.

**TABLE 11**  
**CLASSIFICATION OF ROAD SECTIONS ACCORDING TO AVERAGE DAILY TRAFFIC AND WASHBOARD FORMATION**

Washboard Formation	Number of Sections Having Stated Washboard Formation for Average Daily Traffic of:				Row Totals
	0-24	25-49	50-74	75-99	
No washboarding	163 (144)	191 (190)	46 (57)	16 (25)	416
Slightly wash-boarded	27 (36)	43 (47)	19 (14)	14 (6)	103
Severely wash-boarded	19 (29)	42 (39)	17 (11)	7 (6)	85
Column totals	209	276	82	37	604

Note: Expected frequencies are shown in parentheses below the observed frequencies.

**TABLE 12**  
**CLASSIFICATION OF ROAD SECTIONS ACCORDING TO TYPE OF SURFACE MATERIAL AND WASHBOARD FORMATION**

Washboard Formation	Number of Sections Having Stated Washboard Formation for the Two Types of Surface Material:		Row Totals
	Gravel	Crushed Stone	
No washboards	179 (211)	181 (149)	360
Slightly wash-boarded	77 (59)	24 (42)	101
Severely wash-boarded	56 (42)	16 (30)	72
Column totals	312	221	533

Note: Expected frequencies are shown in parentheses below the observed frequencies.

**Width of Road Surface vs Washboard Formation**

The contingency table giving observed and expected frequencies corresponding to width of road surface and washboard formation is shown in Table 13. The computed value of chi-square is 18.68. The number of degrees of freedom is 6. With six degrees of freedom, the computed chi-square shows a probability value less than 0.01 ( $P=0.005$ ). The result indicates a significant association between the width of road surface and washboard formation. Comparing the observed frequencies with the expected frequencies Table 13 shows that wider road surfaces had more severe formations of washboards than narrower road surfaces.

TABLE 13

**CLASSIFICATION OF ROAD SECTIONS ACCORDING TO WIDTH OF ROAD SURFACE AND WASHBOARD FORMATION**

Washboard Formation	Number of Sections Having Stated Washboard Formation for Road Surface Width (ft) of:				Row Totals
	21-24	17-20	13-16	9-12	
No washboarding	35 (43)	150 (165)	163 (149)	49 (40)	397
Slightly wash-boarded	17 (12)	56 (47)	35 (43)	6 (12)	114
Severely wash-boarded	12 (9)	41 (35)	25 (31)	6 (9)	84
Column totals	64	247	223	61	595

Note: Expected frequencies are shown in parentheses below the observed frequencies.

TABLE 14

**CLASSIFICATION OF ROAD SECTIONS ACCORDING TO CONDITION OF CROWN AND WASHBOARD FORMATION**

Washboard Formation	Number of Sections Having Stated Washboard Formation for Crown Condition of:			Row Totals
	High Crown ( $\frac{1}{2}$ in. or more per ft)	Low Crown ( $\frac{1}{4}$ in. per ft)	No Crown	
No washboarding	284 (255)	105 (122)	50 (62)	439
Slightly wash-boarded	73 (88)	52 (42)	26 (21)	151
Severely wash-boarded	51 (65)	38 (31)	23 (16)	112
Column totals	408	195	99	702

Note: Expected frequencies are shown in parentheses below the observed frequencies.

### Condition of Crown vs Washboard Formation

The relationship between the condition of crown and washboard formation is studied on the basis of the data contained in Table 14. From the data the chi-square computed is 21.77. With the number of degrees of freedom 4, a probability value less than 0.01 is indicated in the chi-square table ( $P=0.000$ ). The result shows the relationship between the condition of crown and washboarding is significant, the formation of washboards being aggravated as the crown is lowered.

### Condition of Shoulders vs Washboard Formation

The observed and expected frequencies with regard to condition of shoulders and washboard formation are shown in Table 15. The computed value of chi-square is 9.87. The number of degrees of freedom is 4. Entering these values in the chi-square table, a probability value more than 0.01 is indicated ( $P=0.043$ ). Since the probability value is above the accepted level of significance, there is no reason to suppose that the hypothesis is incorrect, or postulate that washboard formation is related to the condition of shoulders.

