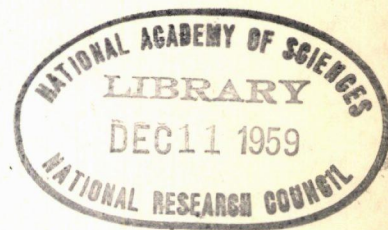


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Bulletin 231

***Lime and Lime-Flyash***  
***as***  
***Soil Stabilizers***



**National Academy of Sciences—**

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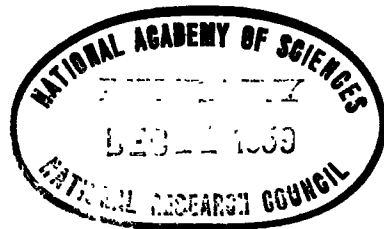
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# Pozzolanic Reactivity Study of Flyash

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In recent years much interest has been shown in soil stabilization methods utilizing the pozzolanic reaction. Lime-flyash stabilization is probably the best known and most thoroughly investigated of these methods. Unfortunately, little is known of the nature of the reaction or reactions involved, or of the reaction products that are formed. This information is needed for evaluating the reactants and improving the reaction.

This paper describes a program of basic research on the nature of the pozzolanic reaction between lime and flyash. Six flyashes of various physical and chemical properties were investigated. The reaction between the flyashes and calcium hydroxide was followed by means of an electroconductivity procedure. The reaction was studied at several temperatures and for several concentrations. The reaction products were investigated by means of x-ray diffraction and differential thermal analysis techniques.

Calcium hydroxide absorption tests, using electroconductivity procedures, on flyashes gave essentially the same evaluation results as those obtained from compressive strength results. The influence of temperature on the reaction was most significant in the 20 to 60 deg C range. A crystalline reaction product was detected, but this may be a secondary product.

An explanation is offered for the mechanism of the pozzolanic reaction. The rate limiting step of the reaction is explained on the basis of diffusion.

●DUE TO increased interest in pozzolans for soil stabilization and other purposes, there have been many investigations of certain materials possessing pozzolanic activity. (The term pozzolanic activity is used to describe the ease with which lime reacts at ordinary temperature with any natural or artificial siliceous or aluminous material in the presence of water to form stable, insoluble compounds possessing cementing properties.) For largely economic reasons flyash has received much attention and study as a pozzolan. However, the majority of the investigations have dealt only with the strength contributing properties of the flyashes and not with the fundamental chemical aspects of the pozzolanic reaction.

Basic studies of the pozzolanic activity of flyashes and other pozzolanic materials are necessary for both practical and theoretical reasons. Explanations for the behavior of a given flyash under given field conditions, such as density and temperature, are of practical importance. Theoretically, if the mechanism and the nature of the reaction products were better known, it might be possible to improve the reaction.

A few basic studies have been made of pozzolans (9, 10, 13). Among these studies, Lea (9, 10) and Moran and Gilliland (14) are noteworthy. Many of the basic concepts of pozzolanic activity postulated by Lea over twenty years ago have not been improved upon. Lea theorized that the reaction was a surface reaction between the dissolved lime and the solid resultant product. He also stated that equilibrium would not be reached except after long periods of time.

Many of the factors that influence the pozzolanic reaction have been recognized, but have not been investigated thoroughly. Recognized factors of the pozzolanic reaction are: temperature, nature of the pozzolan, surface area, carbon content, alkali and



sulfate content, carbon dioxide, hydrogen ion concentration, lime variables, including the calcium to magnesium ratio moisture and time.

The identification of the reaction products of the pozzolanic reaction has been a difficult problem and it has not been completely solved. The problem is analogous to the identification of the hydration products of portland cement. Because of the minute size of particles, the difficulty in isolating them, and the scarcity of necessary identification equipment, very few of the reaction products have been determined. Complex hydrated solid solutions of CaO, SiO<sub>2</sub> and Al<sub>2</sub>O<sub>3</sub> are believed to be the major constituents.

TABLE 1  
PHYSICAL PROPERTIES OF FLYASHES

Flyash No.	Total <sup>1</sup> Specific Gravity	Ignited <sup>2</sup> Specific Gravity	Passing <sup>1</sup> 325 Sieve (%)	Mean <sup>3</sup> Diam. (mm)	Specific <sup>4</sup> Surface, (cm <sup>2</sup> /gm)
10	2.56	2.65	93.1	0.0305	3479
11	2.25	2.53	48.2	0.0380	N.D.
12	2.42	2.55	81.0	0.0310	3370
13	2.93	3.21	65.1	0.0320	N.D.
14	2.67	2.96	94.3	0.0265	3470
15	2.43	2.94	82.3	0.0333	3649

<sup>1</sup> Total sample as received was used.

<sup>2</sup> Total sample ignited in a muffle at 600 deg C was used.

<sup>3</sup> Based on hydrometer and sieve analysis of ignited samples.

<sup>4</sup> Supplied by Robert W. Hunt Co., Chicago, Ill. Based on a gas adsorption method, total sample used.

## PROPERTIES OF THE MATERIALS USED

Six samples of flyash were selected for this study. These samples did not represent all of the property variations found with this material, but it was felt that the major factors were represented. Physical and chemical properties of the flyash used are given in Tables 1 and 2.

It will be noted that the mean diameter values are based on hydrometer and sieve analyses. The difficulty in determining the particle size distribution of flyashes has been discussed by Holton and Reynolds (7); on the basis of their conclusions, the order of magnitude of the mean diameters is prob-

ably close enough for the comparisons made in this study. Ignited samples were used because the combustible portion of flyash has a specific gravity of one or less.

In addition to the physical and chemical analysis, a sodium hydroxide solubility test similar to that used by the Bureau of Reclamation was performed on the flyashes. The results of these tests are shown in Table 3.

Because of the impurities present in commercial limes, reagent grade calcium hydroxide was used to make up all of the lime solutions used in this study.

## TEST PROCEDURE

Absorption methods were explored to determine the rate of reaction between the lime and the flyashes. An electroconductivity method was used to determine the rate of calcium absorption. This method enabled the reaction rate of a single sample to be traced without altering the solution or the material being tested. At certain periods the absorption of calcium was checked by titration procedures.

The test procedure was to place a known amount of material along with 90 ml of lime solution of known concentration in a 10 by 1 in. glass test tube. Each test tube was sealed with a two-holed rubber stopper equipped with two glass tubes. One tube reached about 7 in. into the test tube so that the lime solution could be withdrawn, tested, and returned to the test tube. The other tube was rather short and acted as a vent while the lime solution was being withdrawn and returned to the test tube. Both tubes extended about 1/2 in. above the top of the stopper. A piece of rubber tubing connecting the upper ends of the glass tubes served to keep the test tubes air tight during the reaction period. During testing periods the rubber tube served as a connector between the test tube and the conductivity pipettes. The vent tube was connected to a calcium chloride tube containing barium hydroxide to prevent the formation of calcium carbonate in the solution. Figure 1 shows a test tube in position for testing.

TABLE 2  
CHEMICAL PROPERTIES OF FLYASHES USED

Chemical Analysis <sup>1</sup> Reported As	FLYASH NO.					
	10	11	12	13	14	15
C	3.20	27.67	10.18	2.90	2.10	15.59
SO <sub>3</sub>	3.04	0.41	1.47	2.93	2.00	1.86
CaO	7.30	2.52	5.34	11.22	8.36	6.89
MgO	0.43	1.16	0.77	2.65	0.52	0.85
Fe <sub>2</sub> O <sub>3</sub>	19.30	13.41	21.23	10.75	21.20	19.63
Al <sub>2</sub> O <sub>3</sub>	20.10	13.23	18.39	29.25	22.92	18.19
SiO <sub>2</sub>	43.40	39.18	41.16	39.20	38.90	35.94
H <sub>2</sub> O	0.30	N.D.	N.D.	N.D.	0.17	N.D.
pH <sup>2</sup>	11.50	12.30	12.10	11.80	11.60	11.90

<sup>1</sup> Reported as percent by weight.

<sup>2</sup> Based on 10 gm of total sample in 25 ml of water.

Because of the decrease in solubility of  $\text{Ca}(\text{OH})_2$  with increased temperature and alkalinity, the concentration of  $\text{Ca}(\text{OH})_2$  solutions used were well below the maximum solubility values. This was done to insure against the precipitation of  $\text{Ca}(\text{OH})_2$  in the test tube during the reaction period. Concentrations of about 1.1, 1.0, 0.9 and 0.8 gm per liter were used for reaction temperatures of 20, 40, 60 and 80 deg C respectively. At first deionized water was used to make up the solutions, but later it was found that more satisfactory solutions could be made by double filtering  $\text{Ca}(\text{OH})_2$  solutions made with distilled water. Filtering was necessary to remove small amounts of  $\text{CaCO}_3$  which was formed by calcium combining with small amounts of  $\text{CO}_2$  in the distilled water.

After filtering, the concentrations of the  $\text{Ca}(\text{OH})_2$  solutions were checked by titration methods.

Test tubes were exposed to concentrated  $\text{Ca}(\text{OH})_2$  solutions at the specified temperature before being used. A check of the reactions by electroconductivity methods showed very little reaction between the pre-treated glass and  $\text{Ca}(\text{OH})_2$ .

The samples were placed in constant temperature baths and ovens. Constant

temperature baths were used to maintain reaction temperatures of  $20 \pm 1$  deg C and  $40 \pm 1$  deg C. Ovens were used to maintain the higher temperatures of  $60 \pm 1$  deg C and  $80 \pm 1$  deg C.

Cementation of particles of the reactive material had to be prevented so that the complete sample would remain exposed. In order to prevent cementation the samples were rodged and shaken frequently during the testing period.

Conductivity readings were taken at specified intervals by means of a pipette type conductivity cell (Cenco 700-72). The cell was partially encased in a small water tight plastic box (Fig. 1). Water from the constant temperature bath was pumped through the box to insure a constant temperature during the conductivity reading operation. Samples in the constant temperature baths were tested at their respective temperatures, whereas samples from the ovens were placed in the  $40 \pm 1$  deg C bath and tested at this temperature.

The set-up for testing a sample is shown in Figure 1. Conductivity readings were taken using a null point conductivity bridge (Industrial Instruments Type RC 16 B). Readings were taken at 1,000 cycles per sec in order to prevent reactions at the electrodes. All air coming in contact with the solution passed through calcium chloride tubes containing barium hydroxide to prevent the formation of calcium carbonate during the testing period. Immediately after testing, the 60 and 80 deg C samples were placed in their respective ovens. About two hours was required for the oven samples to reach the bath temperature and be tested and returned to the oven.

Cell constants of the conductivity pipettes were determined using KCl solutions of known specific conductance. Curves giving the relationship between measured resistances and  $\text{Ca}(\text{OH})_2$  concentrations were prepared for each testing temperature. Specific conductivity data given by Lea (10) for  $\text{Ca}(\text{OH})_2$  solutions were utilized for these curves.

TABLE 3

PERCENT REDUCTION IN ALKALINITY OF THE FLYASHES

Flyash No.	Percent Reduction in Alkalinity, $R_a$
10	62.80
11	39.36
12	50.60
13	61.34
14	59.79
15	56.17

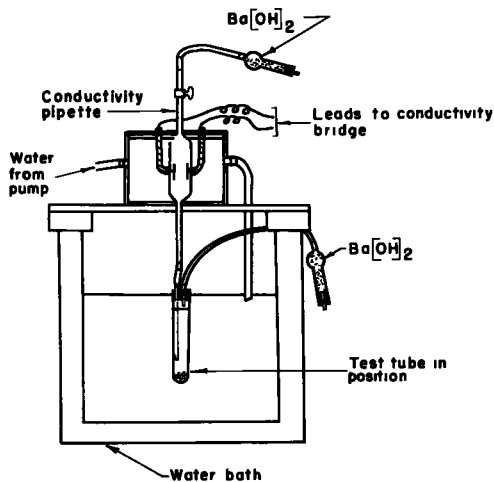


Figure 1. Calcium absorption test apparatus in position for testing.



For a given reaction temperature, three test tubes of each material were prepared. These samples were used to obtain a set of average readings for the initial reaction period and to provide samples for testing the solutions at given time intervals, usually 15, 30 and 45 days after being exposed to the solid material. At the end of the given time intervals, a sample was removed and judiciously filtered to separate the solution from the solid material. The lime concentration of each filtered solution was determined by means of Versenate titrations. The solid material was oven dried at 60 deg C in contact with barium hydroxide to prevent the formation of calcium carbonate.

## DISCUSSION OF RESULTS

Three gram samples of each flyash were allowed to react with  $\text{Ca}(\text{OH})_2$  solutions at 20 deg C for a period of one year. The absorption of lime was followed by electroconductivity methods and checked by Versenate titrations at 90, 180 and 365 days. The values obtained by these two methods checked to within 5 percent. The results of these absorption tests are shown in Figures 2 and 3.

Flyash 10 absorbed the most lime over the 365 day test period and also had the highest rate of lime absorption. The differences between the flyashes were less apparent at the end of the test period than in the initial stages because of the original concentration factor. The absorption by flyash 11, the high carbon flyash, was particularly slow during the initial period, but at the end of 365 days it had absorbed approximately the same amount of lime as samples 12, 13, 14 and 15. The amounts absorbed were limited by the amount of calcium put into solution. Thus, the major differences between the flyashes at this temperature were shown most readily during the initial lime absorption testing period of about 45 days.

It will be noted that negative values of lime absorption were observed. These negative values can be explained by the solubility of compounds in the flyash and perhaps by cation exchange. Sodium, magnesium and calcium compounds reported in flyashes are soluble in partially saturated  $\text{Ca}(\text{OH})_2$  solutions. According to Lerch (12), alkali solutions cannot exist in equilibrium with saturated lime solutions. Referring to Table 2, it is seen that flyash 13 has the largest amount of calcium and

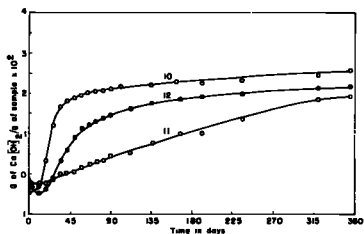


Figure 2. Absorption of calcium by flyash samples 10, 11 and 12 at 20 deg C over a one year period.

magnesium compounds, 13.87 percent (reported as oxides); the greatest amount of negative absorption is observed with sample 13. Sample 11 has the smallest amount of calcium and magnesium oxides, 3.68 percent, and only a slight amount of negative absorption was observed for sample 11. Samples 10, 12, 14 and 15 had approximately the same amounts of calcium and magnesium compounds, 7.73, 6.11, 8.88 and 7.45 percent respectively, and these have intermediate negative absorption values.

If the absorption results are compared with the compressive strength results shown in Figure 4, it will be noted that the initial 45 day absorption results seem to agree with the compressive strength results. In some cases the compressive strength and absorption curves are very similar in shape. The corresponding similarity between the slopes of the compression curves and the absorption curves is particularly striking.

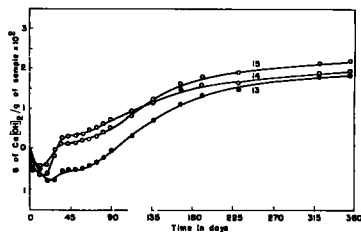


Figure 3. Absorption of calcium by flyash samples 13, 14 and 15 at 20 deg C over a one year period.

It is evident that flyash 10 is the best pozzolan on the basis of compressive strength and lime absorption. The second best pozzolan appears to be sample 12; this conclusion is reached from both the absorption and the compressive strength results. Sample 15 had a rather flat and constant slope in both absorption and compressive strength curves. Sample 11 did not achieve an apparent equilibrium until after about 300 days. The very shallow constant slope of the absorption curve for sample 11 agrees very well with the slope of the strength curve for this flyash.

From the compressive strength and absorption data it appears that a quantitative or semi-quantitative relationship exists between calcium absorption by flyash and its compressive strength in lime-flyash

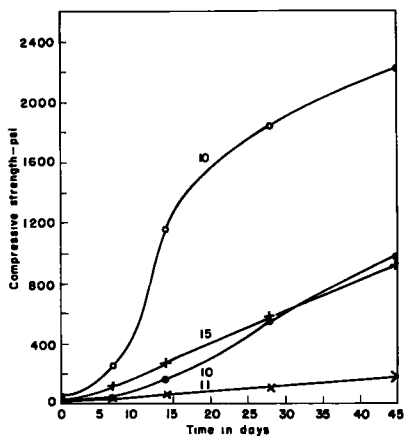


Figure 4. Unconfined compressive strengths of four flyash mortars with 8 percent  $\text{Ca}(\text{OH})_2$  cured at 20 deg C (5). Strengths are for 2 in. diameter by 2 in. specimens at near standard Proctor density which were moist cured at 20 deg C for indicated times, then immersed in water 24 hours before testing.

mixtures. By comparing the maximum slope of the compressive strength curves to the maximum slope of the absorption curves in the figures previously discussed, the relationship given in Figure 5 was obtained. The relationship given in this figure is significant in that it indicates a correlation between absorption and strength; however this relationship is likely to vary with temperature and concentration factors and other physical factors. If a relationship is to be established it should be on the basis of standardized tests.

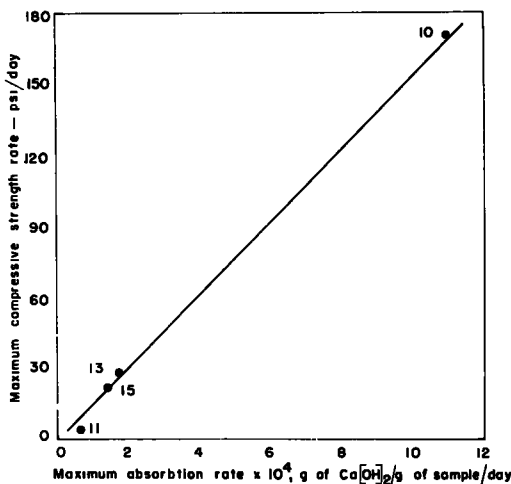


Figure 5. Maximum rate of compressive strength development versus maximum rate of  $\text{Ca}(\text{OH})_2$  absorption based on standardized tests.

### INFLUENCE OF TEMPERATURE

Each sample of flyash was tested by means of the lime absorption method at 20, 40, 60 and 80 deg C. The results of these tests are shown in Figures 6 through 13.

Although 20 deg C absorption curves for the six flyashes have been discussed previously, the curves presented in Figures 6 and 7 represent another set of test data. One day readings are shown in these curves. It will be noted that there was an apparent initial absorption of calcium before the negative absorption that was evident in Figures 2 and 3. There are two plausible explana-

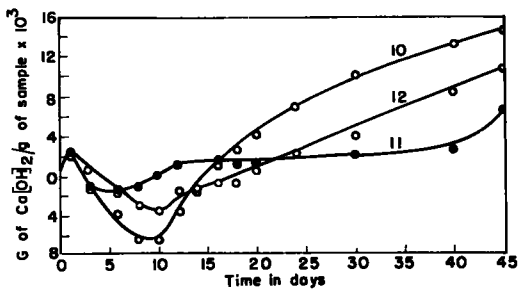


Figure 6. Absorption of calcium at 20 deg C by flyash samples 10, 11 and 12 (3 gm samples).

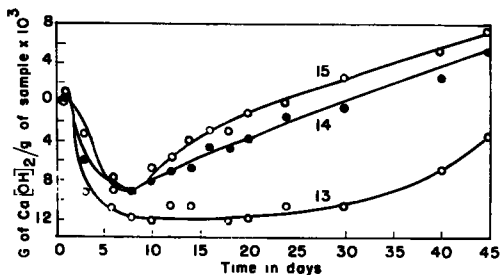


Figure 7. Absorption of calcium at 20 deg C by flyash samples 13, 14 and 15 (3 gm samples).

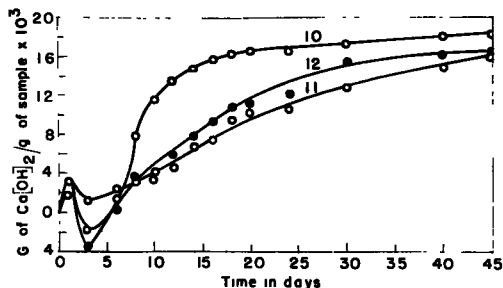


Figure 8. Absorption of calcium at 40 deg C by flyash samples 10, 11 and 12 (3 gm samples).

tions for the initial up-take of calcium. Possibly it is due to an ion exchange reaction, such as calcium ions replacing other ions in the flyash. Another possibility is that it is due to charge deficiencies that exist with the flyash as a consequence of its non-crystalline glassy nature.

The influence of temperature on the rate of absorption of  $\text{Ca}(\text{OH})_2$  by the flyashes was very pronounced with all flyashes. In all cases, after initial reactions, the rate of absorption increased with temperature. After examining these figures, it is obvious that the pozzolanic reaction or reactions observed were endothermic reactions, that is, flyash  $\text{Ca}(\text{OH})_2$ , and  $\text{H}_2\text{O}$  pozzolanic reaction products. Increasing the amount of heat drives the reaction faster to the right. This increase in reaction rate due to an increase in temperature has been termed the Arrhenius effect (6).

It will be noted that the periods of initial and negative absorption decrease as the

temperature increases. At 60 deg C very short periods of negative absorption were observed. Neither initial reactions or negative absorptions were evident during the 80 deg C test period. All curves at 80 deg C have the same shape and the maximum ordinates are essentially the same. High temperature results are of little use for the selection of desirable pozzolans because of the rapidity of the reaction.

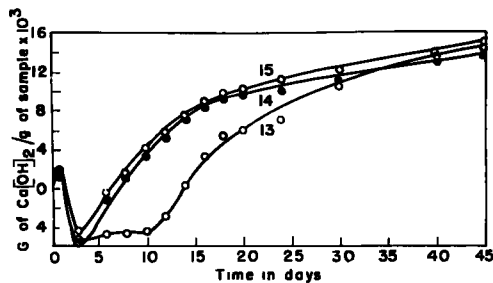


Figure 9. Absorption of calcium at 40 deg C by flyash samples 13, 14 and 15 (3 gm samples).

The curves shown in Figure 14 illustrate the over-all effect of the temperature range used on the six flyashes studied. Because of the initial reactions and solubility effects observed, the times given for one-half of the original  $\text{Ca}(\text{OH})_2$  concentrations to be depleted are not true half lives. By means of Figure 14 it is possible to estimate the effect of temperature on the pozzolanic reaction for the flyashes beyond the range of temperature used in this study. Relatively speaking, little was gained by raising the reaction temperature above 50 deg C. However,

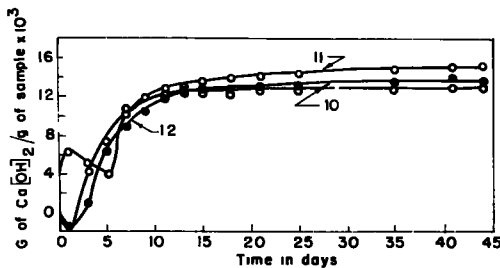


Figure 10. Absorption of calcium at 60 deg C by flyash samples 10, 11 and 12 (3 gm samples).

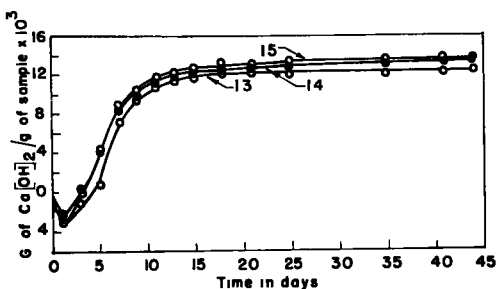


Figure 11. Absorption of calcium at 60 deg C by flyash samples 13, 14 and 15 (3 gm samples).

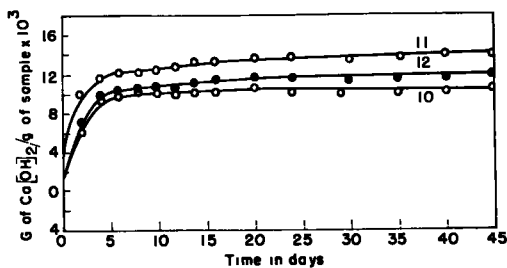


Figure 12. Absorption of calcium at 80 deg C by flyash samples 10, 11 and 12 (3 gm samples).

between 20 and 50 deg C the influence of temperature on the reactions was quite noticeable. Only at temperatures less than 50 deg C was it possible to distinguish between the flyashes from their absorption rates. The curves shown in Figure 14 asymptotically approach time infinity as the temperature decreases below 20 deg C. For practical purposes, at temperatures below 20 deg C most of the flyashes can be assumed to be almost non-reactive.

Compressive strengths of lime-flyash mortars at 60 deg C using four of the six flyashes are shown in Figure 15. The strength differences between the flyashes at this temperature are more pronounced than at 20 deg C (Fig. 4). From the figure it is

noted that samples 10 and 11 are respectively the most satisfactory and the most unsatisfactory pozzolans; this is also the case when the mortars were cured at 20 deg C. Unlike the compressive strengths at the 20 deg C curing temperature, all samples reached about 90 percent of their strengths in about seven days. From Fig-

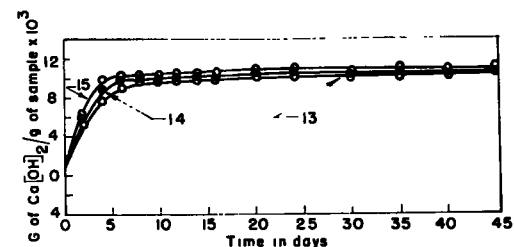


Figure 13. Absorption of calcium at 80 deg C by flyash samples 13, 14 and 15 (3 gm samples).

ures 10 and 11, it can be seen that the major portion of the lime absorption occurred within the first ten days. The rates of initial strength gains correspond with the rates of initial calcium absorption for each flyash. However, the maximum ordinates of the absorption curves are nearly the same, due to calcium depletion whereas the maximum ordinates of the compressive strength curves are considerably different. Therefore, it is evident that there is a relationship between the absorption of lime and the compressive strength, but the total strength development of lime-pozzolan mortars is dependent on factors in additions to the pozzolanic reaction.

#### INFLUENCE OF PARTICLE SIZE ON THE REACTION

The influence of particle size on the reaction was studied by using material passing the number 200 sieve (0.074 mm) and the number 325 sieve (0.044 mm) from flyashes

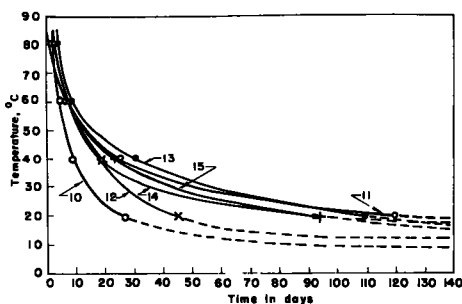


Figure 14. Effect of temperature on time required for one-half of  $\text{Ca(OH)}_2$  to be absorbed.

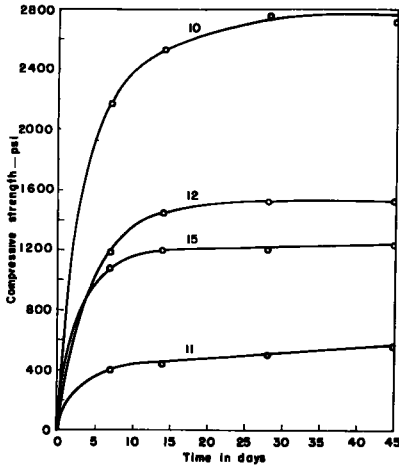


Figure 15. Unconfined compressive strengths of four flyash mortars with 8 percent  $\text{Ca}(\text{OH})_2$  cured at 60 deg C (5). Strengths are for 2 in. diameter by 2 in. specimens at near standard Proctor density which were moist cured at 60 deg C for indicated times, then immersed in water 24 hours before testing.

The majority of the larger particles in flyashes are composed of unburned coal and coke which is reported as loss-on-ignition in Table 4. From this table the decrease in organic content with grain size is very evident. The larger flyash particles are not very active. Also, the larger particles have low specific gravity, less

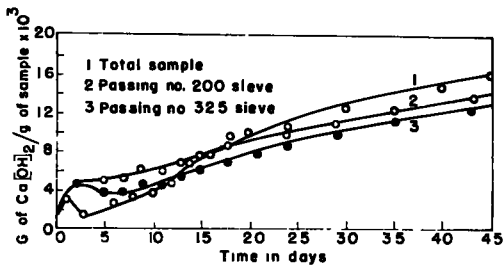


Figure 17. Influence of grain size of flyash sample 11 on calcium absorption at 40 deg C.

curves obtained had essentially the same characteristics as the 3 gram sample curves. However, at any time interval, the amount of  $\text{Ca}(\text{OH})_2$  absorbed per gram by the  $1\frac{1}{2}$  gram samples was nearly twice that absorbed by the 3 gram samples. The influence of concentration on the reaction will be discussed on the basis of diffusion later in this report.

10 and 11. Flyash 10 was used because all tests indicated it was the most reactive flyash of the six tested, whereas flyash 11 was used because all tests indicated it was unsuitable as a pozzolan.  $\text{Ca}(\text{OH})_2$  solutions used at each temperature were of equal concentrations. Three gram samples were used.

Figures 16 and 17 are two sets of curves given to illustrate the influence of particle size on  $\text{Ca}(\text{OH})_2$  absorption at a given temperature. Practically speaking, there was little difference between the rates of absorption and the maximum amounts of absorption for the different fractions at a given temperature.

An increase in reaction rate with increasing surface area was not evident in these tests; this may be explained on the basis of another property of the flyashes.

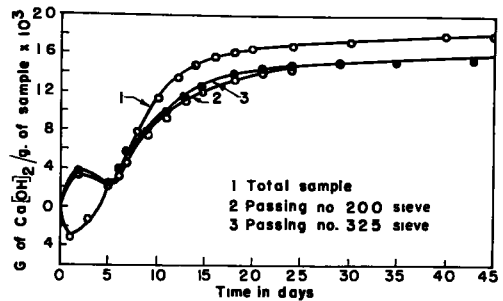


Figure 16. Influence of grain size of flyash sample 10 on calcium absorption at 40 deg C.

than 1.0. Hence, there was little change in the amount of active surface area present when the particle size samples used were compared on the basis of weight. Although the presence of organic material had little effect on the chemical activity of the flyashes, there is reason to believe that it influences the compressive strength results; this will be discussed later.

#### INFLUENCE OF THE LIME-FLYASH RATIO

The influence of the lime-flyash ratio on the reaction was studied by decreasing sample weights one-half while keeping  $\text{Ca}(\text{OH})_2$  concentrations constant. The curves obtained had essentially the same characteristics as the 3 gram sample curves. However, at any time interval, the amount of  $\text{Ca}(\text{OH})_2$  absorbed per gram by the  $1\frac{1}{2}$  gram samples was nearly twice that absorbed by the 3 gram samples. The influence of concentration on the reaction will be discussed on the basis of diffusion later in this report.

TABLE 4

## INFLUENCE OF PARTICLE SIZE ON LOSS-ON-IGNITION

Size Fraction Finer Than	Percent Loss-on-Ignition	
	Flyash 10	Flyash 11
0.420 mm	3.20	27.67
0.074 mm	2.34	19.55
0.044 mm	2.11	10.32

initial period. Although all of the 45 day percentage increases in compressive strengths are positive they are erratic and do not correspond to the increases in absorption. The compressive strengths of the lime-flyash mixtures increased with increasing amounts of  $\text{Ca}(\text{OH})_2$ , but the amount of increase was dependent on other factors in addition to absorption factors.

## REACTION PRODUCTS

After flyash samples had been tested by means of the absorption test procedure, they were removed from the solutions. The solid material was filtered from the solutions and dried in the presence of barium hydroxide to prevent carbonation. After being dried, the flyash samples were subjected to optical, differential thermal analysis and x-ray diffraction procedures in order to determine the nature of the reaction products formed.

TABLE 5

PERCENT INCREASE IN COMPRESSIVE STRENGTH AT THE END OF VARIOUS TEST PERIODS DUE TO INCREASING  $\text{Ca}(\text{OH})_2$  CONTENT FROM 2 TO 4 PERCENT AT 20 AND 60 DEG C (5)

Flyash Sample No.	Percent Increase in Compressive Strength							
	20 Deg C				60 Deg C			
	Test Period, Days				Test Period, Days			
	7	14	28	45	7	14	28	45
10	-31.1	24.7	41.6	62.0	41.6	54.4	56.7	19.8
11	0.8	2.7	2.2	15.1	43.3	38.0	45.2	36.4
12	-51.2	0.2	6.2	47.2	44.1	49.1	36.5	46.1
15	4.3	7.5	6.3	25.8	15.7	6.3	23.3	15.5

No differences between treated and untreated samples could be ascertained by the use of the petrographic microscope. Previous research in the field of cement chemistry has also indicated that petrographic techniques are of little value for this type of work because of the small amounts of reaction products and their minute colloidal size.

For the most part, the differential thermal analysis results were not as satisfactory as the x-ray diffraction results. The presence of unreacted flyash in the absorption samples partially masked the peaks of the small amounts of reaction products. Also, the lack of established data about the reaction products as determined by this method tended to limit the applications of the apparatus.

X-ray diffraction patterns of the  $\text{Ca}(\text{OH})_2$  treated flyash samples were compared with patterns for untreated samples. All samples were treated with ethylene glycol before x-ray examination.

Flyash samples that had been exposed to  $\text{Ca}(\text{OH})_2$  in solution for 365 days at 20 deg C

Percent increases in compressive strengths due to doubling the amount of  $\text{Ca}(\text{OH})_2$  at two temperatures are shown in Tables 5 and 6. At 20 deg C negative values were obtained for some of the 7 and 14 day test periods. This probably indicates that cementation had not taken place to a significant extent during the test period and that the  $\text{Ca}(\text{OH})_2$  decreased the shear strength by acting as a lubricant during the

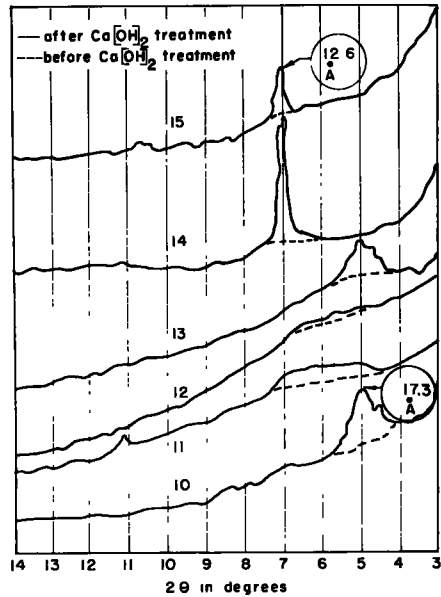


Figure 18. X-ray spectrometer curves for flyash samples used in the absorption tests.



yielded x-ray diffraction patterns that indicated a reaction product. The significant peaks obtained for the six flyashes tested are shown in Figure 18. The dotted lines indicate the x-ray diffraction pattern obtained from untreated samples of the same fly-ash. From the figure, it is obvious that a reaction product was formed and that it is crystalline, as shown by the definite peaks indicating an orderly arrangement of atoms. Two distinct spacings are shown in Figure 15, one at  $12.6\text{\AA}$  and another at  $12.3\text{\AA}$ ; these spacings may be transitional as evidenced by the broad peaks obtained from samples 11 and 12. In addition to the peaks in Figure 18, faint peaks were obtained for samples 10 and 14 at 29 deg. This corresponds to a spacing of about  $3.08\text{\AA}$ . Carbonate peaks that were obtained from the untreated samples were not obtained from the treated samples.  $\text{Ca}(\text{OH})_2$  peaks were not obtained from the x-ray diffraction of the treated samples.

