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# *Lime and Lime-Flyash Soil Stabilization*



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# *Lime and Lime-Flyash Soil Stabilization*

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# Durability of Soil-Lime-Flyash Mixes Compacted Above Standard Proctor Density

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Lime-flyash stabilized Kansas dune sand, Iowa silt (loess) and Texas coastal plain clay show a definite increase in durability when compacted to densities above standard Proctor. In fact, silt and clay mixes gained strength more rapdily through artificial weathering than after sustained moist curing.

Two-in. by 2-in. cylindrical specimens prepared with 25 percent lime-flyash and optimum ratios of lime to flyash (1:9 for sand and silt, and 2:8 for the clay) were cured for 14 days at near 100 percent relative humidity and 70 F prior to being subjected to cycles of freezing and thawing or wetting and drying. Moisture absorption, swelling and unconfined compressive strength of the specimens, after various cycles of freeze-thaw and wet-dry were used as a means of analyzing the durability of the stabilized soils.

The increases in strength during wet-dry and freeze-thaw tests over normal moist curing, are attributed to improved intimacy of contact between lime and flyash grains following dissolution and reprecipitation of the lime.

● STUDIES BY Goecker et al (3) have indicated that compaction to a density greater than standard Proctor greatly improved the resistance of soil-lime-flyash to wetting and drying or freezing and thawing. The present study was undertaken to check the resistance of lime-flyash stabilized soils compacted to densities above standard Proctor and within the capabilities of present-day compaction equipment.

Three compactive efforts were used: (1) between standard and modified Proctor density, (2) equivalent to modified Proctor and (3) above modified Proctor. These are listed in Table 1.

The criteria used to evaluate the effects of increased density on durability of the specimens after various cycles of wetting and drying or freezing and thawing were unconfined compressive strength, moisture absorption and average increase in height of specimens, the latter being an indication of volume change or swelling.

## MATERIALS

### Soils

Soils selected for this study are the same as those used by Viskochil et al (5). The silt is a friable, calcareous loess from western Iowa; the clay is a deltaic deposit from the coastal plain region of Texas; and the sand is from a stable dune area associated with the Arkansas River in south central Kansas. Tables 2 and 3 give field information and the various physical and chemical properties of the soil samples.

### Lime and Flyash

The lime used in this study is a hydrated calcitic (high calcium) lime from the Linwood Stone Products Co., Buffalo, Iowa. A laboratory analysis furnished by the manufacturer is shown in Table 4. The flyash used is from Paddy's Run Station, Louisville Gas and Electric Co., Louisville, Kentucky. A chemical analysis of the flyash was obtained from the Robert W. Hunt Co., Chicago, Illinois, and is shown in Table 4.

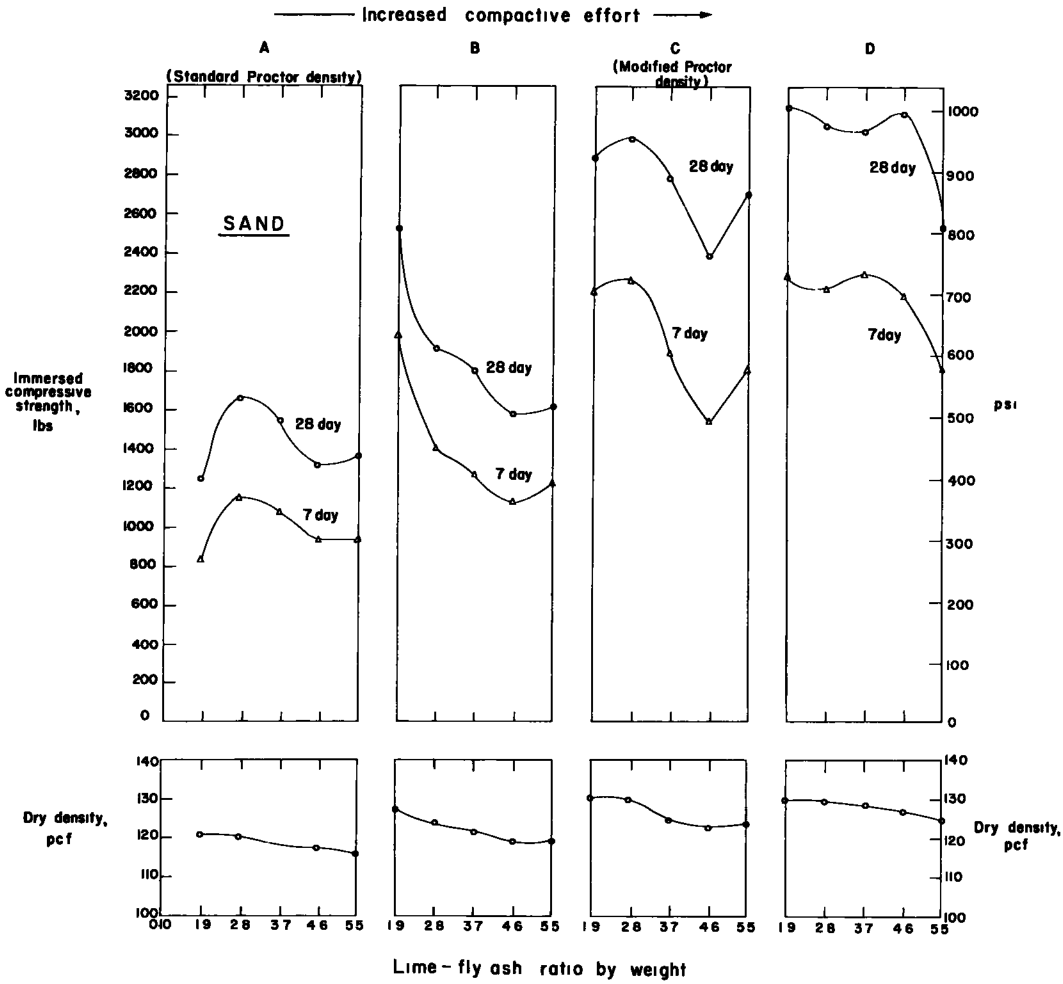


Figure 1. Effect of variations in lime to flyash ratio, compactive effort, and curing on compressive strength and density of lime-flyash stabilized sand.

## METHOD OF TEST

### Mixing

The soils were air dried, pulverized and screened through a No. 10 sieve. Each soil was dry mixed by hand with the various amounts of lime and flyash. Predetermined amounts of distilled water were then hand mixed into the blend and mixing was completed with a Hobart, Model C100, mixer at moderate speed for three minutes.

TABLE 1

DESIGNATIONS OF COMPACTIVE EFFORT[a]

| Compaction | Density Obtained |
|---|---|
| A | Standard Proctor density |
| B | Between standard and modified density |
| C | Modified Proctor density |
| D | Above modified density |

[a]After Viskochil et al (5)

### Molding

Two-in. diameter by 2-in. high specimens were molded at each density using a drop hammer molding apparatus (5). Compaction of 2-in. by 2-in. specimens to standard and modified Proctor densities with this apparatus has been correlated very closely with recognized laboratory compactive procedures (1, 5).

## Curing

Curing was accomplished in a humidity cabinet at approximately 70 F and near 100 percent relative humidity. After designated lengths of curing the samples were measured for height and weight.

TABLE 2

FIELD INFORMATION ON SOIL SAMPLES[a]

| | Kansas Sand | Iowa Silt | Texas Clay |
|---|---|---|---|
| Geological origin | Recent dune sand from the Great Bend tract | Wisconsin age loess from near Missouri River | Deltaic (Beaumont clay) from coastal plain |
| Soil Series | Pratt | Hamburg | Lake Charles |
| Horizon | C | C | C |
| Location | 28 mi. S. of Great Bend | In the town of Missouri Valley | South of Houston |
| Sampling depth, ft | $1\frac{1}{2}$-$3\frac{1}{2}$ | 49-50 | $3\frac{1}{4}$-12 (Composite) |

[a]After Viskochil et al (5)

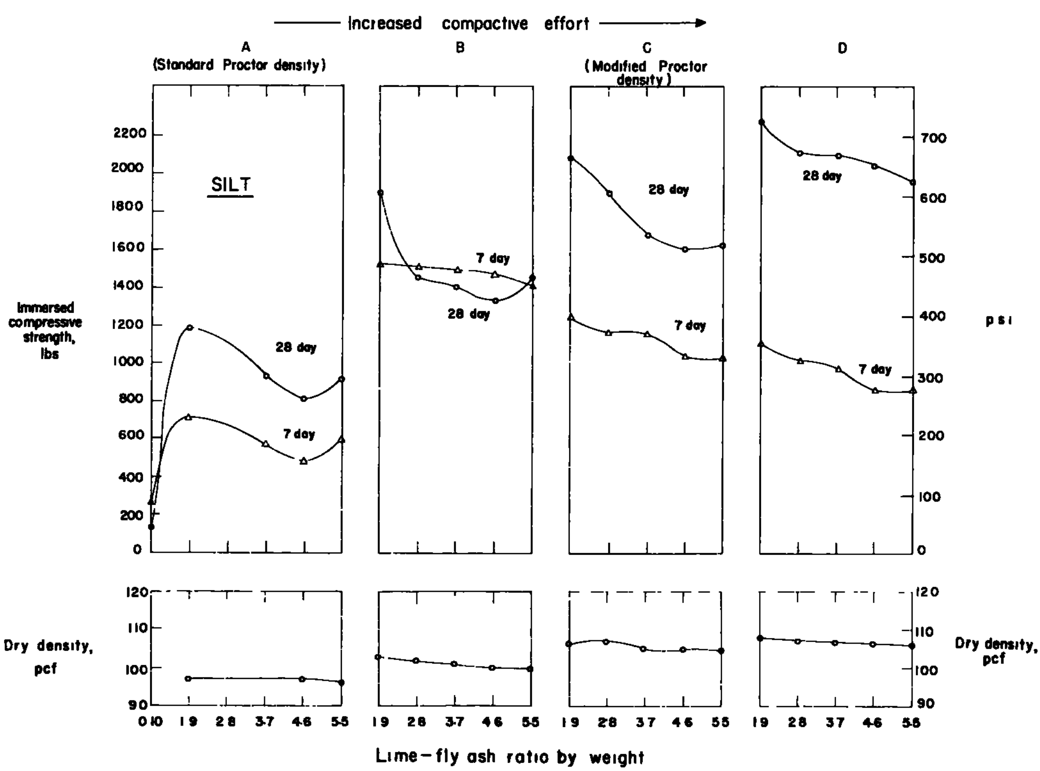


Figure 2. Effect of variations in lime to flyash ratio, compactive effort, and curing on compressive strength and density of lime-flyash stabilized silt.

TABLE 3

PROPERTIES OF SOIL SAMPLES[a]

| | Kansas Sand | Iowa Silt | Texas Clay |
|---|---|---|---|
| Textural composition, percent | | | |
| Gravel (> 2 mm) | 0 | 0 | 0 |
| Sand (2 - 0.074 mm) | 86.4 | 0.7 | 7.7 |
| Silt (74 - 5μ) | 4.0 | 78.3 | 48.2 |
| Clay (< 5μ) | 9.6 | 21.0 | 44.1 |
| Colloids (< 1μ) | 8.6 | 15.8 | 36.8 |
| Predominant clay mineral[b] | Montmorillonite | Ca montmorillonite | Ca montmorillonite |
| Specific gravity 25C/4C | 2.67 | 2.68 | 2.67 |
| Chemical properties: | | | |
| Cat. ex. cap., m.e./100 gm[c] | 7.3 | 13.4 | 25.5 |
| Carbonates, percent[d] | 0 | 10.5 | 0 |
| pH | 5.6 | 7.8 | 5.9 |
| Organic matter, percent[c] | 0.4 | 0.2 | 0.6 |
| Physical properties, percent : | | | |
| Liquid limit | - | 32 | 57 |
| Plastic limit | - | 25 | 20 |
| Plasticity index | NP | 7 | 37 |
| Shrinkage limit | 18 | 25 | 14 |
| Centrifuge moist. equiv. | 5 | 15 | 21 |
| Field moist. equiv. | 21 | 26 | 21 |
| Classification: | | | |
| Textural | Sand | Silty clay loam | Clay |
| Engineering (AASHO) | A-2-4(0) | A-4(8) | A-7-6(20) |

[a] After Viskochil et al (5)
[b] From x-ray and differential thermal analysis of whole soil
[c] Fraction passing No. 40 sieve
[d] From differential thermal analysis

## Wet-Dry Testing

The method of wet-dry test adopted was as follows:

1. Specimens were prepared at the designated density and optimum moisture content, then moist cured for fourteen days.
2. Specimens were air dried for 24 hours at room temperature and then completely immersed in distilled water for 24 hr. This completed one cycle of wetting and drying. Further cycles were a repetition of this step.
3. After designated cycles of wetting and drying specimens were wiped with a towel to a surface dry condition, measured for height and weight, and tested for unconfined compressive strength.

## Freeze-Thaw Testing

The method of freeze-thaw test adopted was as follows:

1. Specimens were prepared at the designated density and optimum moisture content. After moist curing for fourteen days, specimens were placed on ½-in. thick felt pads set in approximately ¼ in. of water.
2. Specimens on moist felt pads were placed in a freezer at - 10 F for 24 hr.

3. After removal from the freezer, specimens were allowed to thaw in open air at room temperature for two hours.

4. Specimens were placed in a humidity cabinet at approximately 70 F and 100 percent relative humidity for 22 hr. This completed one cycle of freezing and thawing. Further cycles were a repetition of steps 2, 3, and 4.

5. The specimens to be tested were measured for height and weight and tested for unconfined compressive strength.

## Compressive Test

After completion of curing and/or various cycles of wet-dry or freeze-thaw, all specimens were tested for unconfined compressive strength. The rate of deformation of the testing machine was held constant at 0.05 in. per min. per in. of specimen height.

## Absorption and Volume Change

The percentage of moisture absorbed during the wet-dry and freeze-thaw test was determined by subtracting the weight of the specimen after molding from the weight after immersion and dividing by the oven-dry weight of the specimen. Though the actual volume change was not measured, an easily determinable indicator of volume change was used—the average increase in height of specimens. This was determined by subtracting the height of the specimen after molding from the height after various cycles and dividing by the height after molding.

# EVALUATION OF TEST RESULTS

## Selection of Mixes

Research by Viskochil et al (5) indicates that increasing the density of lime-flyash stabilized soils greatly increases strengths, as shown in Figures 1, 2 and 3. It will be noticed that as lime content increases, the density and immersed compressive strength at each compactive effort in general tend to decrease, probably because of increased clay aggregation by the lime (5).

Figures 1, 2 and 3 indicate that optimum ratios of lime to flyash at densities greater

TABLE 4

PROPERTIES OF LIME AND FLYASH[a]

| | Linwood hydrated lime | Louisville Flyash |
|---|---|---|
| Specific gravity | 2.29 | 2.67 |
| Fineness | | |
| Percent passing No. 325 sieve | 99.00 | 94.30 |
| Specific surface, sq cm/gm | | 3470 |
| Chemical analysis, percent | | |
| Total $Ca(OH)_2$ | 97.82 | |
| Available $Ca(OH)_2$ | 97.38 | |
| MgO | 0.49 | 0.52 |
| $CaCO_3$ | 0.77 | 8.36 |
| Fe and Al oxides | 0.82 | |
| SiO | 0.80 | 38.90 |
| $Al_2O_3$ | | 22.92 |
| $SO_3$ | | 2.00 |
| Free water | | 0.17 |
| Loss on ignition | 24.56 | 2.10 |

[a]After Viskochil et al (5)

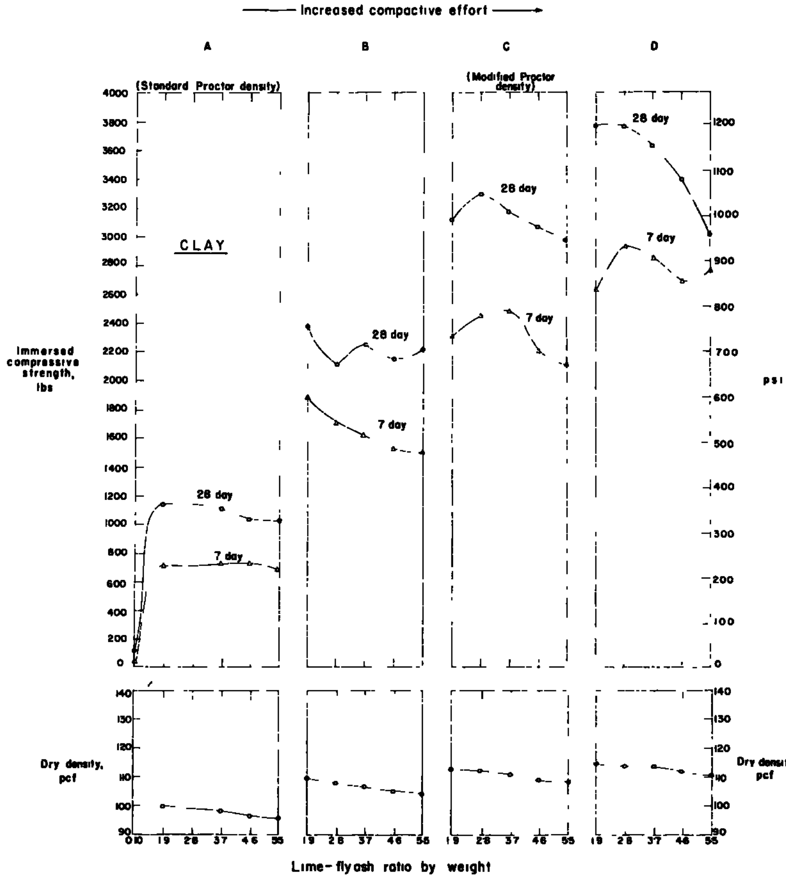


Figure 3. Effect of variations in lime to flyash ratio, compactive effort, and curing on compressive strength and density of lime-flyash stabilized clay.

than standard Proctor are 1:9 for the sand and silt, and 2:8 for the clay. These ratios were used in the wet-dry and freeze-thaw testing described in this paper. On the basis of previous studies, a mix with 25 percent lime-flyash was chosen as being satisfactory and economical. All test points were run in duplicate or in triplicate.

## Wet-Dry Tests

Silt. Wet-dry test results with silt are shown in Figure 4, along with freeze-thaw results with the same soil. The wet-dry cycles apparently cause a general increase rather than a decrease in strength. Similar trends have been reported elsewhere for lime and lime-flyash stabilized soils (2, 4). Strength curves in Figure 4 for different compactive efforts tend to diverge after 12 cycles, but this was accompanied by a similar divergence in the data for each point, indicating greater statistical error. Therefor no particular significance is attached to the upturn or downturn of the curves after 12 cycles. On a strength basis alone there is an advantage to compacting the silt to modified Proctor density (effort C), but the wet-dry tests show little benefit from compacting beyond this.

Curves for moisture absorption and expansion during wet-dry cycles are also shown in Figure 4. During early cycles all specimens absorbed water and expanded. Increased compaction reduced expansion but tended to increase the absorption of water, perhaps due to improved capillarity. After five cycles the absorption by specimens molded to compactive effort D is drastically reduced—this could be due either to reduced permeability or increased cementation tending to hold the specimen together.