### Condition of Side Ditches vs Washboard Formation

The contingency table showing observed and expected distributions of road sections according to their washboard formation and according to the condition of side ditches is shown in Table 16. The calculated chi-square is 1.89; and the number of degrees of freedom is 4. Entering these values in the chi-square table, a probability value more than 0.01 is noted ( $P=0.756$ ). Since the probability value falls above the accepted level of significance, there is no cause to assume that the washboard formation varies with the condition of side ditches.

### Type of Subgrade Soil vs Washboard Formation

The contingency table classifying road sections in accordance with three subgrade soil groups, namely, DH, DM and ZS, and washboard formation is shown in Table 17. The chi-square computed from the frequencies in Table 17 is 2.06. The number of degrees of freedom is 4. Entering these values in the chi-square table, a probability value more than 0.01 is indicated ( $P=0.725$ ). The result does not ascertain that washboard formation is related to the type of subgrade soil, since the probability value

TABLE 15

### CLASSIFICATION OF ROAD SECTIONS ACCORDING TO CONDITION OF SHOULDERS AND WASHBOARD FORMATION

Washboard Formation	Number of Sections Having Stated Washboard Formation for Shoulders in:			Row Totals
	Good Condition	Fair Condition	Poor Condition	
No washboarding	121 (107)	62 (61)	253 (268)	436
Slightly wash-boarded	22 (28)	14 (16)	79 (71)	115
Severely wash-boarded	10 (18)	11 (10)	52 (45)	73
Column totals	153	87	384	624

Note: Expected frequencies are shown in parentheses below the observed frequencies.

associated with the computed chi-square is above the accepted level of significance.

### General Discussion

The circumstances that have been ascertained to be associated with washboard formation include (a) volume of traffic, (b) width of road surface, (c) type of surface material, and (d) condition of crown. Although all these circumstances have previously been found to be also related to pothole formation, their controlling influence upon the performance and serviceability of the roads investigated seems all the more evident.

Both the volume of traffic and the width of road surface, as noted in the discussion in connection with pothole formation, are indicative of the cyclic stresses imposed by wheels of moving vehicles upon a road surface. When the traffic volume is small and the total stresses are less than those that can be safely tolerated by the road material, the surface remains relatively intact. As the traffic increases, the weakest places in

TABLE 16

CLASSIFICATION OF ROAD SECTIONS ACCORDING TO CONDITION OF SIDE DITCHES AND WASHBOARD FORMATION

Washboard Formation	Number of Sections Having Stated Washboard Formation for Side Ditches in:			Row Totals
	Good Condition	Fair Condition	Poor Condition	
No washboarding	251 (246)	82 (83)	102 (106)	435
Slightly wash-boarded	73 (73)	25 (24)	30 (31)	128
Severely wash-boarded	49 (54)	19 (19)	28 (23)	96
Column totals	373	126	160	659

Note: Expected frequencies are shown in parentheses below the observed frequencies.

TABLE 17

CLASSIFICATION OF ROAD SECTIONS ACCORDING TO SUBGRADE SOIL GROUPS AND WASHBOARD FORMATION

Washboard Formation	Number of Sections Having Stated Washboard Formation for Subgrade Soil Group:			Row Totals
	DH	DM	ZS	
No washboarding	66 (63)	231 (237)	186 (183)	483
Slightly wash-boarded	13 (16)	70 (63)	45 (49)	128
Severely wash-boarded	12 (12)	44 (45)	36 (35)	92
Column totals	91	345	267	703

Note: Expected frequencies are shown in parentheses below the observed frequencies.

the road give way first and the surface begins to deteriorate. Since the destructive forces are inherent in the vehicles using the road, the developed failure, in whatever form it may occur, will be accentuated by increasing frequency of loading or growing traffic.

The effect of traffic on soil-aggregate roads, as far as washboard formation is concerned, has long been recognized. Ladd reported in 1924, for instance, in connection with his investigation concerning washboard formations in roads in Maine, New Hampshire, Connecticut, New Jersey, Michigan, Wisconsin, and Oregon that gravel roads subjected to a traffic of not more than 200 or 300 vehicles per day remained practically free from washboards if occasionally dragged. As soon, however, as traffic reached 400 to 450 vehicles per day washboards developed very rapidly (7).