TABLE 6  
PERCENT INCREASE IN COMPRESSIVE STRENGTH AT THE  
END OF VARIOUS TEST PERIODS DUE TO INCREASING  
 $\text{Ca}(\text{OH})_2$  CONTENT FROM 4 TO 8 PERCENT AT  
20 AND 60 DEG C (5)

Flyash Sample No.	Percent Increase in Compressive Strength							
	20 Deg C				60 Deg C			
	Test Period, Days				Test Period, Days			
7	14	28	45	7	14	28	45	
10	-3.7	-10.8	6.0	11.0	6.9	17.6	19.0	36.6
11	-4.5	9.8	15.8	30.0	11.7	21.2	17.7	27.3
12	8.6	34.4	12.5	18.2	25.4	34.4	78.0	86.0
15	60.8	32.0	33.4	40.3	45.9	39.9	39.6	73.1

investigators in connection with portland cement hydration products (2, 11). It is frequently observed at temperatures below 100 deg C with various molar ratios of  $\text{CaO}$  to  $\text{SiO}_2$  (2). The diffuse scattering of x-rays by calcium silicate hydrate I has been attributed to water. Previous research indicates that it is a layer type mineral similar in structure to montmorillonite and other swelling clay minerals (2). The basal reflections vary with the amount of water between the layers. Treatment of the samples with ethylene glycol permits a shift in the C-dimension of the mineral due to the absorption of the organic molecules between the individual layers. This usually results in sharper x-ray diffraction peaks.

Examination of x-ray diffraction patterns for the flyash samples from the higher temperature absorption tests did not reveal a crystalline reaction product. Also, reaction product peaks were not observed in the diffraction patterns for 20 deg C absorption samples that were removed from the  $\text{Ca}(\text{OH})_2$  solutions at 45 days and less. All absorption test results indicated that the reaction period for most samples was over at the end of 45 days. Furthermore, normally under hydrothermal conditions, crystals grow in thickness with time and the largest crystal growths occur, though more slowly, at ordinary temperatures. Hence, it appears that a crystalline product did not form as the reaction proceeded and that the crystalline product developed from a non-crystalline reaction product. This intermediate product is probably a gel. Aside from the existence of regular atomic order in the calcium silicate hydrate crystallites, there probably is little difference between the crystallites and the amorphous gels. According to Bernal (2), the size of the particles that make up some classical gels are well within the range of the calculated dimensions of calcium silicate hydrate I.

#### FACTORS AND MECHANISM OF THE REACTION

The presence of cementitious reaction products was proved indirectly by the compressive strength tests and the absorption tests. Apparently the compressive strength of lime-flyash mortars is dependent on the amount and number of contacts of the cementitious material. The presence of coke and other organic, non-reacting materials

The x-ray diffraction pattern obtained for the 365 day samples is similar to the two major peaks of calcium silicate hydrate I; the spacings of this compound are given in Table 7. Calcium silicate hydrate I has been studied and reported by several

TABLE 7  
POWDER X-RAY DIFFRACTION PEAKS FOR  
 $\text{CALCIUM SILICATE HYDRATE I}$  (11)

d Spacings in $\text{\AA}$	Relative Intensity
9-14 (highly variable)	10
3.06	10
2.81	8
1.83	8
1.67	4
1.53	2
1.40	4
1.17	1
1.11	2
1.07	1

TABLE 8  
K CONSTANTS FOR FLYASH SAMPLES

Flyash Sample No.	Solution Temperature, deg C	Constant $K \times 10^7$ cm <sup>2</sup> /day/gm/liter	
		3 gm Sample	1½ gm Sample
10	20	5.88	16.52
	40	7.37	21.50
	60	9.80	37.40
	80	14.15	55.40
11	20	1.43	3.25
	40	3.02	9.60
	60	6.10	20.60
	80	8.53	27.60
12	20	3.43	7.64
	40	4.66	16.00
	60	6.40	20.90
	80	9.10	25.60
13	20	0.00	0.00
	40	4.91	19.58
	60	9.98	24.90
	80	12.00	31.00
14	20	3.48	14.45
	40	5.18	18.20
	60	8.05	32.50
	80	11.25	49.80
15	20	3.95	14.10
	40	7.02	18.45
	60	9.98	29.80
	80	14.03	49.10

prevents contacts of the cementitious material that forms on the surface of the pozzolan and may be a gel that later forms a crystalline product.

If the desired cementitious product forms on the surface, it follows that in order for any more reaction product to be formed, the necessary calcium must pass through the reacted layer to react with the enclosed pozzolan. Thus, the mechanism of the pozzolanic reaction is one of simultaneous diffusion and chemical reaction of the calcium. The activity of the reacting material is limited by the rate of diffusion of calcium through the reaction product.

The mathematical theory of diffusion in isotropic substances, according to Crank (4) and Barrer (1), is based on the hypothesis that the rate of diffusion through a unit area of section is proportional to the concentration gradient normal to the section, that is,  $\frac{dQ}{dt} = -D \frac{c}{x}$  where  $\frac{dQ}{dt}$  is the

rate of transfer per unit area of section,  $c$  is the concentration of the diffusing substance,  $x$  is the space coordinate measured

normal to the section, and  $D$  is called the diffusion coefficient.

It is assumed that the rate-limiting step is diffusion of calcium through the reaction zone. The reacting material is assumed to be in the form of a sphere. It is also assumed that the thickness of the reaction zone is negligible in comparison to the diameter of the reacting material so the surface can be considered a plane, and the following mathematical development can be made.

- Let  $x$  = thickness of reacting zone  
 $Q$  = amount of  $\text{Ca}(\text{OH})_2$  which has reacted/unit area  
 $\bar{V}$  = molar volume of the product  
 $C$  = concentration of  $\text{Ca}(\text{OH})_2$  at any time  
 $C_0$  = initial concentration of  $\text{Ca}(\text{OH})_2$   
 $R$  = total amount of  $\text{Ca}(\text{OH})_2$  reacting  
 $S$  = specific surface of solid

If  $x = Q\bar{V}$

then  $\frac{dQ}{dt} = \frac{DC}{x} = \frac{DC}{Q\bar{V}}$ .

The amount removed by reaction will be  $\bar{V}(C_0 - C)$ . By reaction stoichiometry

$$\bar{V}(C_0 - C) = \alpha QS$$

where  $\alpha$  is a "ratio of small whole numbers." Hence

$$C = C_0 - \frac{\alpha QS}{\bar{V}} = C_0 - \beta Q$$

where  $\beta = \alpha \frac{S}{\bar{V}}$

hence  $\frac{dQ}{dt} = \frac{D(C_0 - \beta Q)}{Q\bar{V}}$

thus  $\frac{Dt}{\bar{V}} = \frac{-Q}{\beta} \frac{-C_0}{\beta^2} \ln \left( 1 - \frac{\beta Q}{C_0} \right)$ .

Expanding the equation by MacLaurin's power series the equation becomes

$$\frac{Dt}{\bar{V}} = \frac{Q^2}{C_0} \left[ 0 + \frac{1}{2} \left( \frac{\beta}{C_0} \right) \frac{Q}{3} + \dots + \left( \frac{\beta}{C_0} \right)^{n-2} \frac{Q}{n} n^{-2} + \dots \right]$$

If  $\frac{\beta Q}{C_0}$  is less than 0.1 then all terms greater than the third term are negligible.

Then 
$$\frac{Dt}{\bar{V}} = \frac{Q^2}{2C_0}$$

and 
$$Q = \left( \frac{2C_0 D}{\bar{V}} \right)^{1/2} (t)^{1/2}$$

If 
$$K = \left( \frac{2C_0 D}{\bar{V}} \right)^{1/2}$$

and 
$$R = QS.$$

The following relationship is obtained

$$R = KS (t)^{1/2}$$

Assuming that the reacting particles are spherical in shape and heterogeneous, the following equation may be used for the specific surface.

$$S = \frac{4 \pi r^2}{\frac{4}{3} \pi r^3 \lambda} = \frac{3}{\lambda r}$$

where  $\gamma$  and  $r$  are, respectively, the specific gravity of the material and the mean radius of the reacting matter. Hence, the following relationship can be obtained

$$R = \frac{3K(t)^{1/2}}{\lambda r}$$

This equation is valid only for cases where the concentration is high. The  $K$  values given in this report are for the initial portion of the reaction. The relationship for the lower concentrations can be developed using the adjustable parameters  $\beta$  and  $D$  instead of  $K$ .

Absorption data for the flyashes in  $\text{Ca}(\text{OH})_2$  solutions satisfied the relationship given previously since linear plots of  $R$  versus  $\sqrt{t}$  were obtained until the concentration of  $\text{Ca}(\text{OH})_2$  dropped to about 0.1 gm/liter. From the slopes of these plots, average values of  $K$  were calculated. The  $K$  values obtained for the flyash samples are given in Table 8.

The  $K$  values indicated are not diffusion coefficients, but they do indicate the trend of the diffusion coefficients. It will be noted, from Table 8, that the rate of diffusion increases with temperature. The relationship

$$\log KC_0^{1/2} = C_1 + \frac{C_2}{T}$$

where  $T$  is the absolute temperature, may be used to express the influence of temperature on the rate of diffusion. Secondly, the rate of diffusion was increased by decreasing the amount of reacting sample while keeping the concentration of the  $\text{Ca}(\text{OH})_2$  constant.

It will be further noted that the better flyashes, as determined by the other tests, also give the higher  $K$  values. Probably, this results from the fact that there is normally more reactive material in these samples and they have a higher specific surface area.

## CONCLUSIONS

The rate of compressive strength development by lime-flyash mortars is directly related to the rate of lime absorption of the flyash. The greater the rate of lime ab-

sorption, the greater the rate of compressive strength development. The rate of lime absorption is limited by the rate of diffusion of the calcium through the reaction product. The rate of diffusion, which is increased by increasing the temperature and the lime concentration, varies with the type of flyash used. At temperatures less than 20 deg C most flyashes may be considered non-reactive.

The source of the strength of lime-flyash mortars is the reaction products that form as the result of the pozzolanic reaction. From x-ray diffraction studies, it appears that a crystalline product does not form at first, but develops from a non-crystalline reaction product. The initial product is probably a gel. The final crystalline product is believed to be calcium silicate hydrate I, a reaction product that has been found in set portland cement pastes.

Unburned organic material that is found in most flyashes is nonreactive with calcium hydroxide and its presence seems to indicate a flyash of coarse grain size in both the organic and inorganic phases. Since the total mortar strength developed depends on the number of contacts of the cementitious reaction products, the organic material breaks the continuity of this system and thus decreases the total strength.

#### ACKNOWLEDGMENTS

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#### REFERENCES

1. Barrer, Richard M., "Diffusion In and Through Solids," Univ. of Cambridge Press, Cambridge, (1951).
2. Bernal, J. D., "The Structures of Cement Hydration Compounds," Proc. of the Third Symposium on the Chem. of Cement, London, 1952:216-261, (1954).
3. Bessey, G. E., "The Estimation of Free Calcium Hydroxide in Pozzolan Mixes," Proc. of the First Symposium on the Chemistry of Cement, Stockholm, 1938:484-488, (1939).
4. Crank, J., "The Mathematics of Diffusion," Oxford on the Clarendon Press, London, (1956).
5. Davidson, D. T., Delbridge, Norman C., Jr., and Sheeler, J. B., "Reactivity of Four Types of Flyashes with Lime," Highway Research Board, Bull. 193, (1958).
6. Frost, Arthur A., and Pearson, Ralph G., "Kinetics and Mechanism," John Wiley and Sons, New York, (1953).
7. Holton, W. C., and Reynolds, Dana F., Jr., "A Comparison of Size-Consistent Determinations of Duplicate Samples of Flyash," Combustion, 14:41-46, (1954).
8. Klug, H. P., and Alexander, L. E., "X-ray Diffraction Procedures for Polycrystalline and Amorphous Materials," John Wiley and Sons, New York, (1954).
9. Lea, F. M., "The Chemistry of Pozzolans," Proc. of the First Symposium on the Chemistry of Cement, Stockholm, 1938:460-490, (1939).
10. \_\_\_\_\_, "Investigation on Pozzolans," Great Britian Dept. of Scientific and Industrial Building and Research, Tech. Paper No. 27, (1940).
11. \_\_\_\_\_ and Desch, C. H., "The Chemistry of Cement and Concrete," St. Martins Press, New York, (1956).
12. Lerch, W. C., "Studies of Some Method of Avoiding Expansion and Pattern Cracking Associated With the Alkali-Aggregate Reaction," Am. Soc. for Testing Materials, Spec. Tech. Pub., No. 99, pp. 153-178, (1950).
13. Mielenz, R. C., Ramaley, D., Benton, E. J., and Geier, F. H., "Pozzolans and Cement-Pozzolan Reactions," U. S. Bureau of Reclamation, Petrographic Laboratory Report No. 102, (1952).

14. Moran, W. T., and Gilliland, J. L., "Summary of Methods for Determining Pozzolanic Activity," Am. Soc. for Testing Materials, Spec. Tech. Pub. No. 99, pp. 109-131, (1950).

## Discussion

Z. C. MOH, Massachusetts Institute of Technology— In the past few years considerable increase in the interest in using pozzolanic materials, both natural and synthetic, has led to the recognition of the importance of understanding the nature of the pozzolanic activity and the reaction products. The authors have made an important contribution in this respect. However, some of the conclusions described in the paper seem to be somewhat inconsistent with the data presented. The writer wishes to discuss these inconsistencies.

The authors interpreted the products of the reaction between lime and flyash after one year as calcium silicate hydrate I (CSH-I) based on their x-ray diffraction pattern. However, the evidence, as shown in Figure 18, is not conclusive. The x-ray patterns reveal only the presence of one line, the 12.6 Å line, which is only one of the strong lines of CSH-I (Table 9). The two strongest lines of CSH-I, 3.06 Å and 2.80 Å (15, 16, 19), are not present in the authors' data. Furthermore, the 17.3 Å of flyash No. 10 does not correspond with any calcium silicate hydrate pattern reported in the literature. Moreover, CSH-I has a very large exothermic peak at about 830 to 900 deg C on the DTA pattern (17). The authors did not indicate such.

According to Bogue (16), Taylor (19), and others, the calcium silicate hydrates formed in a lime solution of concentration well below saturation are low in lime. If one accepts the authors' interpretation of the end products as CSH-I, this naturally would be the low lime hydrous silicate, that is,  $\text{CaO} \cdot \text{SiO}_2 \cdot \text{aq}$ . The maximum lime absorption after one year at 20 deg C, as reported in the paper, was 2.5 gm  $\text{Ca}(\text{OH})_2/\text{gm}$  of flyash  $\times 10^2$  (flyash No. 10 in Fig. 2). This value is equivalent to 0.0189 gm  $\text{CaO}/\text{gm}$  of flyash. With  $\text{CaO}$ -to- $\text{SiO}_2$  ratio of one, this means only 0.0435 gm/gm of the flyash was reacted (flyash No. 10 has 43.4 percent  $\text{SiO}_2$ ); that is, only 4 percent of the pozzolan participated in the reaction in a period of one year. Furthermore, in

the authors' experiments the flyash was prevented from cementation. The total surface area exposed to the reaction would undoubtedly be much greater than that in a compacted lime-flyash mortar. In other words, if the experiments described really represent the type of reaction postulated by the authors in a lime-flyash mixture, it is inconceivable that such a small percentage of reactants was able to bind all the fine particles providing fairly high compressive strength. (Although there are no strength data of mixtures with 2 to 4 percent lime presented in the paper, however, it can be deduced from Figure 4 and from Tables 5

TABLE 9

X-RAY DIFFRACTION PATTERNS OF CALCIUM SILICATE I

References	d Spacing in Å	Relative Intensity
Taylor (15)	10.0 - 11.5	Strong
	3.03	Very strong
	2.80	Medium strong
	1.81	Medium strong
Bogue (19)	13.72	100
	3.06	100
	2.80	100
Bernal (16) <sup>1</sup>	10 - 15	7
	3.1	10
	2.8	9

<sup>1</sup>Interpreted from line diagram.

and 6 that the one-year strength of such mixtures would be fairly high.)

Furthermore, the decreases in strength (20 deg C) at 7 and 14 days by increasing the calcium hydroxide contents in the mixture (Tables 5 and 6) contradict the absorption data discussed. The increase in absorption by lime at all curing ages would indicate to the writer an increase in the total amount of reaction product at all curing ages. Hence, higher strength would be expected—as stated by the authors: "The strength of lime-flyash mortars is dependent on the amount and number of contacts of the cementitious materials." Therefore, it seems likely that the absorption test is not adequate to account for the proposed mechanism.

The authors explained the reaction rate on the basis of diffusion. It is clearly stated

by the authors that rate of diffusion is a function of  $\text{Ca}(\text{OH})_2$  concentration. Therefore, at higher lime content the rate of calcium ion diffusion is faster; the reaction rate between lime and flyash should then be faster. In other words, the strength development should also be faster at high lime content than at low lime content, but the data in Tables 5 and 6 do not agree with this explanation.

The negative absorption and also the initial absorption at temperatures above 20 deg C cannot be satisfactorily explained by exchange reaction or due to soluble components of the flyash as stated by the authors. Otherwise, the conductivity measurement would not represent the lime concentration in solution and definitely would not agree with titration data if titrations were carried out at those ages. The reverse absorption phenomena may be explained by the view of uneven distribution of ions on the flyash particles as described for cements (18).

The majority of large particles in flyashes are composed of unburned coal and coke which generally have little effect on the chemical reactivity of the pozzolans, as stated by the authors. Yet, as shown in Figure 6, Sample No. 10 has a pronounced negative absorption by using the entire sample, while the No. 200 and No. 325 fractions exhibited positive absorption (the sample has 93 percent passing No. 325). It would seem that a more plausible explanation for this difference of 8 gm of  $\text{Ca}(\text{OH})_2$  absorption per gm of flyash  $\times 10^3$  is due to the small percentage of the coarse fraction.

#### REFERENCES

15. Bernal, J. D., "The Structure of Cement Hydration Compounds," Proc. Third Int. Sym. on the Chemistry of Cement, London, pp. 216-261, (1952).
16. Bogue, R. H., "The Chemistry of Portland Cement," Reinhold Pub. Corp., New York, (1955).
17. Kalousek, G. L., "Discussion on the Reactions and Thermochemistry of Cement Hydration at Ordinary Temperature, by H. H. Steinour," Proc., Third Int. Sym. on the Chemistry of Cement, London, pp. 296-311, (1952).
18. Pike, R. G., and Hubbard, D., "Miscellaneous Observations on the Alkali-Aggregate Reaction and the Ionic Charge on Hydrated Cement," Highway Research Board, Bull. 171, pp. 16-33, (1958).
19. Taylor, H. F. W., "Hydrated Calcium Silicates, Part I: Compound Formation at Ordinary Temperatures," Jour. Chem. Soc., London, pp. 3682-3690, (1950).

R. J. LEONARD and D. T. DAVIDSON, Closure — Although Mr. Moh's discussion does not make known any new information about the lime-flyash pozzolanic reaction, it is appreciated for it offers the authors an opportunity to clarify and discuss further some of the major points of their study.

Although x-ray spectrometer curves were obtained for the absorption test flyashes for a range of 20 of 2 to 100 deg, only that portion showing the interesting peaks at 12.6 Å and 17.3 Å were shown in Figure 14. The other significant peaks that were observed at 3.08 Å and 2.81 Å were reported in the text of the paper. The data for CSH(I) reported in Table 7 of the paper are more recent than that presented by Mr. Moh and it may be significant to give Dr. Lea's addendum to these data, "This is the data for the poorly crystalline material. The long spacing can vary considerably and may also be undetected." It should be noted that the peaks appear to be transitional; this is particularly evident with sample 11. Sample pretreatment and the small amount of reaction product available in any particular sample probably play major roles in the location of the first peak. On the basis of the evidence of the 12.6 Å to 17.3 Å peak and the presence of the second peaks it seems reasonable to identify one of the reaction products as CSH(I).

The size of the DTA exothermic peak at about 830 to 900 deg C reported by Kalousek (22) for CSH(I) is a function of the mass of this reaction product. Since only a small amount of reaction product was available for each flyash sample this affected the DTA results. Peaks were observed at 830 to 900 deg C, but they were not very significant.



Also, the large amounts of unburned carbonaceous material in some of the flyashes caused some drift despite the fact that a nitrogen atmosphere was used in the furnace. In addition to Kalousek's (22) work, Greenberg (21) and others have done DTA work on CSH(I); however, most researchers prefer the use of x-ray diffraction procedures for the examination of crystalline minerals of this type. None of these reports include information on the reaction of lime with flyash and the resulting products; also most of the reports dealt with pure mixtures of  $\text{Ca}(\text{OH})_2$  with silica gel or minerals that had been exposed to solutions of high concentrations of  $\text{Ca}(\text{OH})_2$  for long periods of time.

At the 38th Annual Meeting of the Highway Research Board, Benton (20) of the Bureau of Reclamation, presented some information on the reaction products of pozzolans. Mr. Benton indicated that there were two reaction products of the lime-flyash reaction, one product is CSH(I) and the other is a calcium-alumino-hydrate. The latter product may or may not contribute to the strength of the lime-flyash mixture.

The lime-absorption reported in Figure 2 should not be misconstrued to be the maximum amount of  $\text{Ca}(\text{OH})_2$  that these flyashes can possibly absorb in that length of time and at that temperature. The curves shown in Figures 2 and 3 represent the amounts absorbed from an original  $\text{Ca}(\text{OH})_2$  solution of slightly less than saturated concentration. At 20 deg C a saturated  $\text{Ca}(\text{OH})_2$  solution is 0.165 gm per 100 gm of water (23). The concentrations of the solutions used were kept below the saturated concentration because of the presence of soluble constituents in the flyashes. Any precipitation of calcium in the solution would make the absorption test meaningless.

Since the pozzolanic reaction is between calcium and the reactive constituents that occur on the surface of the flyash particles, total exposure of the flyash surface area is necessary for the absorption test results to be significant. Although No. 10 flyash has 43.4 percent  $\text{SiO}_2$  not all of this  $\text{SiO}_2$  will be available for immediate reaction. The amount reacted for a particular pozzolan will be dependent on the surface area of the flyash, the rate of diffusion of calcium through the reaction product that forms on the surface of the particles and the availability of calcium in the system. Lea (24), among others, has estimated that under normal conditions a pozzolan will not react in a year with more than about 20 percent of its weight of lime.

Due to concentration restrictions only a small amount of the flyash was reacted in the absorption tests, nevertheless, there was enough calcium present to cause cementation of the particles. The samples were stirred to prevent this cementation from forming aggregates. Some of the flyashes have enough calcium to provide significant compressive strength without the addition of lime.

Mr. Moh has confused the term mechanism; it is not clear whether he means chemical or mechanical mechanism. The authors sought to explain the chemical mechanism of the pozzolanic reaction. By the use of absorption tests and x-ray spectrometer methods the chemical reactions were investigated. If there were no reactions that produced the cementitious material there would be no strength development, hence the importance of the chemical mechanism should be obvious.

There are various physical factors in addition to the chemical factors that affect the development of compressive strength. The strength of lime-flyash mortars is dependent on the amount and number of contacts of the cementitious material; therefore, anything that affects either the amount or number of contacts will affect the strength. Physical effects such as the depletion of lime at certain points in the mortar, or an excess of lime may even prevent the cementitious material from making contact; these factors may arise from inadequate mixing of the mortars. Non-reactive material may prevent contact of the cementitious material. Density is another physical factor that may affect the strength (26); increasing the density of the mortar increases its compressive strength. However, despite the physical effects it seems reasonable there must be a relationship between the chemical reactions and the strength development of the lime-flyash mortars.

The rate of compressive strength development of lime-flyash mortars at ordinary temperatures generally is low initially, then the strength development is rapid and finally the rate decreases significantly, but the strength continues to increase; this was shown in Figure 4 of the paper. The rate of absorption of flyashes in  $\text{Ca}(\text{OH})_2$  solutions proceeds in the same manner. The rapid increase in strength is due to the cementitious

material produced by the lime-flyash reaction, and the decrease in compressive strength is due to the local depletion of lime and limitation due to the diffusion of calcium through the reaction products. By increasing the concentration of the lime the rate of strength development will be increased after the initial period, but the rate of strength development will not be exactly the same as the rate of absorption. This is due to the physical factors that are not involved in the absorption tests. It was noted in the paper that flyashes having a high rate of calcium absorption also have a high rate of compressive strength development. Therefore, the correlation between the rate of absorption and the rate of strength development must be made on the basis of standardized tests if it is to have interpretation meaning.

The negative absorption can be satisfactorily explained by the presence of soluble components in the flyashes. The presence of calcium compounds in the flyashes will be noted in the chemical analysis of the flyashes given in Table 2 of the paper. Some of the flyashes possess enough soluble calcium to be weakly cementitious without the addition of lime. The solubility of these flyash compounds is of little importance to the pozzolanic chemical mechanism. The inconsequential initial absorption values may be due to charge deficiencies that are associated with glasses, or other phenomena; they do not enter into any of the calculations. Lea (24) chooses to ignore the initial reactions in his studies.

From the physical data given in Table 4 and the discussion given in the paper it should be obvious that the minor differences are due to the coarse fraction; the major role of the coarse fraction, containing the majority of the unburned coal and coke, is not in the chemical reactivity of the pozzolans, but in the physical factor of preventing the contact of cementitious material. The presence of unburned coal and coke also generally indicates a coarser flyash and hence a lower specific surface.

#### REFERENCES

20. Benton, E. J., "Cement Pozzolan Reactions," Highway Research Board, Bull. 239, (1960).
21. Greenberg, Sidney A., "The Chemisorption of Calcium Hydroxide by Silica," Jour. Phys. Chem., 60:325-330, (1956).
22. Kalousek, George L., "The Reactions of Cement Hydration at Elevated Temperatures," Proc. of the Third Sym. on the Chemistry of Cement, London, 1952:334-356, (1954).
23. Lange, Norbert A., "Handbook of Chemistry," 9th Edition, Handbook Publishers, Sandusky, Ohio, (1956).
24. Lea, F. M., "The Chemistry of Pozzolana," Proc. of the First Sym. on the Chemistry of Cement, Stockholm, 1938:460-490, (1939).
25. Lea, F. M., and Desch, C. H., "The Chemistry of Cement and Concrete," St. Martins Press, New York, (1956).
26. Veskoehil, Ralph H., "Effect of Density on Unconfined Compressive Strength, Absorption and Volume Change of Lime and Flyash Stabilized Soils," Unpublished M. S. Thesis, Iowa State College Library, Ames, Iowa, (1956).

# Impact of Stabilization of Loess with Quicklime on Highway Construction

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The possibilities of improving the properties of highly water-sensitive loess through the addition of quicklime (CaO) are discussed. Results of laboratory tests are given to show the influence on the mechanical and physical properties of the soil through the addition of lime. These include the modification of grain size distribution (coagulation), modification of plasticity, change in compaction characteristics, and influencing of water absorption capacity and dependent strength properties. Addition of 3 percent by weight of CaO improves the soil so that a new and better construction material results.

Improvements in construction progress result by using lime stabilization. Whereas construction work as a rule would have to be stopped during periods of rain, building activities can be continued during such periods when lime is used. The cost of lime stabilization is only a fraction of the losses which result from such delay.

Use of lime stabilization has resulted in an increase of quality, building progress, and efficiency in the case of the road construction work done on the section of the Northeim-Hannover, Autobahn.

● IT HAS BEEN NOTED that major earth construction difficulties are experienced in loess areas when this very water sensitive soil changes to a liquid state through precipitation or through a water bearing stratum. The haul roads become impassable and also expert placing and compacting of the overly wet loess into embankments is no longer possible.

A radical solution consists in removing such soils and replacing them by some other suitable material. But this solution is as uneconomical as it is technically unsatisfactory even if such materials are available in sufficient quantity close to the construction site. If such a substitution cannot be made, another solution must be found in any case.

For the construction of the section Northeim-Seesen of the North-South Autobahn from Hamburg to Goettingen, the local loess has improved and stabilized with lime. The following is a report on the experience gained on this job.

## GEOLOGICAL SURVEY OF THE JOB SITE

The Autobahn section Northeim-Seesen runs through the foreland of the Harz which is a hilly country. The hills belong to the Triassic and Jurassic formations. The lower plains, where the Autobahn runs entirely, are covered by immense loess deposits with underlying marly clays.

## PROPERTIES OF THE RAW SOIL

Based on test results, the properties of a soil sample, typical for the above-mentioned job site, are described in the following:

Table 1 shows the chemical and mineralogical composition of the tested soil.

Figure 1 shows the grain size distribution. According to this the tested loess is almost single-size silt. It should be noted that the raw soil contains 15 percent of clay minerals (Table 1) although size fraction "clay" amounts to 6 percent only.

TABLE 1  
CHEMICAL AND MINERAL COMPOSITION  
OF THE RAW SOIL

Constituent	Percent by Weight	Constituent	Percent by Weight
Chemical		Mineral	
Loss on ignition	6.32	Quartz	54
SiO <sub>2</sub>	75.45	Feldspars	18
Fe <sub>2</sub> O <sub>3</sub>	10.83	Calcite	8
CaO	5.18	Dolomite	5
MgO	1.36	Kaolinite and Illite	15
Not determined	0.86	Total	100

0.09 mm and the following chemical composition (in percent by weight related to the oven-dried material):

Loss on ignition	3.78	
CO <sub>2</sub>		2.01
Non-soluble in HCL	0.94	
Total CaO	92.39	
Available CaO		89.83
MgO	0.55	
SiO <sub>2</sub>	1.29	
Fe <sub>2</sub> O <sub>3</sub>	0.52	
Al <sub>2</sub> O <sub>3</sub>	0.25	
Not determined	0.28	

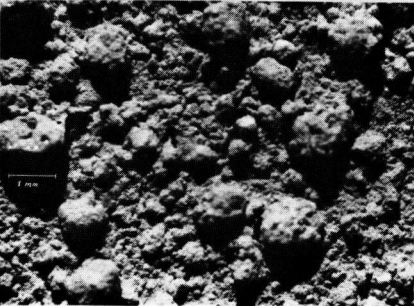
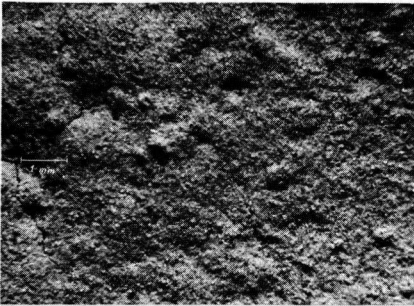


Figure 2. Flocculation of loess by lime. Above: Raw soil. Below: Soil treated with 3 percent lime.

The average specific gravity was determined to be 2.585 g/cm<sup>3</sup>.

The plasticity test (according to Atterberg) gave the following results:

Liquid Limit	LL = 22.8 percent
Plastic Limit	PL = 18.0 percent
Plasticity Index	PI = 4.8

#### PROPERTIES OF LIME USED

The lime used was a calcitic quicklime according to German specification DIN 1060 with 97.75 percent passing sieve

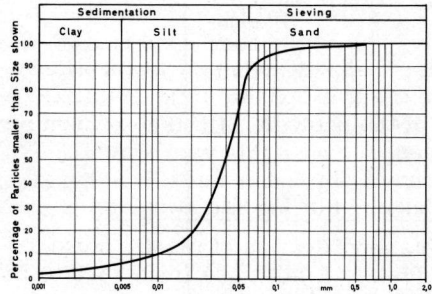


Figure 1. Grain-size distribution.

#### MODIFICATION OF LOESS PROPERTIES BY ADDITION OF LIME

In order to study the possibilities of soil property modification, the raw soil was mixed with 1, 3 and 5 percent respectively of the calcitic quicklime described in Properties of Lime Used. Then the properties of the soil-lime mixtures were examined.

#### Modification of Grain Size Distribution (Flocculation)

Immediately after mixing lime with the humid raw soil, a considerable flocculation was observed (Fig. 2). As shown in Figure 3, this flocculation proved to be

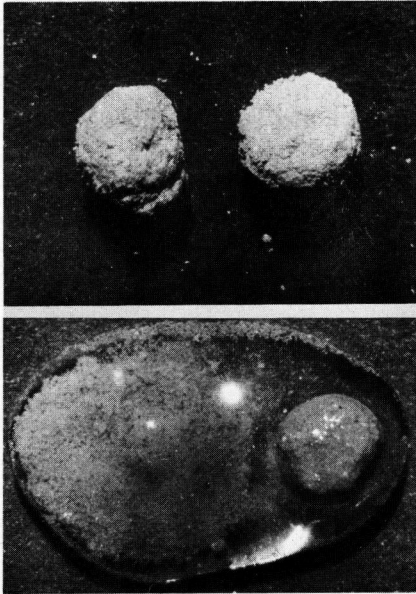


Figure 3. Water resistance of flocculation (Left: raw soil; Right: lime treated soil) Above: Particles before influence of water. Below: Particles 5 min after influence of water.

Figure 5 shows the ready absorption of water by the raw soil (left half of the photograph) while the hydrophobic lime-stabilized soil repels the water drop like quicksilver (right half of the photograph). The reason for this behavior could not yet be satisfactorily explained.

Modification of Plastic Behavior

The modification of plastic behavior through addition of lime is shown in Figure 6. The Plastic Limit rose from 18.0 to 22.0, the Liquid Limit rose from 22.8 to 24.0 which reduced the Plasticity Index from 4.8 to 2.0. The soil-lime mixtures were tested 24 hours after addition of lime.