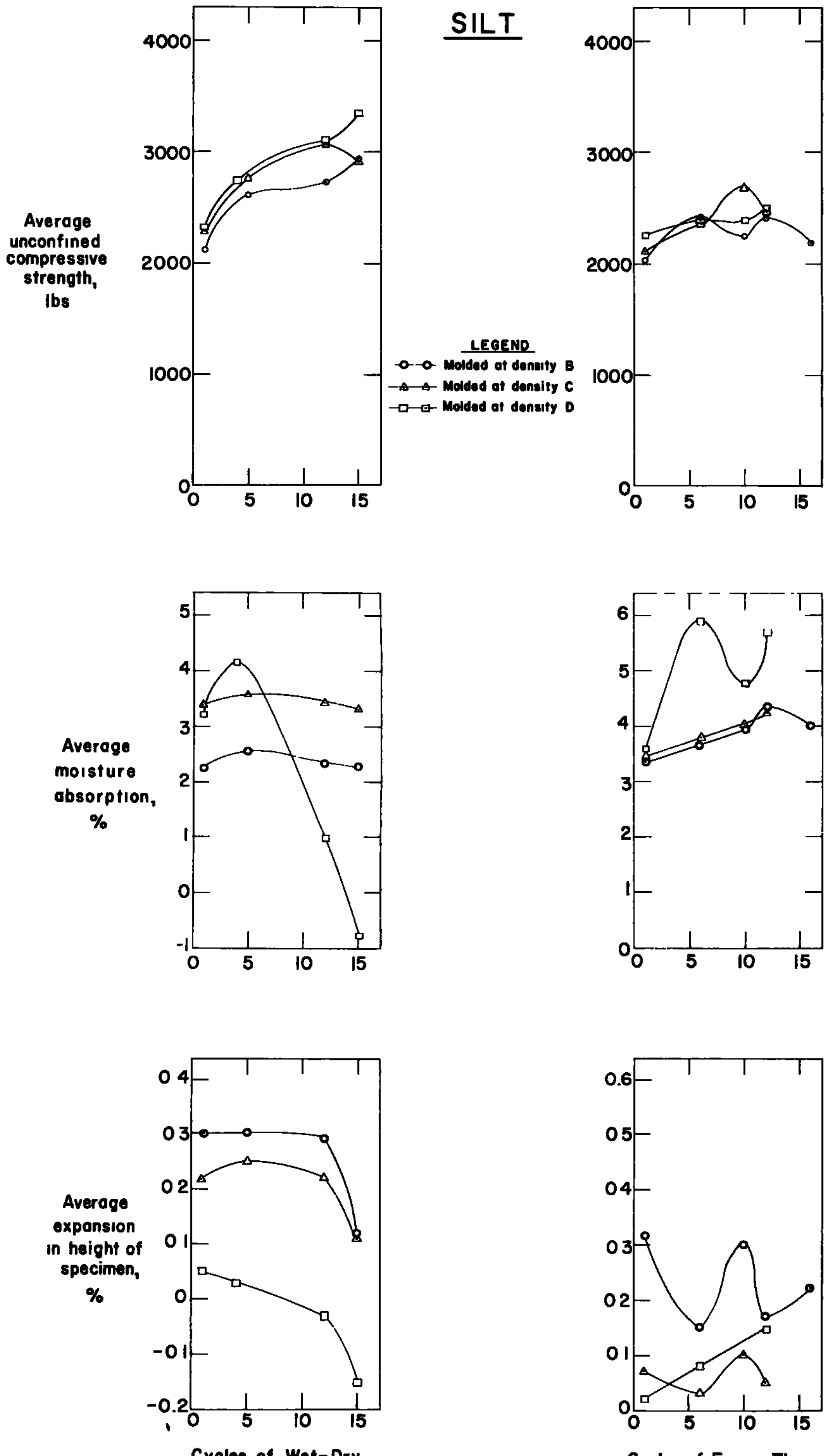


Figure 4. Effect of compactive effort, wetting and drying, and freezing and thawing on compressive strength, moisture absorption and expansion of lime-flyash stabilized silt.

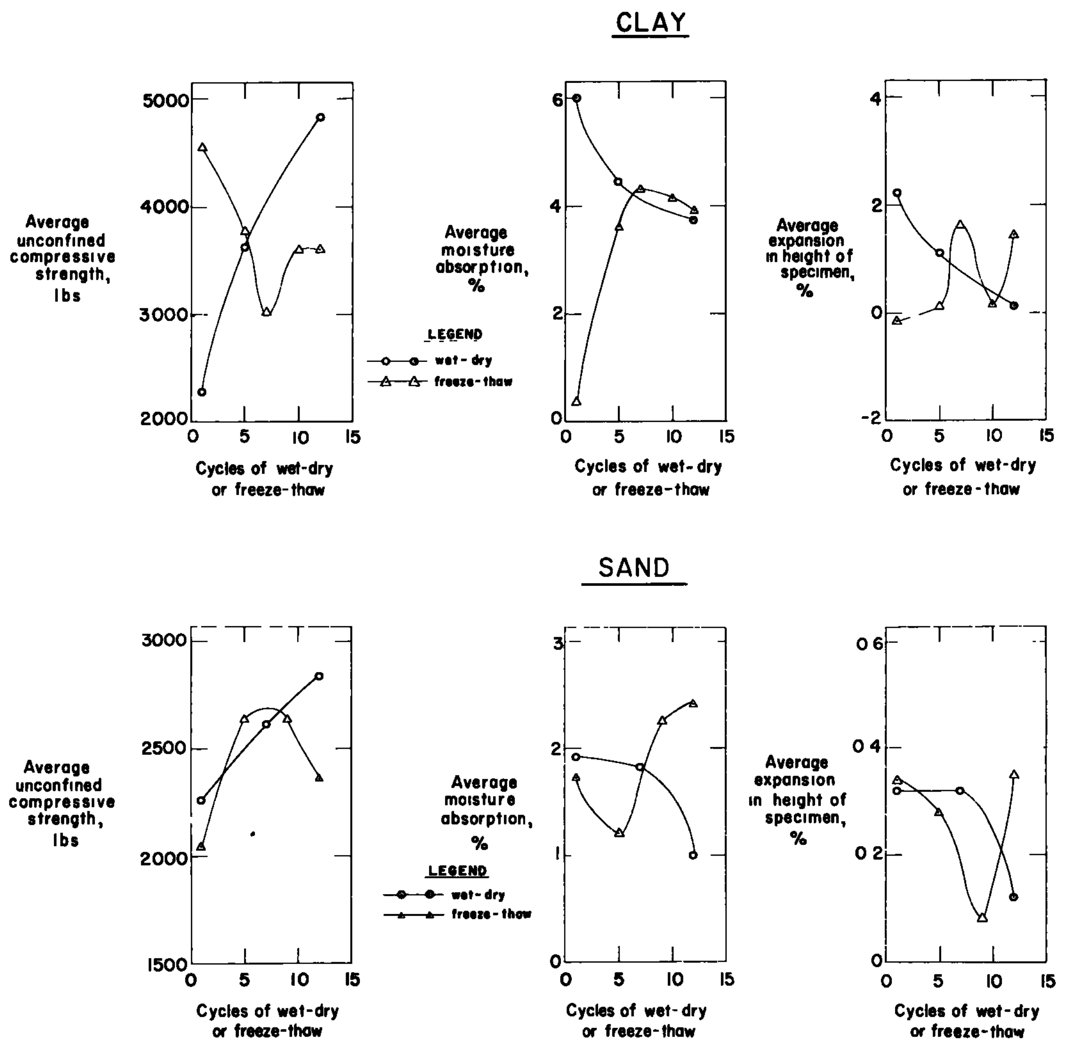


Figure 5. Effect of wetting and drying, and freezing and thawing on compressive strength, moisture absorption, and expansion of lime - flyash stabilized clay and sand compacted to modified Proctor density.

Since the latter effect does not appear in the compressive strength, one can conclude that pozzolanic reaction products may be plugging the pores or at least rendering them impermeable to water.

Clay and Sand Soils. The clay and sand were tested after compaction to modified Proctor density (effort C). Results are presented in Figure 5. Both clay and sand show a uniform increase in strength through the wet-dry cycles, and moisture absorption and expansion both are reduced.

Comparison to normal moist curing. Compressive strengths after normal moist curing are plotted in Figure 6 for the silt and Figure 7 for the sand and the clay. The silt and the clay are considerably benefited by wet-dry cycles, whereas the sand is not. Similar data by Goecker et al (3) show that clay and silt were benefited by prolonged soaking, but again the sand was not.

## Freeze-Thaw Tests

Silt. All silt specimens gained strength through the first 10 or 12 freeze-thaw cycles, then took the more logical trend downward (Fig. 4). A satisfactory resistance is in-

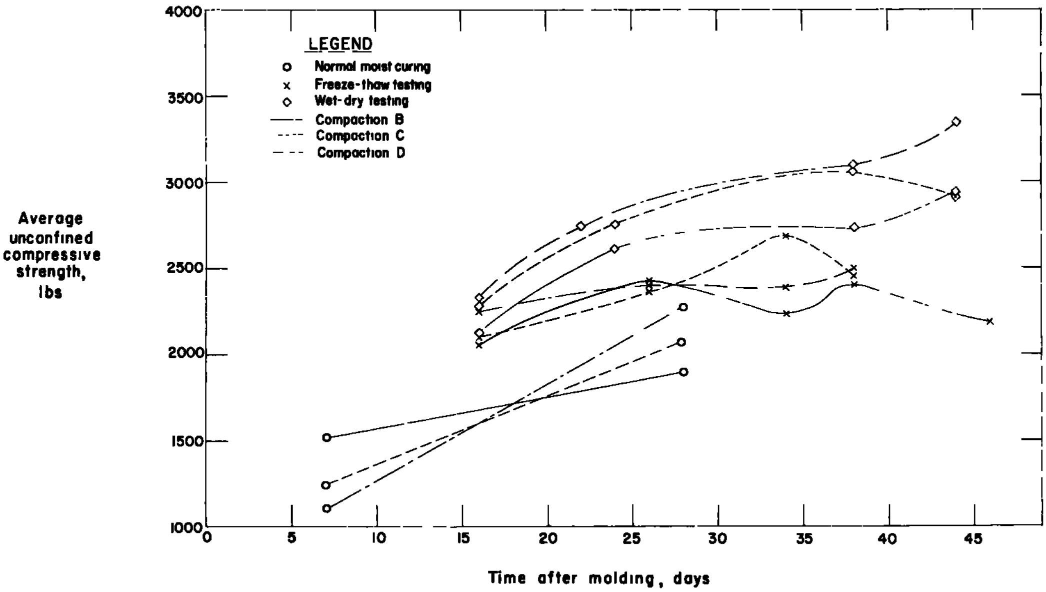


Figure 6. Relationship of effect of compactive effort, curing, wetting and drying, and freezing and thawing on compressive strength of lime-flyash stabilized silt.

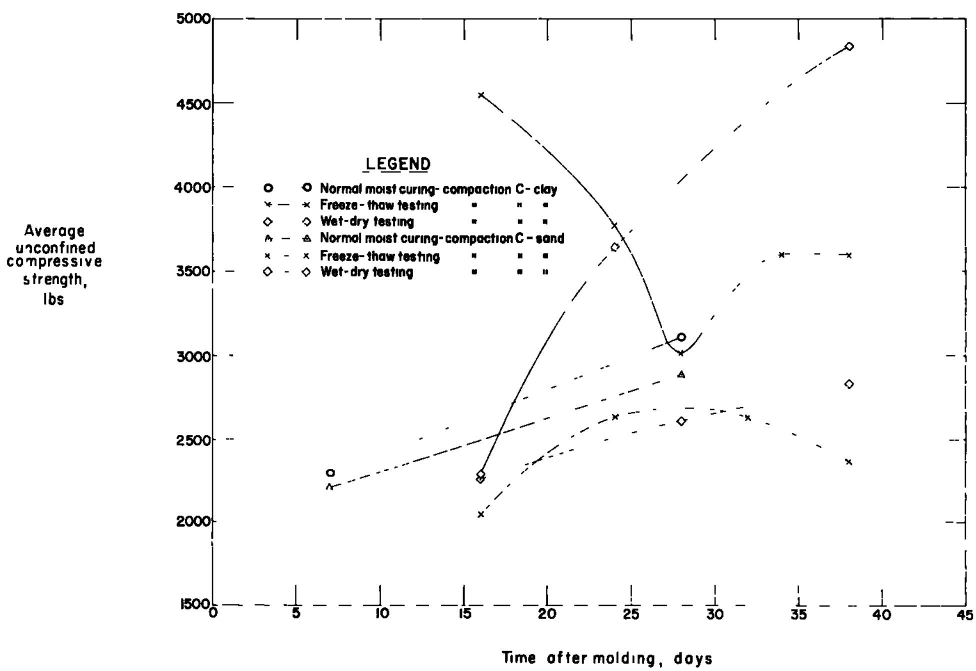


Figure 7. Relationship of effect of curing, wetting and drying, and freezing and thawing on compressive strength of lime-flyash stabilized clay and sand compacted to modified Proctor density.

dicated for all three compactive efforts, since the strengths after weathering are higher than those before weathering started. Moisture absorption trends upward, indicating progressive failure during freezing and thawing. Expansion remains small, but the silt compacted with effort D shows a gradual increase, suggesting overcompaction.

Overcompaction was previously noted for effort D with this soil, but had disappeared after 28 days normal curing (5, conclusion 3).

Clay and Sand Soils. Strengths of clay and of sand specimens during cycles of freeze-thaw correlate well with the moisture absorption (Fig. 5). The clay shows a drastic increase in absorption up to 5 cycles, after which the specimens slowly lose water. Strengths drop about 50 percent from the first to the fifth cycle, after which there is a slow gain. A slight volume expansion also takes place after the fifth cycle.

Curves for the sand are somewhat reversed to those for clay, but show the same relationships. Moisture absorption increases on the ninth cycle, and coincident with this the strengths go down. A sharp increase in volume is noted after the eighth cycle.

Comparison to Normal Moist Curing. Freeze-thaw cycles benefit the strengths of the silt and are somewhat deleterious to the strengths of the sand (Fig. 6 and 7). The same results were found with wetting and drying. The clay is uniquely benefited by freezing and thawing for one cycle. After this the strength progressively decreases until it approximates that obtained during normal moist curing; then strengths start back up.

## CONCLUSIONS

The obvious conclusion is that high density does improve durability of soil-lime-flyash, to the extent that after an initial moist cure, clay and silt soils gain strength even more rapidly during wet-dry or freeze-thaw cycles than they do in a continued moist cure. The sand soil gave comparable strength gains in either weathering cycles or moist cure. The comparatively high durabilities were realized by compacting to modified Proctor density. Previous studies showed the durability of soil-lime-flyash to be questionable after compaction to standard Proctor (3).

The uniqueness of a strength gain during a supposedly destructive testing program deserves more than a passing remark. Wetting and drying could result in periodic dissolution and redistribution of part of the lime, giving greater intimacy of contact and promoting the reactions with flyash. Prolonged soaking in water can apparently give the same mobility, as strengths are then high also (3). Similar results from lime stabilization indicate the importance of contact between lime and soil grains.

Beneficial effects of freeze-thaw cycles are more problematical and have not been noted before. The extreme case was with the clay, where strength was doubled by one cycle. After this, destruction started, but the trend again reversed after the fifth cycle. Benefits were less marked with the silt and nil for the sand, further emphasizing the importance of surface reactions not only with flyash, but also with soil. In dolomitic lime, the solubility of $MgO$ and $Mg(OH)_2$ increases with a rising temperature whereas the solubility of $Ca(OH)_2$ decreases. Therefore, a critical redistribution of lime may result from a single freeze-thaw cycle. The redistribution is apparently of lesser importance with coarse-grained soils, which have lower surface area for cementation reactions.

## ACKNOWLEDGMENTS

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Special acknowledgment is also given Capt. R.H. Viskochil, U.S. Corps of Engineers and R.J. Leonard, former graduate students of Civil Engineering, Iowa State College, for their assistance in conducting the testing phases of this investigation.

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# Structural Properties of Lime-Flyash-Aggregate Compositions

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● LIME-FLYASH-AGGREGATE compositions in highway and airfield base courses and highway shoulders have been in existence for periods up to seven years. Reports have been made previously on the properties, construction, and performance of these compositions. It is the purpose of this paper to present additional information concerning these materials with particular regard to their structural properties or their load transmission characteristics, as measured by triaxial and CBR tests. The paper contains a discussion of the use of the CBR method for designing lime-flyash-aggregate base courses for highways and airfields. A further evaluation of existing installations is presented. Also included is a comparison between lime-flyash-aggregate compositions and lime-soil mixtures.

Previous work (1) has shown that the magnitude of the pressure transmitted through a base course is dependent on the materials used in the base course. Smaller pressures occurred beneath good base and subbase materials than occurred beneath poor materials. It was further illustrated that the relative effectiveness of the materials could be predicted by means of the triaxial test. It is logical to assume that this same qualitative relationship could be determined by other tests such as the CBR and stabilometer. Previous work (2) has also shown that the curing conditions, particularly those involved with wetting and drying, have a substantial effect on the properties of the compositions. Considerations have been given to the use of various methods of measuring the durability of lime-flyash-aggregate mixtures when subjected to freezing and thawing tests at early ages.

A broad analysis of the use of lime-flyash and aggregate mixtures has indicated that in many instances the compositions possess characteristics differing substantially from the products that are formed in typical soil stabilization operations. In order to evaluate the engineering features of lime-flyash and aggregate mixtures, a program has been established which is concerned with both the development of adequate test procedures and the utilization of the data for design purposes. This program includes studies involving the effect of aging of the compositions under various conditions of wetting and drying, high or low humidity, and a study of the resistance of the compositions to freezing and thawing also under varying curing conditions. Furthermore, an effort is being made to evaluate the proportions and properties of the aggregates and aggregate mixtures which are effective in producing compositions of the desired properties.

The work which is presented in this report covers only the initial phase of the investigation and describes tests which have been made on specimens that have been tested at early ages. It includes only the work done with mixtures of lime and lime-flyash with natural soils and does not include data on the modification of the natural aggregate with supplementary aggregate material to improve gradation. A paper will be presented later describing some of these other studies which are currently underway.

The initial laboratory work reported is considered to be useful for establishing the relative effectiveness of base courses of lime-flyash-aggregate compositions over those of natural soil for reducing the amount of pressure transmitted to the subgrade. The data presented give comparative results of triaxial and CBR tests performed on the compositions. It is to be emphasized that the evaluation of the lime-flyash-aggregate base material has been carried out before any pozzolanic reaction had taken place.

## TESTING PROCEDURE

### Triaxial Test

The triaxial tests were performed on an A-4 silt and on a mixture of lime and flyash with the A-4 soil as an aggregate. The soil is a type found in widespread areas in the Commonwealth of Pennsylvania. It is characterized by its relative fineness (64 percent through the No. 200 sieve, in this case) and its moderate to low plasticity. The composition was a mixture of 90 parts of the A-4 soil, 10 parts of fly ash, and 5 parts of hydrated lime, by weight.

Three series of tests were performed. Each series differs from the others only in the length of time allowed for saturation. The first series of tests was performed after the samples were completely saturated. The time required for 100 percent saturation was rather long (from 5 to 7 days), and the results with these test cylinders indicated that some cementing action had taken place during this period. Since it was desired to evaluate the mixtures before any appreciable pozzolanic reaction had occurred, two other series of tests were then run; one after one day of saturation and one after two days of saturation.

All test cylinders were 1.4 in. in diameter and 2.8 in. high. They were carefully molded at optimum moisture by static compaction in a split mold. The static load was held on each sample until it was felt that a uniform density was obtained. Two different densities were used for the cylinders that were 100 percent saturated. Some of the cylinders were at standard AASHO density and some were at a density determined by a compaction test in which the 10 lb. rammer was substituted for the 5.5 lb. rammer. All the cylinders for the one and two day tests were made using the greater density. The samples were not cured except during the saturation period.

The triaxial tests were run to failure at lateral pressures of 2.5, 5.0, and 7.5 psi. The rate of loading was .015 in. per min.