Carpenter and Dana reported in 1927 in a discussion of the relation of volume of traffic to formation of washboards that a traffic of 300 or 400 vehicles per day did not usually cause serious formation of washboards (1). On the other hand, it has also been reported that, based on more recent experience, washboards might be encountered in some roads if the volume of traffic exceeded 50 vehicles per day, and that only by intensive maintenance could a greater number of vehicles be handled without serious washboards formed (5). While there were effects other than traffic upon the formation of washboards, the disagreement in reports regarding the critical amount of traffic effecting the formation of washboards would be expected. It is speculated, however, that considering that weights and speeds of vehicles have been considerably increased during the past 30 years, a traffic of 50 vehicles per day can probably be regarded at present as the average volume that would initiate the nuisance of washboarding in a fairly well constructed road even under favorable climatic conditions.

The washboard formations are characterized by the presence of transverse waves on the road surface with approximately equal amplitudes and uniform spacings. Just how the traffic causes or accentuates these formations has been a problem of interest for many years, and the indications are that the periodic vibrations and impacts of moving wheels have much to contribute to the problem. As described by Dana,

...a small obstruction, or unevenness in the road surface, the lurch of the vehicles, the vibrations of the engine, or any of a number of different circumstances, separately or combined, may cause the wheel ...to jump from the road surface.....When it descends, the cushioning of the impact by the air pressure in the tire will cause the wheels to bounce again. This bouncing will continue until the original energy imparted to the system has all been absorbed....The wheels of the next vehicles, hitting the same obstruction, or being influenced by similar circumstances, will repeat the previous performance. Thus the impacts of successive tires in nearly the same spots will begin to break and loosen the....surface, and the air currents caused by the moving tires and by the car itself will lift part of the loosened binder and fine material and carry it away, at the same time shaping the remainder into successive waves ...which in turn promote still more and deeper washboards (3).

The fact that washboard formations occur in all sections of the country makes it evident that nearly all soil-aggregate roads, if they serve sufficient traffic, are subject to the development of this nuisance even though the methods of construction and kinds of material vary widely. However, the facility by which the washboard formations are formed appears to depend to a considerable extent upon the strength characteristics of the road material. From the experience gained with this type of construction, it is generally agreed that to resist adequately the periodic vibrations and impacts of moving vehicles, the surface material must be such that it would consolidate sufficiently by rolling or under traffic and remain relatively stable under all weather condi-



tions. Such a material calling upon a combination of the high internal friction of the aggregates and the beneficial cohesion of the binder soil is generally attained through properly controlling the gradation or relative amounts of components of the mixture, and by controlling the binding properties of the binder soil. If a material is lacking in adequate binder soil, compacting the material to a desirable state of consolidation and maintaining it in this state are extremely difficult. The strength of the mixture has to resort primarily to the internal friction produced by the interlocking and mutual support of adjacent particles in the mass. Although a mass of angular particles may produce some resistance, a loose granular mass composed of rounded particles is evidently incapable of withstanding the destructive forces of traffic, and, in the words of Ladd, "washboards are formed by the kick-back of surface materials arising from the spin of . . . wheels of automobiles as they descend after a bounce over some obstacle or depression" (7, p. 18). On the other hand, if a mixture contains an excessive amount of binder soil, the material will become soft and plastic in the presence of water and, according to Ladd, washboards are formed by the squeezing action of wheels resulting from impact (7, p. 19).

It is evident that the type of surface material bears a definite relation to the formation of washboards. Ladd reported as early as 1924 that, in the opinion of some highway officials, washboard trouble would be reduced should angular aggregate materials be used. It was reported that in northern New Hampshire some roads that were built of a material which had resulted from the decomposition of granite rocks and was full of angular quartz were said to give better service, as far as washboards were concerned, than those built in the same area with glacial gravels. Highway engineers, according to Ladd, notably in Wisconsin and Oregon, objected to any clay binder in the gravel and preferred only the fines produced by crushing and from surface wear to prevent washboard formation (7, p. 18).