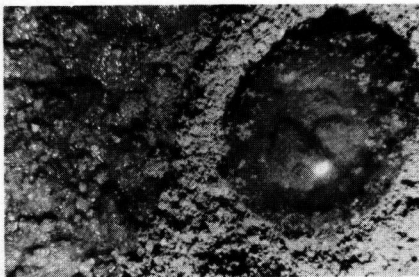


Figure 5. Water repellency of soil by lime treatment. Left: Raw soil; Right: Lime treated soil.

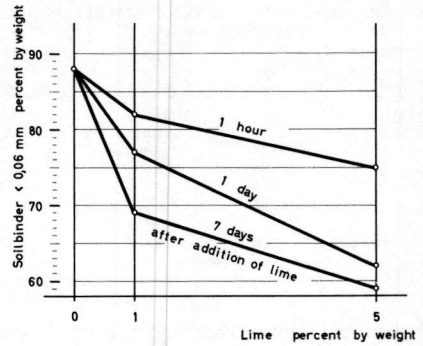


Figure 4. Flocculation of loess by addition of quicklime.

water-resistant. Figure 4 shows the relation between intensity of flocculation and the amount of lime added as well as curing time. The uncompacted soil-lime mixture was cured in airtight containers until sieving; then sieving with a 0.06 mm sieve took place under water. As shown in Figure 4, the soil-binder content was reduced considerably by the addition of lime.

In the course of this examination it turned out that the soil had acquired hydrophobic properties through the addition of lime.

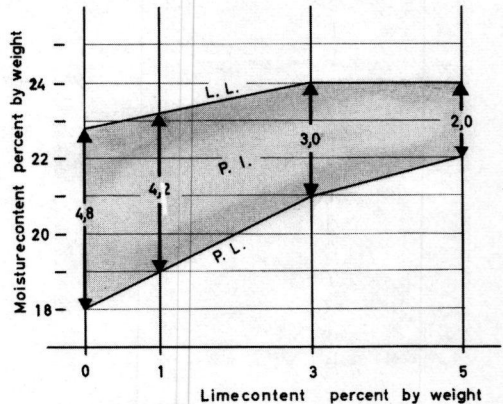


Figure 6. Change of plastic properties by addition of lime.

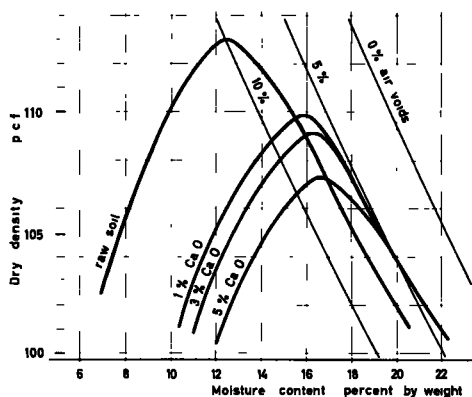


Figure 7. Moisture-density relationship (proctor test).

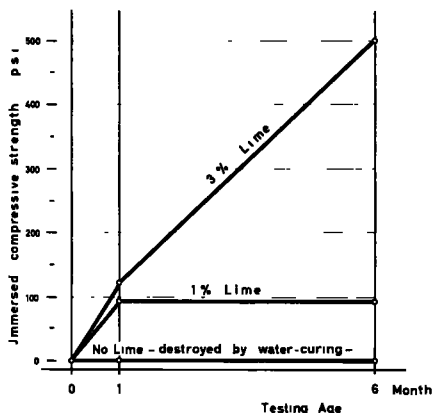


Figure 8. Immersed unconfined compressive strength related to age and amount of lime.

### Modification of Dry Density-Moisture Content Relationship

The moisture-density curves of the Proctor test are plotted in Figure 7 for the raw soil and the tested soil-lime mixtures. The mixtures were tested 24 hours after addition of lime.

Maximum dry densities were somewhat reduced by lime whereas the optimum moisture contents were noticeably increased. For high water contents, soil-lime mixtures show a greater compactibility than the raw soil.

### Modification of Immersed Strength

Cylinders, 3 by 3 in., were used as specimens. The raw soil and the soil-lime mixtures were compacted at optimum water content to 100 percent Proctor density. The specimens were cured at first in moisture-saturated air and immersed in water prior to the unconfined compression test. Loading speed during the unconfined compression test was 14 psi per sec.

In Figure 8, immersed strength is plotted against curing time. The raw soil specimens were decomposed without exception during immersion in water. The specimens treated with 1 percent lime were water resistant and showed a compressive strength of 98 psi after 28 days; upon further curing, no additional increase of compressive strength was noticed. This indicates that the small amount of 1 percent of lime had already been used up for flocculation and an initial hydraulic strengthening. For a further hydraulic strengthening no more lime was available (1). The specimens treated with 3 percent lime showed a steady further strengthening up to 500 psi after six months and it is safe to assume that this process would have continued further.

The above test results clearly show that loess may be improved considerably by adding small amounts of lime. In particular, the following properties can be improved: (a) soil structure; (b) plasticity; (c) compactibility at high water content; (d) water resistance; and (e) strength and bearing capacity.

In the following the effects of a soil treatment with lime upon the performance at the above mentioned earth-work job will be described.

### EFFECTS OF LIME STABILIZATION ON CONSTRUCTION PERFORMANCE

Exact data on the climatic conditions, earthwork performance and use of lime at the job site are plotted in Figure 9.

During the months of August and September 1957 there had been very heavy rains, amounting to up to 224 percent of the mean value taken over a number of years and reached the unusual level of 245 mm in September. Therefore, the earthworks had to



be stopped because of impassability of the haul roads. The soil was soaked to a point where, even after several sunny days, no construction traffic was possible. The water content of the soil had increased from a normal amount of 14 to 16 percent to 20 to 22 percent, in some cases even 26 percent, and therefore exceeded the Liquid Limit (Fig. 6).

At the beginning of October, lime stabilization was used in order to allow a resumption of work. Areas of application were as follows: (a) construction of haul roads; (b) improvement of soaked soil for embanking; and (c) stabilization of the subgrade for fine grading.

**Stabilization of Loess with Quicklime**  
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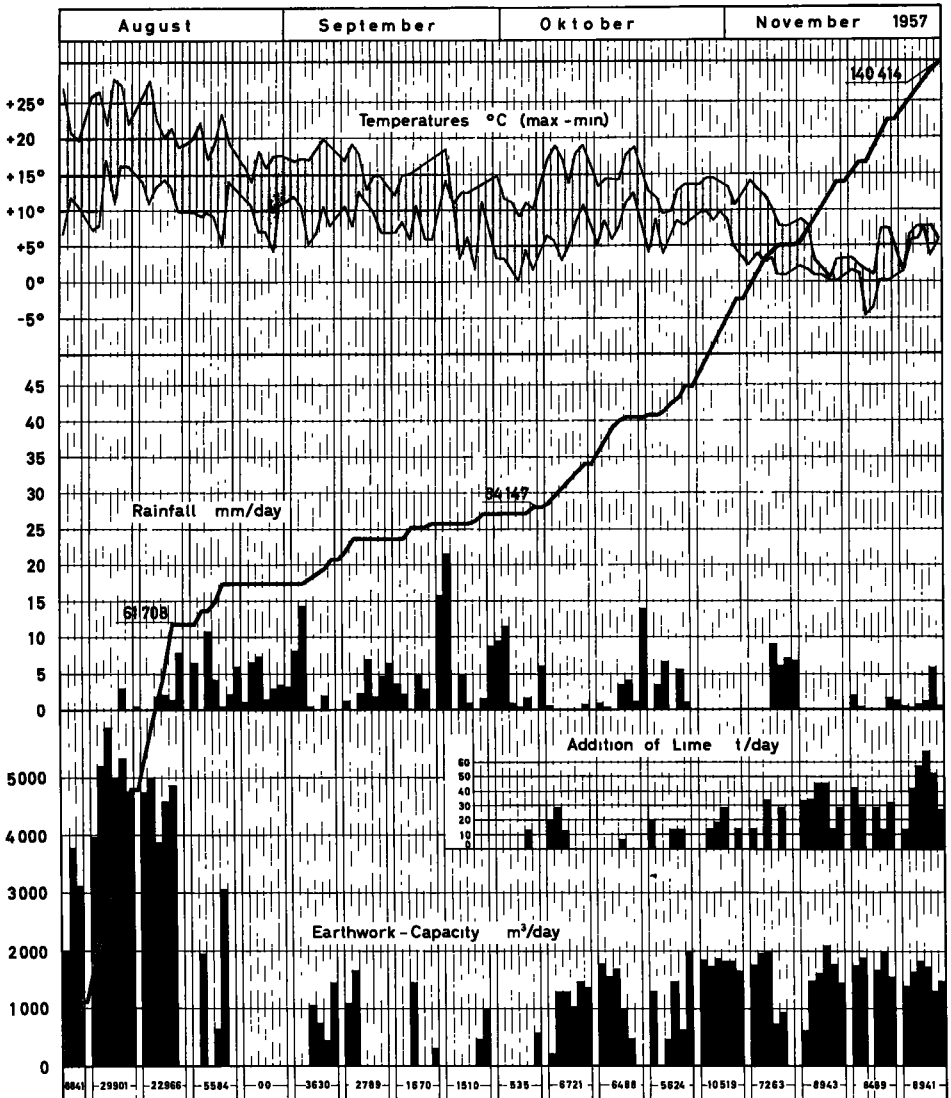


Figure 9. Effects of lime stabilization on construction performance.



Figure 10. Situation of the job site after rainfalls (untreated soil).

The water content of the soaked soil had been reduced by about 5 percent by adding 3 percent of calcitic quicklime. Desiccation was effected by slaking of the lime and the heat generated by this as well as the improved aeration through flocculation. While the raw soil at 21

percent moisture content was nearly in liquid state ( $LL=22.8$  percent) and incompactible (Optimum Moisture Content = 12.5 percent), the soil-lime mixture with 3 percent of lime at 16 percent moisture content was in solid state ( $PL=21$  percent) and in optimum condition for compaction (Optimum Moisture Content = 16 percent).

One hour after mixing soil with lime, the haul roads were passable for heavy trucks, the lime-treated soil used for embankments was in optimum condition for compaction, and the required quality of fine grading was assured. As shown in Figure 9, it was made possible by lime stabilization to resume continuous construction work. Independent of rainfalls and greatly lowered temperatures, a soil could be employed as construction material which ordinarily would not have been used under the prevailing weather conditions. The decline in daily average performance as compared to the August performance was caused by the suppression of double shift work because of the earlier fall of darkness and a reduction in equipment.

A comparison of the additional costs arising from lime stabilization with the plant hire rates for temporary shut-down of the construction site proved that there are important economic advantages for the contractor as well as for the sponsor in employing lime stabilization.



Figure 11. Lime-stabilized haul road after rainfalls.

#### REFERENCES

1. Brand, Walter, "Die Bodenstabilisierung mit Kalk," (Soil Stabilization with Lime). *Strasse und Autobahn*, (Nov. 1958).
2. Unger, \_\_\_\_\_, "Verbesserung von Schluff durch Kalk," (Improvement of Silt by Lime). *Strasse und Autobahn*, (Nov. 1958).

# Experimental Lime Stabilization in Nebraska

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The Nebraska Department of Roads in 1956 performed an experiment involving the use of hydrated lime in the stabilization of plastic soils, and in the upgrading of inferior base course materials. The experiment included a preliminary laboratory study and a field construction project.

This paper reports and summarizes the tests performed on the various materials before and after adding hydrated lime, and presents the results of deflection measurements at half-year intervals since the construction of the field project.

● WIDESPREAD experimentation in the use of hydrated lime for stabilization of soils and base materials has apparently established that this material has a place in the construction of highways. Each year, new research on this subject is reported. The principal uses of the hydrated lime appear to be in two categories as follows: (a) to improve highly plastic subgrade soils and (b) to improve the performance of inferior base course materials.

The State of Nebraska has large areas within its borders where the subgrade and embankment soils used in the construction of highways are highly plastic, and poor in load supporting ability, when wet. The two principal areas of undesirable soils are: (a) glacial till area of eastern Nebraska and (b) the clay and shale areas in the northern part of the state.

In general, base course materials available in Nebraska are considered to be of mediocre quality. Materials which must be used in the construction of highways include principally the following:

1. The rounded coarse sands and gravels of the Platte and other rivers, and those of glacial origin.
2. Wind-blown, fine sands.
3. Mortar beds (low quality limey sandstone) from tertiary deposits.
4. Gravels composed of soft, limey sandstone.
5. Limited quantities of limestone.

Of these base materials, only the limestones of the eastern part of the state are considered to provide base courses of superior quality.

If the permanent stabilization of the heavy plastic soils of Nebraska is possible by the addition of small percentages of lime, or if it is found that the inferior base course materials can be improved by this method, a great benefit would result for the highway program. For this reason, it was proposed that an experiment be conducted using hydrated lime in the improvement of subgrades and base courses.

The experiment was a combined effort of the engineering staffs of the Bureau of Public Roads and the Nebraska Department of Roads. The authors appreciate the opportunity to summarize and report the results of this cooperative effort.

## PRELIMINARY INVESTIGATION

### Preliminary Field Investigation

The experimental project (Project No. F-43(4)) is located in Johnson County, about 60 mi south and east of Lincoln, in the southeastern part of Nebraska. This project, a part of State Highway No. 3, begins about one mi north of Vesta and extends in an easterly direction for a distance of about seven mi, ending at the concrete pavement on the west edge of Tecumseh. About two mi were included in the experiment.

TABLE 1

**LABORATORY TEST RESULTS FOR PRELIMINARY SUBGRADE  
SOIL SAMPLES, LIME-TREATED SUBGRADE SOIL SECTION**

Location <sup>1</sup>	Plasticity Tests			Hydrometer Analysis			% Ret. No. 200 Sieve	AASHO Soil Class.
	LL	PL	PI	Sand	Silt <sup>2</sup>	Clay		
1047+00	45	19	26	34	29	37	29	A-7-6(15)
1056+00	50	23	27	20	38	42	13	A-7-6(17)
1059+00	51	21	30	26	33	41	18	A-7-6(18)
1077+00	48	19	29	32	36	32	16	A-7-6(17)
1086+00	46	20	26	30	34	36	20	A-7-6(16)
1092+00	51	21	30	17	46	37	5	A-7-6(18)
1101+00	49	21	28	18	43	39	10	A-7-6(17)
1107+00	45	21	24	25	39	36	11	A-7-6(15)

<sup>1</sup>Depth of samples approximately 7 to 31 in. below finish grade elevation.

<sup>2</sup>0.005 mm to 0.05 mm.

The area traversed by the project is in a diversified farming region, with corn, sorghums, and small grains as the principal crops. The temperatures in the area range from 103 F in the summer to -10 F in the winter, with an annual average of 51 F. The annual precipitation averages about 31 in. of which about 7 in. is in the form of snow. The frost normally penetrates from 12 to 18 in., however, during prolonged cold periods the frost may penetrate to as much as 24 in.

Upon completion of the subgrade survey and laboratory tests of the subgrade soils, study indicated that the soils and situation between Station 1047 and Station 1107 and between Station 1148 and Station 1200 were typical, and sufficiently uniform to serve as the experimental sections.

The 6,000 ft section between Station 1047 and Station 1107 was selected for the lime treated subgrade soil section. The terrain traversed by this section is a hilly upland plain, with good surface drainage due to the hills and poor subsurface drainage due to the impervious soils. The soils encountered in the subgrade, through this section were predominantly glacial clays. Table 1 shows the laboratory test results of the preliminary samples in this section.

TABLE 2

**LABORATORY TEST RESULTS FOR PRELIMINARY SUBGRADE SOIL  
SAMPLES; LIME-TREATED BASE COURSE SECTION**

Location <sup>1</sup>	Plasticity Tests			Hydrometer Analysis			% Ret. No. 200 Sieve	AASHO Soil Class.
	LL	PL	PI	Sand	Silt <sup>2</sup>	Clay		
1149+00	58	25	33	4	45	51	2	A-7-6(20)
1155+00	58	24	34	2	47	51	1	A-7-6(20)
1164+00	70	26	44	4	41	55	3	A-7-6(20)
1173+00	50	25	25	7	47	46	2	A-7-6(16)
1191+00	61	23	38	7	45	48	2	A-7-6(20)

<sup>1</sup>Depth of samples approximately 14 to 38 in. below finish grade elevation.

<sup>2</sup>0.005 mm to 0.05 mm.

**TABLE 3**  
**LABORATORY TEST RESULTS FOR PRELIMINARY SAMPLES**  
**COARSE SAND AND AFTONIAN SILT**

Coarse Sand Gradation Ranges												
Sieve Number	1 in.	3/4 in.	3/8 in.	4	10	20	30	40	50	80	100	200
Percent Ret.	0-4	0-9	0-20	0-30	1-42	4-65	11-74	19-81	44-90	69-94	71-95	74-96
Percent Calcareous (est.)..... Nil												
Percent Shale (est.)..... Nil												

Aftonian Silt							
Plasticity Tests			Hydrometer Analysis			Grad. % Ret.	
LL	PL	PI	Sand	Silt <sup>1</sup>	Clay	No. 100	No. 200
28-31	22-25	4-7	33-46	38-51	14-18	0	1-4

<sup>1</sup>0.005 mm to 0.05 mm.

The 5, 200 ft section between Station 1148 and Station 1200 was selected for the lime treated base course section. Surface drainage between Station 1148 and Station 1192 is slow due to the level terrain, while drainage between Station 1192 and Station 1200 is fair due to the slope toward the Big Nemaha River. Subsurface drainage throughout the section is poor due to the impervious soils. The soils in this section are subsoils of glacial origin and parent glacial clays, Table 2 shows their laboratory test results.

At the time of the soil survey the area in the vicinity of the project was thoroughly prospected for local aggregates. The available aggregates include fine sand, glacial coarse sand, soil binder and aftonian silt. Of these, the coarse sand and aftonian silt were selected for use in the lime treated base course mixtures. Table 3 shows the test results for the preliminary samples of these two materials. It will be noted that the coarse sand is not a favorable material for use in base course construction. Aftonian silt is an interglacial wind-blown layer which was deposited between the Nebraskan and Kansan glacial stages. As encountered at this location it was a white flour-like material.

**TABLE 4**  
**ROUTINE METHODS OF TEST PERFORMED ON**  
**UNTREATED AND LIME-TREATED SOILS**

Test	Method
Preparation of sample	AASHO T87-49
Sieve analysis	AASHO T11-49
Hydrometer analysis	AASHO T88-54
Liquid limit	AASHO T89-54
Plastic limit	AASHO T90-54
Plasticity index	AASHO T91-54
Shrinkage limit, shrinkage ratio, lineal shrinkage, and volumetric change	AASHO T92-54
Field moisture equivalent	AASHO T93-54
Maximum density and optimum moisture	AASHO T99-49
	(Except that new material was used for each point on the curve)
Capillarity, absorption failure, and cementation	Nebraska procedure (see Appendix A)

### Preliminary Laboratory Testing

Upon completion of the preliminary field investigation a comprehensive laboratory study was undertaken. This investigation was divided into the following parts:

1. Treatment of a fine-grained sub-grade soil with hydrated lime.
2. Treatment of sand-aftonian silt mixtures with hydrated lime.
3. Comparison of four pozzolans (aftonian silt, volcanic ash, flyash and peorian loess) when mixed with sand and hydrated lime.

In the following paragraphs each of the three laboratory studies is discussed.

TABLE 5

## LABORATORY TEST RESULTS FOR PRELIMINARY SOIL-LIME MIXTURES

Tests	One Hour Cure					2 Day Cure					14 Day Cure					240 Day Cure						
	0% Lime <sup>2</sup>	1% Lime <sup>2</sup>	3% Lime <sup>2</sup>	6% Lime <sup>2</sup>	10% Lime <sup>2</sup>	0% Lime <sup>2</sup>	1% Lime <sup>2</sup>	3% Lime <sup>2</sup>	6% Lime <sup>2</sup>	10% Lime <sup>2</sup>	0% Lime <sup>2</sup>	1% Lime <sup>2</sup>	3% Lime <sup>2</sup>	6% Lime <sup>2</sup>	10% Lime <sup>2</sup>	0% Lime <sup>2</sup>	1% Lime <sup>2</sup>	3% Lime <sup>2</sup>	6% Lime <sup>2</sup>	10% Lime <sup>2</sup>		
Sieve Analysis, % Ret.																						
No. 10	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	1	1	1	1	1	1	1	1	2	5	1	1	1	3	6	1	1	2	13	17		
40	4	4	4	4	4	4	3	5	9	16	4	4	4	10	16	4	4	8	28	31		
100	7	6	6	6	7	8	9	8	15	22	7	7	7	15	22	7	7	15	35	39		
50	14	13	13	14	17	18	14	13	18	27	14	13	16	28	35	13	14	30	49	53		
200	19	18	18	20	24	24	20	18	27	36	18	18	23	37	43	19	20	42	60	62		
Hydrometer Analysis																						
Sand	31	27	29	48	67	74	29	31	57	78	77	27	30	56	76	77	26	30	70	83	90	
Silt <sup>3</sup>	33	38	42	42	30	24	35	45	34	21	23	37	47	34	27	23	37	47	25	16	10	
Clay	36	35	29	10	3	2	36	24	9	1	0	36	23	10	2	0	37	23	5	1	0	
Colloids	31	32	15	5	1	0	33	11	4	0	0	33	10	4	1	0	34	10	2	0	0	
Specific Gravity	2.63	2.70	2.70	2.70	2.66	2.66	2.67	2.69	2.70	2.69	2.69	2.69	2.69	2.67	2.67	2.67	2.67	2.67	2.66	2.67	2.66	
Liquid Limit	47	51	44	35	32	32	45	36	35	NP	NP	48	37	36	NP	NP	49	38	35	NP	NP	
Plastic Limit	21	21	23	27	31	32	21	23	29	34	37	20	23	28	NP	NP	22	23	30	NP	NP	
Plasticity Index	26	30	21	8	1	NP	24	15	6	NP	NP	28	14	8	NP	NP	27	15	5	NP	NP	
Capillarity	5'4"	4'26"	12'4"	23'52"	8'37"	6'39"	6'43"	14'03"	18'48"	5'02"	5'09"	5'29"	15'02"	18'55"	6'01"	3'32"	5'28"	12'11"	15'54"	6'16"	2'49"	
Absorption Failure	5'4"	4'26"	12'34"	2 hr+	2 hr+	2 hr+	6'43"	14'56"	2 hr+	2 hr+	2 hr+	5'29"	31'02"	2 hr+	2 hr+	2 hr+	5'28"	55'28"	2 hr+	2 hr+	2 hr+	
Cementation	200+	200+	200+	200+	180	145	200+	200+	200+	101	119	200+	200+	200+	147	108	200+	200+	200+	200+	169	
Shrinkage Limit	12.5	11.8	16.3	26.5	27.5	29.7	11.1	18.9	25.8	27.3	30.7	11.3	15.1	25.7	27.6	28.9	9.2	12.9	22.8	26.2	27.9	
Shrinkage Ratio	1.87	1.91	1.74	1.48	1.46	1.45	1.93	1.68	1.51	1.45	1.39	1.92	1.81	1.50	1.45	1.43	1.99	1.85	1.58	1.49	1.38	
Lineal Shrinkage	5.1	6.3	4.9	1.9	3.9	3.1	6.2	2.1	1.1	3.3	3.3	6.8	5.7	1.9	2.6	4.1	9.8	9.5	5.1	5.4	5.6	
Volumetric Change	17.1	21.7	16.2	6.07	12.83	9.9	21.3	6.6	3.4	10.6	10.7	23.5	19.2	5.8	8.2	13.4	35.8	34.2	16.7	17.9	18.8	
Field Moist. Equivalent	21.6	23.1	25.6	30.6	36.6	36.5	22.1	22.8	28.0	34.6	38.4	23.6	25.7	29.5	33.2	38.3	27.0	31.4	33.4	38.2	41.5	
Optimum Moisture, %	19.5	—	—	—	—	—	19.0	19.0	21.0	21.0	22.0	—	—	—	—	—	—	—	—	—	—	
Maximum Density, gm/cc	1.65	—	—	—	—	—	1.67	1.65	1.62	1.56	1.54	—	—	—	—	—	—	—	—	—	—	
Stability, Total Load, lb	—	278	635	942	1185	1483	263	712	1497	2013	2498	292	812	2130	3360	4020	198	362	1458	3133	5027	
Percent Moist, as Tested	—	18.1	18.0	18.0	18.1	17.9	18.8	18.5	18.6	18.8	18.4	18.6	18.6	17.6	17.3	17.8	18.8	18.6	18.3	18.5	18.6	
Unconfined Compressive Strength, 4- by 4-in. Cylinders, P. S. I.	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	45	62	253	586	758	
Percent Moist, as Tested	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	19.2	69.1	19.4	19.2	18.3	
Unconfined Compressive Strength, 2- by 2-in. Cylinders, P. S. I.	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	50	74	346	772	1377	
Percent Moist, as Tested	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	19.0	19.0	18.8	19.3	19.0	
AASHO Soil Class	A-7-6 (16)	A-7-6 (18)	A-7-6 (13)	A-4(8)	A-4(8)	A-4(8)	A-7-6 (15)	A-6(10)	A-4(8)	A-4(6)	A-4(4)	A-7-6 (17)	A-6 (10)	A-4 (8)	A-4(6)	A-4 (4)	A-7-6 (17)	A-6 (10)	A-4 (5)	A-4(1)	A-4 (1)	

<sup>1</sup> Test results of soil as received in laboratory.

<sup>2</sup> Lime added, percent by weight.

<sup>3</sup> 0.005 mm to 0.05 mm.



**Experimental Treatment of a Fine-Grained Subgrade Soil with Hydrated Lime.** — The first of the three studies presents data showing changes in test results observed when hydrated lime was added to plastic soil in the following percentages by weight: 1, 3, 6 and 10. All tests were made to duplicate, as nearly as possible, the actual field conditions of the soils as to moisture, density and curing.

The soil used in this phase of the study was sampled from the left backslope at Station 1085. It was a plastic reddish-brown glacial clay AASHO soil classification A-7-6(16) representative of the subgrade soil on this project. Table 4 indicates the procedures used in the soil tests.

Upon receipt of the soil sample at the laboratory it was dried in an oven for approximately 48 hours at 140 F, then pulverized to pass the No. 4 sieve. A representative sample was taken from this material and routine soil tests including moisture-density determination, were performed on the soil sample as prepared in the laboratory. These test results are shown in Table 5.

Upon completion of the routine soil tests, soil-lime mixtures were prepared by dry mixing 1, 3, 6 and 10 percent hydrated lime (by weight) with four portions of the soil previously prepared to pass the No. 4 sieve. Cylinders (4- by 4-in.) were molded from each soil-lime mixture, as well as the untreated soil, to the approximate optimum moisture content and the maximum density previously determined on the untreated soil. After molding, each cylinder was immediately wrapped in aluminum foil to prevent loss of moisture and placed in closed containers in the moist room at a temperature of 70 F + 2 F, for curing. The cylinders were then removed to perform the routine soil tests, as outlined in Table 4, (excluding the maximum density-optimum moisture tests), at intervals of one hour, 2 days, 14 days and 240 days. The results of these tests are shown in Table 5 and Figures 1 through 8.

It will be noted from Table 5 that the addition of hydrated lime to plastic soil has a marked effect on the soil character. Small amounts change the nature of the plastic soils somewhat, while the addition of 6 or 10 percent shows a radical change in their physical characteristics.

Figures 1 through 4 indicate that the lime treatment apparently causes the agglomeration of some of the silt and clay sized particles with the net result that the soil is somewhat coarsened. The biggest textural changes caused by the lime appear to have taken place in the particle size range covered by the hydrometer test. This physical change reflects not only the percentage of lime added but also the length of curing time. This phenomenon is such that a clay was so changed by the addition of lime that after 14 days it was reclassified as loam to sandy loam, and after 240 days' curing it was classified as clay loam to sand. However, it should be pointed out that the particles

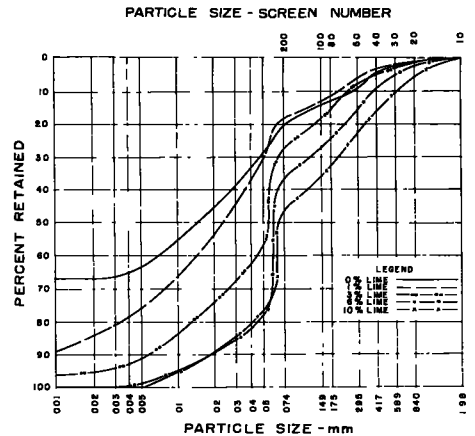
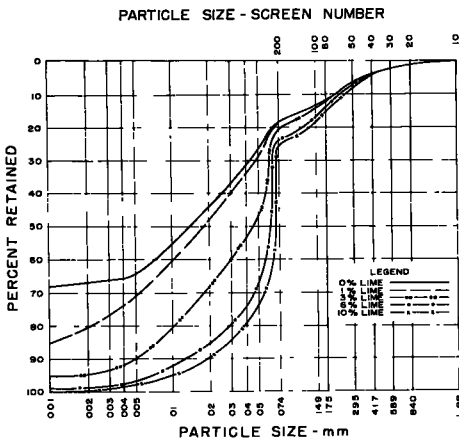


Figure 1. Effect of lime-treatment on grain size of soil—1 hour cure.

Figure 2. Effect of lime-treatment on grain size of soil—2 day cure.

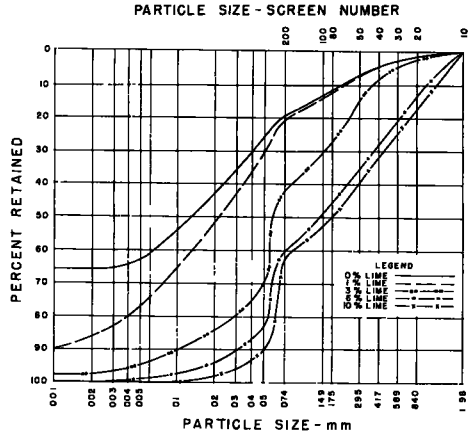
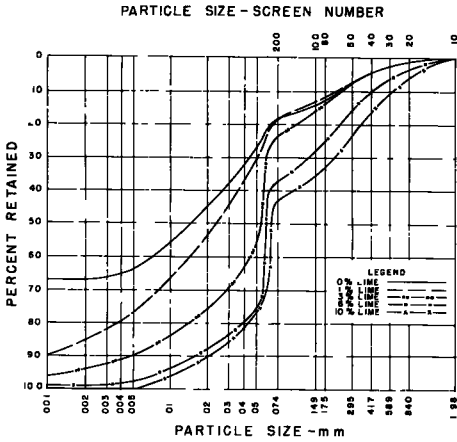


Figure 3. Effect of lime-treatment on grain size of soil—14 day cure.

Figure 4. Effect of lime-treatment on grain size of soil—240 day cure.

appeared to be weakly bonded. For this reason, it was necessary to establish a standard 5-min agitation period in the mechanical mixer as part of the sieve analysis procedure.

One of the most publicized features of lime is its ability to reduce the plasticity

index of soils. The laboratory tests on this experiment show that the addition of as little as 6 percent lime reduces the plasticity index from 30 to 1 in only one hr and to 0 in 2 days (Fig. 5). The addition of 10 percent lime reduced the plasticity index of the plastic soil to 0 in one hr. This reduction in plasticity index is the result of lowering the liquid limit and raising the plastic limit.

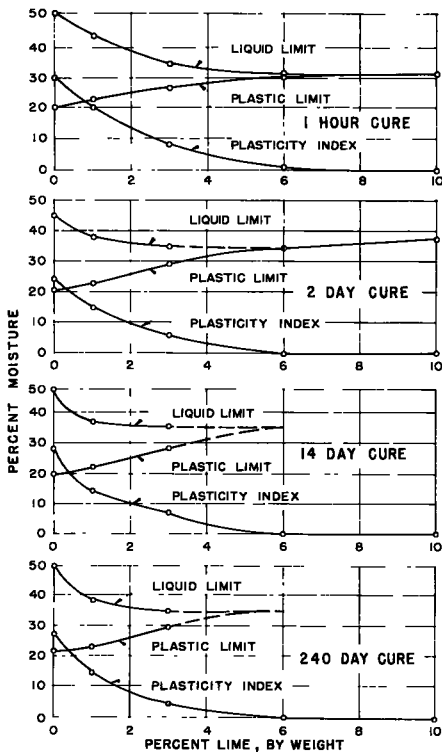


Figure 5. Plasticity tests of lime-treated soil.

when added to a plastic soil on the field moisture equivalent, capillarity time, shrinkage ratio, and volumetric change. In Figure 6 it will be noted that the capillarity time of 1 and 3 percent lime-treated specimens show an increase over the untreated soils, while the 6 and 10 percent specimens show very little change over the untreated soil. The field moisture equivalent tests show an increase as greater percentages of lime are added, but shrinkage ratio values are lowered by the addition of lime. It is interesting to note that 3 percent lime reduces the volumetric change of the soil more than 6 or 10 percent lime.

It will be noted on Figures 6 and 7 that the patterns of the changes in soil test results due to the lime are similar, regardless of this period of cure. However, the curves for the 240 days' cure do show some

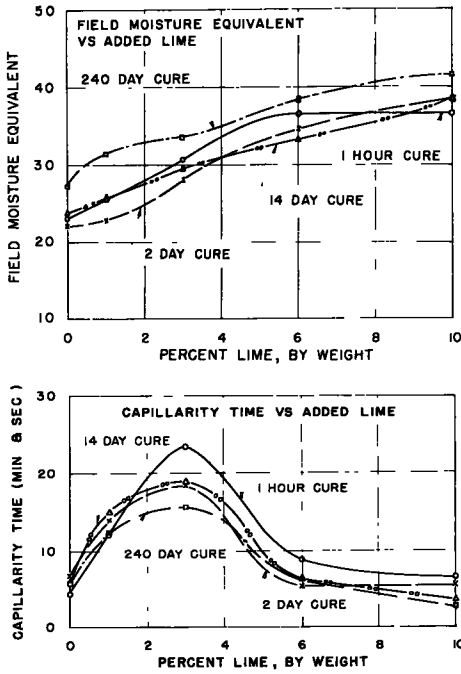


Figure 6. Effect of lime on the field moisture equivalent and capillarity on plastic soil.

noticeable in the volumetric change curve and this is the accumulative effect of the higher field moisture equivalent and shrinkage ratio values and the lower shrinkage

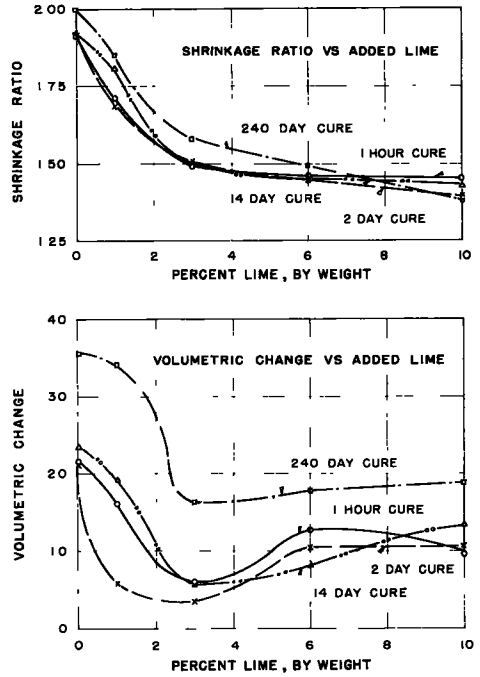


Figure 7. Effect of lime on the shrinkage properties of plastic soil.

limit values. A possible explanation for the deviations of the 240-day tests is that they were performed by a different laboratory technician, than the tests for the other curing periods.