### California Bearing Ratio

Comparative CBR tests were performed on four different soils; on the lime-flyash-aggregate mixtures using the four soils as aggregates and on lime-soil mixtures. Of the four soils used, two were A-4 and two were A-2-4. In each case, the composition consisted of 90 parts of soil, 10 parts of flyash, and 5 parts of hydrated lime, by weight. Three percent lime, by weight, was added to the soil in the lime-soil mixtures.

The penetration tests were performed on samples that were compacted to 100 percent of CBR density, soaked for four days, and drained for 30 min prior to testing. The loading rate was .05 in. per minute.

The properties of the soils used in both the triaxial and CBR tests are shown in Table 1.

## TEST RESULTS

### Triaxial Tests

An examination of Figure 1 shows that the vertical pressures at failure after one and two days of saturation, of the lime-flyash-aggregate compositions were of a magnitude greater than those of the raw soil cylinders at the same lateral pressures. The additions of lime and flyash to the A-4 soil changed the shearing properties. Failure of the natural soil was evidenced by a uniform bulging of the cylinders while failure of the compositions was evidenced by a shearing failure along a single plane. The inconsistencies which are apparent in the vertical pressures at failure shown in Figure 1 for varying lateral pressures on supposedly identical test cylinders are believed to be due to the difficulty of reproducing exact moisture conditions short of 100 percent saturation. Regardless of these inconsistencies, the relationship between lime-flyash mixtures and the natural soil cylinders is clearly shown.

Figure 2 shows the stress-strain curves that were derived from triaxial tests performed on 100 percent saturated cylinders. The test cylinders were molded and cured under different conditions and tested at various lateral pressures, as shown in Table

TABLE 1

PROPERTIES OF SOILS USED IN INVESTIGATION

| | Triaxial Test | California Bearing Ratio Test | | | |
|---|---|---|---|---|---|
| Soil Number | CT-2 | CB-7 | CB-8 | CB-9 | CB-10 |
| Mechanical Analysis Percent Passing: | | | | | |
| ⅜ in. | 100.0 | 100.0 | 100.0 | 100.0 | - |
| No. 4 | 99.0 | 99.5 | 100.0 | 98.7 | 86.3 |
| No. 10 | 96.8 | 99.4 | 100.0 | 95.4 | 80.1 |
| No. 20 | 91.9 | 97.0 | 99.5 | 85.9 | 67.0 |
| No. 40 | 86.8 | 91.0 | 92.9 | 75.0 | 54.5 |
| No. 60 | 82.3 | 86.9 | 76.2 | 68.7 | 47.7 |
| No. 140 | 70.1 | 78.9 | 15.7 | 57.0 | 33.6 |
| No. 200 | 63.7 | 73.6 | 12.9 | 51.9 | 26.8 |
| Liquid Limit | 34 | - | - | 34 | 28 |
| Plasticity Index | 5 | NP | NP | 8 | 4 |
| Standard AASHO Density | 112.0 | 104.3 | - | - | - |
| Optimum Moisture | 17.8 | 17.8 | - | - | - |
| CBR Density | - | - | 123.2 | 113.0 | 132.1 |
| Optimum Moisture | - | - | 11.6 | 16.0 | 11.0 |
| BPR Classification | A-4 | A-4 | A-2-4 | A-4 | A-2-4 |

TABLE 2

MOLDING AND TESTING CONDITIONS FOR TEST CYLINDERS OF FIGURE 2

| Curve Designation | Composition of Cylinder | Density | Curing | Lateral Pressure - psi |
|---|---|---|---|---|
| A | Natural soil | Standard | None | 2.5 |
| D | Natural soil | Modified | None | 2.5 |
| E | Natural soil | Modified | None | 5.0 |
| F | Natural soil | Modified | None | 7.5 |
| U | Lime, flyash, soil | Standard | 7 days at elevated temp. | 7.5 |
| W | Lime, flyash, soil | Modified | 7 days at elevated temp. | 5.0 |
| Y | Lime, flyash, soil | Standard | None | 5.0 |
| Z | Lime, flyash, soil | Standard | None | 7.5 |

2. An approximate average for the saturation time for the test cylinders was seven days. The curves fall into two distinct groups. The stress-strain curves for the lime-flyash cylinders are grouped along the vertical axis and those for the natural soil cylinders are along the horizontal axis. It is evident that the vertical pressures at failure are far greater for the lime-flyash mixtures than for the natural soil.

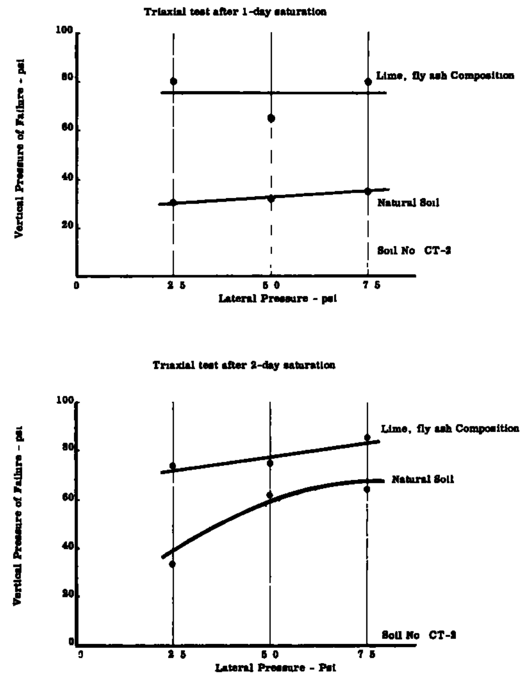



Figure 1. Relationship between vertical and lateral pressure.

California Bearing Ratio Tests

The CBR values obtained using standard test procedures are shown in Figure 3 and 4. Considerable improvement can be noted when additions of lime and flyash or lime only are made to the natural soil. Since these samples remained completely submerged in water from the time they were molded until shortly before the penetration was performed, it is felt that little or no pozzolanic reaction occurred.

Discussion of Test Results

The results obtained in this investigation again indicate the immediate improvement in soil properties brought about by additions of hydrated lime and flyash. The comparative triaxial tests show that the compositions of lime-flyash if used as a base course would be more effective than the natural soil in distributing the load over the subgrade. This superiority shows up as soon as one day after molding. The effectiveness of the composition increases with time even though adverse moisture conditions are encountered. The vertical pressures at failure of the 100 percent saturated cylinders, which were tested approximately seven days after molding, ranged from two to four times as much as the vertical pressures for one and two day cylinders. Some difficulty was encountered in reproducing accurately the results in the triaxial test. It is felt that, unless further investigation should produce a technique for improving the reproduceability of results, the triaxial method may not be as suitable for base course design for lime-flyash compositions as in the CBR method.

In three of the four cases tested in the CBR investigation the values of 168, 190, and 151 show that the compositions would be suitable for use in a road base immediately below the wearing course. The CBR tests on the compositions were found to be reproducible and easily performed. Figure 5 shows a typical stress-strain curve for one of the A-4 soils used in the investigation together with the curves for the lime-flyash and the lime-soil mixture.

## DESIGN METHOD FOR LIME-FLYASH-AGGREGATE COMPOSITIONS

It is proposed that the CBR design method for flexible pavements be used to determine the thickness of the lime-flyash-aggregate base required under given conditions of traffic volume, load, and subgrade conditions. In designing a flexible pavement, it is usually desired to evaluate the components of the cross-section under the most adverse conditions that occur in the actual road. With lime-flyash this would involve subjecting the material to saturating conditions immediately after it has been placed in the road. At the end of four days of soaking, with no initial curing, it is felt that the lime-flyash-aggregate mixture would be in the most critical physical state that it would encounter during its life in the pavement. A design thickness based on its prop-

TABLE 3

DESCRIPTION OF FIVE MAJOR LIME, FLY ASH, AGGREGATE INSTALLATIONS

| LOCATION | TYPE OF CONSTR. | DATE CONSTR. | TRAFFIC | | | WEARING SURFACE | | BASE | | | SUBBASE | | | SUB-GRADE |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Type Vehicle | Weight Vehicle | Density | Type | Thick-ness | Compo-sition | Aggre-gate | Thick-ness | Compo-sition | Aggre-gate | Thick-ness | |
| Camden N.J. | City Street | July 1954 | 97% Passenger 3% heavy truck | --- 22000# Single axle | ADT 5000 (1954) | Bit. Conc. | 2" | Lime, Fly Ash, Aggregate | A-1-b | 6" | ---------- | ---------- | ------ | A-1-b |
| Wings Field, Pa. | Runway Taxiway Parking area | Aug. 1954 | Aircraft | 12000# Single wheel load | 52000 move-ments per year | Asphalt and stone chips | --- | Lime, Fly Ash, Aggregate | A-5 + coarse agg. | 6" | Lime Stabilized | A-5 | 6" | A-5 |
| New Castle, Del. | Auto Parking area | Aug. 1956 | Passenger cars | --- | --- | Bit. Conc. | 2" | Lime, Fly Ash, Aggregate | A-4 + coarse agg. | 6" | Lime Stabilized | A-4 | 3" to 4" | A-4 |
| International Airport Phila., Pa. | Auto Parking Area | Oct. 1953 | Passenger cars | | 1500 cars daily | Bit. Conc. | 2" | Lime, Fly Ash, Aggregate | A-1-b | 6" | Compacted Gravel | | | A-1-b |
| Salem, N.J. | County Road | Sept. 1956 | Passenger and Trucks | 22,000# Single axle | ADT 1650 (1957) | Bit. Conc. | 1-5/8" | Lime, Fly Ash, Aggregate | A-1-b | 6" | ---------- | ---------- | ------ | A-1-b |

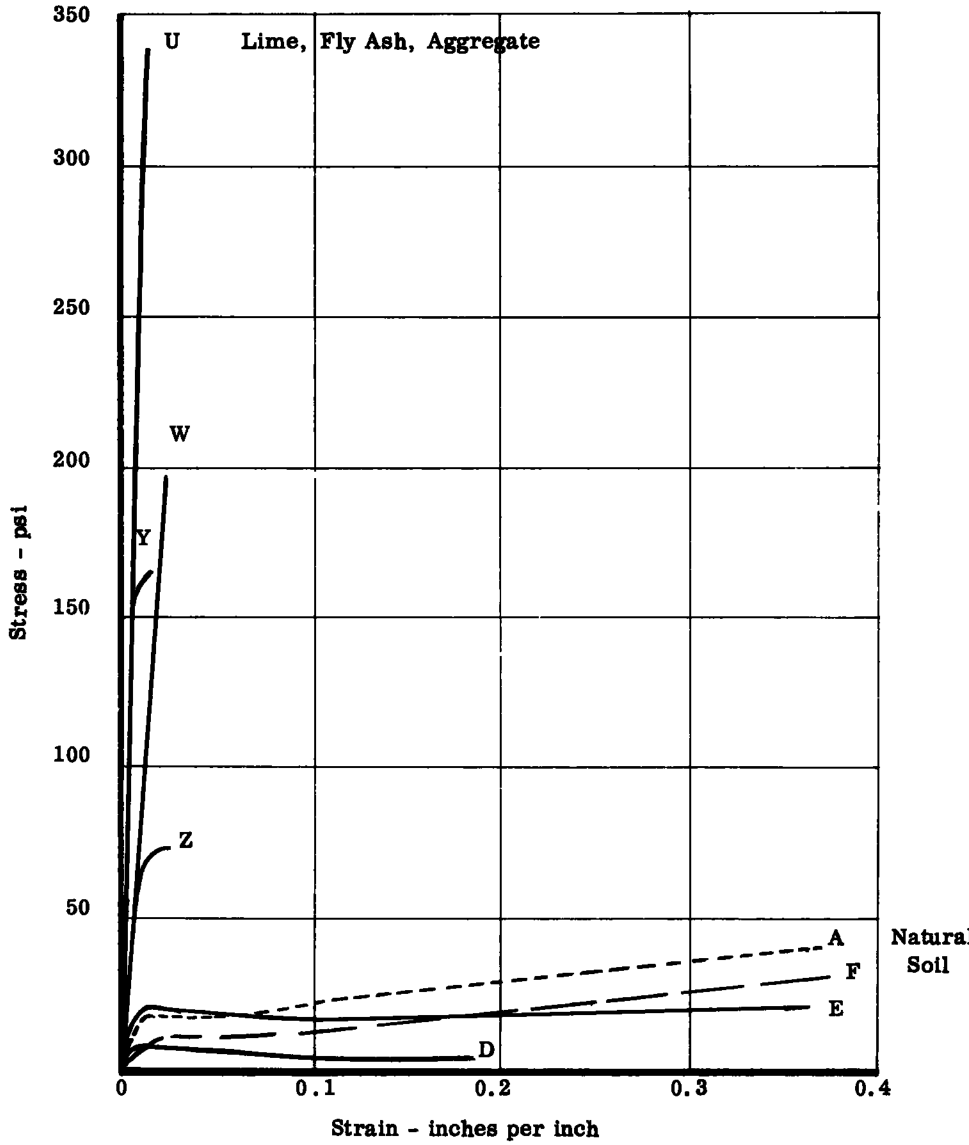



Figure 2. Stress-strain curves for triaxial test, samples 100 percent saturated, Soil No. CT-2.

erties at this critical time should be amply safe. The question now arises as to how the cementing action, which occurs in the mixture affects its performance in a road designed as a flexible pavement.

First, let us briefly review the strength characteristics of lime-flyash-aggregate compositions as they are known at this time. Ordinary compression tests on cubes and Proctor size cylinders prepared in the laboratory have shown compressive strengths ranging between 200 and 1,900 psi. Samples removed from a highway shoulder this past summer, three years after construction of the shoulder, gave compressive strengths as high as 3,360 psi. when broken in an oven dry condition. Within the past two years, flexural tests have been performed on lime-flyash aggregate beams (3). The aggregate used in the beam was an A-2-4 silty sand. Tests with this particular aggregate have indicated a relatively low strength in flexure. The estimated tensile strength at

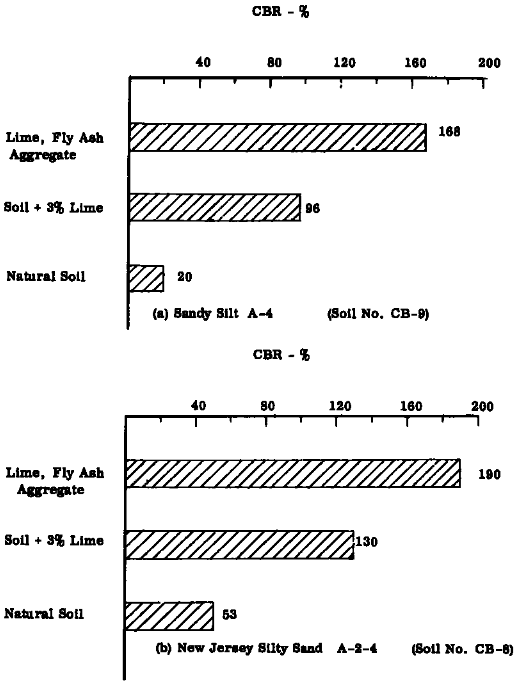



Figure 3. Comparative results of CBR tests. All samples were soaked for four days before testing.

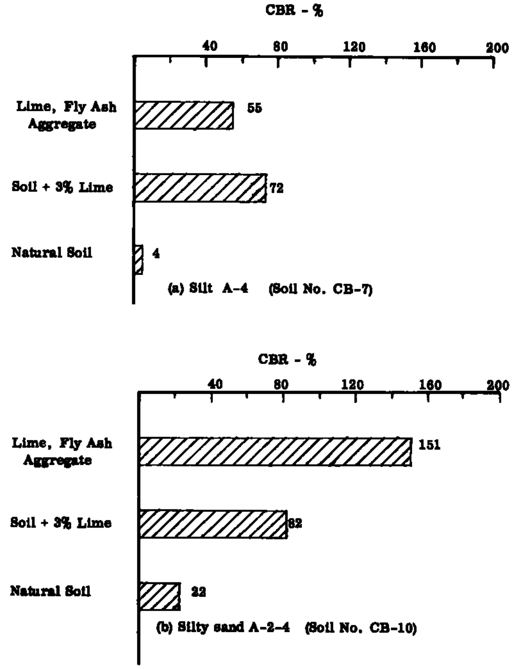



Figure 4. Comparative results of CBR tests. All samples were soaked for four days before testing.

failure was found to be 15 to 20 psi. The compressive strength of these mixtures was approximately 500 psi. Additional testing of the flexural strength of lime-flyash-aggregate compositions, to clearly define this property, is warranted.

The significance of a compressive strength value in connection with a road base material is questionable. An increase in compressive strength in any given material does, however, indicate an increase in cohesion. Strength in compression is also an indication that the individual particles which make up the composition are cemented togather. The permanence of this bonding is dependent upon the leaching effects caused by weathering forces such as ground water, freezing, etc. The ability of a material to resist shearing stresses is increased by increased cohesion and increased bonding of its particles. It would seem, therefore, that its ability to distribute a load over a considerable area would also be improved. Compressive strength is then, at least, a qualitative index of the load transmission characteristics of a base material.

In lime-flyash-aggregate compositions the individual particles are more or less cemented together, but the compositions at early ages apparently possesses low strength in flexure. The lack of any significant strength in bending would insure action as a flexible base rather than a rigid one.

The concept of designing a structure using a material which will be many times stronger as its age increases is a rare and in many ways comforting situation. The uncertainties involved in the structural design of a highway are well known. One of the least predictable factors is the amount, type, and weight of traffic that a road will be required to carry during its lifetime. The ability of lime-flyash-aggregate compositions to become stronger over a long period of time tends to compensate for the possibility of the road having to carry more severe loads then those for which it was designed.

Two simple examples will show clearly the proposed application of the CBR method to the design of a flexible pavement with a lime-flyash-aggregate base and/or subbase.

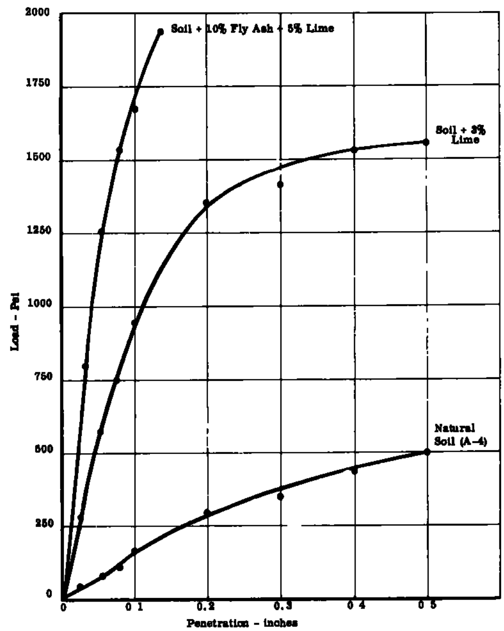



Figure 5. Stress strain curves, CBR test, Soil No. CB-9.

Example 1. CBR of subgrade = 4 percent; maximum single-axle load = 22,000 lb; traffic—heavy.

Using design curves of the Asphalt Institute ("Thickness Design, Flexible Pavements for Streets and Highways," The Asphalt Institute, Fig. 2, pp. 14 and 15) the total thickness of flexible pavement would be 15 in. A lime-flyash-aggregate composition of the subgrade soil with 10 percent flyash and 5 percent lime would furnish a CBR value of 55 percent. This would be suitable for a subbase. A 7-in. layer of this material would leave 8 in. for base and wearing course. A small percentage of screenings or coarse aggregate added to the lime-flyash-aggregate of the subbase would produce a mixture acceptable for the base (CBR 80 percent or better). A 5½-in. lime-flyash-aggregate base and 2½-in. course of asphaltic concrete would complete the cross-section as shown in Figure 6.