More recently Jahn in examining road failures in Flensburg, Germany, reported among other things that serious defects, either in the form of washboards or potholes, were observed in roads where the material showed either an excessive amount of binder soil or an excessive amount of aggregates (6).

The result of the present study agrees in general with the trends indicated above. Although washboards were found in road sections surfaced with gravel as well as crushed stone, the road sections surfaced with gravel showed definitely a closer association with washboard formation than those surfaced with the other material. It may be recalled that the gravel material was a pit-run product, consisting of rounded aggregate particles with more than enough binder material which tended to expand and to act as a lubricant when wet. Such a material would readily soften in the presence of water and be squeezed by the periodic vibrations and impacts of moving wheels, resulting in the formation of washboards. The crushed stone, on the other hand, consisted entirely of angular aggregates, highly segregated and in general deficient in binder content. It is postulated that the mechanical advantage due to the interlocking of angular particles enhanced the resistance of the material to the bouncing and squeezing action of traffic, and restrained more or less the formation of washboards. However, the segregated material automatically incorporated many weak spots in the road surface and invited localized destruction under traffic in the form of potholes which showed no apparent regularity as washboards.

Although a road material containing an excessive amount of binder is generally regarded unsatisfactory in soil-aggregate construction, the detrimental effects are often unsuspected if there is no excessive moisture present. Ladd has reported, for instance, that a section of road built of soft gravel which was high in clay-silt bond was practically free from washboards throughout summer, but later during the wet season became nearly impassable (7, p. 20). Because washboard formations in a soil-aggregate surface depend upon the strength characteristics of the road material and because the strength characteristics, as indicated previously, are adversely affected by increasing moisture content, the development of washboards can often be curbed by protecting the road material from moisture through effective drainage. As a road surface a soil-aggregate material may occasionally be weakened by ground water or capillary water, but more often it is weakened by uncontrolled rain and storm water standing upon the

surface. Consequently, an effective crown must be provided to shed rapidly the surface water away from the traveled way so that the stability of the road material will be well preserved. As an important drainage factor, the condition of crown would be expected to bear a definite relationship to the development and condition of washboards. The pattern has been brought out by the result of the present investigation, in that road surfaces with good or adequate crowns were less associated with washboards than those which had poor or inadequate crowns for surface drainage.

Among the road characteristics studied in this investigation, three have not been ascertained to be associated with washboard formation, namely, (a) condition of shoulders, (b) condition of side ditches, and (c) type of subgrade soil. It must be again emphasized, in this connection, that the results give no indication that these characteristics are unrelated to washboard formation. Although washboard formations have been reported to be affected by the bearing value and drainage requirements of the subsoil (2) and the condition of shoulders has indeed been ascertained previously to be associated with pothole formation, it is within the bounds of possibility that one or more of these characteristics are related to washboard formation. However, all these characteristics gave a probability value above the accepted 0.01 level of significance, implying that any possible correlation could have arisen solely due to chance and giving no evidence to reject the hypothesis that these characteristics were unrelated to washboard formation. It is pertinent to conclude, nonetheless, that as far as the evidence is concerned, the condition of shoulders and side ditches and the type of subgrade soil were homogeneous with respect to the formation of washboards.

## SUMMARY AND CONCLUSIONS

### Results

In this investigation a number of road characteristics observed and noted in a condition survey of soil-aggregate roads were studied for association by chi-square test. Certain circumstances have been proved to be definitely associated with the development and condition of potholes and washboards in these roads. The results are summarized as follows:

1. Both the pothole formation and the washboard formation were associated with volume of traffic. Road sections with larger volume of traffic appeared to have more serious problems of potholes and washboards than those with smaller volume of traffic.
2. Both the pothole formation and the washboard formation were associated with the width of road surface. Road sections with wider surfaces were more severely potholed or washboarded than those with narrower surfaces. Inasmuch as wider surfaces were also associated with larger volume of traffic, the correlation was, in effect, an indication of the relationship between the volume of traffic and pothole or washboard formation.
3. Potholing was more severe in road sections surfaced with crushed stone than in those surfaced with gravel. Washboarding was more severe in road sections surfaced with gravel than those surfaced with crushed stone.
4. A very definite relationship was shown to exist between the condition of crown and shoulders as to drainage and the pothole formation. Road surfaces with poor or inadequate crowns and shoulders showed more severe potholing than well drained surfaces. A significant relationship was also found between the condition of crown and washboard formation. Road surfaces with poor or inadequate crowns tended to have more serious washboard problems than those with high or adequate crowns.