In connection with the capillarity test, attention is invited to the absorption failure test (Table 5). It will be noted that the absorption failure time has increased from a matter of minutes to over two hours, with the addition of 3 or more percent lime. This indicates that the addition of lime to a plastic soil increases its resistance to slaking.

Maximum density and optimum moisture values were determined on the soil-lime mixtures, as well as on the untreated soil, which had cured in the moist room for a period of 48 hr. The results of these tests are shown in Table 5. It will be noted that as greater percentages of hydrated lime are added the maximum density decreases and the optimum moisture increases.

The final test performed on the soil-

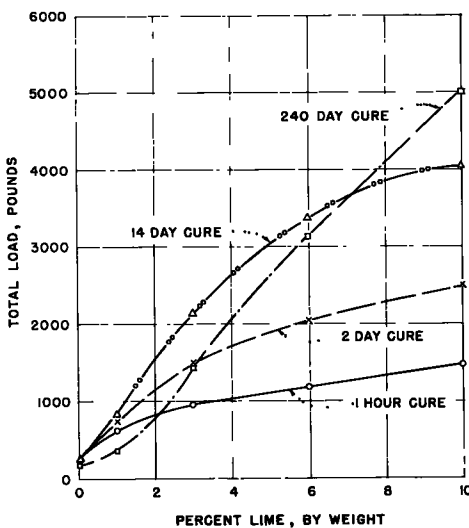


Figure 8. Extrusion (strength) tests of soil-lime mixtures.

TABLE 6  
TEST RESULTS FOR MATERIALS USED IN PRELIMINARY  
LABORATORY MIXTURES

Tests	Aftonian Silt	Sand	Hydrated Lime
<b>Sieve Analysis: (AASHTO T11-49)</b> (Total percent retained)			
3/4 In.	—	0	—
3/8 In.	—	1	—
No. 4	—	2	—
10	—	5	—
20	—	14	—
40	—	61	—
50	—	81	—
100	0	90	—
200	8	92	0
<b>Hydrometer Analysis: (AASHTO T88-54)</b>			
Sand, percent	55	—	—
Silt, percent	36	—	—
Clay, percent	9	—	—
Colloids, percent	6	—	—
Specific Gravity	2.66	—	—
Liquid Limit (AASHTO T89-54)	24	—	—
Plastic Limit (AASHTO T90-54)	23	—	—
Plasticity Index (AASHTO T91-54)	1	—	—
Cementation (Nebr. Method)	15	—	—
<b>Chemical Composition: (ASTM C25-47)</b>			
Calcium oxide, CaO, percent	—	—	97.8
Magnesium oxide, MgO, percent	—	—	0.1
Silica, SiO <sub>2</sub> , percent	—	—	0.9
Iron oxide, Fe <sub>2</sub> O <sub>3</sub>	—	—	0.1
Aluminum oxide, Al <sub>2</sub> O <sub>3</sub>	—	—	0.2
Loss on Ignition, percent	—	—	30.8
Carbon Dioxide, CO <sub>2</sub> , percent	—	—	5.7

<sup>1</sup>0.005 mm to 0.05 mm

lime mixtures was the strength or stability test. This test was performed as nearly as possible in accordance with ASTM Designation D915-49T. The following exceptions in equipment were made:

**Forming Mold**—A forming mold with an inside diameter of 2.00 in. and 5.00 in. in height replaced the specified mold.

**Compaction Tamper**—A compaction tamper weighing 1.930 gm, 12 in. in length, with a tamping face diameter of 1.10 in. replaced the specified compaction tamper.

Soil-lime mixtures were prepared by mixing 1, 3, 6 and 10 percent by weight of hydrated lime, with portions of soil pulverized to pass the No. 10 sieve. Cylinders were then molded from each mixture and from untreated soil using the optimum moisture and maximum density values determined in the routine compaction tests of the soil alone. Initial compaction was accomplished by placing the material in the mold in two equal layers and giving each layer 12 blows with the compacting tamper. The mold was then placed in the testing machine and molded to a height of 2.00 + 0.02 in. at a rate of 0.1 in. per min. The final load was held for 2 min.

Each cylinder was then wrapped in aluminum foil and placed in closed containers in the moist room at a temperature of 70 F + 2 F for curing. The cylinders were removed from the moist room at intervals of 1 hr, 2 days, 14 days and 240 days for strength or extrusion testing. Upon removal from the moist room each cylinder was weighed and measured, then loaded at a rate of 1 in. per min. The extrusion or strength values were taken at the maximum load required to cause failure of the test specimen. A moisture sample was taken from each cylinder at completion of the test to determine actual moisture content. The strength values are shown in Table 5 and Figure 8. It will be noted from Figure 8 that the addition of lime greatly increases the strength of the soil when tested in the extrusion test.

TABLE 7  
TEST RESULTS FOR SAND-AFTONIAN SILT-LIME MIXTURES 16 DAY CURING TIME

Composition of Mixture Percent by Weight			Density as Molded gm/cc	Moisture Percent		Unconfined Compressive Strength, Pounds	Computed Gradation, Percent Retained							
Hydrated Lime	Aftonian Silt	Sand		As Molded	As Tested		Sieve Number							
						3/4 In.	4	10	20	40	50	100	200	
0	20	80	2.00	10.5	10.3	160	1	2	4	11	48	64	71	75
2	20	78	1.97	10.4	9.9	500	1	2	4	11	48	63	70	74
4	20	76	1.96	10.6	10.2	703	1	2	4	11	46	62	68	72
7	20	73	1.94	10.5	10.6	1110	1	2	4	10	45	59	66	69
0	15	85	1.99	10.5	10.2	150	1	2	4	12	52	68	76	79
2	15	83	1.98	10.5	9.9	550	1	2	4	12	51	67	75	77
4	15	81	1.97	10.5	10.2	620	1	2	4	11	49	66	73	76
7	15	78	1.94	10.7	10.1	855	1	2	4	11	48	63	70	73
0	10	90	1.84	7.1	7.1	80	1	2	4	13	55	73	81	84
2	10	88	1.97	10.8	9.7	430	1	2	4	12	54	71	79	82
4	10	86	1.98	10.5	10.0	525	1	2	4	12	52	70	77	80
7	10	83	1.95	10.4	10.0	698	1	2	4	12	51	67	75	77

**Experimental Treatment of Sand-Aftonian Silt Mixtures with Hydrated Lime.** — The second of the preliminary laboratory investigations was concerned with the use of hydrated lime to improve a base course material of inferior quality. This type of material (coarse sand) has been used extensively in Nebraska in the construction of sub-bases. The object was to determine if a satisfactory base course material can be developed by the addition of hydrated lime and a pozzolan to the coarse sands. Since in this case an aftonian silt was found on the project, it was decided to use it as the pozzolan. Another phase of the laboratory work was concerned with the comparison of this material with other available materials for use as a pozzolan.

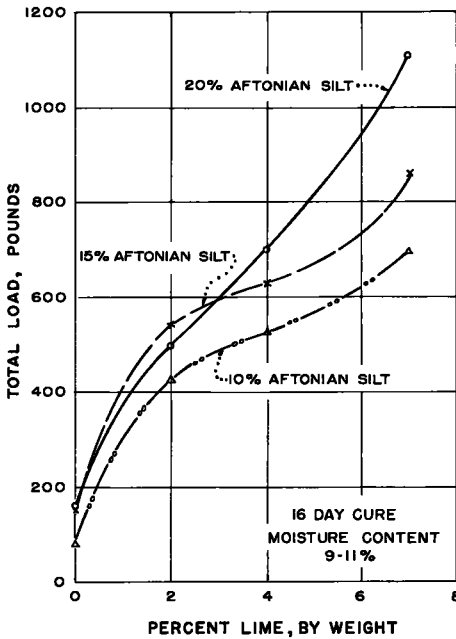


Figure 9. Unconfined compression tests of sand-aftonian silt-lime mixtures.

and maximum density values determined in the tests mentioned in the preceding paragraph. The percentages of each material in these cylinders are shown in Table 7.

After molding, each cylinder was wrapped in aluminum foil to prevent loss of moisture and placed in closed containers in the moist room at a temperature of  $70\text{ F} + 2\text{ F}$ , for curing. The cylinders were cured in this manner for a period of 16 days at which time their unconfined compressive strengths were determined. The results of these tests are shown in Table 7 and Figure 9. The moisture content of the specimens at the time of the test were very nearly the same as those at the time of molding. From Figure 9, it will be noted that 20 percent aftonian silt was probably more than the optimum, if only 2 percent hydrated lime is added, but when 4 and 7 percent of hydrated lime was used, the highest percent of aftonian silt resulted in the highest strengths.

**Unconfined Compressive Strength of Sand-Lime Mixtures Combined with Aftonian Silt, Volcanic Ash, Flyash or Peorian Loess.** — This part of the laboratory investigation covers tests which were conducted to compare the unconfined compressive strengths which develop when aftonian silt, volcanic ash, flyash, and peorian loess are mixed separately with standard ottawa sand and various percentages of hydrated lime.

The sand used in all of the unconfined compressive strength tests was a graded standard ottawa sand. The aftonian silt, volcanic ash, and peorian loess were obtained from various locations in the state. The flyash and hydrated lime were obtained from commercial sources. The analyses of these materials are shown in Table 8.

With the exception of the hydrated lime the materials used in this part of the preliminary investigation were obtained from sources located in the vicinity of the project. The test results for the samples actually used in the laboratory experiments are shown in Table 6.

The first step in this investigation was to prepare three maximum density-optimum moisture curves using the coarse sand and 10, 15 and 20 percent aftonian silt with 4 percent hydrated lime added to each mixture. These tests were conducted in accordance with AASHTO Designation T 99-49. The 10 percent mixture had an optimum moisture of 10.3 percent and maximum density of 1.98 gm per cc. The 15 percent mixture had an optimum moisture of 10.2 percent and maximum density of 1.98 gm per cc and the 20 percent mixture had an optimum moisture of 9.8 percent and maximum density of 1.96 gm per cc.

The second phase of this part of the laboratory experiment was to mold 4- by 4-in. cylinders, in accordance with AASHTO Designation T 99-49, using coarse sand and aftonian silt mixtures with and without hydrated lime. These cylinders were molded to the approximate optimum moisture

The first phase of the laboratory study was to prepare eight dry mixtures having the compositions shown in Table 9.

Optimum moisture-maximum density determinations were then made on each of the dry mixes. These tests were conducted in accordance with the procedure outlined in AASHO Designation: T 99-49. The results of these tests are shown in Table 9.

Using the approximate maximum density and optimum moisture values determined in the first phase of this part of the laboratory investigation, 2- by 2-in. cylinders were molded with each mixture. These cylinders were molded in accordance with the procedure described in ASTM Designation: D 915-49T. This procedure and exceptions to the standard procedure were previously described in the section of this report concerning the experimental treatment of a fine-grained subgrade soil with hydrated lime. After molding, the cylinders were wrapped in aluminum foil, and placed in closed containers in the moist room at a temperature of  $70\text{ F} \pm 2\text{ F}$ , for curing.

The cylinders were removed from the moist room for unconfined compressive strength tests at intervals of 2, 7, 14 and 90 days. Upon removal from the moist room each cylinder was weighed and measured and immediately tested in unconfined compression at a loading rate of 0.1 in. per min. The results of the unconfined compressive strength tests are shown in Table 9 and Figure 10. At the 90 day testing period the compressive strength developed in both flyash mixtures is considerably greater than that developed in the other mixtures. From Figure 10 it will also be noted that in all cases the strength developed in the mixtures with 6 percent of hydrated lime is greater than the strength developed in mixtures having 4 percent hydrated lime.

TABLE 8  
TEST RESULTS FOR MATERIALS USED IN PRELIMINARY LABORATORY MIXTURES

Tests	Hydrated Lime	Aftonian Silt	Flyash	Volcanic Ash	Peorian Loess	Ottawa Sand
Siege Analysis: (AASHO T11-49)						
(Total Percent Retained)						
No. 20	—	—	—	1	—	0
30	—	—	—	1	—	2
40	—	—	—	2	—	35
50	—	—	0	5	0	69
100	—	0	1	19	1	98
200	0	8	5	40	1	100
Hydrometer Analysis: (AASHO T88-54)						
Sand	—	—	13	54	11	—
Silt <sup>1</sup>	—	36	71	42	67	—
Clay	—	9	16	4	22	—
Specific Gravity	—	2.66	2.45	2.44	2.70	—
Liquid Limit: (AASHO T89-54)	—	24	27	NP	35	—
Plastic Limit: (AASHO T90-54)	—	23	26	NP	24	—
Plasticity Index: (AASHO T91-54)	—	1	1	NP	11	—
Cementation (Nebr. Procedure)	—	15	2	8	200+	—
Chemical Analysis						
Silicon Dioxide, Percent	0.9	—	44.3	—	—	—
Aluminum Oxide, Percent	0.2	—	24.1	—	—	—
Ferric Oxide, Percent	0.1	—	17.2	—	—	—
Calcium Oxide, Percent	97.8	—	4.3	—	—	—
Magnesium Oxide, Percent	0.1	—	0.5	—	—	—
Sulfur Trioxide, Percent	—	—	1.4	—	—	—
Loss on Ignition, Percent	30.8	—	3.5	—	—	—
Carbon Dioxide, Percent	5.7	—	—	—	—	—
Insoluble Residue, Percent	—	—	74.2	—	—	—
Ratio of $\text{Al}_2\text{O}_3$ to $\text{Fe}_2\text{O}_3$	—	—	1.4	—	—	—
Tricalcium Aluminate, Percent	—	—	35	—	—	—
Sodium Oxide, Percent	—	—	0.67	—	—	—
Potassium Oxide, Percent	—	—	2.18	—	—	—
Total Alkali, Percent	—	—	2.85	—	—	—
Equiv. Alkali, Percent	—	—	2.10	—	—	—
Water Soluble Alkali, Percent	—	—	0.12	—	—	—
Phosphorus Pentoxide, Percent	—	—	0.05	—	—	—
Manganic Oxide, Percent	—	—	0.08	—	—	—
Chloroform Soluble Organic Substances, Percent	—	—	0.0005	—	—	—
Free Lime, Percent	—	—	0.6	—	—	—
Free Carbon, Percent	—	—	2.8	—	—	—

<sup>1</sup>0.005 mm to 0.05 mm

TABLE 9  
TESTS RESULTS FOR SAND-LIME MIXTURES COMBINED WITH AFTONIAN SILT, VOLCANIC ASH, FLYASH OR PEORIAN LOESS

Mix No.	Composition of Mixes				Maximum Density of Mixture, gm/cc	Optimum Moisture of Mixture, Percent	2 Day Curing Time			7 Day Curing Time			14 Day Curing Time			90 Day Curing Time		
	Sand Percent	Hydrated Lime, Percent	Other Ingredient Percent	Type of Material			Den. as Molded, gm/cc	Moist. as Tested, Percent	Unconfined Compressive Strength, Pounds	Den. as Molded, gm/cc	Moist. as Tested, Percent	Unconfined Compressive Strength, Pounds	Den. as Molded, gm/cc	Moist. as Tested, Percent	Unconfined Compressive Strength, Pounds	Den. as Molded, gm/cc	Moist. as Tested, Percent	Unconfined Compressive Strength, Pounds
1	80	4	16	Aftonian Silt	1.93	9.0	1.91	8.0	45	1.92	8.4	63	1.92	8.7	80	1.93	8.2	467
2	70	6	24		2.00	8.0	1.99	8.0	177	1.99	7.9	252	1.99	8.2	362	2.00	7.7	1442
3	80	4	16	Flyash	1.96	9.5	1.95	8.6	87	1.96	8.2	172	1.96	8.6	445	1.97	8.3	4353
4	70	6	24		1.95	9.5	1.95	8.3	268	1.96	8.3	445	1.95	8.6	1245	1.96	8.7	6113
5	80	4	16	Volcanic Ash	1.84	11.0	1.80	10.7	25	1.81	10.5	90	1.83	10.4	198	1.83	10.3	787
6	70	6	24		1.82	11.6	1.81	11.6	112	1.81	11.3	353	1.82	11.3	575	1.82	11.1	1760
7	80	4	16	Peorian Loess	2.00	10.0	1.95	8.9	55	1.96	8.8	85	1.97	8.7	122	1.98	8.4	785
8	70	6	24		1.98	10.3	1.95	10.3	153	1.97	9.8	242	1.97	9.7	310	1.98	9.3	1373

## DESIGN AND PRECONSTRUCTION SAMPLING OF EXPERIMENTAL PROJECT

### Design of the Experimental Lime-Treated Sections

The data obtained in the preliminary field and laboratory studies seemed to justify the construction of an experimental project, including a lime-treated subgrade soil section and a lime-treated base course section.

The standard design thicknesses of the bases and surfacing for the proposed project

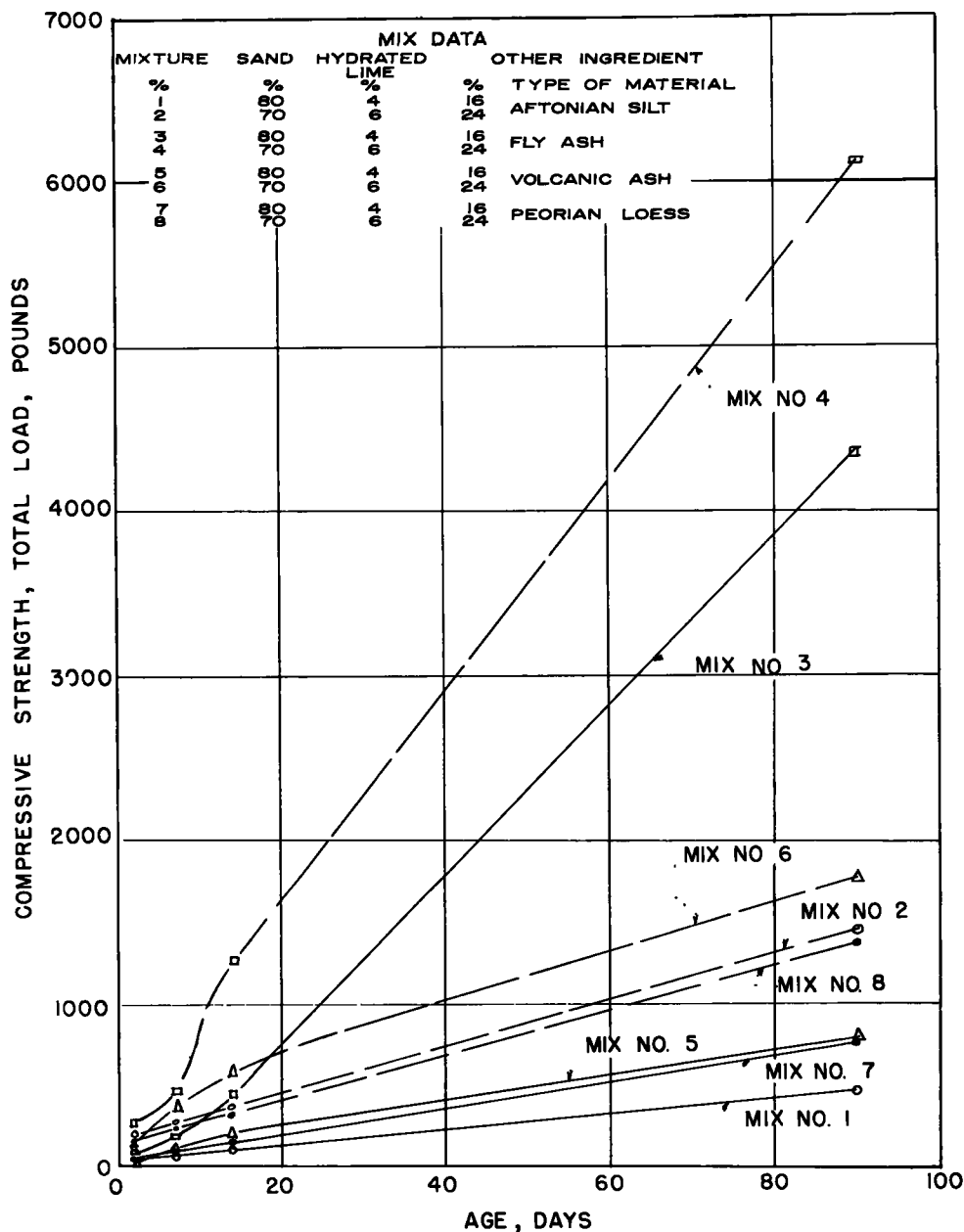


Figure 10. Unconfined compressive strength of sand-lime mixtures combined with aftonian silt, volcanic ash, flyash, or peorian loess.



TABLE 10  
TRAFFIC INFORMATION

1956 Daily Average – Estimated Total Vehicles	485
1956 Daily Average – Estimated Total Commercial Vehicles, Including Trucks, Semi-Trailers, Truck-Trailer Combinations and Busses	105
1956 Daily Average – Estimated Total Semi-Trailers and Truck-Trailer Combinations (included in the above figure)	12
30th Highest Hour of the Year (1956) Estimated	70
Estimated Number of Axles per Day exceeding 5 Tons	22
Estimated Number of Axles per Day exceeding 7 Tons	11
Estimated Number of Axles per Day exceeding 8 Tons	5
Estimated Number of Axles per Day exceeding 9 Tons	1

were based on the Nebraska flexible pavement thickness curves, which take into consideration the soils, traffic, situation and precipitation. These design data were: (a) group index of 20 (AASHO Classification); (b) estimated total of 485 vehicles per day (Table 10); (c) good to fair surface drainage and poor subsurface drainage; and (d) approximately 31 in. of precipitation per year. From this information the standard design thickness for the base was 11 in. and for the surface course 3 in. It was further decided that these thicknesses would be held constant throughout the experimental sections (Fig. 11).

Table 11 shows the location and design of each section in the experimental portion of the project.

Subdivisions No. 1 and 5 were established as control sections in order that the performance of the sections in which lime was used could be compared with the standard design for the project. Each section also includes a subdivision, No. 3 and 7, where the lime addition is about equal to the average amount recommended in the literature. Included, also, in each of the two experimental sections are subdivisions having lime percentages significantly higher and lower than the average for the material.

#### Pre-Construction Sampling

Prior to construction permanent test sites were selected and appropriately marked in each subdivision of the experimental sections. The locations of these permanent test sites are shown in Tables 23 and 24.

At the time the permanent test sites were selected, samples of the subgrade soil were taken at each site. In the lime-treated subgrade soil section the subgrade soil was sampled at two depths: (a) 7 to 14 in. below finish grade elevation; and (b) 14 to 26 in. below finish grade elevation.

The 7- to 14-in. depth represents the subgrade soil that would be lime-treated and all soil tests as outlined in Table 4, including unconfined compressive strength tests were performed. The moisture-density relationship was determined on the untreated soil as well as on the lime-treated soil (lime-soil-water mixtures were allowed to cure for 24 hr in the moist room prior to compaction). The results of these tests are shown in Tables 17, 18 and 19. The subgrade soil samples from the 14- to 26-in. depth represent the subgrade immediately below the lime treatment. Table 23 shows the results of the tests of these subgrade samples.

In the lime-treated base course section the subgrade soil was sampled between the depths of 14- and 26-in. below finish grade elevation. This represents the subgrade immediately below the granular subbase course. The results of tests of these samples are shown in Table 24. The moisture-density relationships of the coarse sand-aftonian

silt-lime mixture were also determined (the mixture was allowed to cure for 24 hr in the moist room prior to compaction). The curve data are shown in Table 20.

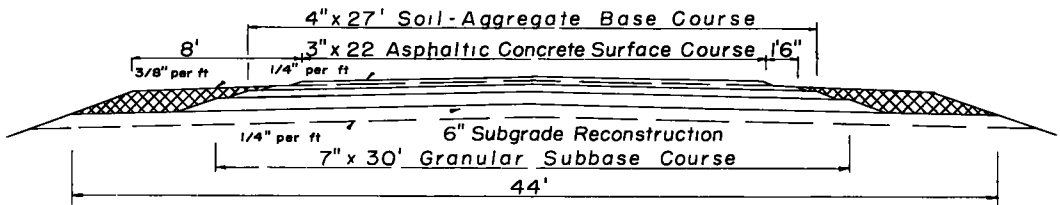
### CONSTRUCTION PROCEDURES AND COST DATA

#### Lime-Treated Subgrade Soil

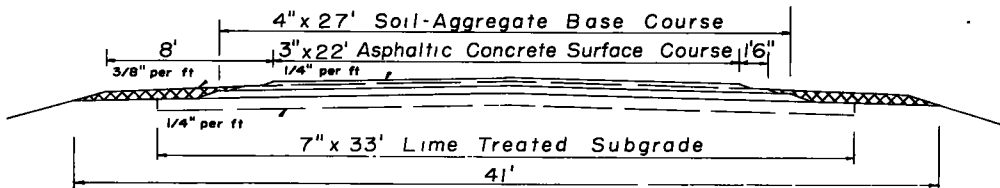
On August 8, 1956 construction began on the lime-treated subgrade soil section. Each of the three lime-treated sections were constructed separately. In the following paragraphs the construction procedures are discussed.

Scarifying and Pulverizing the Subgrade Soil.—In preparing the subgrade for lime stabilization the width of the roadbed was adjusted to a uniform 41 ft. A scarifier attached to a motor grader then loosened the subgrade soil the entire width of the roadbed

### STANDARD DESIGN



### EXPERIMENTAL LIME TREATED SUBGRADE SOIL SECTION



### EXPERIMENTAL LIME TREATED BASE COURSE SECTION

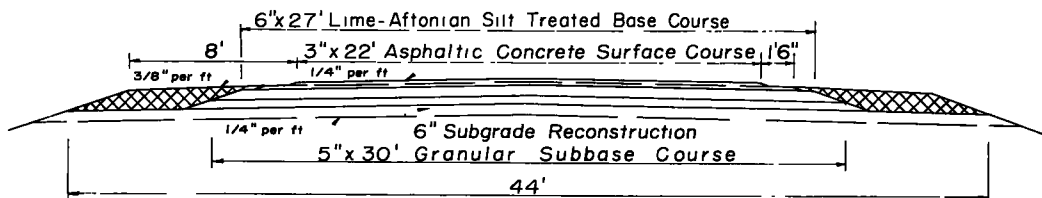


Figure 11. Typical cross-sections.

TABLE 11  
DESIGN OF EXPERIMENTAL SECTIONS

Lime-Treated Subgrade Soil Section				
Subdivision Number	1	2	3	4
Station to Station	1047 to 1062	1062 to 1077	1077 to 1092	1092 to 1107
Surface Course	3 in. Asph Conc	3 in. Asph Conc	3 in. Asph Conc	3 in. Asph Conc
Base Course	4 in. Soil Agg	4 in. Soil Agg	4 in. Soil Agg	4 in. Soil Agg
Subbase Course	7 in. Granular	None	None	None
Subgrade Treatment	None	7 in.	7 in.	7 in.
Percent Lime	None	3%	6%	10%
Remarks	Standard Design	—	—	—

Lime-Treated Base Course Section				
Subdivision Number	5	6	7	8
Station to Station	1148 to 1161	1161 to 1174	1174 to 1187	1187 to 1200
Surface Course	3 in. Asph Conc	3 in. Asph Conc	3 in. Asph Conc	3 in. Asph Conc
Soil Agg. Base Co.	4 in.	None	None	None
Lime-Aftonian Silt Base Course	None	6 in.	6 in.	6 in.
Percent Aftonian Silt	None	15%	20%	20%
Percent Lime	None	2%	4%	7%
Granular Subbase Course	7 in.	5 in.	5 in.	5 in.
Remarks	Standard Design	—	—	—

to a depth of approximately 6 in. The remainder of the soil was loosened during mixing operations. After scarification the loosened soil was pulverized with a rotary tiller to the extent that no chunks remained larger than 3 in. in greatest dimension, and so that not more than 40 percent of the soil remained in chunks which would be retained on the 1-in. sieve. After pulverization the soil was placed in a windrow.

**Spreading Lime.**—Prior to placing the lime the windrow was flattened to a 30-ft width. The 50-lb lime bags were then spotted by hand, on the flattened windrow in transverse rows, in conformity with the individual lime requirements for the particular subdivisions. After 300 to 400 ft of bags had been placed, the bags were split open and the contents dumped in piles. Table 12 shows the distribution of the soil and lime.

Since this project was of an experimental nature and considerable control was desired when placing the lime, the method employed was suitable. However, if constructing a project of considerable length this method might be too inefficient and faster methods of handling the lime would be desirable.

**Mixing.**—The first step in the mixing procedure consisted of pulling a farm harrow through the piles of lime to level and spread it evenly across the flattened windrow. A motor grader with scarifier attachment made one pass through the flattened windrow, and a small amount of water was added. This procedure was followed to reduce wind loss. After several passes through the dry material with a rotary tiller, motor graders were used to bring up the sides of the flattened windrow. Some water was added to the mixture at this time. After the mixture had been windrowed the motor graders cut out small windrows for further mixing with the tiller. Enough water was added during this operation so that the moisture content was between the optimum value and two percentage points above. This procedure was followed until the windrow had been completely turned. Check of the windrow showed that the lime and soil were well mixed and that the moisture content was within the special provision limits.

**Compaction.**—After the soil-lime mixture had cured in the windrow for approximately 12 hr, motor graders were used to spread the mixture uniformly across the roadbed and the sheepfoot rollers were started. Water was added during this operation to compensate for evaporation loss. The special provisions required that the mixtures be compacted to not less than 95 percent of maximum density. No difficulties were encountered in attaining the required density except in Subdivision No. 2 (3 percent lime). In this subdivision too much water was added and the resulting moisture contents were between 23.1 and 25.8 percent. At this high moisture content the density did not rise above 90 percent of maximum. Subsequently the compacted mixture was scarified to a depth of about 4 in., dried, and recompactd. In-place density tests taken after recompacting showed that the material was compacted to the requirements of the special provisions. From experience it is believed that the moisture content of the mixture during compaction should be no greater than 2 percent above optimum. Table 13 shows the range of final densities obtained in the compacted material in this section.

**TABLE 12**  
**DISTRIBUTION OF SOIL AND LIME IN**  
**THE LIME-TREATED SUBGRADE SOIL SECTION**

Subdivision Number	Hydrated Lime		Subgrade Soil	Combined Windrow
	Percent By Weight	Ton/Sta	Approx. No. of Tons/Sta	Approx. No. of Tons/Sta
2	3	2.8	90.2	93
3	6	5.3	83.7	89
4	10	8.8	79.2	88

**Curing.**—Upon completion of laydown operation in the lime-treated subgrade subdivisions no loaded vehicles, other than sprinkling equipment, were allowed on the subgrade for a 5-day curing period. During this period the surface was sprinkled with water at frequent intervals to offset the effects of evaporation. Due to the extremely hot dry weather, sprinkling was continued until the soil aggregate base course was placed.

#### Lime-Treated Base Course

On August 30, 1956, construction began on the lime-treated base course. Each of the three lime-treated sections were constructed separately. In the following paragraphs the construction procedures are discussed.

**Placing the Sand-Lime-Aftonian Silt.**—In the method employed the sand was placed and windrowed. The aftonian silt was windrowed alongside the sand and the two materials combined and thoroughly mixed. After the sand and aftonian silt materials had been mixed and windrowed, the windrow was flattened to about a 30-ft top. Fifty lb bags of hydrated lime were then placed, opened and dumped in the same manner as described in the lime-treated subgrade soil section. Table 14 shows the percentages and tonnages of each of the materials used in each subdivision.

**Mixing.**—After dumping the lime bags a farm harrow was pulled through the piles of lime to level and spread it evenly across the flattened windrow. The surface was lightly sprinkled and the material bladed in from the edges to prevent lime loss by wind. Dry mixing was accomplished with a rotary tiller and motor grader and it was continued until the entire windrow was a homogeneous mixture. Upon completion of dry mixing, water was added uniformly and mixing was continued until all the material was approximately at optimum moisture content. Prior to laydown the mixture was allowed to cure in the windrow for approximately 14 hours.

**Compaction.**—In this section the mixture was spread evenly across the roadbed, sprinkled to maintain the moisture at near optimum and compacted with a pneumatic-tired roller. The special provisions required that the density attained be not less than 100 percent of maximum. The range of densities on the compacted material is shown in Table 15.

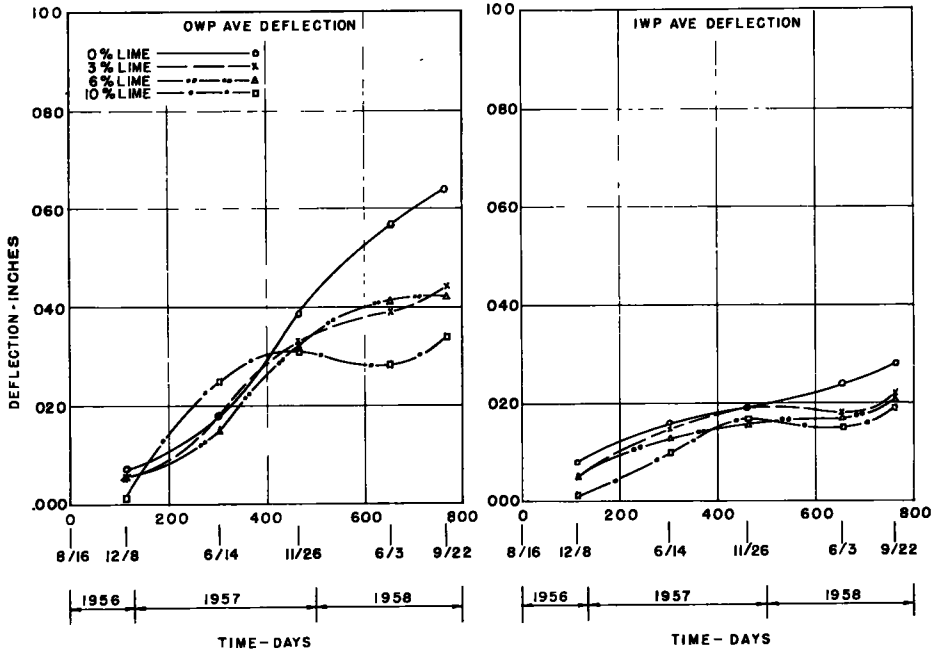


Figure 12. Average flexible pavement deflection; lime-treated soil section.