Example 2. CBR of subgrade = 3 percent ; maximum single-axle load = 8,000 lb; traffic—light.

The design curves show a thickness of 6 in. of flexible pavement is needed. A 4-in. layer of a lime-flyash-aggregate composition with a CBR of 80 percent or better and a 2-in. layer of asphaltic concrete will provide this thickness. For the subgrade material in this example a suitable composition might consist of 60 percent of the subgrade soil, 30 percent gravel or sand, and 10 percent flyash with a 5 percent lime additive. The cross-section is shown in Figure 6.

## Evaluation of Existing Installations

Data are given in Table 3 on five major lime-flyash-aggregate installations. These jobs were constructed between 1953 and 1956. The types of installations represented here are varied. Two are automobile parking areas, one an urban street, one a rural road, and one a runway, taxiway, and parking aprons at a commercial airport for light planes.

One of the more interesting projects has been the runway and taxiway at Wings Field, Pa. The runway was constructed in August 1954. The subgrade soil is an A-5. A 6-in. base was constructed on top of a 3 to 4 in. subbase of lime stabilized soil. A 1-in. asphalt wearing surface was provided. Thus, the total thickness of flexible base that was provided was between 10 and 11 in.

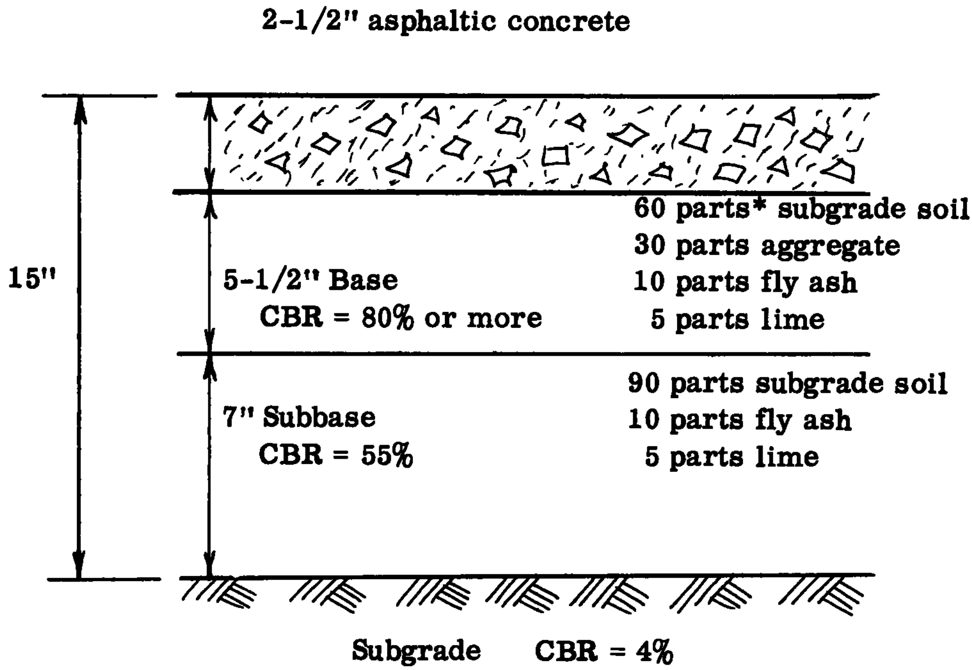
The traffic at Wings Field consists primarily of light and medium planes with maximum single-wheel load of 12,000 lb (Fig. 7). An estimated total at 52,000 plane movements are made per year. A takeoff or landing constitutes one plane movement.

An estimated value of the CBR for the A-5 subgrade soil is 7 percent. Using this value and the CBR design charts developed by the U.S. Corps of Engineers for taxiways and runways, the thickness of flexible pavement required would be approximately 13 in. for a taxiway and 12 in. for a runway.

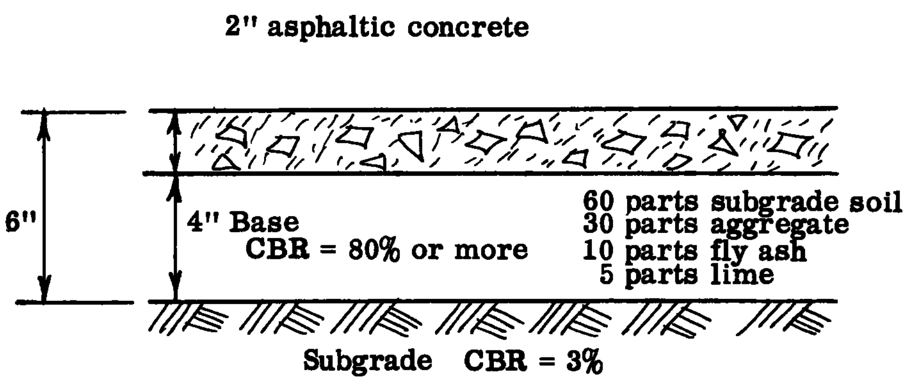
The existing thickness of 10 to 11 in. has been in use for more than three years and continues to perform satisfactorily. A recent visual inspection of the site showed the runway to be in excellent condition (Fig. 7).

Another lime-flyash aggregate base was constructed in the summer of 1956. This was the base for a 3½-mi section of two lane county road in Salem Co., New Jersey. The average traffic volume for this highway in 1957 was 1,650 vehicles per day. The

Example (1). Heavy Traffic, 22,000 lb. axle load



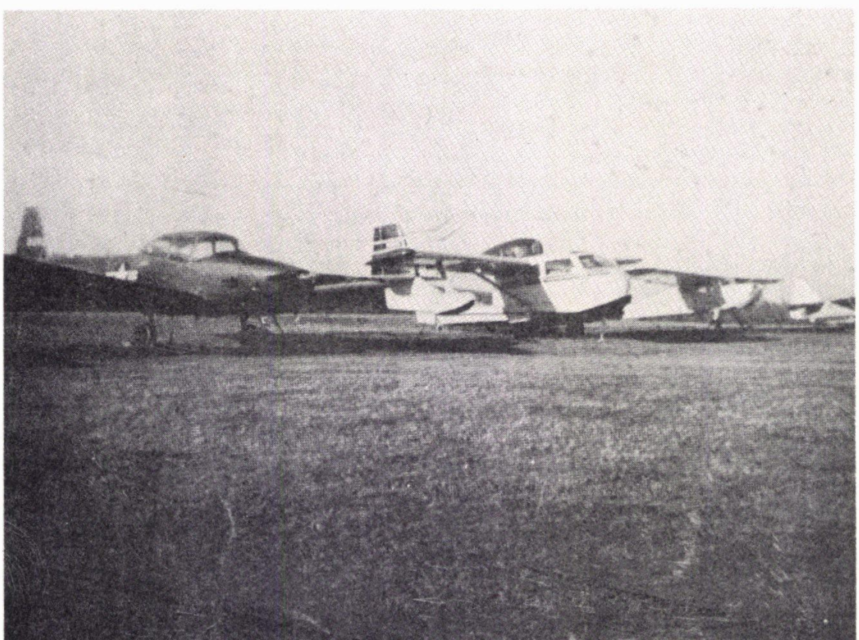

Example (2). Light Traffic, 8,000 lb. axle load




*Proportions of compositions are by weight.

Figure 6. Typical cross-sections using lime-flyash-aggregate.

(A)



(B)

Figure 7. (A) Runway at Wings Field, Pa.; (B) Typical aircraft using runway at Wings Field, Pa.

traffic is mixed passenger cars, light trucks, and heavy trucks and would be classified as "heavy". The recommended minimum thickness of flexible base and wearing course is 8 in. for this case according to the criteria of The Asphalt Institute. The actual thickness used was between 7½ and 8 in. and consisted of 6 in. of lime flyash and 1½ to 2-in. of asphaltic concrete. The subgrade soil is an A-1-b. After more than one year of service, this highway is in excellent condition with no evidence of deterioration.

The other three projects are also subjected to heavy traffic volumes. A comparison of the pavement thicknesses actually being used with that required by a CBR design would show that the actual would be slightly less than the required. They continue to give satisfactory service after three and four years.

## COMPARISON OF LIME-FLYASH-AGGREGATE COMPOSITIONS WITH LIME-SOIL MIXTURES

The fundamental difference between the product of lime soil stabilization and lime-flyash-aggregate compositions is in the cementing action or pozzolanic activity. It is well known that straight lime additives to many soils will enhance their properties which are of importance as far as their suitability as construction material. The action of the lime on the soil is virtually immediate although some cementing effects can be developed later as a result of recrystallization and carbonation of the hydrated lime. It is doubtful that any significant pozzolanic reaction occurs between lime and natural soils. The pozzolans which are produced in nature are usually of volcanic origin although methods have been evolved to process certain select soils, such as shale, by calcination and thereby impart pozzolanic properties to the soil. It has been reported that additions of lime to clay soils have reduced high plastic indexes to a more reasonable value. The supporting power of the soils, as measured by CBR test, has been effectively increased. Numerous field installations, which have been in existence for many years, have performed in a more satisfactory manner than their natural soil counterparts. In essence the addition of lime to fine-grained soils, particularly clay, has transformed the properties to those more nearly of a coarse-grained material. In so doing many separate benefits are realized. The effect of additions of lime is a nearly complete reaction almost immediately except in those soils which contain additions of pozzolanic materials. In those cases there is substantial evidence of a continued cementing process which occurs over a long period of time. The dependence of this long range benefit on the presence of pozzolanic materials with the lime is one of the basic reasons for adding flyash. The addition of flyash insures the presence of a pozzolan in the mixture and thus insures that the mixture will increase in strength over a long period of time. Compressive strengths of samples removed from various installations show increases in strength over periods as long as three years. In two of the three cases analyzed the strength of the compositions has increased.

The comparative CBR tests performed during this investigation show a higher ratio for the lime-flyash mixtures than the lime-soil mixtures in two cases and just the reverse for one other case. It is to be pointed out that the CBR values of the mixtures may be considerably improved after an extended period of aging under wetting and drying conditions. However, the results are significant in that they indicate the abilities of the mixtures to develop considerably improved bearing characteristics over that of natural soil. The composition in which A-4 silt is used requires addition of supplementary aggregate materials to develop optimum bearing power. Experience has shown that where this is done, particularly with lime and flyash, formulations are possible in which heavy proportions of silt may be used in the final compositions.

The results of this study confirm the theory that lime, flyash, aggregate compositions are superior base materials even before any pozzolanic set has occurred. It is felt that pavement designs using lime-flyash aggregate can be made with confidence by the CBR method. Existing installations continue to perform satisfactorily and indications are that they are still gaining strength as predicted.

## ACKNOWLEDGMENT

The support of this investigation by G. & W.H. Corson, Inc., Plymouth Meeting, Pa. is gratefully acknowledged.

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2. Minnick, L. John and Williams, Ralph, "Field Evaluation of Lime - Flyash - Soil Compositions for Roads," Highway Research Board Bulletin 129.
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# Reactivity of Four Types of Flyash with Lime

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The variables of flyash that affect the pozzolanic reaction between lime and flyash were studied by means of unconfined compressive strength. The samples used in this study were molded from various mixtures of lime, four types of flyash, and water. All specimens were moist cured for a specific period prior to testing.

The pozzolanic activity of flyash was found to be dependent upon its carbon content and its degree of fineness. The rate of the pozzolanic reaction was considerably influenced by the conditions of temperature and humidity under which the samples were cured. The study also revealed that the unconfined compressive strength increases with increased lime contents.

● A POZZOLAN is defined in ASTM Standard Definitions of Terms Relating to Hydraulic Cement as a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties (1). Although the principle products of the reaction between a pozzolan and calcium hydroxide are considered to be calcium silicates and aluminates, there is some evidence that more complex compounds are formed.

The use of a pozzolan with lime to produce cementation has been known since the time of the early Roman Empire. The Romans utilized the pozzolanic action of volcanic ash with calcined limestone in the construction of such historic landmarks as the Appian Way, the Colosseum and the Pantheon. Since then various other natural substances and some artificially produced materials have been found to possess various degrees of pozzolanic activity (5).

Flyash is an artificial pozzolan which results from burning pulverized coal. The coal, of which about 80 percent passes a No. 200 sieve, is blown into a furnace with primary air, and the combustion of the organic material in the suspended particles occurs almost instantly. The unburned inorganic materials form minute molten globules at a temperature of approximately 2,800 F. These globules congeal into spherical particles about 75 microns in diameter as they leave the zone of high temperature (5, 6). Some partially burned organic particles result and are of a more irregular shape and are somewhat larger. These particles are considered to be mostly carbon. After passing through the super heater, economizer and preheater, the ash (containing both incompletely burned and unburned particles) is separated from the exhaust gas stream by various methods. Collection of flyash in stack gas is usually accomplished through the use of electrical precipitators or mechanical collectors (8).

The design of power plant boilers generally falls into three basic categories: dry bottom, wet bottom and cyclone. The total ash produced in the operation of dry bottom boilers is approximately 90 percent flyash; the remaining 10 percent consists of larger particles (bottom ash) which fall out by gravity. Wet bottom boilers produce about 50 percent of total ash as flyash, whereas cyclone equipment produces only about 15 percent as flyash (8).

Electrical precipitators are more efficient than mechanical collectors and usually remove a higher percentage of the flyash from the flu gases. Flyash collected by electrical precipitators contains a high percentage of fine particles and therefore has a high specific surface which is considered conducive to high pozzolanic activity (3, 4, 9).

Each power plant produces flyash of a relatively different character, that is, it varies in particle size and chemical composition. These variations are due to the type of

TABLE 1

SOURCE AND COLLECTION DATA OF FOUR FLYASHES (14)

| Flyash | Type Boiler | Coal | Collecting Equipment | Collection Efficiency, percent |
|---|---|---|---|---|
| No. 10 | Dry bottom | Western Kentucky | Electrical | 98[a] |
| No. 11 | Wet bottom | 50 percent eastern Kansas and 50 percent petroleum coke | Electrical | Less than 90[b] |
| No. 12 | Dry bottom | Western Kentucky and southern Illinois | Mechanical | Less than 70[c] |
| No. 15 | Dry bottom | Southern Illinois | Electrical | 95[d] |

[a] Electric precipitators were used and since this station is located close to the heart of a city the combustion chamber and coal pulverizing equipment were designed for extremely efficient burning The precipitators are oversize in order to obtain the high collection efficiency.

[b] This unit has electric precipitators but for the last year has burned a 50-50 blend of coal and petroleum coke. The coke does not fully burn in the short time that it is in the combustion chamber, therefore increasing the loss due to ignition (carbon content).

[c] Low efficiency here is due to the use of mechanical precipitators. The loss on ignition runs on the order of 8 to 12 percent. Most of this loss was evident in the material retained on the No. 200 sieve while the relatively finer material retained on the No. 325 sieve was very low in loss on ignition.

[d] Although electric precipitators are used in this unit the loss on ignition is fairly high. This is due to general overloading of the boilers (approximately 110 percent of rated capacity) resulting in incomplete combustion of the coal

coal, treatment prior to combustion, method of combustion, amount of recirculation and method of collection. Studies of the use of flyash in portland cement mortar and concrete have indicated that fineness and carbon content are possible criteria for differentiating flyashes. Analyses of flyashes include the term "loss on ignition" which is expressed as a percentage of the total flyash and approximately represents the carbon content. The loss on ignition is determined by oxidation, at high temperatures, of the organic material in the flyash. There are several methods of determining fineness, one of the more common is sieving the flyash through a No. 325 sieve to determine the percent passing.

Weinheimer (9), in 1944, conducted an extensive investigation of the chemical properties of flyashes. Chemical analyses of different size fractions of flyash indicated that the non-combustible $SiO_2$, $Al_2O_3$ and $Fe_2O_3$ tend to be concentrated in the finer fractions. The residual carbon, as determined by loss on ignition tests, predominates in the coarser particles. Photomicrographs showed that the carbon in flyash exists as irregular, porous, coke-like particles. The non-combustible particles generally have a characteristic spherical shape, although a small portion of these particles are thin walled polyhedrons called cenospheres.

There has been little information published about the effects of flyash properties on its reactivity with lime. In this study, four flyashes having different properties were used to investigate these effects. The unconfined compressive strengths of lime-flyash mortars were used to evaluate reactivity, the assumption being that strength is a positive function of reactivity.

## MATERIALS

### Flyashes

The sources and properties of each flyash are tabulated and explained in Tables 1 and 2. The flyashes have been assigned the arbitrary numbers shown in the tables and will be referred to by these numbers.

Photomicrographs of flyashes No. 10, 11, 12 and 15 are shown in Figures 1 through 4. These X 100 photomicrographs tend to corroborate Weinheimer's findings. Comparison of the photomicrographs of No. 10 and No. 11 flyashes is particularly interesting as these flyashes contain the least and the most carbon respectively. Notice the greater degree of fineness and the relative absence of carbon in flyash No. 10, whereas the particles in flyash No. 11 appear to be somewhat aggregated and coated by the more abundant carbon.

TABLE 2

PROPERTIES OF FLYASHES

| | | Flyash | | | |
|---|---|---|---|---|---|
| | | No. 10 | No. 11 | No. 12 | No. 15 |
| Specific Gravity | | 2.56 | unknown | 2.30 | 2.24 |
| Fineness | Residue passing a No. 325 sieve, percent by weight | 93.1[a] | 60 3[b] | 81.0[a] | 82.3[a] |
| Chemical Analysis, percent by weight | Silicon dioxide ($SiO_2$) | 43.40 | 39 19 | 41.16 | 35.94 |
| | Aluminum oxide ($Al_2O_3$) | 20.10 | 13.23 | 18.39 | 18.19 |
| | Ferric oxide ($Fe_2O_3$) | 19 00 | 13.41 | 21.23 | 19.63 |
| | Calcium oxide (CaO) | 7 30 | 2 52 | 5.54 | 6.89 |
| | Magnesium oxide (MgO) | 0.43 | 1.16 | 0.77 | 0.85 |
| | Sulphur trioxide ($SO_3$) | 3.04 | 0.41 | 1 47 | 1.86 |
| | Loss on ignition | 3 20 | 27.67 | 10.18 | 15.59 |

[a] Method of test ASTM Designation C204-46T.

[b] Method of test· Material was screened until the percent passing the No. 325 sieve was less than 0.5 percent after five minutes in a mechanical sieve shaker.

### Lime

The lime used was laboratory reagent powdered calcium hydroxide. The manu-

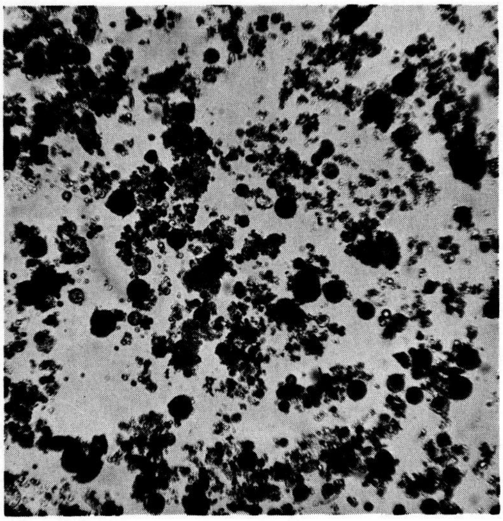

Figure 1. Photomicrograph of No. 10 flyash: X 100.