### Conclusions

On the basis of the foregoing data, the following general conclusions have been drawn:

1. The service of soil-aggregate road surfaces is confined to roads carrying a relatively small volume of traffic, because the capacity of the surface material to resist wear and tear of traffic is much limited. When the traffic volume is relatively

low and the climatic condition favorable, the road surface is decidedly an economic type. As the traffic increases, potholes and washboards start to develop, and the cost of maintenance required to keep the road in serviceable condition begins to mount. When the traffic reaches a certain limit, potholing and washboarding will become incorrigible and the maintenance cost will approach the amount which is no longer economically justified to spend. Although conditions of local material, climate, and method of construction vary, the effect of traffic should be realized by the intensity of maintenance required and by its cost.

2. To minimize or to better control the nuisance of potholes or washboards in soil-aggregate road surfaces, attention must be exercised in selecting the proper material for surface construction. From this study it is difficult to state whether gravel or crushed stone was more desirable since crushed stone sections showed more severe potholing and gravel sections showed more severe washboarding. It is appropriate to conclude, however, that a soil-aggregate mixture with proper amount of aggregate and binder that combines the high internal friction of the aggregate with the beneficial cohesion of the binder soil should minimize the formation of either potholes or washboards.

3. Inasmuch as soil-aggregate surfaces are generally irregular and highly absorbent, a proper amount of crown and sloped shoulders to readily drain the surface water away from the traveled way is a virtual necessity to satisfactory performance of these surfaces. Both potholes and washboards are developed and accentuated if the surface material has been softened by standing water on the surface. It is evident that in order to get constantly rid of the water from the surface as quickly and fully as possible, adequate drainage must be provided which will largely minimize the formation of potholes and washboards.

Although the circumstances related to the formation of potholes or washboards may be many, this paper outlines only those that have been noted in a road condition survey. Since the primary objective of this survey was to locate road sections for special studies, much of the needed data for a correlation study was not collected. Consequently the scope of the study was limited. It is believed, however, that while the formation of potholes and washboards remains a problem of interest for future study, the findings of this investigation at the least provide certain information toward further understanding of these formations.

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The work reported herein was done under the general supervision of Ellis Danner, Director of the Illinois Cooperative Highway Research Program and Professor of Highway Engineering, University of Illinois; and Theodore F. Morf, Engineer of Research and Planning, and W. E. Chastain, Sr., Engineer of Physical Research, Illinois Division of Highways.

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*Discussion*

J. W. SPENCER, Highway Research and Extension Engineer, Department of Agricultural Engineering, Cornell University — Professor Huang's presentation of some relationships between various road characteristics and the occurrence of potholing and washboarding is very helpful — especially as these relationships serve to inspire deeper thinking about the causes of these rather common "diseases" of soil-aggregate roads.

Although there is some danger that lack of independence of a particular characteristic and a particular disease may lead the uninitiated to assume a cause-effect relationship, the chi-square test helpfully indicates where relationships appear definitely to exist. While the chi-square approach provides an objectivity not possible in mere observation of data, there seems to be one contingency table (Table 5) where the indicated significance of relationship between volume of traffic and pothole formation is not readily apparent to the observer.

The author suggests that of the circumstances associated with the formation of potholes, the volume of traffic is perhaps the most remarkable. He amplifies this by stating, "The chi-square test indicated convincingly that the formation of potholes was related to the volume of traffic." The experience of those who have long observed the performance of soil-aggregate roads must confirm an apparent relationship between traffic volume and potholing, but it does not seem that the data in Table 5 indicate a strong relationship for roads carrying at least 50 vehicles per day. Comparing the observed frequencies with expected frequencies, it appears that for an average daily traffic of up to 50 vehicles, increasing traffic had a marked relationship to degree of potholing, but for increases beyond 50 vehicles per day, there appears to be little if any difference between expected and observed frequencies. Perhaps this observer has misinterpreted the data or perhaps the statements were based on data not included in the paper.