**Curing.**—After the base course was compacted and shaped to the typical cross section, it was sprinkled for a period of 4 days. Following this curing period the base was allowed to dry for 3 days, during which time it was sprinkled only enough to prevent cracking or raveling of the surface. When the moisture content of the base course was satisfactory a prime coat was applied.

**Prime Coat.**—The prime coat, Mc-1 asphaltic oil, was applied at the rate of 0.30 gal. per sqyd. However, there was practically no penetration of the prime oil into the base course material. Since there might be detrimental effects to the asphaltic concrete from the excess prime oil, it was bladed off and the base was re-primed. The rate of application of the new prime coat was approximately one-half the original or 0.15 gal. per sqyd. With the reduced rate no excess asphalt remained on the surface. It appears that a lime stabilized base will absorb very little asphaltic oil, and the amount should be reduced by approximately 50 percent of that required on a conventional stabilized base.

### Cost Estimate

A cost study on the square yard basis was made on the experimental lime subdivisions and the standard design subdivisions (Table 16). When reviewing this estimate it should be kept in mind that the lime stabilization in Nebraska was of an experimental nature, and therefore, the bid prices were probably higher than normal.

TABLE 13  
CONSTRUCTION DENSITIES OF THE LIME-TREATED SUBGRADE SOIL

Subdivision Number	Percent Lime	Optimum Moisture	Maximum Density	Range of Density (Percent of Max)	Range of Moisture (Percent)
2	3	17.7	1.72	96% to 98%	18.2% to 20.6%
3	6	18.8	1.66	96% to 105%	18.1% to 19.1%
4	10	19.3	1.64	96% to 103%	21.0% to 21.2%

TABLE 14  
DISTRIBUTION OF AGGREGATES IN THE LIME-TREATED BASE COURSE SECTION

Subdivision Number	Lime		Coarse Sand		Aftonian Silt		Combined Windrow (Tons/Sta)
	%	Tons/Sta	%	Tons/Sta	%	Tons/Sta	
6	2	1.7	83	68.9	15	12.4	83
7	4	3.3	76	63.1	20	16.6	83
8	7	5.8	73	60.6	20	16.6	83

TABLE 15  
CONSTRUCTION DENSITIES OF THE LIME-TREATED BASE COURSE

Subdivision Number	Percent Lime	Optimum Moisture	Maximum Density	Range of Density (Percent of Max)	Range of Moisture (Percent)
6	2	10.5	1.92	105% to 106%	6.3% to 8.0%
7	4	9.5	1.93	104% to 107%	8.6% to 9.1%
8	7	10.0	1.97	101% to 107%	6.4% to 6.8%

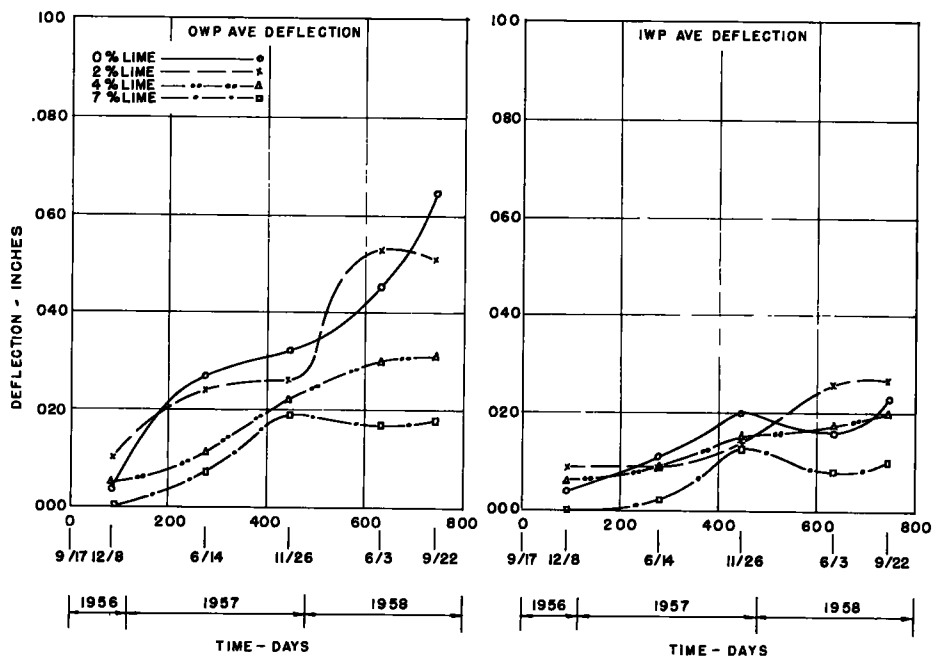


Figure 13. Average flexible pavement deflection; lime-treated base course section.

TABLE 16  
COST DATA BASED ON BID OF LOW BIDDER FOR PROJECT

Lime-Treated Subgrade Soil Section				
Type of Construction	Subdivision No. 1 <sup>a</sup>	Subdivision No. 2	Subdivision No. 3	Subdivision No. 4
	STA 1047 To STA 1062	STA 1062 To STA 1077	STA 1077 To STA 1092	STA 1092 To STA 1107
	0% Lime	3% Lime	6% Lime	10% Lime
Granular Subbase Course				
Dimension	7 in. x 30 ft	—	—	—
Cost per Sq Yd	\$0.45	—	—	—
Soil Aggregate Base Course				
Dimension	4 in. x 27 ft	4 in. x 27 ft	4 in. x 27 ft	4 in. x 27 ft
Cost per Sq Yd	\$0.52	\$0.52	\$0.52	\$0.52
Lime Treated Subgrade Soil				
Dimension	—	7 in. x 33 ft	7 x 33 ft	7 in. x 33 ft
Cost per Sq Yd	—	\$0.43	\$0.65	\$0.96
Total Cost per Sq Yd	\$0.97	\$0.95	\$1.17	\$1.48
Lime-Treated Base Course Section				
Type of Construction	Subdivision No. 5 <sup>a</sup>	Subdivision No. 6	Subdivision No. 7	Subdivision No. 8
	STA 1148 To STA 1161	STA 1161 To STA 1174	STA 1174 To STA 1187	STA 1187 To STA 1200
	0% Lime	2% Lime	4% Lime	7% Lime
Granular Subbase Course				
Dimension	7 in. x 30 ft	5 in. x 30 ft	5 in. x 30 ft	5 in. x 30 ft
Cost per Sq Yd	\$0.45	\$0.35	\$0.35	\$0.35
Soil Aggregate Base Course				
Dimension	4 in. 27 ft	—	—	—
Cost per Sq Yd	\$0.52	—	—	—
Lime Treated Base Course				
Dimension	—	6 in. x 27 ft	6 in. x 27 ft	6 in. x 27 ft
Cost per Sq Yd	—	\$0.64	\$0.81	\$1.07
Total Cost per Sq Yd	\$0.97	\$0.99	\$1.16	\$1.42

<sup>a</sup> Standard Design

## POST CONSTRUCTION SAMPLING, TESTING, AND DISCUSSION OF RESULTS

### Sampling

The first series of samples of the lime-treated subgrade soil were taken 16 days after construction. At this time an attempt was made to take undisturbed cores of the material. This attempt was unsuccessful but disturbed samples, as well as in-place density tests, were taken. The results of these tests are shown in Tables 17, 18 and 19. No attempt was made to take cores of the lime-treated base course at a similar curing period.

In December 1956, another unsuccessful attempt was made to take undisturbed cores of both the lime-treated subgrade soil and lime-treated base course. No samples of either material were obtained at this time.

No further sampling was attempted until April 1958 (600 day curing time). At this time there was moderate success in obtaining partial cores of the material in both lime-treated sections. The undisturbed pieces of the material were large enough for density determination and some cores were sufficient in size so that 2- by 2-in. cubes could be

TABLE 17  
TABULATION OF LABORATORY TEST RESULTS SUBDIVISION NO. 2 STATION 1062+00 TO STATION 1077+00  
3 PERCENT LIME-TREATED SUBGRADE SOIL

Tests	Test Site No. 2a (Fill) Station 1066+00 <sup>1</sup>						Test Site No. 2b (Cut) Station 1076+00 <sup>1</sup>					
	Untreated Soil	3% Lime-Treated Subgrade Soil				Untreated Soil	3% Lime-Treated Subgrade Soil					
	Prior to Const.	16 Days After Const.		600 Days After Const.		Prior to Const.	16 Days After Const.		600 Days After Const.			
	9' Rt.	Cents	9' Rt.	14' Rt.	9' Rt.	14' Rt.	9' Lt.	Cents	9' Lt.	14' Lt.	9' Lt.	14' Lt.
Sieve Analysis, % Ret.												
1 in.	—	—	—	—	—	—	0	—	—	—	0	—
3/8 in.	0	—	—	—	1	—	1	—	—	—	2	—
No. 4	2	0	0	0	3	—	3	0	0	0	2	0
10	2	1	1	1	5	0	7	3	2	1	4	2
20	4	12	8	8	8	7	10	11	11	10	9	12
40	7	24	18	18	13	18	13	19	21	21	15	23
50	9	29	24	24	17	25	16	24	27	27	18	28
100	15	40	35	35	24	37	22	33	37	38	27	38
200	19	47	42	42	30	45	27	40	45	47	34	45
Hydrometer Analysis												
Sand	24	57	51	50	66	50	33	51	55	61	64	66
Silt <sup>2</sup>	37	32	37	38	28	42	36	37	33	31	32	30
Clay <sup>3</sup>	39	11	12	12	6	8	31	12	12	8	4	4
Specific Gravity	2.67	2.66	2.66 <sup>3</sup>	2.66 <sup>3</sup>	2.67 <sup>3</sup>	2.66 <sup>3</sup>	2.66	2.66	2.66 <sup>3</sup>	2.66 <sup>3</sup>	2.67 <sup>3</sup>	2.66 <sup>3</sup>
Liquid Limit	47	39	40	39	38	40	44	37	37	37	38	39
Plastic Limit	23	29	29	28	32	28	20	28	30	30	31	32
Plasticity Index	24	10	11	11	6	12	24	9	7	7	7	7
Capillary	10'13"	17'25"	23'22"	28'41"	3'31"	16'11"	12'23"	33'38"	27'4"	32'10"	7'53"	21'7"
Absorption Failure	34'42"	2hr +	2hr +	2hr +	3'35"	2hr +	20'26"	2hr +	2hr +	2hr +	27'42"	2hr +
Cementation	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+
Shrinkage Limit	12.23	21.34	23.21	17.54	19.7	21.2	12.71	19.51	23.44	23.04	21.7	24.6
Shrinkage Ratio	1.86	1.589	1.561	1.639	1.603	1.60	1.84	1.644	1.522	1.541	1.568	1.49
Lineal Shrinkage	7.1	4.8	4.1	6.7	6.9	5.2	6.4	5.2	2.5	4.6	6.1	4.7
Volumetric Change	22.64	15.71	13.47	22.71	23.6	17.1	21.69	17.44	7.90	14.99	20.5	15.3
Field Moist. Equivalent	24.4	31.23	31.85	31.39	34.4	31.90	24.5	30.12	28.63	32.74	34.8	34.9
Chemical Analysis, Lime												
Top Half, Percent	1.0 <sup>4</sup>	2.7 <sup>5</sup>	2.8 <sup>5</sup>	2.9 <sup>5</sup>	2.5 <sup>5</sup>	2.4 <sup>5</sup>	0.7 <sup>4</sup>	4.0 <sup>5</sup>	4.0 <sup>5</sup>	3.2 <sup>5</sup>	3.3 <sup>5</sup>	3.1 <sup>5</sup>
Bottom Half, Percent	—	2.7 <sup>5</sup>	2.6 <sup>5</sup>	2.5 <sup>5</sup>	2.6 <sup>5</sup>	2.6 <sup>5</sup>	—	2.6 <sup>5</sup>	2.6 <sup>5</sup>	3.4 <sup>5</sup>	2.5 <sup>5</sup>	2.5 <sup>5</sup>
Unconfined Compression												
Strength Test												
Total Load, lb	698 <sup>6</sup>	—	—	—	—	—	540 <sup>6</sup>	—	—	—	—	—
P.S.I.	56	—	—	—	—	—	43	—	—	—	—	—
Percent, Moisture	18.7	—	—	—	—	—	17.8	—	—	—	—	—
Optimum Moisture, %	17.5	17.7	17.7	17.7	17.7	17.7	17.5	17.7	17.7	17.7	17.7	17.7
Maximum Density, Gm/cc	1.73	1.72	1.72	1.72	1.72	1.72	1.73	1.72	1.72	1.72	1.72	1.72
Actual Moisture, %	17.4	—	17.7	—	21.2	22.7	9.8	—	17.8	—	20.5	24.2
Actual Density, Gm/cc	1.72	—	1.67	—	1.66	1.61	1.90	—	1.66	—	1.58	1.60
Percent Optimum	99	—	100	—	120	128	56	—	101	—	116	137
Percent Maximum	99	—	97	—	97	94	110	—	97	—	92	93
AASHTO Soil Class.	A-7-6 (15)	A-4(4)	A-6(5)	A-6(5)	A-4(7)	A-6(5)	A-7-6 (14)	A-4(5)	A-4(4)	A-4(4)	A-4(6)	A-4(4)

<sup>1</sup>Depth of sample approximately 7 to 14 in. below finish grade.

<sup>2</sup>0.005 mm to 0.05 mm.

<sup>3</sup>Assumed.

<sup>4</sup>Chemical analysis of percent lime in untreated soil.

<sup>5</sup>Lime content corrected for original lime content.

<sup>6</sup>4 by 4 in. molded cylinder.



sawed from them for unconfined compressive strength tests. Besides the cores enough material was obtained for soil testing. Also at this time in-place density tests and material for soil testing were taken from the soil aggregate base course (Table 21), granular subbase course (Table 22), and subgrade soil (Tables 23, 24, 25 and 26). A volume measure was used in measuring the in-place densities of the two base course materials, while specimens for the density tests of the subgrade soil were taken by Shelby tube.

### Testing

The tests performed on the post-construction samples of the lime-treated subgrade

TABLE 18  
TABULATION OF LABORATORY TEST RESULTS SUBDIVISION NO. 3 STATION 1077 +00 TO STATION 1092 +00  
6 PERCENT LIME-TREATED SUBGRADE SOIL

Tests	Test Site No. 3a (Cut) Station 1084 +00 <sup>1</sup>						Test Site No. 3b (Fill) Station 1089 +00 <sup>1</sup>					
	Untreated Soil	6% Lime-Treated Subgrade Soil					Untreated Soil	6% Lime-Treated Subgrade Soil				
	Prior to Const.	16 Days After Const.		600 Days After Const.		Prior to Const.	16 Days After Const.		600 Days After Const.			
	9'Rt.	Cents	9'Rt.	14'Rt.	9'Rt.	14'Rt.	9'Lt.	Cents	9'Lt.	14'Lt.	9'Lt.	14' Lt.
Sieve Analysis, % Ret.												
1 in.	—	0	—	—	—	—	—	—	—	—	—	—
3/4 in.	—	1	0	0	—	—	—	—	—	—	0	—
No. 4	0	2	1	1	0	0	0	0	0	0	3	0
10	1	7	2	2	3	1	1	4	1	1	16	2
20	2	20	14	18	18	22	3	19	14	8	34	19
40	3	30	26	31	31	36	6	31	26	17	48	33
50	4	35	32	37	36	41	9	37	31	22	54	39
100	7	44	42	48	48	53	16	48	48	33	63	50
200	9	52	52	58	54	61	23	58	50	43	69	59
Hydrometer Analysis												
Sand	22	79	78	78	82	82	38	80	78	84	77	84
Silt <sup>2</sup>	52	21	21	22	18	18	31	20	20	16	20	16
Clay	26	0	1	0	0	0	31	0	2	0	3	0
Specific Gravity	2.66	2.65	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.67 <sup>3</sup>	2.66 <sup>3</sup>	2.67	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.67 <sup>3</sup>	2.66 <sup>3</sup>
Liquid Limit	40	31	32	31	NP	NP	44	33	31	33	34	NP
Plastic Limit	21	NP	31	NP	NP	NP	20	NP	30	NP	31	NP
Plastic Index	19	NP	1	NP	NP	NP	24	NP	1	NP	3	NP
Capillarity	6'12"	5'44"	5'35"	5'36"	2'44"	4'25"	9'05"	7'30"	8'59"	8'42"	20'3"	6'59"
Absorption Failure	6'12"	2 hr. +	2 hr. +	2 hr. +	2 hr. +	2 hr. +	9'05"	2 hr. +	2 hr. +	2 hr. +	21'9"	2 hr. +
Cementation	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+
Shrinkage Limit	17.75	28.39	26.15	23.77	28.5	28.9	12.93	25.86	20.93	28.07	24.24	28.4
Shrinkage Ratio	1.72	1.346	1.369	1.507	1.381	1.34	1.84	1.440	1.576	1.489	1.532	1.37
Lineal Shrinkage	3.8	2.3	5.3	6.3	4.3	5.2	5.2	4.3	6.8	6.1	4.5	4.4
Volumetric Change	10.92	7.02	17.57	20.87	14.1	16.9	18.16	15.54	23.16	20.33	14.6	14.4
Field Moist. Equivalent	24.1	33.59	38.99	37.61	38.7	41.5	22.8	36.25	35.62	36.69	33.9	38.9
Chem. Analysis, Lime												
Top Half, %	0.5 <sup>4</sup>	7.7 <sup>5</sup>	6.9 <sup>5</sup>	6.1 <sup>5</sup>	5.7 <sup>5</sup>	6.2 <sup>5</sup>	1.2 <sup>4</sup>	5.6 <sup>5</sup>	5.6 <sup>5</sup>	6.4 <sup>5</sup>	4.4 <sup>5</sup>	6.1 <sup>5</sup>
Bottom Half, %	—	4.4 <sup>5</sup>	6.0 <sup>5</sup>	5.7 <sup>5</sup>	4.4 <sup>5</sup>	5.4 <sup>5</sup>	—	5.0 <sup>5</sup>	5.6 <sup>5</sup>	6.2 <sup>5</sup>	4.5 <sup>5</sup>	6.5 <sup>5</sup>
Unconfined Compressive Strength Test												
Total Loads, Lbs.	790 <sup>6</sup>	—	—	—	—	1100 <sup>7</sup>	703 <sup>6</sup>	—	—	—	—	—
P. S. I.	63	—	—	—	—	275	56	—	—	—	—	—
% Moisture	16.9	—	—	—	—	19.7	17.0	—	—	—	—	—
Optimum Moisture, %	16.5	18.8	18.8	18.8	18.8	18.8	16.5	18.8	18.8	18.8	18.8	18.8
Maximum Density, Gm/cc	1.76	1.66	1.66	1.66	1.66	1.66	1.76	1.66	1.66	1.66	1.66	1.66
Actual Moisture, %	13.3	—	16.4	—	17.0	19.8	12.2	—	19.1	—	19.0	20.5
Actual Density, Gm/cc	1.83	—	1.63	—	1.67	1.54	1.75	—	1.60	—	1.66	1.60
% Optimum	81	—	87	—	90	105	74	—	102	—	101	109
% Maximum	104	—	98	—	101	93	99	—	96	—	100	96
AASHO Soil Class.	A-6 (12)	A-4(3)	A-4(3)	A-4(1)	A-4(2)	A-4(1)	A-7-6 (14)	A-4(1)	A-4(3)	A-4(4)	A-2-4 (0)	A-4(1)

<sup>1</sup>Depth of sample approximately 7 to 14 in. below finish grade.

<sup>2</sup>0.005 mm to 0.05 mm

<sup>3</sup>Assumed.

<sup>4</sup>Chemical analysis of percent lime in untreated soil.

<sup>5</sup>Lime content corrected for original lime content.

<sup>6</sup>4 by 4 in. molded cylinder.

<sup>7</sup>2 by 2 in. cube cut from core.

soil included those outlined in Table 4 (except AASHO T99-49) and also in-place moisture density determinations and chemical analyses for percentage of lime. Only sieve analysis, unconfined compressive strength tests, and chemical analyses for percentage of lime were performed on the lime-treated base course samples. Tests performed on the soil aggregate base course and granular subbase course samples included sieve and hydrometer analysis, determination of plasticity index, specific gravity, and in-place moisture density tests. Tests of the untreated subgrade soil samples in both lime-treated sections included those outlined in Table 4 (except AASHO T99-49) and in-place moisture density determinations.

TABLE 19  
TABULATION OF LABORATORY TEST RESULTS SUBDIVISION NO. 4 STATION 1092+00 TO STATION 1107+00  
10 PERCENT LIME-TREATED SUBGRADE SOIL

Tests	Test Site No. 4a (Cut) Station 1096+00 <sup>1</sup>						Test Site No. 4b (Fill) Station 1106+00 <sup>1</sup>					
	Untreated Soil	10% Lime-Treated Subgrade Soil				Untreated Soil	10% Lime-Treated Subgrade Soil					
	Prior to Const.	16 Days After Const.		600 Days After Const.		Prior to Const.	16 Days After Const.		600 Days After Const.			
	9' Lt.	Cents	9' Lt.	14' Lt.	9' Lt.	14' Lt.	9' Rt.	Cents	9' Rt.	14' Rt.	9' Rt.	14' Rt.
Sieve Analysis, % Ret.												
1 in.	—	—	—	—	—	—	—	—	—	—	—	—
3/8 in.	—	—	—	—	—	—	—	—	—	—	—	—
No. 4	—	—	0	0	0	0	1	—	—	—	0	1
10	0	0	0	1	2	2	1	0	0	2	2	2
20	1	9	7	11	20	33	1	7	6	5	20	22
40	1	20	17	21	33	46	3	17	14	13	32	37
50	1	24	22	25	37	50	4	22	19	18	37	42
100	3	33	30	33	43	57	10	34	32	29	47	54
200	6	40	39	41	48	63	20	46	44	41	55	63
Hydrometer Analysis												
Sand	20	81	81	80	84	88	33	79	82	82	82	86
Silt <sup>2</sup>	49	19	19	20	14	12	34	21	18	18	17	14
Clay	31	0	0	0	2	0	33	0	0	0	1	0
Specific Gravity	2.67	2.65	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.67 <sup>3</sup>	2.66 <sup>3</sup>	2.65	2.65	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.67	2.66
Liquid Limit	47	36	36	37	NP	NP	40	31	32	30	NP	NP
Plastic Limit	22	NP	NP	NP	NP	NP	22	NP	NP	NP	NP	NP
Plasticity Index	25	NP	NP	NP	NP	NP	18	NP	NP	NP	NP	NP
Capillarity	15'4"	4'25"	4'3"	3'3"	19'12"	8'24"	15'7"	6'5"	5'44"	5'15"	3'13"	5'34"
Absorption Failure	22'7"	2 hr. +	2 hr. +	2 hr. +	2 hr. +	39'8"	2 hr. +	2 hr. +	2 hr. +	2 hr. +	2 hr. +	2 hr. +
Cementation	200+	200+	200+	200+	182	200+	200+	200+	200+	152	200+	200+
Shrinkage Limit	14.96	27.09	27.41	27.82	29.7	33.2	14.03	24.65	25.70	25.99	27.6	28.1
Shrinkage Ratio	1.80	1.449	1.375	1.331	1.358	1.30	1.80	1.361	1.351	1.366	1.314	1.30
Lineal Shrinkage	4.8	5.5	5.6	6.5	5.6	4.2	4.70	5.2	5.00	5.1	5.3	3.6
Volumetric Change	16.27	17.93	18.41	21.89	18.5	13.8	15.60	17.19	16.48	16.62	17.9	11.6
Field Moist. Equivalent	24.0	39.47	40.79	44.26	43.3	43.8	22.70	37.27	37.88	38.15	41.2	37.0
Chem. Analysis, Lime												
Top Half, %	1.0 <sup>4</sup>	10.3 <sup>5</sup>	9.8 <sup>5</sup>	9.8 <sup>5</sup>	8.6 <sup>5</sup>	9.6 <sup>5</sup>	0.5 <sup>4</sup>	7.5 <sup>5</sup>	7.5 <sup>5</sup>	6.9 <sup>5</sup>	6.1 <sup>5</sup>	7.0 <sup>5</sup>
Bottom Half, %	—	9.7 <sup>5</sup>	9.4 <sup>5</sup>	8.7 <sup>5</sup>	5.8 <sup>5</sup>	10.1 <sup>5</sup>	—	7.1 <sup>5</sup>	7.0 <sup>5</sup>	6.8 <sup>5</sup>	6.8 <sup>5</sup>	6.4 <sup>5</sup>
Unconfined Compression												
Strength Test												
Total load, lb	780 <sup>5</sup>	—	—	—	—	—	703 <sup>5</sup>	—	—	—	—	—
P. S. I.	62	—	—	—	—	—	56	—	—	—	—	—
% Moisture	18.7	—	—	—	—	—	17.7	—	—	—	—	—
Optimum Moisture, %	18.0	19.3	19.3	19.3	19.3	19.3	18.0	19.3	19.3	19.3	19.3	19.3
Maximum Density, Gm/cc	1.69	1.64	1.64	1.64	1.64	1.64	1.69	1.64	1.64	1.64	1.64	1.64
Actual Moisture, %	19.3	—	20.6	—	23.5	26.6	10.2	—	21.5	—	18.3	20.5
Actual Density, Gm/cc	1.76	—	1.62	—	1.56	1.37	1.81	—	1.58	—	1.65	1.58
% Optimum	107	—	107	—	122	138	57	—	111	—	95	106
% Maximum	104	—	99	—	95	84	107	—	97	—	101	96
AASHO Soil Class.	A-7-6 (15)	A-4(5)	A-4(5)	A-4(5)	A-4(3)	A-4(0)	A-6 (11)	A-4(4)	A-4(4)	A-4(5)	A-4(2)	A-4(0)

<sup>1</sup> Depth of samples approximately 7 to 14 in. below finish grade.

<sup>2</sup> 0.005 mm to 0.05 mm.

<sup>3</sup> Assumed.

<sup>4</sup> Chemical analysis of percent lime in untreated soil.

<sup>5</sup> Lime content corrected for original lime content.

<sup>6</sup> 4 by 4 in. molded cylinder.

TABLE 20  
LABORATORY TEST RESULTS FOR LIME-TREATED BASE COURSE STATION 1161 TO STATION 1200

Tests	Subd. No. 6, Sta 1161 to Sta 1174, 2% Lime					Subd. No. 7, Sta 1174 to Sta 1187, 4% Lime					Subd. No. 8, Sta 1187 to Sta 1200, 7% Lime				
	Sta 1170	Test Site 6a (Fill) Sta 1167		Test Site 6b (Fill) Sta 1171		Sta 11800	Test Site 7a (Fill) Sta 1178		Test Site 7b (Fill) Sta 1183		Sta 1195	Test Site 8a (Cut) Sta 1192		Test Site 8b (Fill) Sta 1199	
	During Const.	600 Days After Const.		600 Days After Const.		During Const.	600 Days After Const.		600 Days After Const.		During Const.	600 Days After Const.		600 Days After Const.	
	9'Rt	9'Rt	14'Rt	9'Lt	14'Lt	9'Lt	9'Lt	14'Lt	9'Rt	14'Rt	9'Lt	9'Lt	14'Lt	9'Rt	14'Rt
Sieve Analysis, % Ret.															
1 in.	—	—	—	—	0	—	—	—	—	—	—	—	—	—	—
3/8 in.	0	—	—	0	1	0	0	0	0	0	0	0	0	—	—
No. 4	1	0	0	1	1	3	1	1	0	1	1	2	1	0	0
10	3	2	2	3	2	5	3	2	1	2	3	3	4	1	1
20	11	8	11	11	10	12	16	11	9	12	13	14	14	9	6
30	27	25	28	30	27	23	35	26	25	29	24	31	28	23	20
40	55	51	57	54	53	49	56	49	49	56	52	52	51	45	40
50	69	68	72	69	67	63	70	62	65	70	66	67	65	58	55
80	76	76	79	76	74	69	76	69	71	76	72	75	72	66	64
100	77	78	81	77	76	71	78	70	73	78	74	77	74	68	67
200	79	82	84	81	80	73	84	75	78	82	76	83	78	74	72
Chemical Analysis															
Lime, Percent	—	3.8	3.6	4.0	4.2	—	6.1	4.9	6.3	6.1	—	7.8	7.4	11.6	11.2
Optimum Moisture, %	10.5	10.5	10.5	10.5	10.5	10.5	9.5	9.5	9.5	9.5	10.0	10.0	10.0	10.0	10.0
Maximum Density, gm/cc	1.92	1.92	1.92	1.92	1.92	1.93	1.93	1.93	1.93	1.93	1.97	1.97	1.97	1.97	1.97
Actual Moisture, %	8.0	7.1	8.3	8.3	5.4	9.1	8.7	10.7	8.7	9.5	6.7	7.9	9.5	6.6	7.8
Actual Density, gm/cc	2.01	1.92	1.88	1.93	1.98	2.00	1.92	1.95	1.87	1.89	2.03	1.96	1.94	1.88	1.99
Percent of Optimum	76	68	79	79	51	96	92	113	92	100	67	79	95	66	78
Percent of Maximum	105	100	98	100	103	104	99	101	97	98	103	99	99	96	101
Unconfined Compressive Strength Test															
Total Load, lb	—	—	—	660 <sup>1</sup>	—	—	—	1670	—	2980 <sup>1</sup>	—	3180 <sup>1</sup>	—	1620 <sup>1</sup>	—
P. S. I.	—	—	—	165	—	—	—	430	—	745	—	795	—	405	—
% Moisture as Tested	—	—	—	8.9	—	—	—	9.8	—	8.3	—	8.2	—	8.0	—

<sup>1</sup> 2 by 2 in. cube cut from core.



TABLE 22  
TESTS RESULTS FOR GRANULAR SUBBASE COURSE

Tests	Subd. No. 1 Sta 1047 to 1062 <sup>1</sup>						Subd. No. 5 Sta 1148 to 1161 <sup>1</sup>						Subd. No. 6 Sta 1161 to 1174 <sup>1</sup>						Subd. No. 7 Sta 1174 to 1187 <sup>1</sup>						Subd. No. 8 Sta 1187 to 1200 <sup>1</sup>					
	Sta 1050	Test Site 1a (Fill) Sta 1052		Test Site 1b (Cut) Sta 1059		Sta 1150	Test Site 5a (Cut) Sta 1151		Test Site 5b (Fill) Sta 1159		Sta 1170	Test Site 6a (Fill) Sta 1167		Test Site 6b (Fill) Sta 1171		Sta 1180	Test Site 7a (Fill) Sta 1178		Test Site 7b (Fill) Sta 1183		Sta 1190	Test Site 8a (Cut) Sta 1192		Test Site 8b (Fill) Sta 1199						
	During Const.	600 Days After Const.		600 Days After Const.		During Const.	600 Days After Const.		600 Days After Const.		During Const.	600 Days After Const.		600 Days After Const.		During Const.	600 Days After Const.		600 Days After Const.		During Const.	600 Days After Const.		600 Days After Const.						
	9'Lt	9'Lt	14'Lt	9'Rt	14'Rt	9'Lt	9'Lt	14'Lt	9'Rt	14'Rt	9'Lt	9'Rt	14'Rt	9'Lt	14'Lt	9'Lt	9'Lt	14'Lt	9'Rt	14'Rt	9'Lt	9'Lt	14'Lt	9'Rt	14'Rt					
Sieve Analysis, % Ret.																														
1 in.	0	0	—	0	0	0	0	1	0	0	—	—	—	—	0	—	—	—	—	—	0	0	0	0	0	0				
3/4 in.	1	2	0	6	4	3	1	5	8	3	0	—	0	0	0	1	0	1	0	—	2	1	4	1	1	1				
No 4	4	7	3	8	6	6	5	8	9	7	1	0	2	1	3	1	2	2	0	4	5	5	2	2	2	2				
10	10	16	7	16	11	11	11	14	16	14	4	2	3	2	2	7	3	4	5	2	8	8	9	5	6	6				
20	27	34	13	34	24	26	27	30	29	26	10	10	9	8	7	15	11	13	16	10	18	19	17	15	18	18				
40	63	68	66	72	56	63	80	64	57	56	59	56	55	53	51	55	57	61	63	50	57	56	51	54	58	58				
50	76	79	79	83	69	75	73	77	70	68	76	74	74	70	68	68	74	80	79	65	71	75	69	67	70	70				
100	83	85	86	88	76	85	83	84	85	83	83	81	80	78	76	82	82	87	87	72	83	84	77	76	79	79				
200	85	87	88	90	79	88	86	88	88	87	86	84	83	80	77	88	85	90	89	74	85	86	79	80	82	82				
Hydrometer Analysis																														
Sand	88	89	91	94	82	90	91	91	92	91	85	87	87	84	82	90	88	93	91	—	87	89	85	85	88	88				
Silt	5	6	4	3	9	4	5	4	4	4	7	5	5	7	7	4	5	2	3	—	6	5	8	6	5	5				
Clay	7	5	5	3	9	6	4	5	4	5	8	8	8	9	11	6	7	5	6	—	7	6	7	9	7	7				
Specific Gravity	2.66	2.67 <sup>2</sup>	2.67 <sup>3</sup>	2.67 <sup>3</sup>	2.67	2.65	2.67 <sup>2</sup>	2.67 <sup>3</sup>	2.67	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.65 <sup>3</sup>	—	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.65 <sup>3</sup>	2.65				
Liquid Limit	21	22	19	19	24	19	20	19	NP	26	25	24	28	27	19	18	NP	20	—	22	NP	21	25	23	23	23				
Plastic Limit	17	17	16	15	16	17	17	17	NP	16	15	15	16	16	15	16	NP	15	—	18	NP	16	17	15	15	15				
Plasticity Index	4	5	3	4	8	2	3	3	NP	10	10	9	12	11	4	2	NP	5	—	4	NP	5	8	8	8	8				
Optimum Moisture, %	12.3	12.3	12.3	12.3	12.3	12.3	12.3	12.3	12.3	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0	13.0	—	13.0	13.0	13.0	13.0	13.0	13.0				
Maximum Density, gm/cc	1.87	1.87	1.87	1.87	1.87	1.87	1.87	1.87	1.87	1.82	1.82	1.82	1.82	1.82	1.82	1.82	1.82	1.82	1.82	—	1.82	1.82	1.82	1.82	1.82	1.82				
Actual Moisture, %	13.0	5.8	5.8	5.3	8.7	10.5	5.3	7.0	5.3	5.8	11.1	6.4	8.1	7.5	9.3	10.1	7.0	9.3	6.4	10.5	5.3	10.5	2.1	6.0	6.0					
Actual Density, gm/cc	2.01	1.98	1.87	1.88	1.77	2.06	1.90	1.64	1.83	1.85	1.82	2.17	1.97	2.08	1.85	1.76	1.98	1.86	1.97	1.94	2.01	2.10	1.98	1.97	2.16	2.16				
Percent of Optimum	106	47	47	43	71	85	43	57	43	47	85	49	62	58	72	78	54	54	72	49	81	41	81	16	46	46				
Percent of Maximum	107	106	100	100	95	110	102	88	98	99	100	119	108	114	102	97	109	102	108	107	110	115	109	108	119	119				
AASHTO Soil Class.	A-1-b	A-1-b	A-1-b	A-1-b	A-2-4	A-1-b	A-1-b	A-1-b	A-1-b	A-1-b	A-2-4	A-2-4	A-2-4	A-2-6	A-2-6	A-1-b	A-1-b	A-1-b	A-1-b	A-2-4	A-1-b	A-1-b	A-1-b	A-2-4	A-2-4	A-2-4				
	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)	(0)				

<sup>1</sup> Sample depth in subdivision 1 and 5 approximately 7 to 14 in below finish grade, in subdivision 6, 7 and 8 approximately 9 to 14 in below finish grade.  
<sup>2</sup> 0.005 to 0.05 mm  
<sup>3</sup> Assumed.