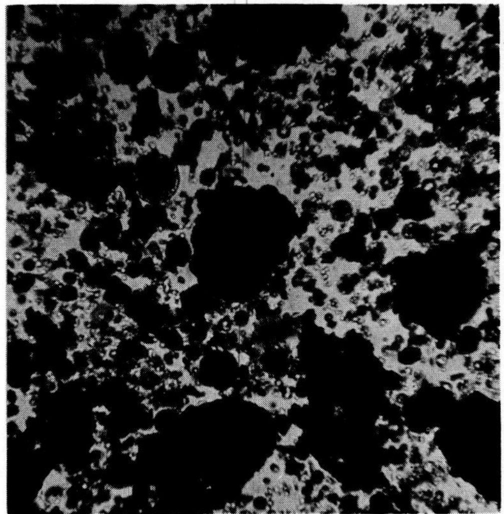

Figure 2. Photomicrograph of No. 11 flyash: X 100.

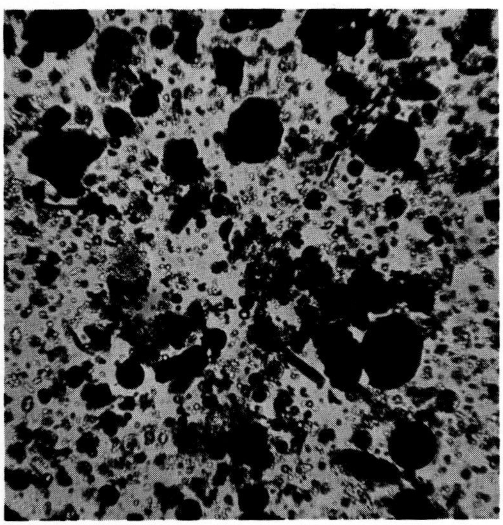

Figure 3. Photomicrograph of No. 12 flyash: X 100.

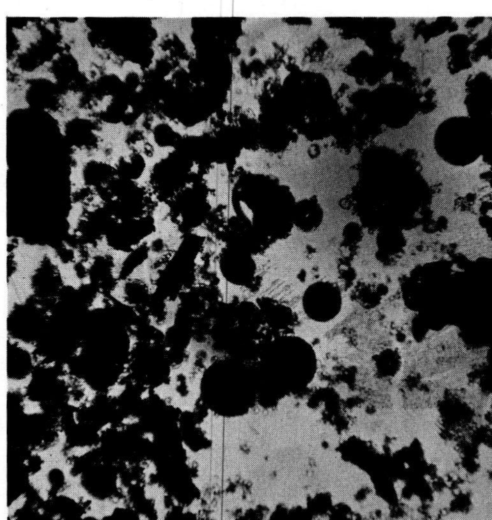

Figure 4. Photomicrograph of No. 15 flyash: X 100.

facturer has listed the maximum limit of impurities as follows:

| | % |
|---|---|
| Insoluble in HCl | 0.03 |
| Chloride | 0.005 |
| Sulfate | 0.10 |
| Heavy metals such as Pb | 0.003 |
| Iron | 0.05 |
| Substances not precipitated by ammonium oxalate | 1.0 |

TABLE 3

OPTIMUM MOISTURE CONTENTS AND CORRESPONDING TRUE MAXIMUM DRY DENSITIES OF THE LIME-FLYASH MORTARS PREPARED WITH THE FOUR FLYASHES

| Lime % | Flyash No. 10 Opt. Moist. | No. 10 Dry Density, lb/ft | No. 11 Opt. Moist. | No. 11 Dry Density, lb/ft | No. 12 Opt. Moist. | No. 12 Dry Density, lb/ft | No. 15 Opt. Moist. | No. 15 Dry Density, lb/ft |
|---|---|---|---|---|---|---|---|---|
| 2 | 29 | 85.0 | 57 | 51.7 | 36 | 68.5 | 45 | 61.4 |
| 4 | 26 | 83.6 | 57 | 52.7 | 36 | 69.1 | 43 | 60.7 |
| 6 | 28 | 82.2 | 57 | 53.3 | 36 | 69.5 | 43 | 63.1 |
| 8 | 28 | 81.8 | 57 | 53.5 | 36 | 70.6 | 42 | 63.8 |

## SAMPLE PREPARATION AND TESTING

### Preparation of Mixtures

The amounts of lime added to each flyash were 2, 4, 6 and 8 percent based on the dry weight of the flyash. The amount of distilled water in each case was sufficient to produce the maximum dry density for standard Proctor compactive effort. Mixtures contained only lime, flyash and distilled water and are referred to as mortars.

Moisture-density curves for four mortar compositions, using flyash No. 10, are shown in Figure 5. Several definite points of maximum density were found in the lower moisture range, but extension of the moisture-density curve into the upper reaches of moisture content revealed a true maximum density. Similar results were obtained with the other flyashes. The true maximum density occurred in all cases slightly below the moisture content at which the mortar began to act as a viscous liquid. The optimum moisture contents and corresponding true maximum densities are presented in Table 3.

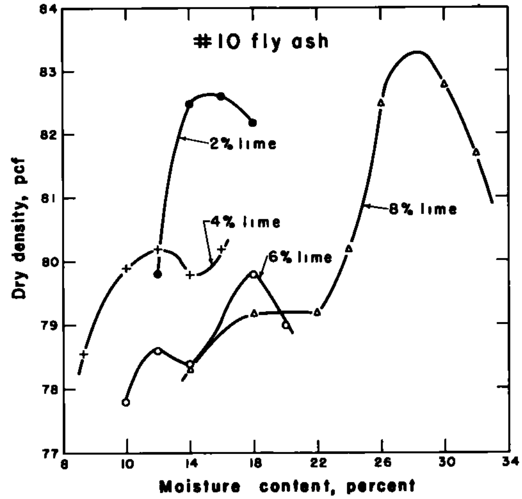


Figure 5. Moisture-density relationship of No. 10 flyash illustrating relative maxima at low moisture contents and the absolute maximum for the 8 percent lime content. These curves are typical for the other three flyashes.

Optimum moisture content tends to decrease with the fineness of the flyash. Davis et al (4) experienced similar results in their investigation of flyash as an additive to portland cement. A correlation of optimum moisture content and loss on ignition is shown in Figure 6. The increase of moisture requirement with increased carbon content is probably due to the porous nature of the carbon. Brink and Halstead (2) found a similar trend in the water requirement of portland cement-flyash mortars.

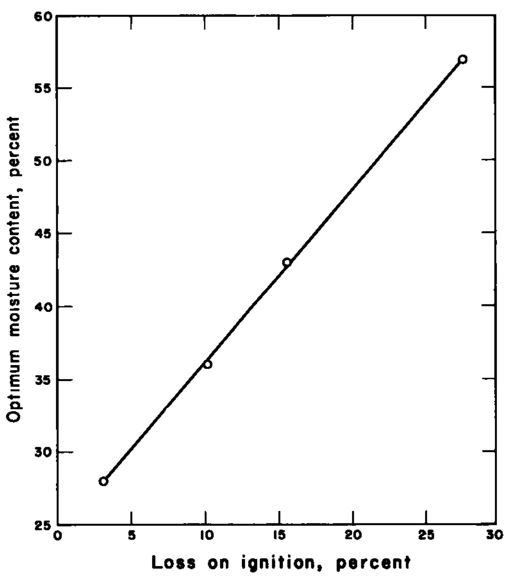


Figure 6. Optimum moisture content for maximum density of the lime-flyash mortars plotted as a function of flyash loss on ignition. The optimum moisture contents are the average values of four lime-flyash mortars.

### Mixing and Molding

Lime-flyash mixtures were proportioned and mixed dry. Optimum water was added and the materials machine mixed for four minutes. Specimens, 2-in. diameter by 2-in. high, for unconfined compressive strength tests were prepared at approximate standard Proctor density with a double plunger drop-hammer molding apparatus.

### Curing

Specimens were cured for various times at two different constant temperatures to study the rate and duration of the pozzolanic reaction. Curing times were 0, 7, 14, 28 and 45 days; curing temperatures were 20 C and 60 C. Each specimen was first wrapped in either Saran Wrap or wax paper, then wrapped with aluminum foil and sealed with Scotch

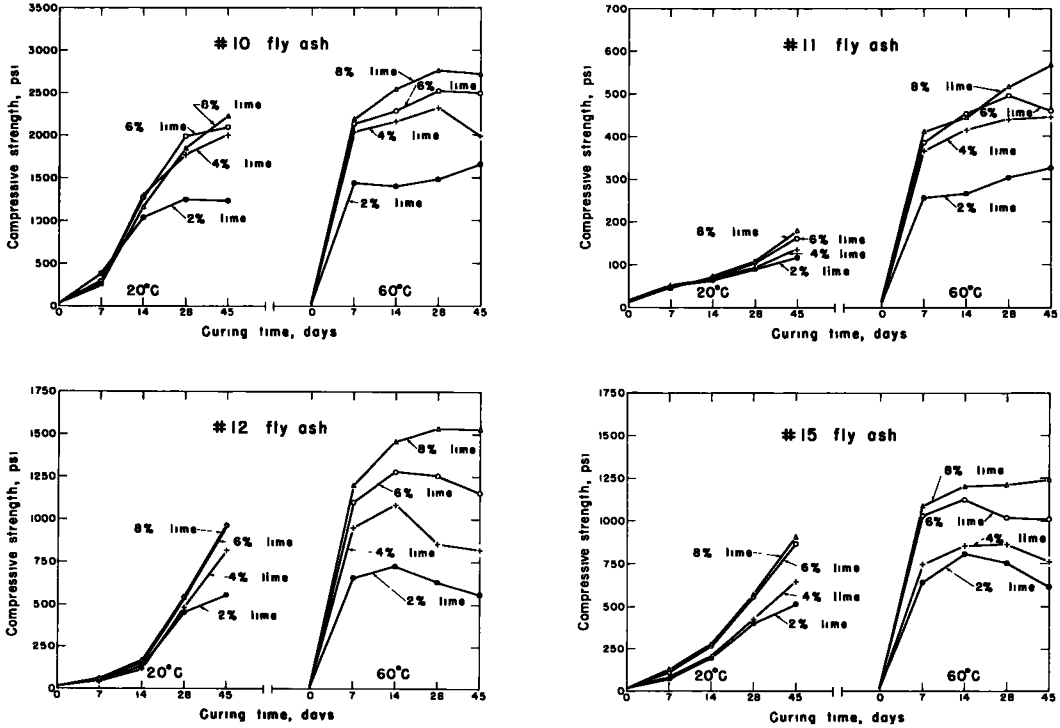


Figure 7. Effect of variation in the amount of lime, curing time and curing temperature on the compressive strength of 2-in. by 2-in. specimens prepared from the four flyashes.

tape. The difference in the inner wrapping should be noted since this inconsistency accounts for a variation in the test results. Specimens cured at 20 C were stored in an atmosphere with a relative humidity of approximately 90 percent. Specimens cured at 60 C were kept in an oven having a non-humid atmosphere; the sealing of each specimen was assumed to be sufficient to prevent any loss of moisture by evaporation.

## Testing

At the end of the curing periods specimens were unwrapped, weighed to determine moisture loss during curing and then tested in unconfined compression using a load travel rate of 0.05 in. per minute. The results reported are the average of three test samples and represent the load at failure, uncorrected for height-diameter ratio.

## DISCUSSION OF RESULTS

The position of the strength-time curves of the lime-flyash mortars fluctuated somewhat during the early stages of curing at 20 C as shown in Figure 7. However, the curves appear to have reached their proper positioning in relation to each other after 45 days curing. The 45 day strength values show that increased lime contents are directly responsible for higher strengths.

Time and temperature are two very significant factors responsible for some of the variations apparent in Figure 7. The slopes of the strength-time curves at 20 C are still definitely positive after 45 days curing, indicating that the pozzolanic reaction has not yet reached completion. The lone exception is shown by the curve for No. 10 flyash with 2 percent lime. Here the reaction appears to be nearly complete after 28 days curing. These data support the validity of the assumption that compressive strength is a criterion for studying the progress of the pozzolanic reaction. Apparently strength develops at a rate that parallels the rate of the reaction. As the lime combines with the flyash, and the amount of free lime decreases, the rate of strength increase gradually slows and the curve tends to become horizontal. This is best shown by the mortars containing 2 percent lime.

Samples cured at 60 C showed a decidedly higher rate of strength development during the first few days than those cured at 20 C. The increase in temperature caused an increase in reaction rate during the first 7 days in all cases. Acceleration of the reaction was anticipated because it has long been known that many chemical reactions may double or treble their velocity with a 10 deg rise in the temperature of the reactants. Arrhenius has given a quantitative relation to this phenomenon through a mathematical description relating reaction rate to absolute temperature.

Figure 7 shows that in all but a few cases, curing beyond 7 days at 60 C caused the strength-time curves to flatten out and then to decrease. The loss of strength is apparently due to an excessive loss of moisture caused by improper choice of interior wrapping material (wax paper). Specimens (No. 10 flyash with 2 percent lime and No. 11 flyash with 2, 4 and 8 percent lime) wrapped with Saran Wrap did not show a decrease in strength or a significant loss of moisture during curing. A comparison of the moisture losses after curing showed an extreme loss of moisture at 28 days for all specimens wrapped in wax paper, for example, 13 grams moisture loss in 45 days for a Saran wrapped specimen as opposed to 30 grams moisture loss in 45 days for a wax paper wrapped specimen.

Other investigators have suggested the use of strength after 7 days curing at 60 C for predicting 28 day strengths of room temperature cured specimens. Comparison of these values in Figure 7 shows that there is no simple relationship between them. However, the strength values after 7 days curing at 60 C place the flyashes in their correct order of reactivity. This suggests the possibility of using accelerated curing for rating flyash reactivity when time does not permit the longer periods of curing required at room temperatures.

Regrouping the curves in Figure 7, so that the curves of identically proportioned

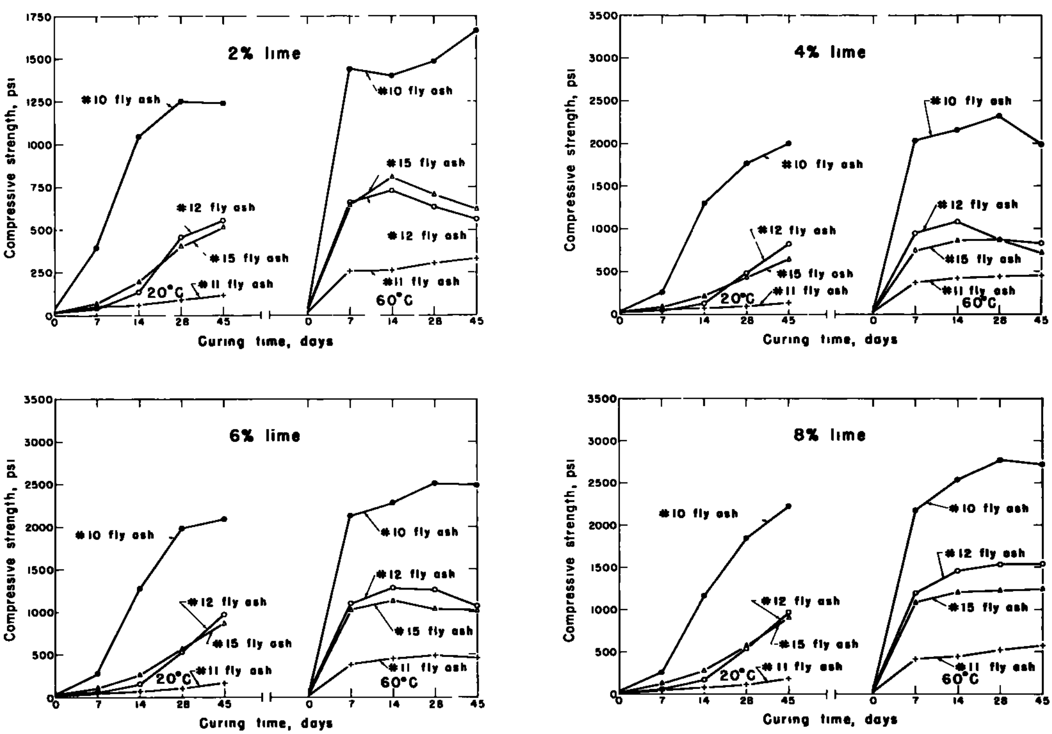


Figure 8. Comparison of compressive strengths of 2-in. by 2-in. specimens prepared from the four flyashes and the indicated percentages of lime.

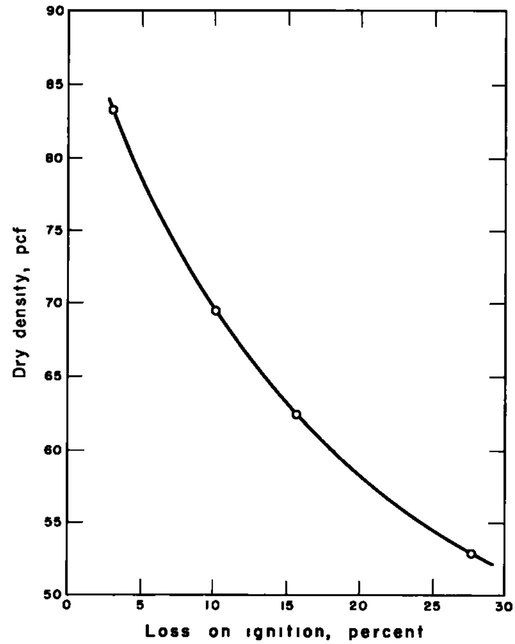


Figure 9. Dry density of lime-flyash mortars plotted as a function of flyash loss on ignition. Each dry density value is an average value for four lime-flyash mortars.

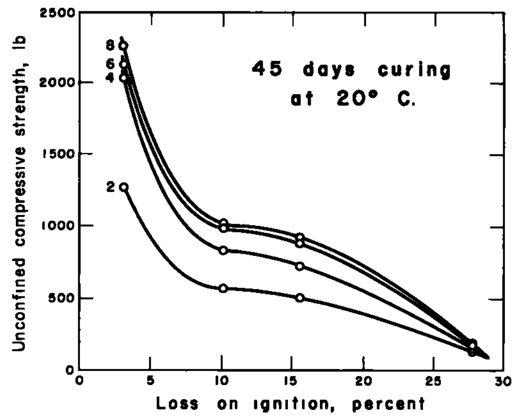


Figure 10. Unconfined compressive strength of lime-flyash mortars plotted as a function of flyash loss on ignition. The lime contents of 2, 4, 6 and 8 percent are shown at the left of each curve.

mortars made from different flyashes are grouped together, reveals the results shown in Figure 8. The highest strength was attained by No. 10 flyash mortars in all cases, which indicates that this flyash is by far the most reactive of those tested.

There are two apparent reasons for the superior performance of No. 10 flyash. Comparison of the photomicrographs of the four flyashes in Figures 1 to 4 shows that No. 10 flyash is much finer and contains less carbon than the other flyashes. Table 2 also verifies this.

Correlations of various mortar properties with the amount of flyash passing the No. 325 sieve were attempted. These correlations were not good but did indicate a rough relationship of density, of optimum moisture, and of strength to flyash fineness. However, by using loss on ignition as the independent variable, better correlations were established. Figure 9 shows the relation between maximum dry density of lime-flyash mortars and loss on ignition. The decrease in density with increase in carbon content cannot be allayed to the difference in specific gravity between carbon and the $Al_2O_3$ and $SiO_2$ replaced by the carbon. A material balance comparison of the highest and lowest densities shows that specific gravity differences account for a density change of only about 2 pcf, whereas the true density difference is about 31 pcf. The difference is thought to be due to aggregating and porosity effects of the carbon.