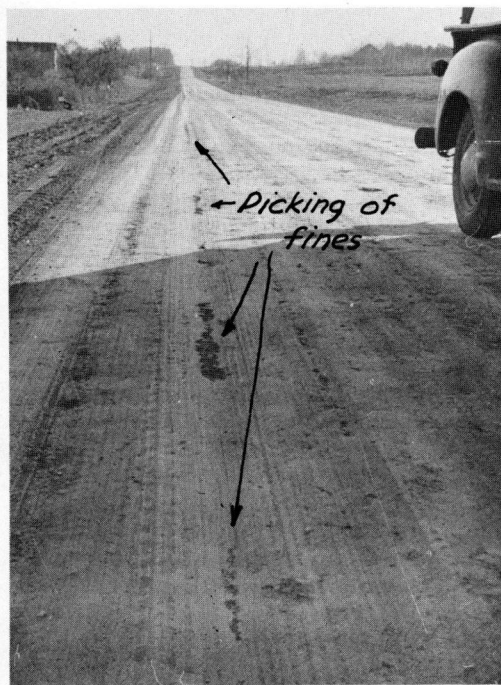


Figure 3. A beginning of potholes; fines at surface of road have picked up under traffic.

The paper helpfully discusses some actions of wheels on a soil-aggregate road which are responsible for the beginnings of potholes. Experience in New York suggests that there may be yet another action which is responsible for the start of many potholes. Figure 3 will help to explain the apparent picking or plucking of fines at the surface of soil-aggregate roads. The surface of many soil-aggregate roads becomes "plastered" with fines during wet weather. A concentration of finer particles at the road surface can result from several actions, but traffic compaction in rainy weather, especially where the soil-aggregate mixture has not previously been fully compacted at a reasonable water content, may contribute to a subsequent pumping of fines to the surface. Such an action cannot be completely controlled by compaction because freezing may slightly "fluff" a soil-aggregate road which has been previously well compacted. Regardless of the cause of a concentration of fines at the road surface, and recognizing that the fines may actually benefit dry-weather road performance, the presence of fines which are adhesive when wet may result in the beginning of potholes.

The section of road shown in Figure 3 had a silt and clay content of about 6 percent in the top 4 in. of the soil-aggregate

mixture; the silt and clay content in the top  $\frac{1}{2}$  in. was about 14 percent. The photograph illustrates how tires had picked up or plucked out fines from the road surface while the material was at a water content which promoted adhesion. Experience indicated that such depressions quite rapidly developed into potholes. Observations have suggested that picking is more likely to result where fines at the surface overlay a rather clean gravel mixture than where the soil-aggregate mixture has a reasonably high silt and clay content for a depth of several inches.

Huang indicates that the formation of washboards might well be promoted by an excess of binder soil under wet weather conditions. Experience in New York suggests that, while potholing is a wet weather disease often encouraged by the action of binder soil, washboarding is largely a dry weather occurrence which is more likely to develop with too little than too much binder soil. It seems that washboarding is promoted by the liberation of granular material which may be displaced by moving traffic and that liberation during dry weather is likely to be hastened by a scarcity of cohesive binder soil.

Although it is understandable that angular aggregates are less likely to be displaced into corrugations or washboards than are rounded aggregates, it is not readily understandable why an excess of binder material would be a major cause of washboarding. It seems that washboarding is very seldom observed on earth (cohesive soil) roads and seldom on roads built with "packing" gravels if such materials have been treated with a palliative to reduce dusting and subsequent raveling. Bank-run materials used for gravel roads in central New York usually contain more than 10 percent minus No. 200 material and classic examples of washboarding are quite difficult to find.

Huang, in indicating that both pothole and washboard formation were associated with width of road surface, explains that wider surfaces were also associated with higher traffic volumes. Perhaps, too, higher vehicle speeds were associated with greater widths of road and had an effect on road performance, especially the washboarding.