**TABLE 23**  
**TEST RESULTS FOR UNTREATED SUBGRADE SOIL BELOW LIME TREATMENT, LIME-TREATED SUBGRADE SOIL SECTION**

Tests	Subd. No. 1 Sta 1047 to Sta 1062 <sup>1</sup>				Subd. No. 2 Sta 1062 to Sta 1077 <sup>1</sup>				Subd. No. 3 Sta 1077 to Sta 1092 <sup>2</sup>				Subd. No. 4 Sta 1092 to 1107 <sup>1</sup>			
	Test Site 1a (Fill) Sta 1052		Test Site 1b (Cut) Sta 1059		Test Site 2a (Fill) Sta 1066		Test Site 2b (Cut) Sta 1076		Test Site 3a (Cut) Sta 1084		Test Site 3b (Fill) Sta 1089		Test Site 4a (Cut) Sta 1096		Test Site 4b (Fill) Sta 1106	
	Prior to Const.	600 Days After Const.	Prior to Const.	600 Days After Const.	Prior to Const.	600 Days After Const.	Prior to Const.	600 Days After Const.	Prior to Const.	600 Days After Const.	Prior to Const.	600 Days After Const.	Prior to Const.	600 Days After Const.	Prior to Const.	600 Days After Const.
	9'Lt	9'Lt	9'Rt	9'Rt	9'Rt	9'Rt	9'Lt	9'Lt	9'Rt	9'Rt	9'Lt	9'Lt	9'Lt	9'Lt	9'Rt	9'Rt
Sieve Analysis, % Ret																
1 in	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
½ in.	—	0	—	—	—	—	—	—	—	—	—	—	—	—	—	—
No. 4	0	1	—	—	—	—	—	—	—	—	0	—	—	—	—	—
10	2	2	0	0	0	0	0	—	—	—	2	0	0	—	0	0
20	5	5	1	1	1	1	1	—	—	—	4	1	1	0	1	1
40	10	10	4	4	2	3	2	—	—	—	9	6	1	1	2	2
50	14	14	7	7	3	4	3	2	—	0	13	13	1	1	3	3
100	23	23	12	12	5	7	6	4	0	1	24	24	2	2	8	8
200	28	28	16	16	7	9	9	8	1	1	31	31	5	4	14	13
Hydrometer Analysis																
Sand	37	33	24	25	13	15	17	19	16	22	39	42	15	18	28	22
Silt <sup>3</sup>	24	31	37	29	46	49	44	45	68	68	24	23	48	51	36	41
Clay	39	36	39	36	41	36	39	36	16	10	37	35	37	31	36	37
Specific Gravity	2.69 <sup>3</sup>	2.67 <sup>3</sup>	2.67 <sup>3</sup>	2.67 <sup>3</sup>	2.66	2.67 <sup>3</sup>	2.67 <sup>3</sup>	2.67 <sup>3</sup>	2.63 <sup>3</sup>	2.69	2.66 <sup>3</sup>	2.67 <sup>3</sup>	2.63 <sup>3</sup>	2.67 <sup>3</sup>	2.63 <sup>3</sup>	2.67 <sup>3</sup>
Liquid Limit	44	47	41	39	44	43	44	42	29	31	41	46	46	47	40	47
Plastic Limit	22	22	21	23	23	24	23	24	24	26	21	22	20	24	19	22
Plasticity Index	22	25	20	16	21	19	21	18	5	3	20	24	26	23	21	25
Capillarity	—	28'42"	—	4'32"	—	6'3"	—	6'24"	—	1'15"	—	7'6"	—	9'19"	—	5'57"
Absorption Failure	—	40'53"	—	4'33"	—	6'3"	—	6'26"	—	2'9"	—	7'6"	—	9'19"	—	6'2"
Cementation	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+
Shrinkage Limit	—	12.2	—	10.1	—	11.4	—	11.2	—	21.7	—	9.3	—	12.1	—	10.4
Shrinkage Ratio	—	1.92	—	1.98	—	1.92	—	1.90	—	1.61	—	1.98	—	1.89	—	1.94
Lineal Shrinkage	—	9.9	—	10.6	—	11.7	—	11.6	—	5.3	—	12.5	—	11.1	—	11.5
Volumetric Change	—	36.4	—	39.6	—	45.0	—	44.1	—	17.5	—	49.1	—	42.3	—	44.4
Field Moist. Equivalent	—	31.2	—	30.1	—	34.8	—	34.4	—	32.6	—	34.1	—	34.5	—	33.3
AASHTO Soil Class.	A-7-6 (14)	A-7-6 (15)	A-7-6 (12)	A-6(10)	A-7-6 (13)	A-7-6 (12)	A-7-6 (13)	A-7-6 (12)	A-4(6)	A-4(8)	A-7-6 (11)	A-7-6 (13)	A-7-6 (18)	A-7-6 (15)	A-6(12)	A-7-6 (15)

<sup>1</sup> Depth of sample approximately 14 to 26 in. below finish grade elevation (below lime treatment).

<sup>2</sup> 0.005 mm to 0.05 mm.

<sup>3</sup> Assumed

TABLE 24  
TEST RESULTS FOR UNTREATED SUBGRADE SOIL BELOW GRANULAR SUBBASE COURSE, LIME-TREATED BASE COURSE SECTION

Tests	Subd No 5 Sta 1148 to Sta 1161 <sup>1</sup>				Subd No 6 Sta 1161 to Sta 1174 <sup>1</sup>				Subd No 7 Sta 1174 to Sta 1187 <sup>1</sup>				Subd No 8 Sta 1187 to Sta 1200 <sup>1</sup>			
	Test Site 5a (Cut) Sta 1151		Test Site 5b (Fill) Sta 1159		Test Site 6a (Fill) Sta 1167		Test Site 6b (Fill) Sta 1171		Test Site 7a (Fill) Sta 1178		Test Site 7b (Fill) Sta 1183		Test Site 8a (Cut) Sta 1192		Test Site 8b (Fill) Sta 1199	
	Prior to Const	600 Days After Const	Prior to Const	600 Days After Const.	Prior to Const	600 Days After Const.	Prior to Const	600 Days After Const.	Prior to Const	600 Days After Const.	Prior to Const	600 Days After Const.	Prior to Const	600 Days After Const.	Prior to Const.	600 Days After Const.
	9'Lt	9'Lt	9'Rt	9'Rt	9'Rt	9'Rt	9'Lt	9'Lt	9'Lt	9'Lt	9'Rt	9'Rt	9'Lt	9'Lt	9'Rt	9'Rt
Sieve Analysis, % Ret.																
1 in.	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
3/8 in.	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
No. 4	0	0	0	0	0	0	0	1	1	—	—	—	—	—	—	0
10	1	1	1	2	1	1	5	4	5	0	1	0	0	1	2	1
40	1	2	2	3	2	2	5	6	8	7	1	1	1	1	6	5
50	2	2	2	3	3	2	6	7	9	8	2	1	2	2	6	7
100	4	4	4	5	3	3	7	9	10	10	3	2	2	3	12	13
200	5	5	5	6	4	5	9	11	11	11	3	4	3	3	16	17
Hydrometer Analysis																
Sand	12	10	11	13	15	14	18	19	20	19	10	7	12	10	27	26
Silt <sup>2</sup>	43	44	43	44	45	47	46	45	43	42	52	53	51	52	45	44
Clay	45	46	46	43	40	39	36	36	37	39	38	40	37	38	28	30
Specific Gravity	2.67 <sup>3</sup>	2.67 <sup>3</sup>	2.67	2.67	2.67 <sup>3</sup>	2.62	2.67 <sup>3</sup>	2.65	2.67 <sup>3</sup>	2.66	2.67 <sup>3</sup>	2.63	2.67 <sup>3</sup>	2.72	2.67 <sup>3</sup>	2.67
Liquid Limit	52	54	51	50	48	49	44	45	47	48	50	49	50	51	40	37
Plastic Limit	31	30	29	28	27	28	23	25	26	27	27	28	27	26	23	21
Plasticity Index	21	24	22	24	21	21	21	20	21	21	23	21	23	25	17	16
Capillarity	—	0'12"	—	5'42"	—	10'16"	—	40'34"	—	19'45"	—	14'57"	—	5'28"	—	21'32"
Absorption Failure	—	6'12"	—	5'42"	—	10'16"	—	43'27"	—	19'45"	—	14'57"	—	5'28"	—	21'56"
Cementation	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+	200+
Shrinkage Limit	—	8.9	—	9.0	—	13.3	—	14.7	—	12.5	—	12.9	—	11.7	—	12.8
Shrinkage Ratio	—	1.98	—	1.97	—	1.81	—	1.79	—	1.85	—	1.84	—	1.90	—	1.85
Linear Shrinkage	—	16.2	—	14.7	—	13.4	—	10.4	—	12.7	—	12.8	—	11.0	—	11.1
Volumetric Change	—	88.7	—	60.3	—	53.4	—	38.8	—	50.0	—	50.8	—	41.4	—	41.8
Field Moist Equivalent	—	43.6	—	39.6	—	42.8	—	36.4	—	39.5	—	40.5	—	33.5	—	35.4
AASHO Soil Class	A-7-5 (15)	A-7-5 (16)	A-7-6 (15)	A-7-6 (16)	A-7-6 (14)	A-7-6 (14)	A-7-6 (13)	A-7-6 (13)	A-7-6 (14)	A-7-6 (14)	A-7-6 (15)	A-7-6 (14)	A-7-6 (15)	A-7-6 (16)	A-6(11)	A-6(10)

<sup>1</sup>Depth of sample approximately 14 to 26 in below finish grade elevation.

<sup>2</sup>0.005 to 0.05 mm

<sup>3</sup>Assumed.

TABLE 25  
MOISTURE AND DENSITY TESTS FOR UNTREATED SUBGRADE SOIL BELOW LIME TREATMENT, LIME-TREATED SUBGRADE SOIL SECTION

Tests	Subd. No. 1, Sta. 1047 to Sta. 1062									Subd. No. 2, Sta. 1062 to Sta. 1077						Subd. No. 3, Sta. 1077 to Sta. 1092						Subd. No. 4, Sta. 1092 to Sta. 1107					
	Test Site 1a (Fill) Sta. 1052			Test Site 1b (Cut) Sta. 1059			Test Site 2a (Fill) Sta. 1066			Test Site 2b (Cut) Sta. 1076			Test Site 3a (Cut) Sta. 1084			Test Site 3b (Fill) Sta. 1089			Test Site 4a (Cut) Sta. 1096			Test Site 4b (Fill) Sta. 1106					
	Prior to Const.		600 Days After Const.	Prior to Const.		600 Days After Const.	Prior to Const.		600 Days After Const.	Prior to Const.		600 Days After Const.	Prior to Const.		600 Days After Const.	Prior to Const.		600 Days After Const.	Prior to Const.		600 Days After Const.						
	9'Lt	9'Lt	14'Lt	9'Rt	9'Rt	14'Rt	9'Rt	9'Rt	14'Rt	9'Lt	9'Lt	14'Lt	9'Rt	9'Rt	14'Rt	9'Lt	9'Lt	14'Lt	9'Rt	9'Rt	14'Rt						
Optimum Moisture, %	17.5	17.5	17.5	17.5	17.5	17.5	17.5	17.5	17.5	17.5	17.5	16.5	16.5	16.5	16.5	16.5	16.5	18.0	18.0	18.0	18.0	18.0	18.0				
Maximum Density, gm/cc	1.73	1.73	1.73	1.73	1.73	1.73	1.73	1.73	1.73	1.73	1.73	1.76	1.76	1.76	1.76	1.76	1.76	1.69	1.69	1.69	1.69	1.69	1.69				
Depth Below Final Grade Elevation, In	14-26	12-24	12-24	14-26	14-26	13-25	14-26	14-26	14-26	14-26	14-26	14-26	13-25	14-26	14-26	14-26	14-26	14-26	13-25	14-26	14-26	14-26	15-27				
Actual Moisture, %	12.4	20.5	22.3	17.2	22.0	22.5	16.8	20.2	24.4	11.9	18.1	14.2	23.4	24.1	12.3	17.8	19.9	18.3	22.5	24.3	11.8	22.2	—				
Actual Density, gm/cc	1.82	1.69	1.67	1.71	1.64	1.64	1.72	1.68	1.60	1.82	1.75	1.84	1.60	1.56	1.78	1.73	1.69	1.72	1.53	1.63	1.82	1.63	—				
Percent of Optimum	71	117	127	98	126	129	96	115	139	88	103	86	142	146	75	108	121	102	125	135	66	123	—				
Percent of Maximum	105	98	97	99	95	95	99	97	92	105	101	104	91	89	101	98	96	102	91	96	108	96	—				
Depth Below Final Grade Elevation, in	—	24-36	24-36	—	26-38	25-37	—	26-38	26-38	—	26-38	26-38	—	25-37	26-38	—	26-38	26-38	—	25-37	26-38	—	26-38				
Actual Moist, %	—	18.7	22.2	—	19.8	21.9	—	23.5	21.5	—	18.7	23.6	—	28.6	28.4	—	14.9	21.4	—	22.0	23.8	—	22.2				
Actual Density, gm/cc	—	1.66	1.69	—	1.67	1.54	—	1.59	1.68	—	1.51	1.63	—	1.46	1.46	—	1.62	1.60	—	1.67	1.59	—	1.65				
Percent of Optimum	—	107	127	—	113	125	—	134	123	—	107	135	—	173	172	—	90	130	—	122	132	—	123				
Percent of Maximum	—	96	98	—	97	89	—	92	97	—	87	94	—	83	83	—	92	91	—	99	94	—	98				
Depth Below Final Grade Elevation, in	—	36-48	36-48	—	—	—	—	—	—	—	—	—	—	—	—	—	38-50	38-50	—	—	—	—	38-50				
Actual Moisture, %	—	22.6	22.3	—	—	—	—	—	—	—	—	—	—	—	—	—	16.8	21.5	—	—	—	—	14.3				
Actual Density, gm/cc	—	1.65	1.65	—	—	—	—	—	—	—	—	—	—	—	—	—	1.70	1.67	—	—	—	—	1.75				
Percent of Optimum	—	129	127	—	—	—	—	—	—	—	—	—	—	—	—	—	102	130	—	—	—	—	79				
Percent of Maximum	—	95	95	—	—	—	—	—	—	—	—	—	—	—	—	—	97	95	—	—	—	—	104				
Depth Below Final Grade Elevation, in	—	48-60	48-60	—	—	—	—	—	—	—	—	—	—	—	—	—	50-62	50-62	—	—	—	—	50-62				
Actual Moist, %	—	20.5	22.3	—	—	—	—	—	—	—	—	—	—	—	—	—	17.3	18.5	—	—	—	—	10.3				
Actual Density, gm/cc	—	1.71	1.68	—	—	—	—	—	—	—	—	—	—	—	—	—	1.62	1.70	—	—	—	—	1.62				
Percent of Optimum	—	117	127	—	—	—	—	—	—	—	—	—	—	—	—	—	105	112	—	—	—	—	87				
Percent of Maximum	—	99	97	—	—	—	—	—	—	—	—	—	—	—	—	—	92	97	—	—	—	—	96				



TABLE 26  
MOISTURE AND DENSITY TESTS FOR UNTREATED SUBGRADE SOIL BELOW GRANULAR SUBBASE COURSE; LIME-TREATED BASE COURSE SECTION

Tests	Subd No. 5, Sta. 1148 to Sta. 1161						Subd No. 6, Sta. 1161 to Sta. 1174						Subd. No. 7, Sta 1174 to Sta. 1187						Subd. No. 8, Sta 1187 to Sta. 1200					
	Test Site 5a (Cut) Sta. 1151			Test Site 5b (Fill) Sta. 1159			Test Site 6a (Fill) Sta. 1167			Test Site 6b (Fill) Sta. 1171			Test Site 7a (Fill) Sta. 1178			Test Site 7b (Fill) Sta. 1183			Test Site 8a (Cut) Sta. 1192			Test Site 8b (Fill) Sta. 1199		
	Prior to Const.		600 Days After Const.	Prior to Const.		600 Days After Const.	Prior to Const.		600 Days After Const.	Prior to Const.		600 Days After Const.	Prior to Const.		600 Days After Const.	Prior to Const.		600 Days After Const.	Prior to Const.		600 Days After Const.	Prior to Const.		600 Days After Const.
	9'Lt	9'Lt	14'Lt	9'Lt	9'Lt	14'Lt	9'Lt	9'Lt	14'Lt	9'Lt	9'Lt	14'Lt	9'Lt	9'Lt	14'Lt	9'Lt	9'Lt	14'Lt	9'Lt	9'Lt	14'Lt	9'Lt	9'Lt	14'Lt
Optimum Moisture, %	18.7	18.7	18.7	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	19.0	20.1	20.1	20.1	20.1
Maximum Density, gm/cc	1.68	1.68	1.88	1.59	1.59	1.59	1.59	1.59	1.59	1.59	1.59	1.59	1.59	1.59	1.59	1.59	1.59	1.59	1.59	1.59	1.59	1.59	1.59	1.59
Depth Below Final Grade Elevation, in.	14-26	12-24	14-26	14-26	14-26	15-27	14-26	13-25	13-25	14-26	12-24	11-23	14-26	12-24	12-24	14-26	13-25	10-23	14-26	13-25	11-23	14-26	12-24	11-23
Actual Moisture, %	16.7	24.7	31.1	17.1	23.3	—	19.0	28.9	—	23.0	—	33.7	22.1	31.0	31.5	20.6	—	15.2	—	29.7	17.9	23.5	9.9	9.9
Actual Density, gm/cc	1.72	1.60	1.46	1.61	1.66	—	1.65	1.36	—	1.49	—	1.24	1.56	1.38	1.37	1.64	—	—	1.76	—	1.43	1.70	1.50	1.76
Percent of Optimum	89	132	166	80	123	—	100	152	—	121	—	177	116	163	166	108	—	—	76	—	148	89	117	49
Percent of Maximum	102	96	87	101	104	—	104	86	—	94	—	78	98	87	86	103	—	—	111	—	90	107	94	111
Depth Below Final Grade Elevation, in.	—	24-36	26-38	—	26-38	27-39	—	25-37	25-37	—	24-36	23-35	—	24-36	24-36	—	25-37	22-34	—	25-37	23-35	—	24-36	23-35
Actual Moisture, %	—	21.4	30.2	—	25.8	20.9	—	26.6	30.1	—	24.4	26.9	—	31.0	21.1	—	29.4	28.7	—	19.4	28.5	—	13.7	11.2
Actual Density, gm/cc	—	1.60	1.43	—	1.57	1.63	—	1.39	1.40	—	1.59	1.53	—	1.53	1.64	—	1.32	1.43	—	1.67	1.38	—	1.85	1.55
Percent of Optimum	—	114	161	—	136	110	—	140	158	—	128	142	—	163	111	—	155	161	—	97	142	—	68	56
Percent of Maximum	—	95	85	—	99	102	—	87	88	—	100	96	—	96	103	—	83	90	—	105	87	—	116	97

### Discussion of the Lime-Treated Subgrade Soil

The laboratory test results for the two testing periods have been tabulated in Tables 16, 17 and 18. Also included in each table are the laboratory test results of the untreated subgrade soils obtained prior to construction. These tests are included for comparison between the untreated and lime-treated soils.

Chemical analysis for lime content was performed on the untreated soil samples and on the lime-treated soils in order to determine the actual percentage of lime added. It will be noted that in general the lime content is close to plan quantity.

While enough undisturbed material was obtained during the 600-day sampling for moisture and density tests, only one core was sufficiently large to cut a 2- by 2-in. cube for unconfined compressive strength tests. The 600-day moisture tests show a general increase in the moisture content (in all but three tests) over the 16-day tests. These changes may be greater than normal due to the unusually high precipitation since construction. The tests show some variance in density between the 16- and 600-day tests. There is no consistent relationship between percentage of lime and loss or gain in density. However, all of the densities under the surfaced area except one (Test Site 2b, 3 percent lime) are above 95 percent of maximum, while four of the six densities taken outside the surfaced area are below 95 percent of maximum.

In general the results of the field tests agree with the findings of the experimental laboratory tests.

### Discussion of the Lime-Treated Base Course

In the lime-treated base course section partial cores were obtained from all test sites. All of the cores were large enough to perform moisture-density tests and in five of the twelve test sites the cores were such that cubes could be cut for unconfined compression strength tests.

In preparing the five samples for unconfined compressive strength tests one 2- by 2-in. cube was sawed from each partial core. These cubes were then placed, uncovered, in the moist room at a temperature of  $70\text{ F} + 2\text{ F}$  for a period of 7 days. Upon removal from the moist room each cube was immediately tested in unconfined compression at a loading rate of 1,000 lb per sq in. per min. The moisture contents of the cubes were also measured and recorded. When reviewing the results of these tests it should be kept in mind that each value represents one test only. It will be noted in Table 20, that the strengths shown in the 4 and 7 percent subdivisions are considerably higher than that of the 2 percent subdivision. From these tests it might be assumed that the strength developed when 7 percent lime is added is not much greater than the strength developed from 4 percent lime. However, it is probable that the strength tests performed on the field specimens are not indicative of the true strength relationships.

Chemical analysis for lime percentage was performed on each sample. It will be noted in Table 20 that the lime percentage, as tested, is considerably higher than the plan quantity. While no original lime content analyses were performed on the coarse sand, such analyses were performed on the aftonian silt. These tests yielded an average of 0.46 percent lime. Since the lime was placed very carefully it is difficult to explain this discrepancy. It is possible that the coarse sand carried one to two percent lime in its natural state.

The tests of the lime-treated base course show that the moisture content of the material is close to that at time of lay-down. There seems to be a slight decrease in some of the densities since construction.

### Benkelman Beam Deflection Tests

Since December 8, 1956 Benkelman Beam deflection tests have been performed on the surface of the pavement twice each year, for a total of five sets of tests. In all cases the deflection readings are taken 30 ft beyond the permanent test sites in order to stay clear of the locations disturbed by sampling. The results of these tests are shown in Table 27. Figures 12 and 13 show the changes in average deflection with time. Each point is the average of the deflections in two test sites. Charts are included for the outer wheel paths (OWP) and inner wheel paths (IWP).

TABLE  
FLEXIBLE PAVEMENT DEFL.  
LIME-TREATED S

Date of Deflection Test	Subdivision No. 1 Sta 1047 to Sta 1062 0% Lime								Subdivision No. 2 Sta 1062 to Sta 1066 0% Lime							
	Test Site 1a (Fill) Sta 1052+30: Lt.				Test Site 1b (Cut) Sta 1059+30: Rt.				Test Site 2a (Fill) Sta 1066+30: Rt.				Test Site 2b (Cut) Sta 1063+30: Rt.			
	Deflection		Temperature		Deflection		Temperature		Deflection		Temperature		Deflection		Temperature	
	OWP	IWP	AIR	MAT	OWP	IWP	AIR	MAT	OWP	IWP	AIR	MAT	OWP	IWP	AIR	MAT
December 8, 1956	0.008"	0.008"	19 F	19 F	0.006"	0.008"	20 F	20 F	0.006"	0.006"	17 F	19 F	0.006"	0.006"	17 F	19 F
June 14, 1957	0.012"	0.010"	74 F	83 F	0.024"	0.022"	73 F	84 F	0.020"	0.020"	73 F	81 F	0.016"	0.016"	72 F	81 F
November 26, 1957	0.042"	0.020"	62 F	70 F	0.036"	0.018"	62 F	71 F	0.036"	0.016"	63 F	72 F	0.030"	0.030"	72 F	81 F
June 3, 1958	0.072"	0.030"	79 F	93 F	0.042"	0.018"	84 F	101 F	0.038"	0.014"	84 F	98 F	0.040"	0.040"	98 F	101 F
September 22, 1958	0.072"	0.032"	80 F	88 F	0.056"	0.024"	83 F	95 F	0.040"	0.024"	81 F	94 F	0.048"	0.048"	94 F	101 F

LIME-TREATED

Date of Deflection Test	Subdivision No. 5 Sta 1148 to Sta 1161 0% Lime								Subdivision No. 6 Sta 1161 to Sta 1167 0% Lime							
	Test Site 5a (Cut) Sta 1151+30: Lt.				Test Site 5b (Fill) Sta 1159+30: Rt.				Test Site 6a (Fill) Sta 1167+30: Rt.				Test Site 6b (Cut) Sta 1164+30: Rt.			
	Deflection		Temperature		Deflection		Temperature		Deflection		Temperature		Deflection		Temperature	
	OWP	IWP	AIR	MAT	OWP	IWP	AIR	MAT	OWP	IWP	AIR	MAT	OWP	IWP	AIR	MAT
December 8, 1956	0.004"	0.004"	20 F	19 F	0.004"	0.004"	20 F	20 F	0.008"	0.008"	18 F	19 F	0.012"	0.012"	18 F	19 F
June 14, 1957	0.024"	0.008"	76 F	82 F	0.030"	0.014"	75 F	88 F	0.022"	0.008"	76 F	86 F	0.026"	0.026"	86 F	93 F
November 26, 1957	0.036"	0.020"	66 F	78 F	0.028"	0.020"	66 F	78 F	0.028"	0.012"	66 F	78 F	0.024"	0.024"	78 F	86 F
June 3, 1958	0.044"	0.014"	92 F	110 F	0.046"	0.018"	92 F	110 F	0.040"	0.016"	92 F	110 F	0.066"	0.066"	110 F	128 F
September 22, 1958	0.074"	0.018"	85 F	99 F	0.056"	0.028"	85 F	98 F	0.038"	0.024"	82 F	98 F	0.064"	0.064"	98 F	116 F

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ONS BY BENKELMAN BEAM  
ADE SOIL SECTION

Lime		Subdivision No. 3 Sta 1077 to Sta 1092 6% Lime								Subdivision No. 4 Sta 1092 to Sta 1107 10% Lime							
Cut) Lt.		Test Site 3a (Cut) Sta 1084+30: Rt.				Test Site 3b (Fill) Sta 1089+30: Lt.				Test Site 4a (Cut) Sta 1096+30: Lt.				Test Site 4b (Fill) Sta 1106+30: Rt.			
Temperature		Deflection Temperature				Deflection Temperature				Deflection Temperature				Deflection Temperature			
R	MAT	OWP	IWP	AIR	MAT	OWP	IWP	AIR	MAT	OWP	IWP	AIR	MAT	OWP	IWP	AIR	MAT
F	19 F	0.006"	0.006"	18 F	19 F	0.006"	0.004"	18 F	19 F	0.002"	0.002"	20 F	20 F	0.000"	0.000"	18 F	19 F
F	80 F	0.012"	0.010"	76 F	83 F	0.018"	0.016"	76 F	82 F	0.012"	0.010"	78 F	84 F	0.038"	0.010"	77 F	83 F
F	72 F	0.026"	0.018"	64 F	72 F	0.038"	0.014"	65 F	72 F	0.016"	0.016"	67 F	74 F	0.046"	0.018"	67 F	74 F
F	104 F	0.022"	0.016"	85 F	107 F	0.080"	0.018"	92 F	107 F	0.020"	0.014"	92 F	114 F	0.036"	0.018"	92 F	109 F
F	94 F	0.028"	0.018"	84 F	102 F	0.058"	0.024"	84 F	96 F	0.024"	0.014"	84 F	100 F	0.044"	0.024"	85 F	97 F

## COURSE SECTION

Lime		Subdivision No. 7 Sta 1174 to Sta 1187 4% Lime								Subdivision No. 8 Sta 1187 to Sta 1200 7% Lime							
Fill) Lt.		Test Site 7a (Fill) Sta 1178+30: Lt.				Test Site 7b (Fill) Sta 1183+30: Rt.				Test Site 8a (Cut) Sta 1192+30: Lt.				Test Site 8b (Fill) Sta 1199+30: Rt.			
Temperature		Deflection Temperature				Deflection Temperature				Deflection Temperature				Deflection Temperature			
R	MAT	OWP	IWP	AIR	MAT	OWP	IWP	AIR	MAT	OWP	IWP	AIR	MAT	OWP	IWP	AIR	MAT
F	19 F	0.006"	0.006"	18 F	18 F	0.004"	0.006"	18 F	18 F	0.000"	0.000"	18 F	18 F	0.000"	0.000"	18 F	18 F
F	85 F	0.010"	0.010"	77 F	86 F	0.012"	0.008"	77 F	89 F	0.010"	0.002"	77 F	81 F	0.004"	0.002"	77 F	88 F
F	77 F	0.020"	0.018"	64 F	75 F	0.024"	0.012"	62 F	73 F	0.020"	0.012"	62 F	73 F	0.018"	0.014"	60 F	71 F
F	110 F	0.032"	0.024"	92 F	108 F	0.028"	0.010"	92 F	112 F	0.016"	0.006"	91 F	108 F	0.018"	0.010"	91 F	108 F
F	95 F	0.040"	0.028"	82 F	98 F	0.022"	0.012"	82 F	93 F	0.022"	0.008"	80 F	93 F	0.014"	0.012"	79 F	90 F

The vehicle used for the deflection tests is a 1948 International dual wheel, single axle, dump truck. The tires are 12 ply 10.00- by 20-in., inflated to 60 lb. The truck is loaded with scale weights and the axle weight for each testing period was as follows:

Period 1 — December 8, 1956	= 18,100 lb
Period 2 — June 14, 1957	= 18,120 lb
Period 3 — November 26, 1957	= 18,140 lb
Period 4 — June 3, 1958	= 18,150 lb
Period 5 — September 22, 1958	= 17,900 lb

It will be noted that the deflections for both lime-treated sections in the first testing period December 8, 1956, were extremely low. While these deflections were taken during a cold period (19 F average air temperature) neither the ground, base courses, nor subgrade were frozen.

Lime-Treated Subgrade Soil Section. — The deflections in the untreated subdivisions (No. 1) show an almost constant increase during all five testing periods. In the lime-treated subdivisions deflection-time curves closely parallel the curve for the untreated section through the first three periods. However, in the last two testing periods the curves for the lime-treated sections seem to be leveling off below the curve for the untreated section. It will be noted, in Table 27, that the deflection in the outer wheel path of Test Site 4b is considerably higher than the deflection in Test Site 4a. Since this high reading has been included in the averages as shown in Figure 12 it is responsible for keeping the outer wheel path deflection curve much higher than it would be if this reading were more in line with that of Test Site 4a. There is no apparent reason for this high deflection. Since it is possible that Test Site 4b may not be representative a new location (designated Test Site 4c) has been tested during the last two testing periods. While these test results are more in line with those of Test Site 4a they are not included in the average deflections nor are they recorded in Table 27. Deflection readings in Test Site 4c may be included in a later report if this location turns out to be more representative than Test Site 4b.

Attention is also invited to the plot of the inner wheel path deflections. These deflections while showing a general increase are considerably lower than the deflections in the outer wheel path. As in the case of the outer wheel path, deflections for the lime-treated subdivisions were lower than those in the control section, during the last two testing periods.

Lime-Treated Base Course Section. — As in the lime-treated subgrade experiment, deflections at all test sites have been increasing with time. In the outer wheel path, the deflections in the untreated section and the 2 percent lime section have been about equal. The rate of increase of deflection with time has been lower for the 4 and 7 percent lime-treated sections and the curves for these sections seem to be leveling off.

Deflection readings for the inner wheel path are smaller than those for the outer wheel path and seem to have a slightly different pattern. It will be noted on this figure that the curves for the standard section, the 2 and 4 percent lime sections are similar and approximately equal. The deflections observed in the 7 percent lime section are definitely smaller than the others.

## CONCLUSIONS

Since this project has been open to traffic for only two years and since the condition of the pavement throughout the entire project is superior, with no cracking or failure, no definite conclusions are warranted at this time. It can be said that to date, the pavements in the lime-treated sections have performed as well or better than the pavements in the standard control sections.

It is planned to continue with the deflection testing program for several years. Additional tests will be made from time to time on samples from the various courses and the subgrade as seem desirable.

When sufficient data become available to justify more extensive conclusions, another report will be submitted.

## REFERENCES

1. Gallaway, B.M., and Buchanan, S.J., "Lime Stabilization of Clay Soils," Texas Eng. Exp. Sta., Bull. 124 (1951).
2. \_\_\_\_\_, "Lime Stabilization of Roads," Natl. Lime Assoc., Washington, D.C. (1954).
3. McDowell, C., "Road and Laboratory Experiments with Soil-Lime Stabilization," Proc., Natl. Lime Assoc., Washington, D.C. (1953).
4. McDowell, C., "The Use of Hydrated Lime for Stabilizing Roadway Materials," Proc., Natl. Lime Assoc., Washington, D.C. (1948).
5. Zube, E., "Experimental Use of Lime for Treatment of Highway Base Course," Tech. Bull. 181, Road Builders Assoc., Washington, D.C. (1952).
6. Goecker, W.L., Moh, Z.C., Davidson, D.T. and Cheu, T.Y., "Stabilization of Fine and Coarse-Grained Soils with Lime-Flyash Admixtures," HRB Bull. 129 (1956).
7. Hoover, J.M., Handy, R.L., and Davidson, D.T., "Durability of Soil-Lime-Flyash Mixes Compacted Above Standard Proctor Density," HRB Bull. 193 (1958).
8. Miller, R.H., and McNichol, W.J., "Structural Properties of Lime-Flyash-Aggregate Compositions," HRB Bull. 193 (1958).
9. Davidson, D.T., Sheeler, J.B., and Delbridge, N.G., "Reactivity of Four Types of Flyash with Lime," HRB Bull. 193 (1958).
10. Jones, W.G., "Lime-Stabilized Test Sections on Route 51, Perry County, Missouri," HRB Bull. 193 (1958).
11. Jones, C.W., "Stabilization of Expansive Clay with Hydrated Lime and with Portland Cement," HRB Bull. 193 (1958).