The unconfined compressive strength after 45 days curing at 20 C has a significant relation to loss on ignition as shown in Figure 10. The curves show that the strength of lime-flyash mortars drops rapidly with increasing carbon content up to about 10 percent carbon, here the curves begin to level off. It is interesting to note that a flyash with a carbon content near 30 percent probably would show little or no pozzolanic activity. Apparently flyashes containing less than about 10 percent carbon are needed to produce lime-flyash mortars having high compressive strength. The advantage of using low carbon content flyashes is evident, but additional work with more flyashes is needed to establish an upper specification limit of carbon content.

Carbon in flyash appears to be deleterious to pozzolanic reactivity and strength of lime-flyash mortars because of its adverse effects on reactive surface area and mortar density. Microscopic examinations of the flyashes showed that the carbon tends to adhere to and partially cover the reactive surfaces, reducing the interfacial area available for pozzolanic reactions with lime. In addition to reducing reactive surface area of individual particles, carbon coatings also act as links between adjacent particles to produce a porous aggregated structure. This structure, in addition to further reducing the available active surface area, reduces the compacted density attainable; the decreased density results in fewer and less intimate contacts between cementitious particles.

## CONCLUSIONS

1. Carbon content as determined by loss on ignition seems to be a reliable indicator of the pozzolanic reactivity of flyashes with lime. The upper limit of carbon content for good pozzolanic cementation appears to be less than 10 percent. Additional work with more flyashes is necessary to establish a specific upper limit.

2. The amount of flyash passing a No. 325 sieve decreases as carbon content increases and is, to a lesser extent, also an indicator of the pozzolanic reactivity of flyash. Evaluation from this criterion was not as reliable as from loss on ignition.

3. The use of lime-flyash mortar strength tests for evaluating flyash reactivity appears to give valid results. The results at both room temperature (20 C) and at 60 C are consistent. Curing at the higher temperature has the advantage of less time requirement for reactivity evaluation.

## ACKNOWLEDGMENT

The subject matter of this paper was obtained as part of the research being done under Project 283-S of the Engineering Experiment Station of Iowa State College. The project is sponsored by the Iowa Highway Research Board as Project HR-1 and is supported by funds supplied by the Iowa State Highway Commission and the U. S. Bureau of Public Roads. Special appreciation is extended to the Walter N. Handy Company, Evanston, Illinois, for information and flyash samples furnished.

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# Lime-Stabilized Test Sections on Route 51, Perry County, Missouri

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Missouri State Highway Department

Test sections to study the performance of lime-stabilized bases were constructed by the Missouri State Highway Department maintenance forces on Route 51, Perry County in the late summer of 1952. The existing roadway materials, consisting of about 2 in. of gravel base, 0.5 in. of bituminous surface, and soil subgrade of clays, silty clays, and silty clay loams were mixed with various types of lime additives for a depth of 6 in. This base was surfaced with a 0.5-in. bituminous seal. Lime additives included (a) hydrated lime, (b) quicklime, (c) a combination of hydrated and quicklime, and (d) a waste product of lime manufacture that was composed of approximately 50 percent lime hydrate, 40 percent calcium carbonate, and 10 percent flyash.

On the basis of preliminary laboratory tests the lime spread was determined to be 4.5 percent where hydrated lime was used, 3.4 percent where quicklime was used, 2.25 percent hydrated lime and 1.7 percent quicklime where the combination was used, and 9.0 percent where the waste lime was applied. Tests made immediately after mixing indicated a satisfactory and sometimes startling reduction in plasticity index with all type additives. It was noted, however, that the hydrated lime, whether in combination with quicklime or not, was consistently more effective in improving the roadway materials.

It appears that either hydrated or quicklime, or a combination of the two, will give adequate results when used as a base stabilizing agent under soil and traffic conditions (77 commercial vehicles per day) as found on this route. By the same measurements the results obtained with waste lime are less satisfactory and its use should probably be limited to base stabilization on very lightly traveled roads or to subbase stabilization on the heavier traveled roads.

● TEST SECTIONS to study the performance of lime-stabilized bases were constructed by the Missouri State Highway Department maintenance forces on Route 51, Perry County, in the late summer of 1952. The construction was in two sections.

Section I starts at the junction of Route J, 9.3 miles south of Perryville, and extends north for 5,141 ft. It is comprised of waste lime treated roadway material in the first 2,600 ft and material treated with a combination of quick and hydrated lime in the remaining 2,541 ft.

Section II starts 2.93 miles north of the Route J junction and extends north for 5,280 ft. It consists of hydrated lime treatment in the first 1,300 ft, quicklime in the next 1,300 ft and waste lime in the remaining 2,680 ft.

The roadway materials that were treated with lime consisted of about 2 in. of gravel base, 0.5 in. of bituminous surface, and soil subgrade of clays, silty clays, and silty clay loams. The waste lime used in the treatment was composed of approximately 50 percent lime hydrate, 40 percent calcium carbonate, and 10 percent flyash.

Preliminary laboratory tests established the optimum lime hydrate content for the roadway materials to be 4.5 percent by weight. On this basis the lime spread was determined to be: (a) 4.5 percent where hydrated lime was used, (b) 3.4 percent where quicklime was used, (c) 2.25 percent hydrated lime and 1.7 percent quicklime where the combination was used, and (d) 9.0 percent where the waste lime was applied.

Tests made soon after mixing indicated a very satisfactory and sometimes startling reduction in plasticity index with all type additives. It was noted, however, that the hydrated lime, whether or not in combination with quicklime, was consistently more

TABLE 1

ROUTE 51, PERRY COUNTY, LIME STABILIZED SECTIONS

| Location | Type of Lime | Date | Passing No. 4 % | Passing No. 200 % | Liquid Limit | Plastic Limit | Plastic Index | Group Classification | Group Index | Optimum Moisture % | Maximum Density lb | Moisture in Sample % | Density lb | % Compaction |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Section I - Starts at Route J and Extends North for 5,141 ft | | | | | | | | | | | | | | |
| Station | None | 1952 | 81.8 | 50.8 | 37.5 | 16.0 | 21.5 | A-6 | 7 | 11.1 | 119.5 | - | - | - |
| 1 + 00 | Waste | 1952 | 73.6 | 29 5 | 31.2 | 30 4 | 0.8 | A-2-4 | 0 | 12.9 | 113.1 | 8.3 | 116.8 | 103.2 |
| | Waste | 1953 | 63.2 | 32 7 | 33.2 | 33.2 | 0.0 | A-2-4 | 0 | 12 2 | 115 9 | 6 7 | - | - |
| | Waste | 1957 | - | - | 34.5 | 31.1 | 3.4 | - | - | - | - | 9.6 | - | - |
| Station | None | 1952 | 91.4 | 77 5 | 38.1 | 20.2 | 17.9 | A-6 | 11 | 14.2 | 113.0 | - | - | - |
| 11 + 00 | Waste | 1952 | 86 0 | 50.6 | 31.5 | 26.8 | 4.7 | A-4 | 3 | 14.9 | 110.3 | 6.9 | 110.5 | 100.2 |
| | Waste | 1953 | 88.9 | 69 6 | 31.4 | 21.8 | 9.6 | A-4 | 7 | 14.6 | 112.6 | 8.2 | 113 0 | 100 4 |
| | Waste | 1957 | - | - | 33.5 | 23.9 | 9.6 | - | - | - | - | 12.5 | - | - |
| Station | None | 1952 | 70.7 | 25.7 | 24.9 | 16.9 | 8.0 | A-2-4 | 0 | 7.2 | 129.7 | - | - | - |
| 24 + 00 | Waste | 1952 | 65.5 | 20.2 | 23.5 | 27.3 | 0.0 | A-1-b | 0 | 7 8 | 126.7 | 8.2 | 129.2 | 102.4 |
| | Waste | 1953 | 60.9 | 18.9 | - | - | N. P. | A-1-b | 0 | 10 1 | 119 8 | 4.1 | - | - |
| | Waste | 1957 | - | - | 27 7 | 27.0 | 0.7 | - | - | - | - | 8.6 | - | - |
| Station | None | 1952 | 76.6 | 49.7 | 38 2 | 16.2 | 22.0 | A-6 | 7 | 12.1 | 118.8 | - | - | - |
| 33 + 00 | Hyd Quick | 1952 | 75.9 | 23.4 | 32.5 | 33.9 | 0.0 | A-2-4 | 0 | 13.4 | 114.7 | 11.6 | 117.9 | 102.8 |
| | Hyd Quick | 1953 | 65.5 | 31.0 | 33.9 | 35.0 | 0.0 | A-2-4 | 0 | 12 8 | 115 1 | 8.5 | - | - |
| | Hyd. Quick | 1957 | - | - | - | - | N. P. | - | - | - | - | 10.4 | - | - |
| Station | None | 1952 | 78.6 | 49.6 | 34 2 | 17.0 | 17.2 | A-6 | 5 | 11.1 | 121 5 | - | - | - |
| 39 + 00 | Hyd. Quick | 1952 | 80.4 | 38.2 | 32.5 | 31.0 | 1 5 | A-4 | 1 | 13.0 | 115.1 | 10.0 | 115.7 | 100.5 |
| | Hyd Quick | 1953 | 72.5 | 32.8 | 33 7 | 32.6 | 1.1 | A-2-4 | 0 | 12 8 | 115.2 | 6.7 | 107.0 | 92.9 |
| | Hyd. Quick | 1957 | - | - | 34 8 | 30.8 | 4.0 | - | - | - | - | 13.1 | - | - |
| Station | None | 1952 | 68.1 | 25.3 | 25 9 | 18.4 | 7.5 | A-2-4 | 0 | 7.5 | 128.4 | - | - | - |
| 42 + 00[a] | Hyd. Quick | 1952 | 71.7 | 23 3 | 28.9 | 30.4 | 0 0 | A-2-4 | 0 | 9 8 | 119.6 | 8.0 | 120.0 | 100.3 |
| | Hyd. Quick | 1953 | 66.7 | 22.4 | 30 4 | 29.6 | 0.8 | A-2-4 | 0 | 10 9 | 118.8 | 5.4 | - | - |
| Section II - Starts 2.93 Miles North of Route J and Extends North for 5,280 ft | | | | | | | | | | | | | | |
| Station | None | 1952 | 86.4 | 69.8 | 34.0 | 18.2 | 15.8 | A-6 | 9 | 13.1 | 114.9 | - | - | - |
| 4 + 00 | Hydrated | 1952 | 85.7 | 39.7 | 32 3 | 33.0 | 0.0 | A-4 | 1 | 16.3 | 107.4 | 13.9 | 108.8 | 101.3 |
| | Hydrated | 1953 | 82 9 | 50.0 | 37.5 | 33.8 | 3 7 | A-4 | 3 | 16 8 | 107.3 | 13.5 | 107.5 | 100.2 |
| | Hydrated | 1957 | - | - | 38.2 | 30.8 | 7.4 | - | - | - | - | 14.1 | - | - |
| Station | None | 1952 | 85.0 | 70.6 | 34.1 | 18.2 | 15.9 | A-6 | 9 | 11 9 | 117 2 | - | - | - |
| 22 + 00 | Quick | 1952 | 89.0 | 55 0 | 32.1 | 27.5 | 4 6 | A-4 | 4 | 15.5 | 110.9 | 14.3 | 114.0 | 102.8 |
| | Quick | 1953 | 87.8 | 60 4 | 34 3 | 29.8 | 4.5 | A-4 | 5 | 15.7 | 109 4 | 10 8 | 110.8 | 101.3 |
| | Quick | 1957 | - | - | 34.1 | 26.9 | 7.2 | - | - | - | - | 11.0 | - | - |
| Station | None | 1952 | 93.8 | 78.1 | 31 7 | 20.8 | 10 9 | A-6 | 8 | 14.4 | 112.2 | - | - | - |
| 32 + 00 | Waste | 1952 | 87.1 | 59.5 | 33.5 | 28.1 | 5 4 | A-4 | 5 | 15.1 | 110.9 | 16.2 | 107.2 | 96.7 |
| | Waste | 1953 | 81.0 | 58.3 | 33 5 | 26.9 | 6 6 | A-4 | 5 | 13.3 | 112.6 | 9.5 | 109.6 | 97.3 |
| | Waste | 1957 | - | - | 34.0 | 24.2 | 9.8 | - | - | - | - | 11.8 | - | - |
| Station | None | 1952 | - | - | - | - | - | - | - | - | | Not Sampled | | |
| 40 + 00 | Waste | 1952 | 77.1 | 40.2 | 32.8 | 27.4 | 5.4 | A-4 | 1 | 10.8 | 118.3 | 12.9 | 111.6 | 94.3 |
| | Waste | 1953 | 70.6 | 31.7 | 33 8 | 30.4 | 3 4 | A-2-4 | 0 | 11.2 | 116.7 | 9.9 | - | - |
| | Waste | 1957 | - | - | 33.4 | 31.9 | 1.5 | - | - | - | - | 11.1 | - | - |

[a] Not sampled in 1957 because material had changed from a modified soil, so to speak, to a structural material showing much hardness. The stabilized material from all locations has hardened (by 1957) to the extent that it will not completely break down when soaked in water. Because of this, it was thought that the mechanical analysis and compaction tests would be of no significance.

effective in improving the roadway materials.

The specified depth of treatment in all instances was 6 in. The thickness determinations made in the completed base at the center and 1 ft from edge at intervals of 300 to 400 ft showed Section I to average 6.2 in. and Section II, 5.9 in. In general, the determination made 1 ft from edge showed less than 6 in. and out of the 52 edge measurements in both sections, 21 measured 5.25 in. or less. In Section I the center averaged 7.2 in., the right edge 5.6 in., and the left edge 5.8 in. In Section II the center averaged 6.3 in., the right edge 5.6 in., and the left edge 5.7 in.

The above measurements indicate that depth of treatment was not too well controlled. These variations in depth can be attributed to the fact that equipment, such as flat bottomed plows, was not available for use in depth control. Motor graders were used for this control, as well as being used to do practically all of the mixing.

Water application was slow and inadequate, especially in Section I, due to a shortage of water distributors.

This was the construction personnel's first stabilization job and their inexperience was reflected in the quality of the work. As the job progressed, however, the quality improved.

All of these factors, which can probably be condensed into two general headings, (1) inexperience of construction personnel, and (2) shortage of equipment, have been reflected to a large extent in the performance of these stabilized sections.

This report describes the condition of the lime sections as they now exist and offers conclusions that have been reached after a 4½-year test period. Also included with

with the report is a table of test results of samples obtained from the stabilized base in 1952, 1953, and 1957. The only tests made on samples obtained the spring of 1957 were for plasticity and moisture. The stabilized material has hardened to the extent that it will not completely break down when soaked in water, so it was thought that mechanical analysis and compaction tests would not be of any significance.

## ROAD CONDITION

### Waste Lime—Station 0 to 26, Section I

The appearance of this portion is classed as "fair" to "poor." About 12 percent of the total area is showing distress in the form of alligator cracks. This includes about 3 or 4 percent of badly cracked area that is confined to the outer 1 to 2 ft for one-fourth of the length. A large part of this edge distress can probably be attributed to thin base. Approximately 600 sq ft (1.2 percent) of complete failure has occurred. Maintenance has consisted of applying light seal in addition to the original seal and placing patches in the failed areas and several other small areas where surface blemish has occurred.

The base does not have much hardness (it could not be cored) but appears dry and flaky. The plastic index of the base shows an increase from 0.8 to 3.4 at one location since the fall of 1952 to the spring of 1957, an increase from non-plastic to 0.7 at another location, and an increase from 4.7 to 9.6 at the third location. The moisture content of the base is higher than that found at any time before but is still 1.5 to 2.6 points below the optimum moisture as determined from samples obtained in 1953. This higher moisture was anticipated since this is the first time samples were taken in

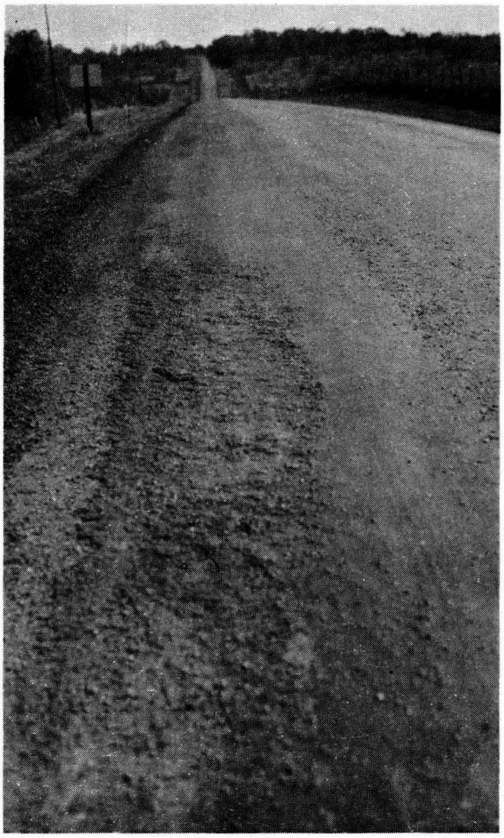

Figure 1. Waste lime, Section I, general view.

Figure 2. Waste lime, Section I, distortion in outer wheel track.

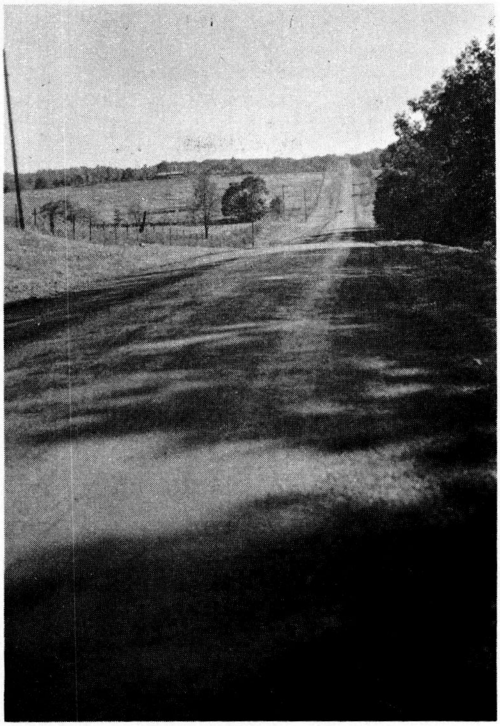

Figure 3. Hydrated-quicklime, Section I, general view.

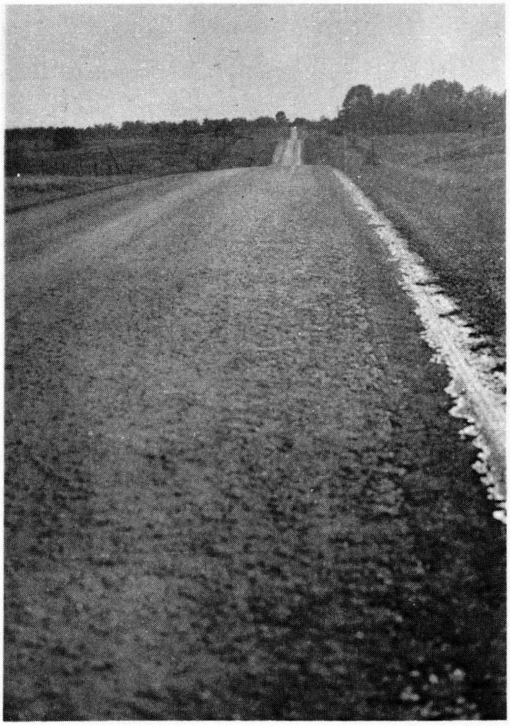

Figure 4. Hydrated lime, Section II, general view.

the spring instead of late summer or fall when the other sampling was done.

This section of waste lime is fairly smooth and is still giving good service, however, it is classed as the poorest of all the stabilized sections. One reason for this is that it was the first built and many construction inadequacies, such as poor edge and depth control, insufficient water in mix, existed because of inexperience of the personnel and shortage of water distributors and mixing equipment.