**CLOSURE, EUGENE Y. HUANG**— The purpose of this paper was to present the results of a study aimed at determining some of the circumstances associated with the occurrence of potholes and washboards on soil-aggregate roads. Although associations between certain road circumstances and these formations are indicated by the chi-square test, there is indeed no evidence in the results that these associations are causal relationships. It must be noted, therefore, that any interpretation in the discussion of the results implying a cause-effect relationship between the associated variables was based upon the author's observation and knowledge of the roads investigated. Since the findings are limited to a certain extent to the types and conditions of the roads studied, the author appreciates the comments made by Mr. Spencer, which apparently came from his experience in this type of road construction, regarding a particular traffic action responsible for the start of many potholes and the phenomenon of washboards on roads in New York State.

With reference to Table 5, the author agrees that "there appears to be little if any difference between expected and observed frequencies" for roads with an average daily traffic exceeding 50 vehicles. However, the scope of the present investigation using a chi-square test was insufficient to indicate an independence between traffic and pothole formation. Consequently, a general statement was made that "road sections with a smaller volume of traffic appear to have less pothole problems than those with a larger volume of traffic." It would be interesting to extend the present investigation further in order to prove the definite relationship between traffic and pothole formation with daily traffic exceeding 50 vehicles.

Regarding the phenomenon of washboards and the type of material on which washboards form, many theories have been advanced. None of these theories have yet been adequately confirmed. The author agrees that a certain amount of "washboarding is promoted by the liberation of granular material which may be displaced by moving traffic and that liberation during dry weather is likely to be hastened by a scarcity of cohesive binder soil." Indeed many road engineers in tropical desert countries have reported the seriousness of these formations, particularly during dry seasons. However, the phenomenon of washboards on different types of materials has also been reported by engineers in other parts of the world. This nuisance is by no means peculiar to a certain kind or condition of material under a given climatic condition. Insofar as the author's observation is concerned, washboarding is ineluctable as long as the road material lacks a sufficient over-all mechanical stability to resist the kick-back or squeezing actions of moving wheels. Thus formations may occur on a material deficient in binder soil in dry weather, as well as on a material with excess binder under adverse moisture conditions such as that shown in Figure 2. The latter case appears similar to the washboard formations commonly observed on bituminous roads where a more or less soft surface material shapes itself into rhythmic undulations under the influence of traffic.

Spencer's suggestion concerning the possible association between vehicle speeds and widths of road and the effect of vehicle speed on road performance is helpful. Unfortunately, no observations are available from this investigation that could confirm this relationship.

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**T**HE NATIONAL ACADEMY OF SCIENCES—NATIONAL RESEARCH COUNCIL is a private, nonprofit organization of scientists, dedicated to the furtherance of science and to its use for the general welfare. The ACADEMY itself was established in 1863 under a congressional charter signed by President Lincoln. Empowered to provide for all activities appropriate to academies of science, it was also required by its charter to act as an adviser to the federal government in scientific matters. This provision accounts for the close ties that have always existed between the ACADEMY and the government, although the ACADEMY is not a governmental agency.

The NATIONAL RESEARCH COUNCIL was established by the ACADEMY in 1916, at the request of President Wilson, to enable scientists generally to associate their efforts with those of the limited membership of the ACADEMY in service to the nation, to society, and to science at home and abroad. Members of the NATIONAL RESEARCH COUNCIL receive their appointments from the president of the ACADEMY. They include representatives nominated by the major scientific and technical societies, representatives of the federal government, and a number of members at large. In addition, several thousand scientists and engineers take part in the activities of the research council through membership on its various boards and committees.

Receiving funds from both public and private sources, by contribution, grant, or contract, the ACADEMY and its RESEARCH COUNCIL thus work to stimulate research and its applications, to survey the broad possibilities of science, to promote effective utilization of the scientific and technical resources of the country, to serve the government, and to further the general interests of science.

The HIGHWAY RESEARCH BOARD was organized November 11, 1920, as an agency of the Division of Engineering and Industrial Research, one of the eight functional divisions of the NATIONAL RESEARCH COUNCIL. The BOARD is a cooperative organization of the highway technologists of America operating under the auspices of the ACADEMY-COUNCIL and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the BOARD are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.

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