*Appendix A*

**OUTLINE OF PROCEDURES FOR PERFORMING TESTS  
DESIGNATED NEBRASKA PROCEDURE**

Cementation Test: Nebraska Procedure

1. Scope — The cementation test is intended to show the relative cohesion of the soil in a consolidated dry state.
2. Apparatus —
  - a. A balance
  - b. Mixing pans
  - c. Flat-bottom pie pans
  - d. Spatula
  - e. Calipers
  - f. Press capable of exerting a load of 132 Kg/sq cm on a 25 mm diameter briquette
  - g. Forming mold having an inside diameter of 25 mm
  - h. Cementation test machine
3. Preparation of Soil Briquettes — A portion of the material passing the No. 40 sieve, shall be mixed with sufficient water to form a plastic mixture. A weighed portion of this mixture shall be placed in a metal die, 25 mm inside diameter, and subjected to a pressure of 132 Kg/sq cm for an instant in a press. The cylindrical briquette resulting shall be exactly 25 mm in height. If it does not measure this height, subsequent samples of the plastic mixture shall be taken in such quantity that the resulting briquette after compression will be exactly 25 mm in height. Three briquettes shall be made and allowed to air-dry for approximately 20 hr after which they shall be oven-dried for 4 hr in an oven at a temperature of 220 F and then cooled in a desiccator for at least 20 min.

4. **Cementation Test** — The three oven-dried briquettes shall be tested in a machine as follows: The machine shall be arranged so that a 1-Kg hammer is raised to a height of 1 cm and allowed to fall freely on a plunger which transmits the shock of the blows of the hammer to the briquette, successive blows being struck by the hammer at a rate of 40 to 70 per min, until the briquette fails, which is indicated by the failure of the plunger or hammer to rebound. The briquette shall be placed on the anvil under the plunger without lateral support, and may be fastened in place on the anvil by a drop of shellac. The average number of blows required to produce failure is the result to be reported, and is the coefficient of cementation.

#### Capillarity and Absorption Failure: Nebraska Procedure

1. **Scope** — The Capillarity Test is performed on soils to show the resistance possessed by the soil to capillarity rise of water. The Absorption Failure Test is performed on soils to show the resistance to slaking. The results of both tests are reported in hours, minutes, and seconds.
2. **Apparatus** — Same apparatus as required in the Cementation Test.
3. **Preparation of Soil Briquettes** — The soil briquettes for these tests are prepared in the same manner as the briquettes for the Cementation Test. Three briquettes will be required to perform these tests.
4. **Capillarity and Absorption Failure Tests** — The three oven-dried briquettes shall be placed in a flat-bottom pan, containing water  $\frac{1}{2}$  in. in depth, so that each briquette is not closer than 2 in. from the sides of the pan or to another briquette. The time required for the water to rise and wet the top of the briquette is recorded as the capillarity time. The time required for the briquette to fall apart or slack down is recorded as the absorption failure. If the briquette falls apart or slacks down before its entire top is wet with capillarity water, the capillarity time is recorded as being longer than the absorption failure time. The time reported shall be the average of the three briquettes.

#### Determination of Lime-Content of Soil-Lime Mixtures: Nebraska Procedure

Dry and grind the entire portion of the soil-lime sample to pass a 40-mesh sieve. Take a 25- to 35-gram, representative portion of the ground material by quartering. Place the representative sample in a weighing bottle and dry to constant weight in a 105 C oven. (Usually an overnight drying period is sufficient).

Weigh out, on an analytical balance, 5 grams of the oven-dried representative sample. Place in a 250-ml beaker, add 50 ml of HCl (1:1), cover, and boil gently for five minutes on a hot plate.

Add 25 ml of hot water to the beaker, stir, allow to settle momentarily, and then decant through a 11-cm Whatman No. 1 (or No. 41) filter paper. The filtrate should be received in a 250-ml volumetric flask. When the liquid has passed through the filter paper, wash the residue once by decantation, using hot water; then transfer the residue to the filter, using a stream of hot water. The beaker should be rapidly polished, the loosened material being transferred to the filter paper. The material on the filter should be washed an additional four times, each washing consisting of 10 to 15 ml of hot water directed in a stream from the wash bottle. Very small amounts of residue will occasionally pass through the filter. These ordinarily may be disregarded. (Excessive boiling, resulting in gelation of the silica, may cause slow filtration. A longer settling time before filtration may help speed the filtering process.)

When washing has been completed, discard the filter, and dilute the filtrate in the volumetric flask to 250 ml with cold water. The temperature of the solution should be near the calibration temperature of the flask. Agitate the flask to thoroughly mix the contents, then pipette a 50-ml aliquot into the original 250-ml beaker. Dilute to

100 ml. Make the solution slightly ammoniacal, boil 1 to 2 min, and allow the hydroxides to settle. (If the sample contains ferrous iron, it is desirable to add a few drops of nitric acid, before precipitation of the hydroxides.)

Filter the hydroxides through a 11-cm Whatman No. 1 (or No. 41) filter paper, receiving the filtrate in a 600-ml beaker. Wash the original 250-ml beaker into the filter once with a stream of hot ammonium nitrate (2 percent) solution and follow by washing the hydroxide precipitate once or twice with hot ammonium nitrate (2 percent) solution. Set the filtrate aside, and place the original 250-ml beaker under the funnel. Puncture the paper with a glass rod, and wash the hydroxides into the beaker, using a stream of hot ammonium nitrate (2 percent) solution to remove most of the precipitate from the filter paper. Treat the paper with 20-ml of hot HCl (1:3), directing the acid over the paper with a glass rod or the lip of a graduated cylinder. Wash the paper several times with hot water, and then discard the paper. Dilute the solution to 75 ml. Make the solution slightly ammoniacal, and boil 1 to 2 min. Allow the precipitate to settle, then decant through a No. 1 (or No. 41) paper as before, receiving the filtrate in the 600-ml beaker previously set aside. Wash and police the beaker, in which precipitation took place, finally washing the precipitate on the filter 3 or 4 times with ammonium nitrate (2 percent) solution. Discard the paper and precipitate.

Add 2 ml of ammonium hydroxide to the filtrate, which will now have a volume of 250 to 350 ml. Heat the solution to boiling and add 10- to 15-ml of hot, saturated ammonium oxalate solution. Boil gently until the precipitate becomes granular, then set aside on the water bath for 30 min or more. Before filtering off the calcium oxalate, verify completeness of precipitation, and make sure that a slight excess of  $\text{NH}_4 \text{OH}$  is present.

Filter the mixture through a 11-cm Whatman No. 42 filter paper making sure that all precipitate is being retained. Thoroughly clean the beaker, in which precipitation took place with a rubber policeman and transfer the contents to the filter paper with a stream of hot water. Wash the filter paper 8 to 10 times with hot water (not over 75 ml) using a stream from the wash bottle.

Transfer the filter paper and contents to a 400-ml beaker containing 125 ml of water and 6 ml of  $\text{H}_2\text{SO}_4$ . Heat the solution to 85 C and titrate with standard 0.1 N potassium permanganate.

Make a blank determination, following the same procedure and using the same amounts of all reagents.

Calculate the lime content of the soil-lime mixture as follows:

$$\% \text{Ca(OH)}_2 = \frac{(A-B) E \times 1.3213}{S} \times 100$$

In which

A =  $\text{KMnO}_4$  solution required for titration of the sample, ml;

B =  $\text{KMnO}_4$  solution required for titration of the blank, ml;

E = CaO equivalent of the  $\text{KMnO}_4$  solution, g per ml =  $0.028 \times \text{KMnO}_4$  normality;

S = weight of sample in aliquot titrated, g; and

1.3213 = gravimetric factor for converting CaO to  $\text{Ca(OH)}_2$ .



# Stabilization of Soils with Lime, Lime-Flyash, and Other Lime Reactive Materials

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The purpose of this report is to discuss desirable testing and construction techniques to be used for construction of successful lime stabilization projects. The report discusses the history of the process, the composition and characteristics of the limes used, and the types of lime stabilization generally used. A brief discussion of some of the chemical reactions explains why and how lime changes physical characteristics of soils. A portion of the report pertains to testing of mixtures and to interpretation of test results.

Discussion of construction practice covers such subjects as application of lime, mixing, compacting, curing, and reworking. Recommendations with regard to these practices and their relationship to cracking and permanence are made. A number of tests to be made subsequent to construction are suggested and the benefits of lime as a stabilizer and of properly constructed lime stabilized projects are summarized.

● **LIME STABILIZATION**, as applied to present day highway construction, is an outgrowth of ancient practices which have been modified by laboratory and field tests to fulfill a variety of stabilization requirements. Since all varieties of this type of stabilization are not entirely satisfactory for all job requirements, and since different construction techniques are necessary for use with various job requirements, a discussion of this subject is indicated. It is the purpose of this report to clarify some of these points by setting forth some of the best concepts and practices known to date. It is believed that this report will provide both background and specific information to those who, for the first time, are employing lime stabilization by presenting some of the history of the process and by explaining why and how soil-lime mixtures have stabilizing characteristics. Since this type of stabilization is growing rapidly, and the present knowledge of the subject is constantly being enlarged upon in both the field and the laboratory, one may expect to elaborate upon this report considerably in the future.

## HISTORY

Broadly speaking, the use of lime stabilization preceded the beginning of clearly recorded history. Perhaps its use in constructing the pyramids of Shensi in the Tibetan-Mongolian Plateau is one of the oldest. These pyramids are much larger and older than the Egyptian pyramids and were constructed of compacted mixtures of clay and lime. They were considered so very old that little was known about them when their first history was recorded over 5,000 years ago.

China and India also have used lime stabilization in various ways throughout their long history. Chinese engineers have used lime-stabilized clay-gravel for massive bridge footings and underground chambers. Engineers from India tell of using lime-clay-sand mortars in construction of tall masonry dams. In their country, this treatment is noted for the prevention of weeping of dams. The use of lime in subbases of the Roman roads dates back more than 2,000 years. The practice had probably existed in this area long before construction of the Roman roads.

Many of these and other countries have continued such practices right up to the

present time. One very noticeable point is that in many instances the practice was and still is a family art handed down from father to son. For this reason there were practically no written procedures or specifications available for modern use; therefore, considerable experience in testing and constructing actual experimental projects is required to adapt techniques to modern day road building. These techniques covered such items as desirable methods for mixing, compacting and curing.

The Texas Highway Department has utilized some of the art known by ancient races in development of lime stabilization for use in present practices of road building. Like many other organizations they have carried out laboratory tests on soil-lime mixtures for the last 25 years, but the first 13 years were spent without observable success. The development of laboratory compaction and triaxial compression methods of testing made it possible to better evaluate such mixtures as early as 1945. In highway work, laboratory experiments without field experiments are usually inconclusive and one without the other can often be misleading. Prior to 1945, field experiments were conducted in a number of states (including the midwest) without consideration of control of mixing, compacting and curing. Most of these jobs were used as open surfaced roads and resulting performances were disappointing. One such job in Texas probably delayed development of lime stabilization in that state for ten years. These same materials that failed on one job in this state, when used on another nearby job, have given splendid performance under medium heavy traffic for a period of 14 years to date. Several hundred miles of this granular type of lime stabilization are giving good service. Treatment of clays with lime started about seven years ago and has increased in demand so rapidly that several hundred miles of this type of stabilized subbase now can be found on this state's highway system. The use of lime for stabilization purposes in this one state alone has reached an average of 9 to 10 thousand tons monthly and is increasing. The percentages of lime used have varied from 1 to 8, based on dry weight of soil, and costs in the southern states exclusive of surfacing have varied from 22 to 50 cents per sq yd for a 6-in. depth. Many other states and countries are building lime stabilization projects which should contribute greatly to the knowledge on this subject.

#### TYPES OF LIMES USED

Most of the lime used for road stabilization to date has been hydrated lime (calcium and/or calcium-magnesium hydroxide) although some quicklime (calcium and/or calcium-magnesium oxide) and waste lime have been used with success. Both high calcium and dolomitic or magnesium limes have been used successfully. Hydrated limes have been applied both in the powder and slurry forms. So far as is known, quicklime has been applied only in the powder form and has been known to burn some workman who were not properly protected. In order to overcome this difficulty, quicklime probably could be slurried sufficiently to avoid severe burns, but for best mixing the lime particles after slurrying should be as finely divided as hydrated lime. Waste lime may contain both quicklime, hydrated lime and impurities, and its use has been limited to small experimental jobs. It is difficult to plan on the use of such limited and erratic sources for stabilization on a scale of any considerable magnitude.

The chemical constituents of lime are quite definite and there should be no doubt as to what lime is; however, the words "lime" and "limestone" have been so loosely used that many people, including engineers, mistake limestone dust or screenings for lime. Limestone is calcium-carbonate and not calcium-oxide or lime; it cannot become a lime except by having its carbon dioxide removed by heating. Some metamorphic limestones may have a trace of free lime available in them due to their past geological history, but their lime content cannot be expected to produce stabilization results which are comparable to those obtained when commercial lime is used.

Some control of the purity of lime to be purchased should be exercised in order to avoid buying inert materials. Lime for soil stabilization has generally been purchased under ASTM Designations C-6-49 or 207-49 with plasticity requirements deleted. The ASTM tests for this purpose, if strictly interpreted, have been found by some to be too complicated and too slow for control of large quantities expected to be transported on short notice. One state has its own test procedure and specification for control of lime quality. When lime stabilization is used frequently in a given state or area, it may

become desirable for control samples to be taken at the lime plants in lieu of samples from individual truck or rail shipments. This can avoid the embarrassing situation of having sub-standard lime on or in the job and wondering what to do about it.

### TYPES OF LIME STABILIZATION

Generally, there are two main types of lime stabilization which can be accomplished out of a variety of combinations. One consists of treating granular or aggregate bearing materials for final course of base upon which a wearing course or surfacing is to be placed. Such mixtures usually contain less than 50 percent soil binder and are treated with from 2 to 4 percent lime based on dry weight of soil aggregate material. The other consists of treating soil materials to serve as subbase. Usually, clay soils are treated for this purpose by the addition of from 4 to 8 percent lime based on dry weight of soil. Due to the effects of weathering conditions, clay-lime mixtures which are adequate for subbase purposes probably will not prove to be satisfactory if used as final course of base upon which a thin surfacing is to be placed. Many types of soils react favorably to lime stabilization regardless of whether they are acid or alkaline.

The treatment of granular soils for final course of base to be surfaced, which do not react well with lime alone, can be enhanced by the addition of flyash, expanded shale fines, volcanic ash, portland cement and bituminous materials. A class of high type pavements probably could be opened up by pursuing the use of aggregates treated with lime and flyash or expanded shale (burned clay) or some volcanic ashes. This type of stabilization could probably be pursued further by using dolomitic or magnesium lime in conjunction with calcium chloride or gypsum to form oxychloride or oxysulphate cementation. If adequate methods of injecting carbon dioxide after completion of compaction could be developed, a different type of hardening and increase in strength could be accomplished.

### CHEMICAL REACTIONS

There are many chemical reactions which take place in soil-lime mixtures and the following discussion probably covers only a few of them. The first reaction which takes place is of a base exchange nature which lowers the plasticity index of the soil and gives the soil a loose friable appearance. Various small amounts of mixing, depending on the nature of the soil, are required to obtain this consistency. This texture is similar to good tilth in farm land and will remain friable unless compacted. If compacted, however, another reaction is believed to be effected which consists of calcium reacting with available silica and alumina (colloidal sizes) to form complicated compounds of non-slaking monocalcium silicates and aluminates. This is a form of slow setting cement which continues to gain strength over long periods of time and is commonly referred to as a pozzolanic reaction. The rate of this cementation varies a great deal with the type of soil being treated and with climatic conditions. The third reaction is the slowest and involves the absorption of carbon dioxide from the air so as to react with the calcium hydroxide to form calcium carbonate or limestone. For either of these last two reactions to produce effective hardening by cementation, the mixture must be thoroughly compacted prior to time of reactions.

### TESTING OF SOIL-LIME MIXTURES

In running tests on such mixtures, it is important to always use a fresh supply of good lime. The soil constants of highly plastic, clay material to be treated with lime should be run as an indication of the degree of mixing required. If lime will not lower the PI of a heavy clay effectively, mixing difficulties may be anticipated. If lime hardens a low PI material, it is not necessary to require a decrease in PI.

The strengths of soil-lime mixtures are usually evaluated by such tests as the CBR, unconfined compression, resistance value, triaxial, etc. In attempting to evaluate the test results, it should be kept in mind that the mixes which are hardest at an early age are not necessarily the best, because some of the differences in strength may be due only to the rate of setting having been speeded up. It should also be kept in mind that

if a mixture has sufficient hardness, strength or toughness to withstand the climates and extensive repetitions of the stresses imposed, it may serve just as well as a much stronger layer. There is a need for fast-setting mixtures where the lack of detours makes it necessary to carry heavy traffic over existing roads. There is danger that such mixtures under heavy traffic may be susceptible to cracking. Regardless of the type of strength test used, it is common practice to plot strength versus percent lime, as shown in Figure 1.

In this case, plotting test results by the aforementioned methods would seem to indicate an optimum lime content of 5, 7 and 9 percent for 7, 21 and 60 day old specimens, respectively. The strength values for the 180 day old specimens show that there really is no optimum lime content to be gained from such an analysis. Strength tests are necessary to see if a proposed mixture will harden sufficiently but experience and good judgment are also essential tools for selecting percentage of lime to be used. There is also a noticeable discrepancy between road and laboratory strengths of clay-lime mixtures. Laboratory strengths may indicate a Class 1 mix but the same clay-lime mix may not obtain such strengths in the road due to delayed mixing, compacting and weathering. Cores cut from old roads consisting of old, lime-granular soil mixtures show that field strengths are higher than are found from laboratory tests on specimens up to 60 days old. In selecting the percentage of lime for use, it should be kept in mind that some lime will be lost in construction operations by blowing, carbonation, etc., so it is advisable to use a little more lime than short-time tests indicate to be sufficient. It will be very advantageous to have some extra lime in mixtures which have to be reworked. Very low percentages of lime are practical to use for temporary effects on haul roads and detours. Of any combination of materials selected for use as base course, Texas test method THD-82 suggests a minimum unconfined compressive strength of 100 psi after 7 days moist curing and 10 days soaking or capillary wetting at room temperature. Mixtures that set slower than this may be difficult to handle, particularly during cold or adverse weather conditions. Laboratory personnel often observe a fast rate of wetting in moist-cured, partially dried specimens. This should not be mistaken as an indication of a high rate of permeability. Capillarity and permeability are two different things. Compacted lime mixtures are particularly effective for repelling water percolation.

### CONSTRUCTION METHODS

The quality and behavior of soil-lime stabilization mixtures are highly dependent

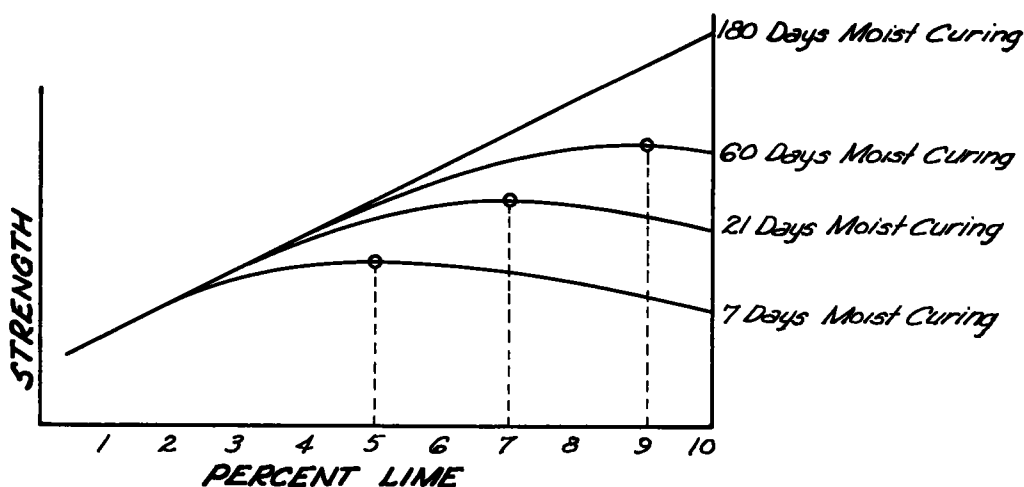


Figure 1. Effect of lime content on strength of soil-lime mixtures.

upon the technique used during construction. If the material to be treated is in place, some scarification and pulverization, prior to addition of lime, are indicated. Rooters, disc harrows and rotary speed pulverizers are very useful tools for these types of operation.

### Application of Lime

As stated before, lime may be added in either the powder or slurry form. The choice of which to use often depends upon the moisture content of the soil to be treated and location of the project with respect to creating a dust hazard to homes and business establishments. Powdered lime may be added by proper spacing of bags or by mechanical spreaders, or with transport trucks equipped with auger unloaders and flexible downspouts. Hydrated lime slurries have been made by blending water to lime with pugmills, compressed air, and circulating pumps as stirring devices. Blending may be effected during or prior to loading truck tanks. The cost of satisfactory plants of this type have ranged from 3 to 7 thousand dollars which becomes a small item if used on several jobs or if parts are converted to other uses. It has been found that lime slurry can be handled through ordinary water tank truck spray bars when 2 or 3 lbs or more of water per lb of hydrated lime are properly blended. Occasionally, truck tanks not equipped with circulating devices will accumulate a layer of lime sediment in the bottom of such tanks. Compressed air is usually effective in stirring up these sediments so the tanks can be flushed out.

### Mixing

In the case of sandy or aggregate bearing soils, lime is usually added to the surface and mixed with motor patrol graders or rotary speed pulverizers. In case lime slurry is being added, thin layers are bladed aside as each increment of slurry is added by sprinkling until windrows are formed. After windrows are bladed back across the road, the mix is usually ready for compaction if its moisture content is near optimum. If the moisture content is below optimum moisture when spread, additional water will need to be added by sprinkling and blade mixing. If the mixture is too wet to roll, further aeration is indicated. This can be accomplished by additional blading or by use of rotary speed pulverizers fixed so as to throw the material into the air. In the case of mixing dry lime with clay soils it is often necessary to blade part of the soil to be treated into windrows and form a sandwich by adding the lime, sprinkling, and covering with soil from the windrows. This procedure protects the lime from the elements and at the same time does not delay application of lime. A delay of mixing for two days when necessary will help to mellow extremely lumpy clays and decrease difficulty of mixing. The remainder of mixing and adding of water may be carried out by sprinkling, and mixing with blade and/or rotary pulverizers over a period of time not to exceed four days.

Although the pugmill type road mixer has not been used frequently in lime stabilization, it is possible that this type of mixer will help to eliminate unequal transverse distribution of lime. Clay-lime mixtures should be pulverized to pass the 1-in. screen and at least 60 percent to pass the  $\frac{1}{4}$ -in. screen (No. 4 sieve) prior to compaction.

### Compaction

Delay of compaction of thin windrows of all types of soil-lime mixtures which dry and become carbonated probably will decrease hardening of the mix subsequent to compaction. For best hardening results, compaction to high density at the proper time is essential for all lime mixtures. Naturally, close control of moisture content will be of great aid to density control. Experience has shown that a few rules for compaction of soil-lime mixtures have been helpful when followed. All soil-lime mixtures, except lime-treated heavy clays, may be compacted promptly after mixing or at any time within two days. Ordinarily, such soil aggregate bearing lime mixtures should not be recompacted if previous rolling is over four days old without adding new water, lime and/or soil. There are two exceptions to this rule: one is when the mixture contains a high percentage of lime and the other is in the case of stabilizing heavy plastic clays

treated with high percentages. In the latter case some delay in compaction after mixing can be advantageous but this delay should not be in excess of four days. Mixtures may be compacted with various types of pneumatic, tamping or vibratory rollers in various layer thicknesses. The layer thickness and weight of roller will usually depend upon the type of soil being stabilized and the supporting power of the various sub-layers. Reworking of clay-lime mixtures which have been compacted for more than 7 days should be avoided unless additional lime and/or soil is incorporated.

### Curing

Proper curing is an essential step in obtaining hardening effects. The lime stabilized base can be cured satisfactorily by sprinkling for 7 days or by sealing with a bituminous treatment within one day after final rolling. Since lime stabilized soils absorb very small amounts of asphalt, the usual quantities of prime used for untreated bases probably will be excessive. Generally, the application of heavy loads should be avoided for a period of 7 days after completion of rolling. If it is necessary to haul over a stabilized layer during this period, least damage will occur if loads are applied immediately after compaction. Treated subbase layers may be sprinkled for periods of time exceeding 7 days or until covered with subsequent material. Light rolling is usually applied during this period to keep the surface knitted together.

### Cracking

Any good, well compacted semi-flexible or cemented mixture is generally more susceptible to cracking than are good flexible bases. This defect becomes increasingly noticeable in the surfacing with time. Much can be done during design and construction to minimize such defects. Causes for cracking may be due to any one or more of the following reasons: (a) swelling of clay subgrade; (b) shrinkage of clay subgrade especially when inadequate shoulders are used; (c) use of inadequate design thicknesses; (d) application of heavy loads from hauling or rolling during curing period; and (e) natural shrinkage of the mixture. The first three causes listed above are common to all types of pavements and every reasonable effort should be made to minimize high volume change characteristics and use of base depths which are too thin. The last two causes listed above become more readily noticeable than the others and are often referred to as shrinkage cracks. Prior to application of asphaltic concrete, the use of a single asphalt surface treatment instead of a prime seems to reduce the seriousness of this problem, perhaps because penetration asphalt does not run down crevices in the base and prevent rehealing as much as a prime or asphaltic emulsion will. Surface treatments also prevent raveling and pot holing prior to paving. This so-called shrinkage type of cracking has occurred on a few lime stabilized jobs. Where it was most severe the base consisted of low PI (8 or less) materials. Higher PI materials when treated with lime appear to be less troublesome in this respect. It is theorized that aggregate bearing mixtures consisting of high PI materials and lime are less fragile and can support loads with less cracking during curing than can low PI or more fragile mixtures. Lime mixtures for base course which are susceptible to cracking probably should be "tight bladed" and sealed promptly after compacting. If such mixtures have already cracked extensively, they probably should be reworked using lime water. If a base is properly designed and constructed, transverse cracks may occur at fairly large distance intervals and they should be so minute that they are difficult to observe.

### Permanence

The history of ancient lime mortars and pozzolanic mixes, such as were used in construction of Roman roads, indicates that some lime mixtures have a high degree of permanence. Like many other mixtures the permanence of such mixtures depends upon good design and construction practices. Design includes the careful selection of good mixtures and adequate thicknesses to withstand repetitional stresses imposed upon or below their surfaces. Proper uses of triaxial, CBR, and resistance value test methods should be very useful in selecting thicknesses of layers to be used. The percentage of

lime used may also be found to influence the life of such roads. Many jobs in south central states up to 14 years old are still in excellent condition. A few jobs in New Mexico and California have withstood severe climatic conditions for several years without damage from freezing and thawing. Final conclusions with respect to this matter will depend upon future experience. It should be emphasized that extensive vertical movements due to swelling or heaving of deep layers of soils are not overcome by lime-treatment or any other treatment applied in thin layers.

#### TESTS TO BE MADE ON COMPLETED ROAD

There are a number of tests which will reveal interesting data on roads as they age under traffic, several are listed as follows:

1. Deflection tests by use of Benkelman beam or plate loading.
2. In place CBR.
3. Compression tests of undisturbed cores.
4. Flexural tests on sawed beams.

#### BENEFITS OF LIME STABILIZATION

There are several characteristics of lime that make it a good stabilizer:

1. It is easy to mix with soil.
2. It quickly reduces the plastic properties of soil when wetted.
3. It sets slowly, the time interval between mixing and compacting is not critical, especially if the mixture is not allowed to remain spread in a thin windrow for long periods of time.
4. Compaction can be done over a period of two or three days, bases need not be rolled all at once allowing time for base to adjust to subgrade.
5. Costs are reasonably low.

There are also a number of benefits being derived from clay-lime subbase treatments. Some are as follows:

1. The lime-treated subbases form a working table upon which contractors can continue construction of pavements shortly after rains.
2. Wetting operations for such treatments transfer enough water into subgrade soils to cause their subgrade to lose some of its swelling characteristics.
3. The treated layers form a water barrier to prevent excessive shrinkage cracking of subsoils due to drying or infiltration of water during rainy weather.
4. Last, but not least, such lime-treated layers form a subsection of the total depth of pavement.

# Activation of the Lime-Flyash Reaction

## By Trace Chemicals

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The results of an investigation of the effect of small amounts of forty-seven different chemicals on the strength of Ottawa sand-lime-flyash mixtures are presented. An explanation is offered for the mechanism of strength improvement by the different groups of chemicals studied.

● ROAD BASES stabilized with lime and flyash may not gain sufficient strength in 7 or 28 days to satisfactorily carry heavy traffic or withstand repeated freeze-thaw cycles. This prolongs the period that the road must be closed to traffic or reduces the construction season in northern climates. Thus, an economical method of speeding up the rate-of-strength gain of lime-flyash stabilized soil would extend its usefulness in road construction.

Heat is known to be a good activator for the lime-flyash (pozzolanic) reaction; 7-day strengths of over 1,000 psi may result from curing compacted specimens of soil-lime-flyash at 140 F in sealed containers. But since high temperature curing of road bases is not economical with presently available fuels, a more practical alternative for activation of the lime-flyash reaction would be with trace chemical additives.

This paper presents the findings of a search for trace chemical activators.

### MATERIALS

#### Soil

Natural monomineralic silica sand from Ottawa, Ill., was used as the soil component of mixtures to eliminate variables due to the complex mineral composition of natural soil. The gradation of the sand met the requirements (ASTM Designation: C 109-56) for graded standard sand:

SIEVE SIZE	PERCENT PASSING
No. 16 (1190-micron)	100
No. 30 (590-micron)	98 + 2
No. 50 (297-micron)	28 + 5
No. 100 (149-micron)	2 + 2

#### Flyash

The flyash was from the Detroit Edison Company St. Clair Power Plant, St. Clair, Michigan. The composition and physical properties of the sample used are:

Silicon dioxide (SiO <sub>2</sub> ), percent	41.9
Magnesium oxide (MgO), percent	1.0
Calcium oxide (CaO), percent	2.7
Aluminum oxide (Al <sub>2</sub> O <sub>3</sub> ), percent	22.5
Iron oxide (Fe <sub>2</sub> O <sub>3</sub> ), percent	25.8
Sulphur trioxide (SO <sub>3</sub> ), percent	0.8

<sup>1</sup> Presently Assistant Professor of Civil Engineering, Indian Institute of Technology, Bombay, India.



Available alkalis (as Na <sub>2</sub> O), percent	0.3
Loss on ignition, percent	3.6
Moisture content, percent	0.2
Specific gravity	2.61
Fineness (percent passing No. 325 sieve)	88.7
Specific surface (sq cm/g)	2720

### Lime

The lime was calcium hydroxide, reagent grade, from the Allied Chemical and Dye Corporation.

### Chemicals

Forty-seven chemicals were evaluated. The selection included chemicals known or suspected to improve the pozzolanic reaction as well as chemicals whose effect on the reaction was unknown (1, 2, 3, 4, 5, 6). The chemicals were reagent, technical or purified grade. They are grouped in Table 1, primarily on the basis of their reactions—basic, neutral or acid. Bases and basic salts, also known as alkalis and alkaline salts, produce hydroxyl ions in water solution to varying extents. Acid salts produce hydrogen ions in water solution to varying extents. Neutral salts in water solution do not upset the natural balance of hydrogen and hydroxyl ions. Unclassified chemicals are in the miscellaneous group.

## METHODS

### Mix Proportions

Lime-flyash mortars were composed of 75 percent Ottawa sand and 25 percent lime and flyash, with the ratio of lime to flyash either 1:9 or 1:4. (75:2.5:22.5 or 75:5:20 Ottawa sand-lime-flyash.) The trace chemical additive was 0.5 and/or 1 percent, based on the dry weight of the mortar. Chemicals were added to the dry mortar as a powder or as a component of the mix water which was close to standard Proctor optimum moisture (ASTM Designation: D698).

### Mixing and Molding

Mixing of batches for preparing test specimens was done in a Hobart kitchen mixer, model C-100, at low speed. The dry ingredients were mixed 25 sec; then the mix water was added and mixing continued for 4 minutes.

Molding was started immediately after a batch was mixed. A double plunger drop-hammer apparatus was used to mold 2 in. diameter by 2 in. high specimens to approximate standard Proctor density.

### Curing

Specimens of each batch were moist cured at near 70 F and 100 percent relative humidity for 7 days, 28 days and 4 months. To better preserve moisture and reduce entry of carbon dioxide from the air, all specimens were wrapped in wax paper sealed with cellophane tape.

### Strength Testing

After each curing period, specimens were unwrapped and immersed in distilled water for one day. Then they were tested for unconfined compressive strength using a load travel rate of 0.10 in. per minute. Tests were run in triplicate and the average strengths are reported.