Hydrated-Quicklime—Station 26 to 51 + 41, Section I

The appearance of this section is classed as "fair" to "good." About 450 sq ft (0.9 percent) of the area is showing distress in the outer 4 ft while another 5 percent of the surface area shows distress in the outer 1 to 2 ft (thin base at the edges is again probably the main factor). A regular shrinkage crack pattern (blocks of about 4 to 6 ft square) occurs throughout this section, the only section showing such pattern. Maintenance has consisted of resealing the section and patching some small areas to correct surface blemishes.

This base is definitely the hardest of all and shows much structural strength. Core drilling was attempted at four locations and cores were obtained at three. Because of the imperfection of the cores, no attempt was made to break them in compression, but after several weeks of soaking these cores remained virtually intact and apparently just as hard as before soaking. At one location the plasticity of the base remained the same, non-plastic, while at another location it increased from 1.5 to 4.0. Moisture in the base at one location was 2.4 points below optimum, at another it was 0.3 points above. (Part of the distress occurring in this section can also be attributed to construction inadequacies.)

Hydrated Lime—Station 0 to 13, Section II

This hydrated lime section is in excellent condition. There are a few minor blemishes in the seal coat. Two small areas of about 16 sq ft each have been patched and a 1- by 25-ft edge patch has been placed. Other maintenance has consisted of a light drag seal.

The base is fairly hard but two attempts at core drilling were unsuccessful. At the location sampled the moisture content was 2.7 points below optimum and the plastic index has increased from 0.0 to 7.4 in the period from 1952 to 1957.

This section has performed the best of all the lime stabilized sections.

Quicklime—Station 13 to 26, Section II

Appearance of this section is rated "good" to "excellent." One area covering 90 sq ft (0.4 percent of total area) has failed and has been patched. Approximately another 1 percent of the area is showing slight distress in the outer 4 ft. There is also a slight amount of edge ravelling and some minor surface blemishes. A light drag seal has been placed by maintenance.

The base is hard but not sufficiently so to permit obtaining a core. It is dry, being 4.7 points below optimum moisture content, and its plasticity index has increased from 4.6 to 7.2 during the 1952 to 1957 period.

Waste Lime—Station 26 to 52 + 80, Section II

The appearance is rated "good." About 600 sq ft (1.1 percent) is showing distress in outer 4 ft. About another 2 percent of the surface area is showing distress in the outer 1 ft along the edges. Much of this distress can be attributed to thin base at the edges. A light drag seal has been placed on the section since the original.

This base shows little hardness and could not be core drilled. At one location the moisture is 1.5 points below optimum while at another location it is practically at op-

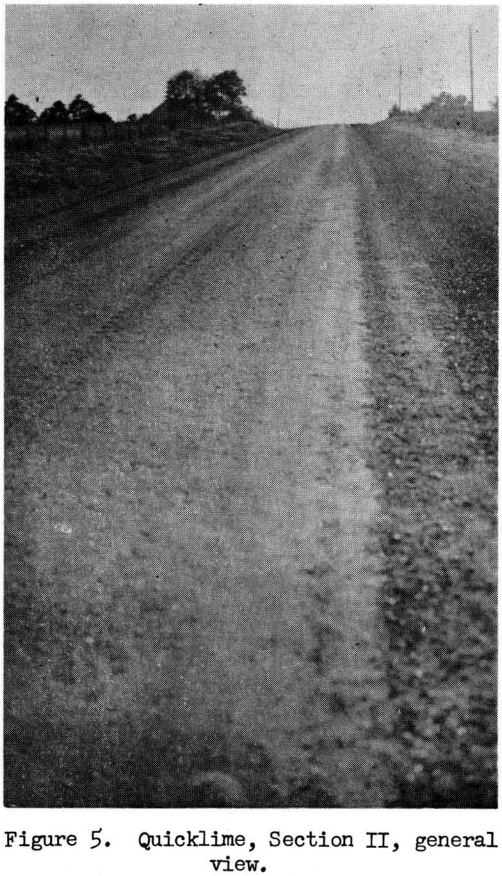

Figure 5. Quicklime, Section II, general view.

Figure 6. Quicklime, Section II, distortion in outer wheel track.

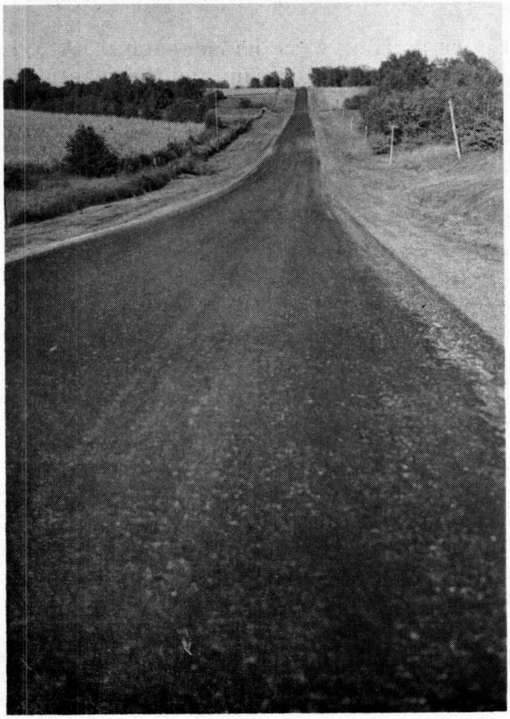

Figure 7. Waste lime, Section II, general view.

Figure 8. Waste lime, Section II, edge distress.

timum. At one location the PI of the base has increased from 5.4 to 9.8 since 1952, while at the other location a decrease from 5.4 to 1.5 was noted.

Blank Section

The 1.95 mile gap between the lime stabilization sections of Route 51 is similar to the lime sections in topographic, drainage, soil, and traffic conditions. In November of 1952 this section was in good to excellent condition because of recent conditioning by the maintenance division. At that time the pavement consisted of an average thickness of 2.13 in. of gravel base topped with an average thickness of 0.95 in. of bituminous surface. This section has required much maintenance since 1952. There have been several resealings and numerous small patches. The surface in April 1957 was very rough and showed much distress in the form of cracked and distorted areas.

## SUMMARY AND CONCLUSIONS

Based on appearances, the sections are rated in order as to the service given during this 4½-year test period.

1. Hydrated lime—Station 0 to 13, Section II.
2. Quicklime—Station 13 to 26, Section II.
3. Waste lime—Station 26 to 52 + 80, Section II.
4. Hydrated-quicklime—Station 26 to 51 + 41, Section I.
5. Waste lime—Station 0 to 26, Section I.

It will be noted that Section II, as a whole, has performed the most satisfactorily. Factors that could enter into this performance, other than the type of lime used, are (1) drainage, (2) subgrade support, (3) the type of material processed, and (4) construction features.

As far as the first three factors are concerned they do not appear to have affected performance to any great extent. Drainage is perhaps slightly better in Section I; generally, subgrade support is better in Section II but there are some parts of Section I showing much distress that have similar subgrade; in general more clay is found in Section I and this type of material apparently reacts more favorably with lime.

The remaining factor, construction features, therefore, appears to be the most important. Section I was the first constructed and there were more construction inadequacies because of the inexperience of all personnel involved and a shortage of equipment. In this section mixing was not as thorough, the edges were not as well defined, the depth of processing was more variable, and less moisture was introduced into the mix. It is believed that Section II is better than Section I, as a whole, mainly because of better construction procedure.

The hardness of the base is rated in the following order: (1) hydrated lime-quicklime—Station 26 to 51 + 41, Section I; (2) quicklime—Station 13 to 26, Section II; (3) hydrated lime—Station 0 to 13, Section II; (4) waste lime—Station 26 to 52 + 80, Section II; and (5) waste lime—Station 0 to 26, Section I.

Actually the base in the hydrated lime-quicklime section is the only portion that shows hardness as expected. The waste product has definitely shown the least hardness while the quicklime appears to show slightly more hardness when used alone than does the hydrated lime.

Why so much more hardness in the section where a combination of quick and hydrated lime was used? This section contains more clay as a whole and it appears that clay reacts more completely and satisfactorily with lime. However, there are some areas within this section that are no more clayey than the material processed in Section II but they still show much hardness. For instance at Station 42, Section I the raw soil sample before lime was added had a PI of 7. 5. This base is hard enough to make a pick ring now and the core obtained at this location remained hard and intact after several weeks of soaking.

Apparently the combination of hydrated and quicklime causes some reaction that produces greater hardness than do the other types of limes used on this project. The next hardest is the quicklime section. Since quicklime is involved in both instances, there is the possibility that the heat generated by the quicklime hastens and carries to further completion the reaction of the lime with the silica in the soil to form insoluble calcium silicate.

The tests of samples show the moisture content of the base to be higher than it was four years ago. This increase is greater in Section I which might account to some extent for the poorer performance of this section. However, in all instances except one, the moisture content of the base is below the optimum moisture as determined on samples obtained in 1953. In this one instance the moisture content is only 0. 3 points above the optimum. In Section I, the moisture content of the base averages 1. 7 points below optimum while in Section II it averages 2. 3 points below. It is believed though that much significance should not be placed on these observations since the slight differences could well be attributed to variations in the material.

In general, the plastic index of the base has increased since construction in 1952.

Station 1 + 00, Section I—PI reduced from 21. 5 to 0. 8 in 1952 which is 3. 7 percent of original PI of soil. At present the PI is 3. 4 or 15. 8 percent of that of the original soil.

Station 11 + 00, Section I—PI reduced to 26. 2 percent of original in 1952, now is 53. 7 percent of original PI.

Station 24 + 00, Section I—Reduced to 0 percent of original in 1952, now is 8. 8 percent of original.

Station 33 + 00, Section I—No change.

Station 39 + 00, Section I—8. 7 percent of original in 1952, at present it is 23. 3 percent.

Station 4 + 00, Section II—Was 0 percent in 1952, at present it is 46. 8 percent.

Station 22 + 00, Section II—Has increased from 28. 9 percent in 1952 to 45. 3 percent in 1957.

Station 32 + 00, Section II—PI showed 49. 6 percent of original in 1952, now shows 90 percent.

Station 40 + 00, Section II—No sample obtained before processing with lime. However, the lime base at this location is the only one showing a decrease in PI. It has decreased from 5.4 to 1.5 during the 4½-year period.

The average daily traffic count on the lime sections as well as the intervening gap is 454 vehicles of which approximately 77 are trucks and busses. In these conditions the 6-in. lime stabilized base has given better service than the intervening gap that consists of 2.13 in. of gravel topped with 0.95 in. of bituminous surfacing. This comparison does not have much significance, however, since the lime sections are twice as thick as that in the intervening section.

A large part of the distress occurring in the lime sections is along the edges where thin base is found. This is especially true in the waste lime sections where a combination of thin base and inadequate hardness is found.

It is believed that it can be definitely concluded that either hydrated or quicklime or a combination of the two is superior to the waste lime product in the quantities used on this project. The quantity of waste lime used was approximately twice the amount of hydrated or quicklime.

It appears that either hydrated or quicklime or a combination of the two will give satisfactory results when used as a base stabilizing agent, in soil and traffic conditions as found on this section of Route 51. This is with the provision that the depth of treatment is adequately controlled and the soil and lime are properly mixed, moistened, and compacted. It is believed, however, that still more satisfactory results would be obtained if a minimum thickness of 2 in. of bituminous surface were used instead of a thin seal coat.

Soil modified by the waste lime should be excellent for use as a subbase and possibly as a base under a 2-in. bituminous surface on lightly (less than 50 commercial vehicles) traveled roads.

There are some rather definite indications that the amount of reduction in plasticity gained immediately after mixing is not permanent. Whether or not the material will regain all the plasticity it possessed before stabilizing remains to be seen. Therefore, it appears that more sampling and testing of the experimental sections is desirable. By further sampling and testing it may be found that the changes in plastic index can be ascribed to sampling since the roadway materials on this project vary to some extent transversely as well as longitudinally.

# Stabilization of Expansive Clay with Hydrated Lime and with Portland Cement

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● THIS PAPER describes procedures and results of laboratory tests on California expansive clay canal soil treated with hydrated lime and with portland cement to show the degree of soil stabilization that could be obtained with these admixtures. Small cylindrical specimens of the compacted soil with lime or with cement in amounts ranging from 0 to 6 percent were subjected to tests to determine expansion upon wetting, shrinkage upon drying, unconfined compression, and wet-dry durability characteristics. The results showed that the presence of either admixture substantially reduced expansion and shrinkage characteristics of the soil and increased soil stability. The cement reduced soil shrinkage upon drying somewhat more than did the lime, but the lime was superior in causing the soil to resist deterioration from wetting and drying action. The recommended amount for the field use of either admixture was 4 percent by weight of the soil.

The laboratory tests described herein were conducted on expansive clay (locally known as "Porterville" clay) from Friant-Kern Canal located in the Central Valley of California near the city of Fresno. The purpose of the tests was to provide a basis for recommending a procedure for stabilizing the canal side slopes and prevent the recurrence of sloughing which has been a maintenance problem in the expansive clay areas. The recommendations called for a trial reconstruction of several sloughed areas with compacted clay containing 4 percent lime or portland cement. Upon further investigation, it was decided that, even if successful, such a method would probably not be used because (a) the sloughing appeared to be on a decrease and (b) the more simple reconstruction measures which consist generally of resloping from the original 1½ : 1 to 2 : 1 with some stabilization with rock were much less expensive. Therefore, the trial test section was not constructed. Estimates did show that if the larger trouble areas could have been located and the admixtures had been incorporated in the slopes during the original canal construction, the cost would have been about the same as the cost required to repair these areas since construction.

The ideas for this proposed canal soil stabilization originated from a knowledge that the general type of stabilization described herein is being investigated for and used to a limited extent in highway pavement subgrades. In some areas where expansive clay soils are encountered, it would be more economical to stabilize the clay subgrade with lime or portland cement to prevent pavement displacement with change of soil moisture than to remove and replace the clay with more suitable, but scarce materials.

## ACTION OF LIME OR PORTLAND CEMENT ON SOIL

Hydrated lime ($Ca(OH)_2$) reacts chemically with some clay soils to change their properties and make them more stable. The details of this reaction are not fully known, but the stabilization is apparently caused by two processes. In one, a base-exchange reaction occurs with a replacement of certain ions, such as the replacement of sodium with calcium. In the other, a cementing agent is formed which acts to bind the soil particles together. The most likely explanation for this is that the calcium of the lime combines with silica and alumina in the soil to form various calcium-alumino-silicate compounds which have cementing properties. Thus, lime has been found to have a stabilizing effect, not only on Na-montmorillonites, but also on other types of montmorillonites and on other groups of the clay family (1).

A previous study (2) by Goldberg and Klein at the University of California included Porterville clay from the Central Valley area as one of several soils treated with lime and investigated in the laboratory. The conclusion from this investigation was that hydrated lime reduced the swelling characteristics of the clay. When the amount of lime used was between 2 and 6 percent, the reduction in expansion was marked and in direct

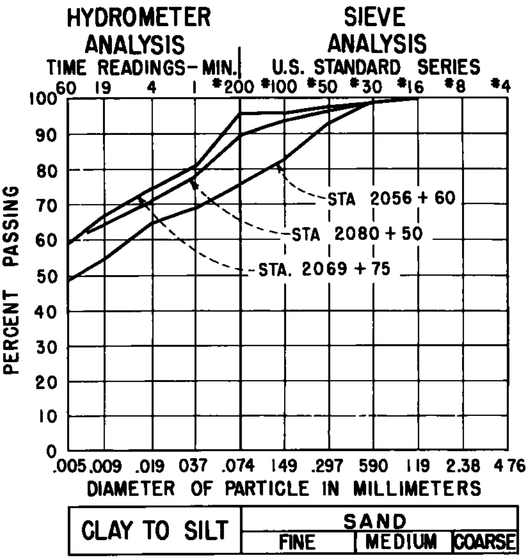


Figure 1. Grading of soil used in lime, and cement stabilization tests--expansive clay from Friant-Kern Canal.

proportion to the amount used; above and below this range for lime, the reduction in swelling was much less noticeable. The stability of the soil was increased as the expansion was decreased principally from the binding action and from a lower soil-water content.

Portland cement which contains a large proportion of lime as one of its original ingredients is believed to have a base exchange and cementing action on soil similar to that of lime. When small quantities of cement on the order of those used in this study are mixed with soil, the resulting material is commonly called "cement modified" soil. This should be distinguished from standard soil cement in which a much larger proportion of cement is used to form a more rigid and durable product. Past studies have shown that a small amount of cement will lower plasticity and shrinkage characteristics of silt-clay soils. This study affords a direct comparison of the two admixtures on the same soil type.

## SOIL SAMPLES

In August 1956, soil samples for the stabilization studies were obtained from the locations on Friant-Kern Canal shown in Table 1.

Table 2 shows the results of Atterberg limits tests on these samples.

The gradation and compaction characteristics of these samples are shown in Figures 1 and 2.

Chemical and petrographic tests made on similar expansive clay from the same area, used in a study of electrochemical treatment (3), showed that the soil is a Ca-beidellite clay with a base exchange capacity ranging between 34 to 54 me/100 grams. The exchangeable base is predominantly calcium. Sodium in amounts from 4 to 10 percent of the total exchange capacity is present.

TABLE 1

| Station | Canal Slope | Distance Above Water[a] |
|---|---|---|
| 2056 + 60 | Left | 6 in. |
| 2069 + 75 | Right | 12 in. |
| 2080 + 50 | Left | 12 in. |

[a] The water elevation at the time of sampling was 439.6.

TABLE 2

| Station | Liquid Limit | Plasticity Index | Shrinkage Limit |
|---|---|---|---|
| 2056 + 60 | 54.7 | 33.7 | 6.9 |
| 2069 + 75 | 63.3 | 36.6 | 6.2 |
| 2080 + 50 | 57.7 | 35.5 | 6.9 |

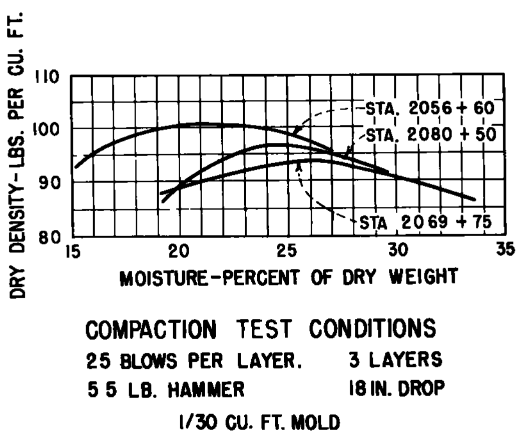


Figure 2. Compaction test curves of soil used in lime and cement stabilization tests--expansive clay from Friant-Kern Canal.

## LIME AND CEMENT USED IN THE TEST PROGRAM

Laguros, Davidson, and Chu (4) have shown that the composition of the lime used for soil stabilization influences the properties of the stabilized soil to a considerable extent. No investigation of this effect was introduced in this study; the lime used was one available in the laboratory at the time. The lime conformed to requirements of ASTM Designation C6-46T (Tentative Specifications for Normal Finishing Hydrated Lime) except for the additional requirement that it "should contain not less than 75 percent of calcium oxide and not more than 5 percent magnesia, based upon the non-volatile portion and shall be of such fineness that the residue on a No. 325 sieve is not greater than 5 percent." A chemical analysis showed that the lime contained 98.16 percent calcium oxide and 0.09 percent magnesia.

The cement used for the stabilization was regular Type I.

## LABORATORY TEST PROGRAM

Besides the gradation and compaction tests shown in Figures 1 and 2, the laboratory test program consisted of the following tests on specimens treated with lime or cement:

1. Atterberg limits, shrinkage, expansion, and unconfined compression tests on specimens containing 0, 2, 4, 6, and 8 percent lime and on specimens containing 2, 4, and 6 percent cement.
2. Triaxial shear and compaction tests on specimens containing 4 percent lime.
3. Wet-dry durability tests on specimens containing 4 percent lime and on specimens containing 4 percent cement.

### Preparation of Specimens

With the exception of some of the unconfined compressive strength specimens which were compacted in molds, the actual test specimens were cut from larger soil specimens of 8-in. diameter by 3-in. height. The larger specimens were prepared by compacting soil, mixed with the desired amounts of lime or cement, at optimum moisture and Proctor maximum density for the untreated soil.

Prior to cutting the smaller test specimens, the larger soil specimens were cured for 28 to 40 days in a 50 percent humidity room. In the room they were suspended over a pan of water and beneath a burlap cover which was supported on a framework. The ends of the burlap were placed in the water to keep it continuously wet and raise the humidity of the atmosphere immediately around the soil.

The smaller test specimens were formed by cutting the soil away by hand beneath a cutting ring of the proper diameter and gently forcing the ring downward over the soil to form the cylindrical specimen. This is a standard Bureau of Reclamation procedure for cutting undisturbed specimens (5).

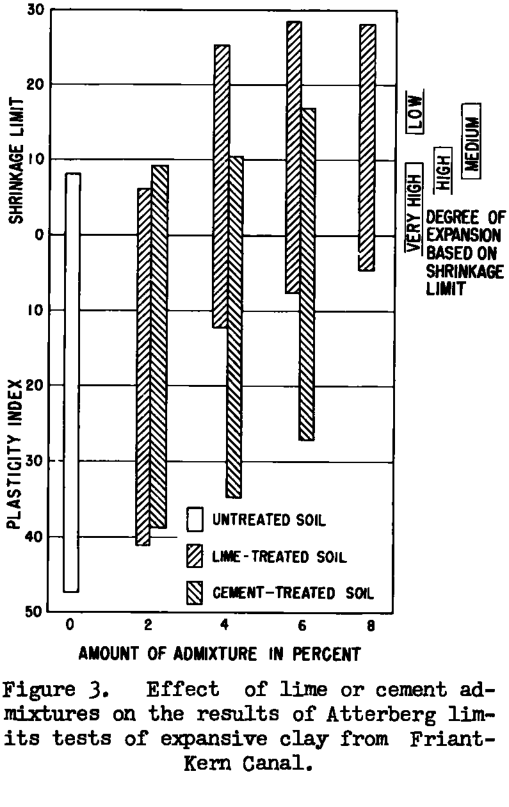


Figure 3. Effect of lime or cement admixtures on the results of Atterberg limits tests of expansive clay from Friant-Kern Canal.

### Atterberg Limits Tests

Atterberg tests for liquid, plastic and shrinkage limits were conducted on the untreated and treated soils after the 30-day curing period. The results of these

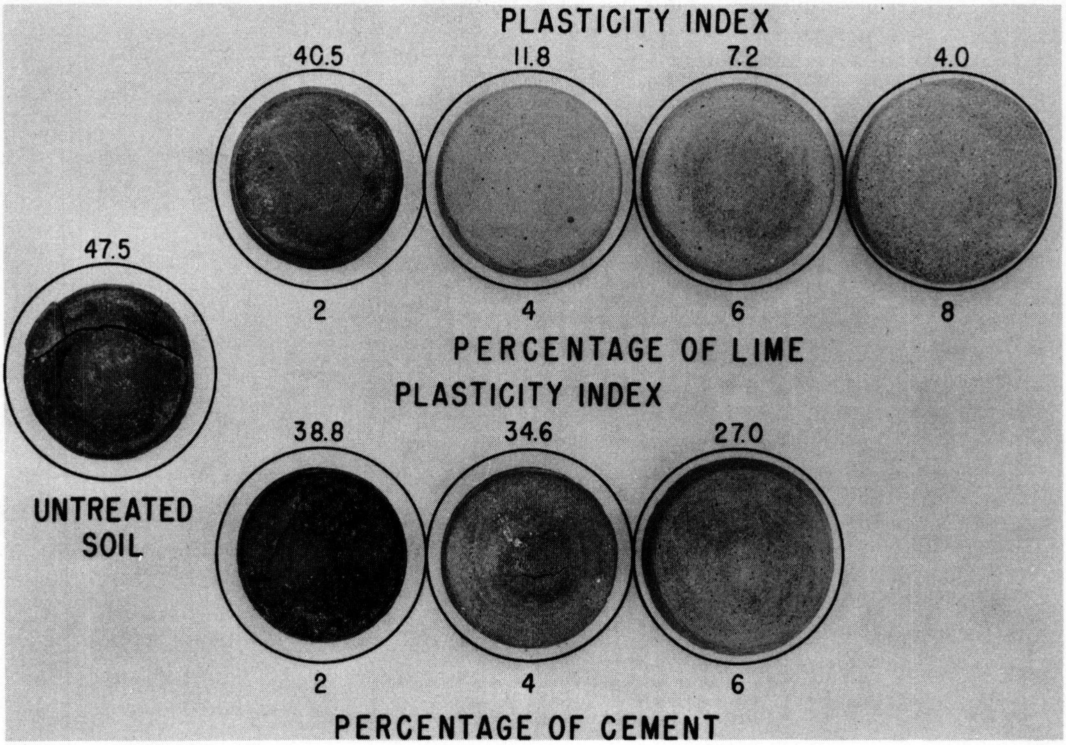


Figure 4. Comparison of sizes of the Atterberg shrinkage limit pats after drying of untreated, lime-, or cement-treated Friant-Kern expansive soil.

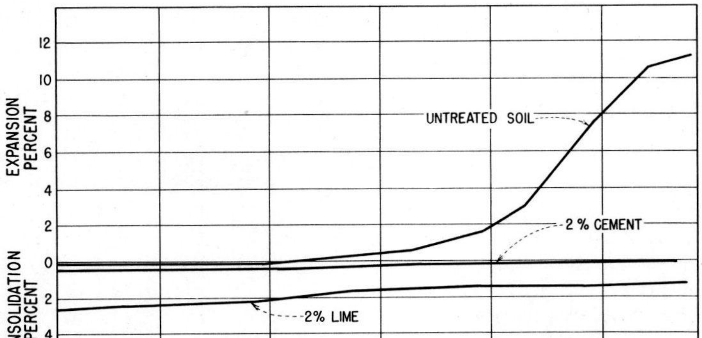


Figure 5. Expansion or consolidation of wetted specimens with time. Friant-Kern Canal expansive clay treated with lime or cement.

tests are plotted in Figure 3. An actual photograph of the dried shrinkage limit pats, for a comparison of the amount of shrinkage after oven drying is shown in Figure 4.

## Expansion and Shrinkage Tests

Expansion tests were conducted on cylindrical specimens 4¼ in. in diameter by 1¼ in. in height in conventional one dimensional consolidometers. For this test, the specimen, which contained the residual moisture after curing under the conditions described above, was subjected to a load of 1 psi and the expansion recorded while an excess of water was kept in contact with the specimen. The results of these tests are shown in Figures 5 and 6.

TABLE 3

| Percent Lime | Percent Cement | Moisture Content: Before Test | After Test: Expansion | After Test: Unconfined Compression: Cut Specimens | After Test: Unconfined Compression: Molded Specimens |
|---|---|---|---|---|---|
| 0 | | 21.8 | 32.4 | 33.9 | 37.8 |
| 2 | | 24.9 | 29.8 | 35.0 | 32.5 |
| | 2 | 27.2 | 27.1 | 29.7 | 32.3 |
| 4 | | 30.0 | 30.2 | 30.6 | 33.9 |
| | 4 | 29.4 | - | - | 31.2 |
| 6 | | 30.5 | 30.4 | 31.8 | 31.2 |
| | 6 | 27.3 | - | - | 28.2 |
| 8 | | 30.3 | 30.2 | 31.3 | 35.9 |

For the shrinkage test, specimens were cut to consolidation specimen size and allowed to dry in laboratory air for a period of 25 days. Then the volumes of the dried specimens were determined by immersion in mercury. The shrinkage, based on the original volume, was computed and expressed as a percentage. The volume change due to expansion and/or shrinkage for each specimen is shown in Figure 6.

## Stability Tests

The stability of the untreated and treated soil was determined by unconfined compressive strength tests and one triaxial shear test on the specimen containing 4 percent lime.

The unconfined compressive strength test specimens were 1⅜ in. in diameter by 2¾ in. in height. The specimens were soaked in water for 7 days prior to testing. The load on the specimens was applied at the rate of 0.005 in. per minute. The triaxial shear tests were conducted in accordance with standard Bureau of Reclamation procedures (5). The results of these tests were plotted in Figure 7a.

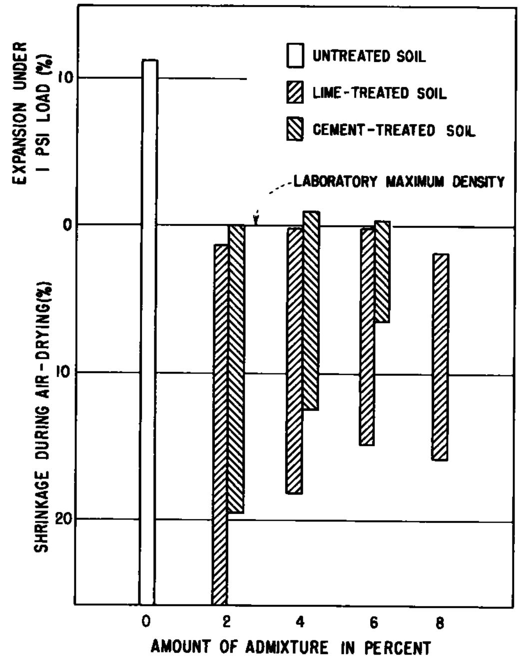


Figure 6. Expansion and shrinkage of untreated, lime-, or cement-treated specimens of expansive clay from Friant-Kern Canal.

## Moisture Content

The moisture content of specimens was determined before and after the expansion tests and the unconfined compression tests. These results are shown in Table 3.

## Wet-Dry Durability Tests

In order to find the stability of the treated soil after periods of wetting and drying, specimens containing 4 percent lime and others containing 4 percent cement were subjected to cycles of wetting and drying and then to the unconfined compressive strength test. These specimens were subjected alternately to 24 hours' submergence in water at approximately 70 F and 24 hours in air at 100 F under an incandescent lamp (Figure 8). Immediately after the final 24-hour submergence

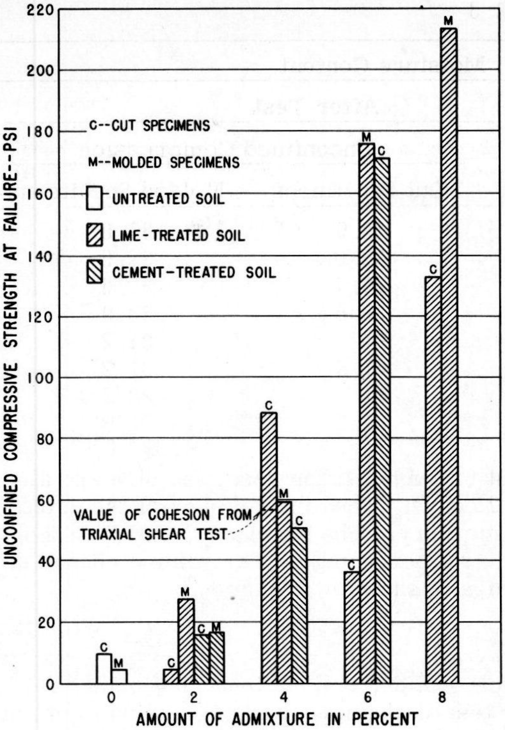


Figure 7a. Unconfined compressive strength of lime- or cement-treated expansive clay from Friant-Kern Canal.

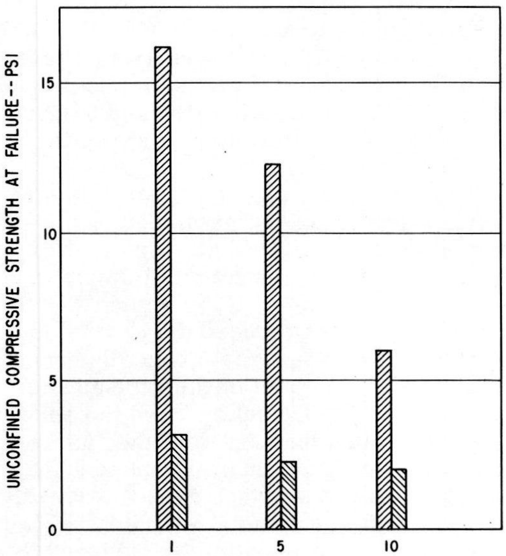


Figure 7b. Unconfined compressive strength of wet-dry test specimens with 4 percent admixture--expansive clay from Friant-Kern Canal.

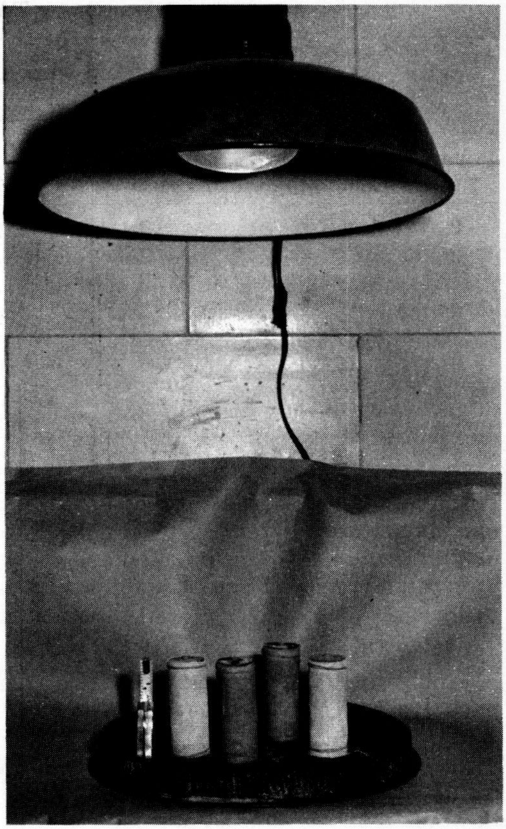

Figure 8. Specimens treated with 4 percent lime or 4 percent cement are being dried under the incandescent lamp at 100 F temperature during the drying cycle of the wet-dry durability tests.

period, unconfined compressive strength tests with a rate of loading of 0.005 in. per minute were conducted on duplicate specimens after 1, 5, and 10 cycles of wetting and drying. The results of these tests are shown in Figure 7b.

## DISCUSSION OF RESULTS

The results of the tests made on lime- or cement-treated Friant-Kern expansive clay specimens show that the presence of either admixture improves the soil for uses such as compacted earth lining or compacted embankment.

Figure 3 shows a plot of the effect of the admixtures on Atterberg limits test results. The use of 2 percent lime or cement reduced the plasticity index of soil only slightly and had practically no effect on the shrinkage limit, but the use of 4 percent or more of lime markedly

TABLE 4

SUMMARY OF TEST RESULTS

| Cement (%) | Lime (%) | Atterberg Limits | | | Unconfined Compression[a] (maximum stress in psi) | | Consolidometer Test | | | Triaxial Shear | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | LL | PI | SL | Cut Specimens | Molded Specimens | Expansion (%) | Shrinkage[b] (%) | Total (%) | Tan $\phi$ | Cohesion (psi) |
| | 0 | 71.9 | 47.5 | 8.1 | 9.3<br>9.3 | 4.7<br>3.2<br>4.0 | 11.2 | 25.8 | 37.0 | | |
| | 2 | 65.4 | 40.9 | 6.5 | 5.5<br>4.3<br>4.9 | 31.7<br>23.7<br>27.7 | -1.3 | 25.8 | 24.5 | | |
| 2 | | 64.1 | 38.8 | 9.4 | 15.3<br>15.8<br>15.9 | 16.4 | 0.1 | 19.6 | 19.7 | | |
| | 4 | 47.2 | 11.8 | 25.3 | 103.6<br>72.7<br>88.1 | 40.1<br>79.1<br>59.6 | -0.1 | 18.2 | 18.1 | 1.23 | 28.5 |
| 4 | | 61.0 | 34.6 | 10.5 | 51.7<br>50.5<br>51.1 | | 0.9 | 12.4 | 13.3 | | |
| | 6 | 43.4 | 7.2 | 28.4 | 33.1<br>39.3<br>36.2 | 122.5<br>229.9<br>176.2 | -0.1 | 14.8 | 14.7 | | |
| 6 | | 53.8 | 27.0 | 17.2 | 173.0<br>172.4<br>172.7 | | 0.3 | 6.5 | 6.8 | | |
| | 8 | 42.1 | 4.0 | 28.4 | 128.7<br>138.1<br>133.4 | 319.0<br>109.3<br>214.1 | -1.7 | 15.8 | 14.1 | | |

[a] Specimens soaked in water for 7 days.
[b] Shrinkage of 4¼-in. diameter by 1½-in. specimens after air drying.

reduced the plasticity index and increased the shrinkage limit; this converts it to a soil of better workability and of less susceptibility to volume change with a change in moisture conditions. As the plot shows, the cement affected these soil properties to a much lesser degree. The difference in the amount of shrinkage of the untreated and various treated soil specimens after the oven drying of the shrinkage limit pats is shown in Figure 4.

As shown by the plot (on the extreme right of Figure 3) of the general range of values of shrinkage limit for soils having a very high to low degree of expansion, as established by previous studies on expansive clays (6), the use of 4 percent or more of the admixtures converted the soil from one of very high expansive potential to one of medium-to-low expansiveness.

It is notable that either 2 percent lime or 2 percent cement reduced the expansion of the soil specimen, subjected to a 1-psi load and a water source, from about 10 percent expansion to practically 0 expansion (Figures 5 and 6). The effect of the lime and cement was about the same in reducing soil expansion, but the cement reduced the shrinkage of air-dried soil specimens about 25 to 50 percent more than did the lime, as seen in Figure 6. This is probably due to the superior cementing action of the cement.

The use of 4 percent or more of lime (also cement, but to a lesser extent) markedly increased the unconfined compressive strength of the soil, as seen in Figure 7a. The value of cohesion from the one triaxial shear test conducted was within the same order of magnitude of cohesion obtained from the unconfined tests if the value of cohesion is considered to be one-half the unconfined compressive strength; the latter relationship has often been used for practical applications. As shown in Figure 7b, the specimens with 4 percent lime resisted the action of wetting and drying better and resulted in much higher compressive strengths at the end of 1, 5, and 10 cycles than did the soil specimens with 4 percent cement.

The cost of cement in Denver is $1.32 for a 94-lb bag while the cost of lime is about $1.00 for a 50-lb bag; therefore, the cost of lime is about 1.4 times the cost of cement on an equal weight basis.

## CONCLUSIONS AND RECOMMENDATIONS

From the results of laboratory tests on specimens of Friant-Kern expansive clay treated with lime or portland cement as admixtures, the following conclusions are drawn:

1. The use of either lime or cement reduces the plasticity, shrinkage, and expansion properties of the soil and increases soil stability, generally in proportion to the amount of admixture used.
2. Although the cement admixture reduced the soil shrinkage under air drying somewhat more than an equal amount of lime (probably because of superior cementing action), the properties of the lime-treated soil were more favorable in other respects, expecially in reduction of plasticity and in increased unconfined strength after wetting and drying action.
3. The recommended amount of lime or cement admixture for use in a field installation is 4 percent.

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THE 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.