### Free-Thaw Testing

Twenty-eight day cured 2- x 2-in. specimens of selected mixtures were subjected to cycles of freezing and thawing. Specimens on water saturated felt pads were frozen

TABLE 1  
CHEMICALS EVALUATED AS ADDITIVES

Group and Chemical	Formula	Manufacturer
<b>BASES</b>		
Potassium hydroxide	KOH	J. T. Baker Chem. Co.
Sodium hydroxide	NaOH	" " " " "
<b>BASIC SALTS</b>		
Lithium carbonate	Li <sub>2</sub> CO <sub>3</sub>	Fisher Scientific Co.
Potassium bicarbonate	KHCO <sub>3</sub>	Mallinckrodt Chem. Works
Potassium carbonate	K <sub>2</sub> CO <sub>3</sub>	J. T. Baker Chem. Co.
Sodium bicarbonate	NaHCO <sub>3</sub>	" " " " "
Sodium borate	Na <sub>2</sub> B <sub>4</sub> O <sub>7</sub> · 10H <sub>2</sub> O	Fisher Scientific Co.
Sodium carbonate	Na <sub>2</sub> CO <sub>3</sub>	J. T. Baker Chem. Co.
Sodium sulfite	Na <sub>2</sub> SO <sub>3</sub> · 7H <sub>2</sub> O	" " " " "
<b>NEUTRAL SALTS</b>		
Lithium chloride	LiCl	J. T. Baker Chem. Co.
Lithium fluoride	LiF	Fisher Scientific Co.
Lithium nitrate	LiNO <sub>3</sub>	" " "
Lithium sulfate	Li <sub>2</sub> SO <sub>4</sub> · H <sub>2</sub> O	" " "
Potassium chlorate <sup>a</sup>	KClO <sub>3</sub>	J. T. Baker Chem. Co.
Potassium chloride	KCl	" " " " "
Potassium dichromate <sup>a</sup>	K <sub>2</sub> Cr <sub>2</sub> O <sub>7</sub>	Fisher Scientific Co.
Potassium permanganate <sup>a</sup>	KMnO <sub>4</sub>	J. T. Baker Chem. Co.
Sodium chloride	NaCl	" " " " "
Sodium dichromate <sup>a</sup>	Na <sub>2</sub> Cr <sub>2</sub> · 3H <sub>2</sub> O	Fisher Scientific Co.
Sodium hypochlorite <sup>a</sup>	NaOCl <sup>b</sup>	" " "
Sodium nitrate	NaNO <sub>3</sub>	J. T. Baker Chem. Co.
Sodium pemanganate <sup>a</sup>	NaMnO <sub>4</sub> · 3H <sub>2</sub> O	Fisher Scientific Co.
Sodium sulfate	Na <sub>2</sub> SO <sub>4</sub>	J. T. Baker Chem. Co.
<b>ACID SALTS</b>		
Aluminum chloride	AlCl <sub>3</sub> · 6H <sub>2</sub> O	J. T. Baker Chem. Co.
Ammonium chloride	NH <sub>4</sub> Cl	Allied Chem. & Dye Corp.
Calcium chloride	CaCl <sub>2</sub>	J. T. Baker Chem. Co.
Calcium hypochlorite <sup>a</sup>	Ca(OCl) <sub>2</sub>	Fisher Scientific Co.
Calcium sulfate	CaSO <sub>4</sub> · 2H <sub>2</sub> O	J. T. Fisher Chem. Co.
Chromic chloride	CrCl <sub>3</sub> · yH <sub>2</sub> O	Fisher Scientific Co.
Cobaltous chloride	CoCl <sub>2</sub> · 6H <sub>2</sub> O	" " "
Cupric chloride	CuCl <sub>2</sub>	" " "
Ferric chloride	FeCl <sub>3</sub>	" " "
Ferric sulfate	Fe <sub>2</sub> (SO <sub>4</sub> ) <sub>3</sub> · xH <sub>2</sub> O	Allied Chem. & Dye Corp.
Ferrous chloride	FeCl <sub>2</sub> · 4H <sub>2</sub> O	Fisher Scientific Co.
Ferrous sulfate	FeSO <sub>4</sub> · 7H <sub>2</sub> O	J. T. Baker Chem. Co.
Magnesium chloride	MgCl <sub>2</sub> · 6H <sub>2</sub> O	Allied Chem. & Dye Corp.
Manganous chloride	MnCl <sub>2</sub> · 4H <sub>2</sub> O	Fisher Scientific Co.
Nickel chloride	NiCl <sub>2</sub> · 6H <sub>2</sub> O	" " "
Stannous chloride	SnCl <sub>2</sub> · 2H <sub>2</sub> O	" " "
Titanium tetrachloride	TiCl <sub>4</sub>	" " "
Zinc chloride	ZnCl <sub>2</sub>	" " "
<b>MISCELLANEOUS</b>		
Ethylene glycol	CH <sub>2</sub> OHCH <sub>2</sub> OH	Wilkens-Anderson Co.
Magnesium oxide	MgO	Allied Chem. & Dye Corp.
Portland cement	—	Hawkeye Portland Cement Co.
Potassium biphthalate	KHC <sub>8</sub> H <sub>4</sub> O <sub>4</sub>	J. T. Baker Chem. Co.
Quadrafos <sup>c</sup>	—	Rumford Chem. Co.
Vanadyl dichloride	VOCl <sub>2</sub>	Fisher Scientific Co.

<sup>a</sup> Strong oxidizing agent. <sup>b</sup> In solution with 5 to 6 percent available chlorine. <sup>c</sup> Trade-name for sodium tetrphosphate (P<sub>2</sub>O<sub>5</sub>, 63.5 percent; Na<sub>2</sub>O, 36.0 percent; H<sub>2</sub>O, 0.5 percent).

at -10 F for 23 hr; then they were removed from the freezer and allowed to thaw in open air for 2 hr and in a moisture (near 100 percent RH) room for 23 hr. This was one cycle. Specimens were tested for unconfined compressive strength after 0, 1, 4, 5, 7, 9 and 12 cycles, (zero cycle specimens were immersed in water for 2 hr before testing for strength). Strengths reported are the average for three specimens.

### GENERAL COMPARISON OF CHEMICALS

The first study of the investigation was a general comparison of the chemicals to determine their relative effects on 7-day, 28-day and 4-month strengths. The amount of each chemical treatment was 0.5 percent, based on the dry weight of the 75 percent

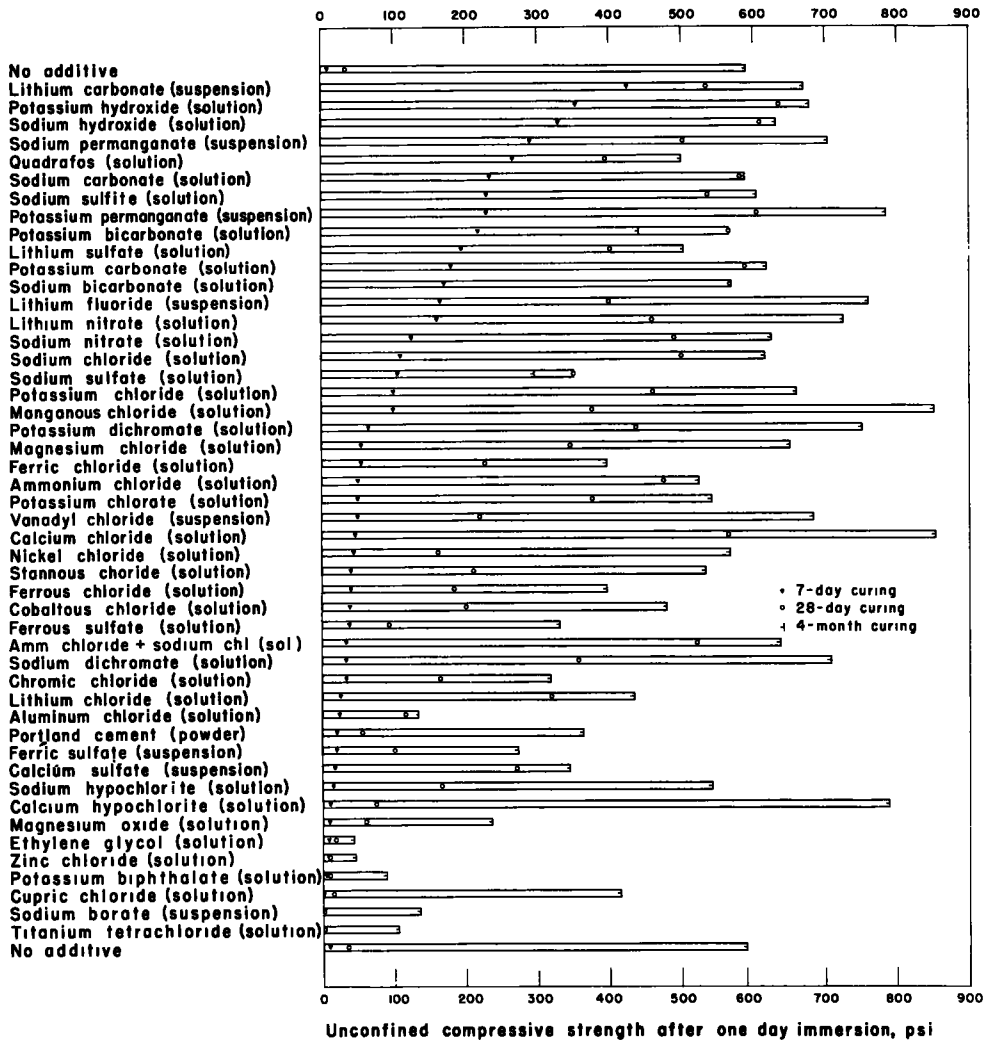


Figure 1. Immersed strengths after 7-day, 28-day, and 4-month normal moist curing of 2 in. diameter by 2 in. specimens prepared at standard Proctor density from 75:2.5:22.5 Ottawa sand-lime-flyash mortar treated with 0.5 percent of each chemical listed. Chemicals are listed from top to bottom in decreasing order of benefit to 7-day strength. All chemicals were added to the mortar dissolved or suspended in the mix water.

Ottawa sand: 2.5 percent lime: 22.5 percent flyash mortar. The chemicals were added dissolved or suspended in the mix water.

Test results are shown by bar graphs in Figure 1. Starting at the top of the figure chemicals are listed in order of decreasing benefit to 7-day strength.

### 7-Day Strength

The 7-day strength of a soil-lime-flyash road base is of much practical significance. Seven days is about as long as newly constructed roads can be kept closed to traffic, and to avoid rutting, base course strength, as measured by the unconfined compression test, may have to reach 100 to 300 psi, depending on soil type stabilized, road usage and thickness of bituminous surfacing. Since the strength gain of soil-lime-flyash road bases is greatly reduced or even halted when the temperature drops to near freezing, 7 days of curing in northern climates may represent the maximum obtainable in late season construction. For adequate freeze-thaw resistance, soil-lime-flyash bases may need a strength of 300 to 500 psi, depending on soil type stabilized, thickness of bituminous surfacing and severity of the climate.

The 7-day strength, after 24 hr immersion, of untreated Ottawa sand-lime-flyash specimens averaged only 9 psi (Fig. 1), illustrating the slowness of the pozzolanic reaction under normal curing conditions. Many of the chemicals greatly increased 7-day strength; these may be classed as accelerators for the lime-flyash reaction. The primary purpose of this investigation was to discover such chemicals.

Best strength acceleration was with lithium carbonate which gave an average 7-day strength of 226 psi, over 47 times the strength of the untreated reference specimens. Potassium hydroxide, sodium hydroxide, sodium permanganate and Quadrafos gave 7-day strengths near or above 300 psi. Sodium carbonate, sodium sulfite, potassium permanganate, potassium bicarbonate, lithium sulfate, potassium carbonate, sodium bicarbonate, lithium fluoride and lithium nitrate also are worthy of special mention, all giving 7-day strengths near or over 200 psi.

A look at the classification of chemicals in Table 1 shows that the two bases were good activators for the pozzolanic reaction. The basic salts, with the exception of sodium borate, gave 7-day strengths over 150 psi. The neutral salts produced variable results; best were the strong oxidizing agents potassium permanganate and sodium permanganate, both giving strengths over 200 psi. The acid salts did not appreciably improve 7-day strength. Of the chemicals in the miscellaneous group, only Quadrafos gave good 7-day strength.

### 28-Day Strength

Twenty-eight days of curing can usually be counted on in late summer or early fall construction and ideally soil-lime-flyash should reach adequate strength before the first freeze. The untreated lime-flyash mortar specimens did not gain adequate strength in 28 days (Fig. 1), the average being only 34 psi.

The chemicals cited as most beneficial to 7-day strength also greatly improved 28-day strength, but it will be noted that the order of merit is somewhat changed. Also, a number of chemicals which did not look promising on the basis of 7-day strength improvement, showed up well in the 28-day tests. Chemicals which gave 28-day strengths near or above 600 psi (about 18 times the 28-day untreated strength) are: potassium hydroxide, sodium hydroxide, potassium permanganate, potassium carbonate, sodium carbonate, calcium chloride, sodium bicarbonate and potassium bicarbonate, listed in order of decreasing merit. Many of the other chemicals gave 28-day strengths of 400 to 500 psi or higher. Among the most promising of this group are: lithium carbonate, sodium permanganate, sodium sulfite, sodium chloride, and the 50:50 combination of ammonium chloride and sodium chloride.

The bases and the basic salts, except sodium borate, again gave best results. The neutral salts, except sodium hypochlorite, gave 28-day strengths over 300 psi. Among the acid salts, calcium chloride, ammonium chloride, magnesium chloride and manganese chloride gave strengths over 300 psi, but as a whole the chemicals in this group rated low. Quadrafos and the 50:50 combination of ammonium chloride and sodium chloride were the most promising in the miscellaneous group.

**4-Month Strength**

The average 4-month strength of the untreated Ottawa sand-lime-flyash specimens was 589 psi, which is very adequate, and demonstrates that the long-term strength of lime-flyash stabilized soil is not a problem. In northern climates, spring or early summer construction is necessary to obtain 4 months of curing. But even when this is possible, sufficient early strength to carry traffic is necessary.

Figure 1 shows that most of the chemicals cited as being very beneficial to 7 and/or 28-day strengths caused little or no improvement of 4-month strength, or even decreased it. The greatest benefit to 4-month strength was obtained with calcium chloride (855 psi) and manganous chloride (852 psi). Both of these chemicals were beneficial to 28-day strength, particularly calcium chloride, but they rated low on the basis of 7-day

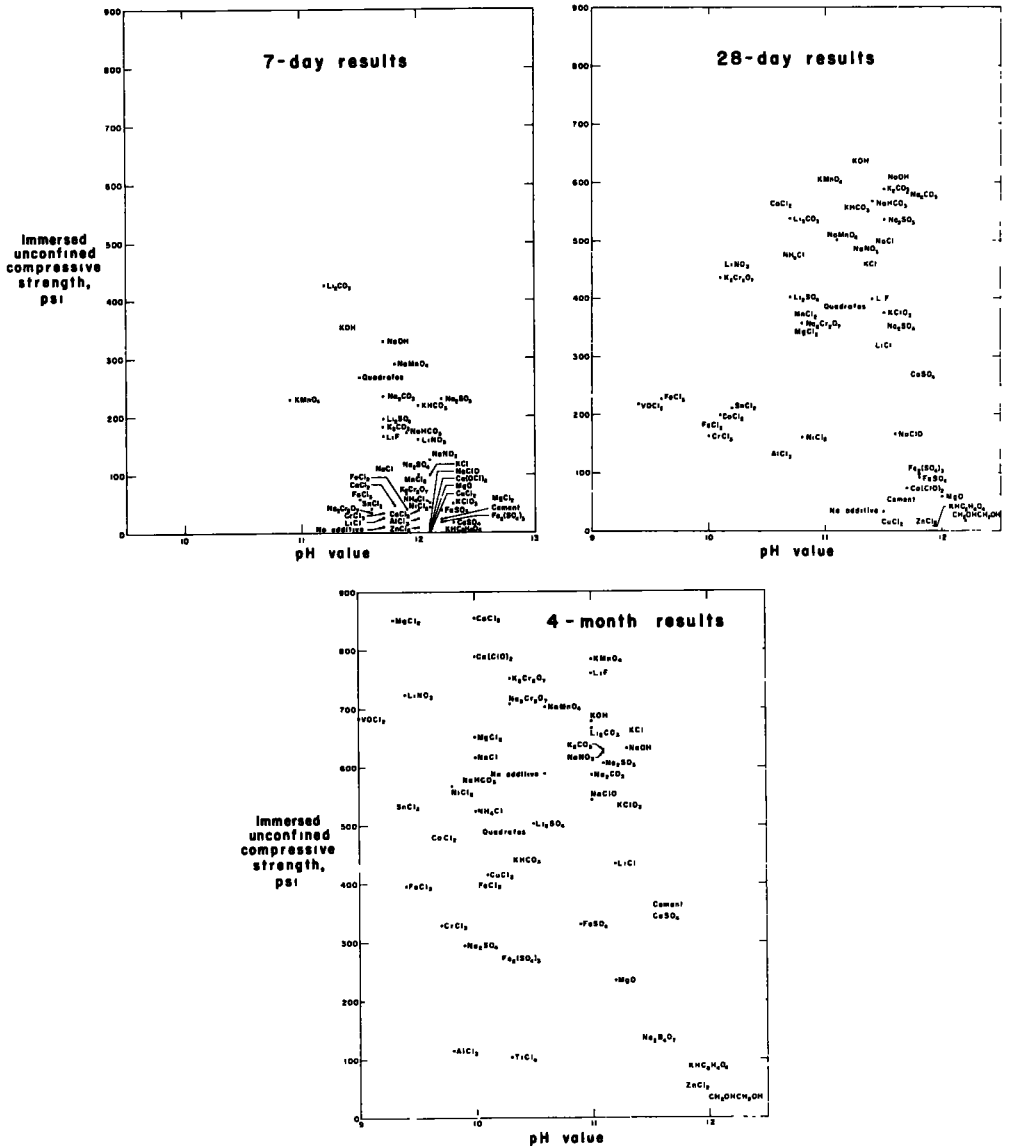


Figure 2. Relationship between strength and pH of 75:2.5:22.5 Ottawa sand-lime-flyash mortar treated with 0.5 percent of the different chemicals.

strength improvement. Calcium hypochlorite gave a 4-month strength of 789 psi, but 7 and 28-day strengths with this chemical were very low. On the basis of strength improvement at all three ages, potassium permanganate might be rated highest. Other chemicals giving 4-month strengths above 700 psi, and which were also noteworthy for 7 and 28-day strength improvements, were: sodium permanganate, lithium fluoride and lithium nitrate. Potassium dichromate and sodium dichromate gave 4-month strengths over 700 psi and also gave good 28-day strengths, but both rated low at 7-days.

The bases and the basic salts did not contribute much to the 4-month mortar strength indicating that the chemicals in these groups act mainly as activators of the lime-flyash reaction. The three chemicals which gave the highest 4-month strengths are acid salts, but several neutral salts also were beneficial; other chemicals in these two groups reduced 4-month strength. Several of the better neutral and acid salts are strong oxidizing agents (Table 1). In the miscellaneous group, vanadyl dichloride and the 50:50 combination of ammonium chloride and sodium chloride were slightly beneficial.

### Density Variations

Some of the chemical additives influenced the compaction characteristics of the 75:2.5:22.5 Ottawa sand-lime-flyash mortar as evidenced by variations in the dry density of test specimens. The maximum variation from the compacted density of the untreated mortar was plus 6.4 or minus 2.6 pcf. However the variations in density do not correlate with the improvements in strength.

### pH

Determinations of pH were made on material from specimens tested for unconfined compressive strength. (The electrometric method was used, employing 15 grams of sample in 30 mm of distilled water.) The object was to find a relationship between strength and pH after each curing period. A significant correlation is not evident to the authors from the data plotted in Fig. 2 although it is seen that mixtures containing the bases and basic salts maintained a relatively constant and high pH during the three curing periods.

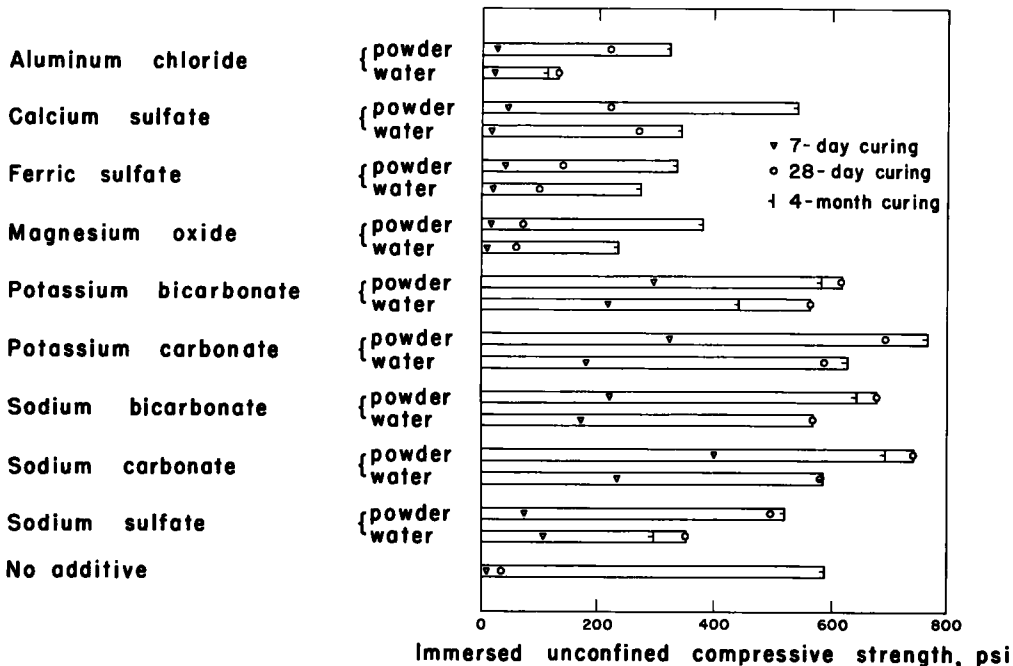


Figure 3. Comparison of powder vs mix water application of chemical additives. The amount of chemical was 0.5 percent; the mortar was 75:2.5:22.5 Ottawa sand-lime-flyash.

All pH values were in the alkaline range due to the presence of calcium hydroxide. The amount of lime present in the mixtures was greater than the maximum solubility (1.2 grams per liter) of calcium hydroxide at 25 C. Enough lime was present to counter-balance the influence of the acid salts used as additives. Decreases of the pH of mixtures with time is presumably due to the lime being used up in the pozzolanic reaction.

**POWDERED VS LIQUID APPLICATION OF CHEMICALS**

Since some of the chemicals could be used in powdered form, and dry mixed with the lime-flyash mortar prior to adding the mix water, a check was made to fine out what effect powdered application would have on mortar strengths. Test results in Fig. 3 show that most of the chemicals gave better results when mixed as a powder. Most noteworthy is powdered sodium carbonate which produce 7-day, 28-day and 4-month

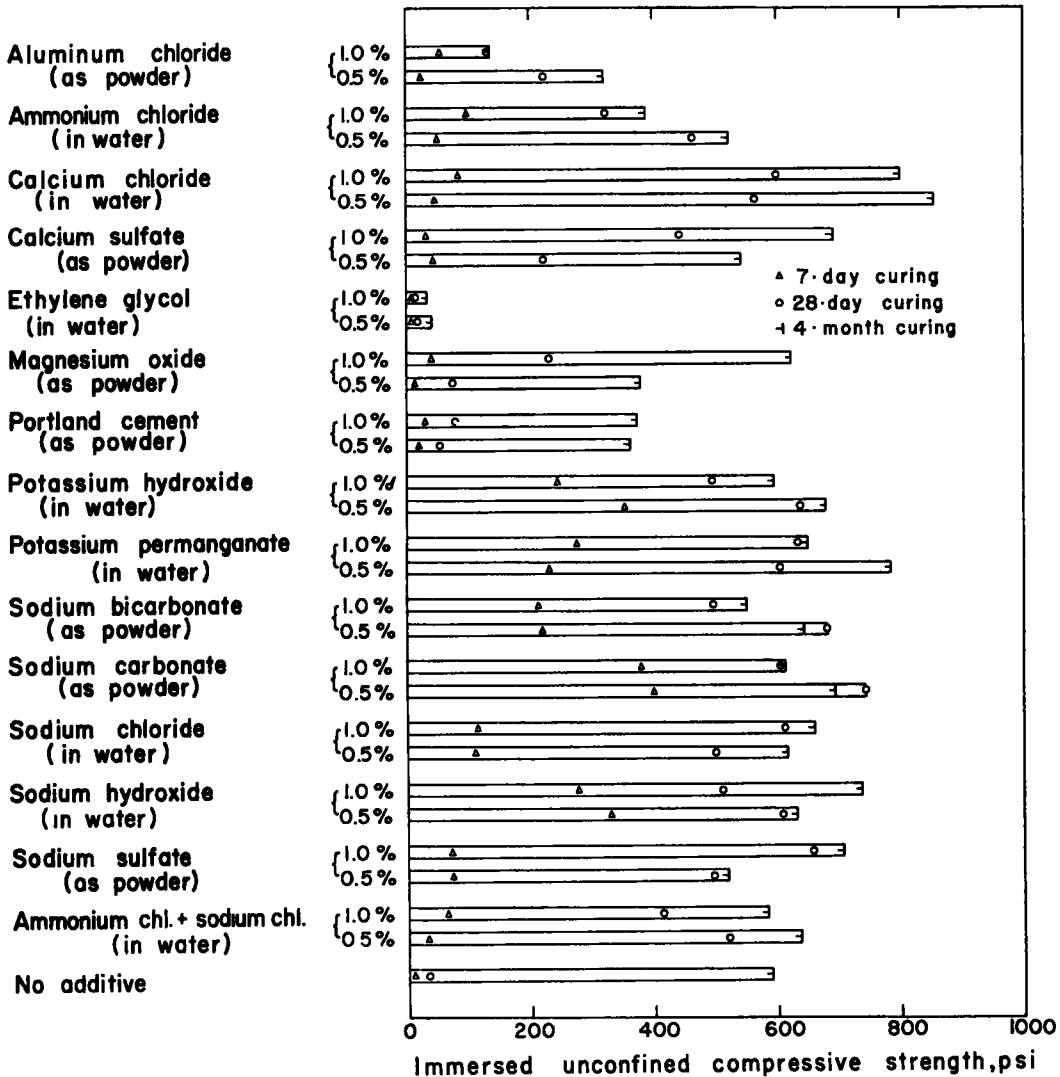


Figure 4. Effect of amount of chemical additive on strength of 75:2.5:22.5 Ottawa sand-lime-flyash mortar.

strengths that are, respectively, 71, 28 and 18 percent higher than the comparable strengths produced by liquid application of sodium carbonate.

### AMOUNT OF CHEMICAL

The use of more chemical additive than 1 percent may not be economical; 0.5 percent represents a more desirable treatment level. A comparison of strength benefits from 1 and 0.5 percent chemical treatments of 75:2.5:22.5 and 75:5:20 mortars are shown in Figs. 4 and 5. It appears that there is little advantage and perhaps a disadvantage in using more than 0.5 percent chemical, particularly of the more promising additives such as sodium carbonate, sodium hydroxide, potassium hydroxide, calcium chloride, aluminum chloride, potassium permanganate and magnesium oxide.

### INFLUENCE OF LIME TO FLYASH RATIO

The ratio of lime to flyash may be an important factor affecting the strength attained by soil-lime-flyash mixtures. In the present investigation the influence of the ratio was studied by comparing the strength gains of chemically treated 75:2.5:22.5 (ratio 1:9) and 75:5:20 (ratio 2:8) Ottawa sand-lime-flyash mortars. The results obtained from 1 and 0.5 percent chemical treatments are shown in Figures 6 and 7 respectively.

#### 7-Day Strength

The 7-day strengths, with a few exceptions, were similar for both ratios. The exceptions are of interest because they involve three of the most promising activators: potassium hydroxide, sodium hydroxide and sodium carbonate. One percent powdered sodium carbonate gave highest strength with the 2:8 ratio mortar, the increase being

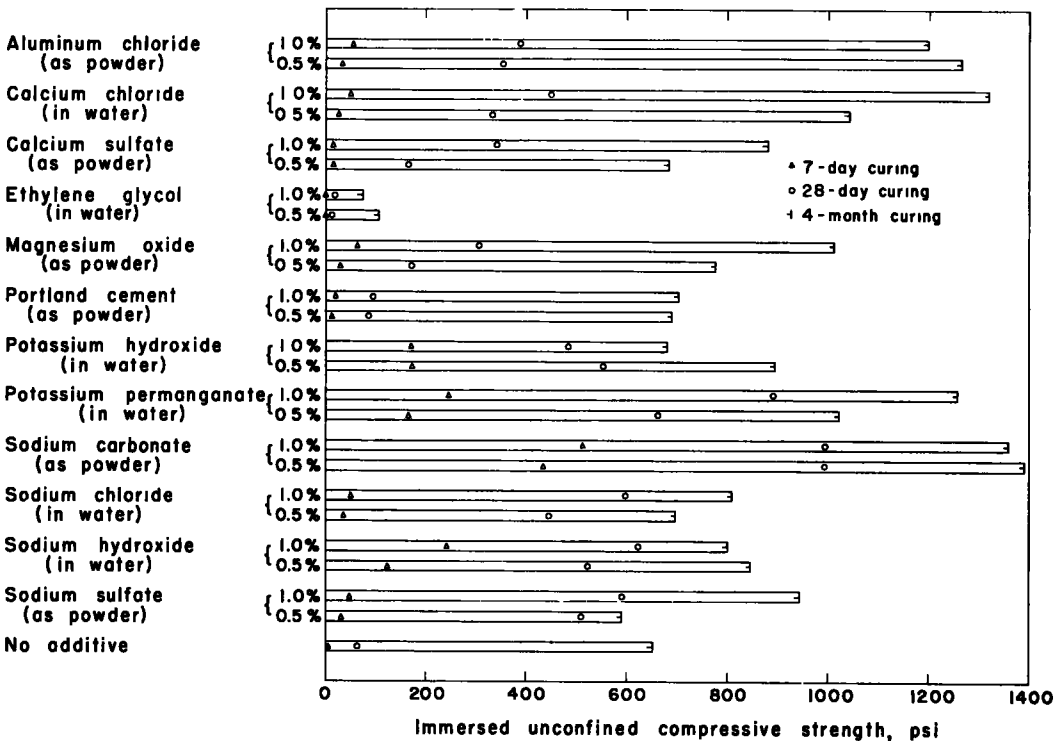


Figure 5. Effect of amount of chemical additive on strength of 75:5:20 Ottawa sand-lime-flyash mortar.



144 psi; with 0.5 percent sodium carbonate the 7-day strength difference between the two mortars was negligible. One-half percent sodium hydroxide gave best results with the 1:9 ratio mortar by about 100 psi, whereas little strength difference was observed for the 1 percent treatment. Potassium hydroxide likewise favored the 1:9 ratio mortar, the strength increase being about 100 psi for the 0.5 percent treatment.

### 28-Day Strength

It is difficult to conclude which lime to flyash ratio gave the best 28-day strength. With the better chemicals previously cited, good strengths were obtained with both ratios. Examples of chemical treatments most sensitive to the ratio are: 0.5 and 1 percent calcium chloride, which gave best results with the 1:9 ratio mortar by about 150 to 230 psi; 1 percent potassium permanganate, which was best with the 2:8 ratio mortar by about 260 psi; and 0.5 and 1 percent sodium carbonate, which produced best strengths with the 2:8 ratio mortar by about 250 to 390 psi.

### 4-Month Strength

The best 4-month strengths without exception were obtained using the 2:8 lime to flyash ratio; in some cases the strength was two or more times that obtained when the ratio was 1:9. The following chemicals produced 4-month strengths above 1,000 psi: aluminum chloride, calcium chloride, potassium permanganate, sodium carbonate and magnesium oxide. Of these, only magnesium oxide was appreciably less effective when the amount used was 0.5 percent.

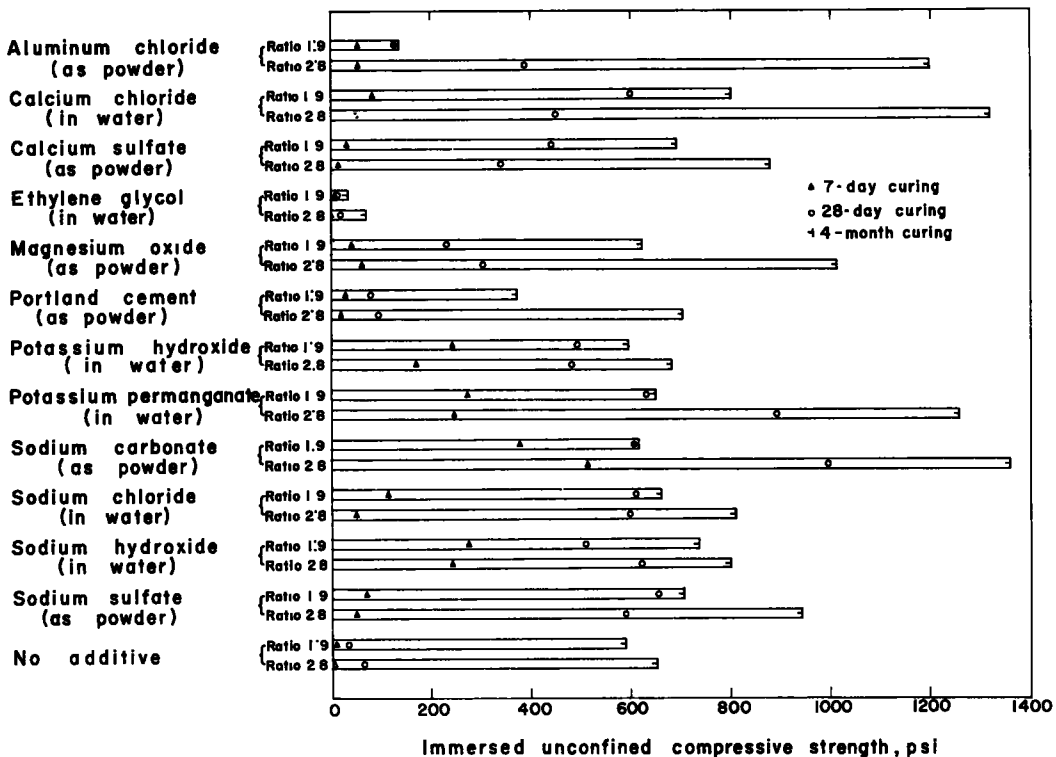


Figure 6. Effect of lime to flyash ratio on strength benefits from 1.0 percent chemical treatments. The mortars were composed of 75 percent Ottawa sand and 25 percent lime and flyash.

## FREEZE-THAW RESISTANCE

The strength retention through 12 cycles of alternate freezing and thawing of 28-day cured 2-in. diameter x 2-in. specimens provides an indication of relative durability. The effects of 0.5 percent of any one of four chemicals on the durability of the 75:2.5:22.5 mortar are shown in Figure 8.

The mortar specimens with no chemical additive showed high strength retention, but initial strength was less than 200 psi. Specimens containing sodium carbonate (added as a powder) had a marked decrease in strength, but after 12 cycles the strength retained was more than 600 psi. Potassium permanganate treated specimens, after a decrease in strength through 4 cycles, showed an abrupt increase to over 600 psi which was retained with little reduction through the last 8 cycles. Specimens containing calcium chloride and sodium chloride failed after the fifth cycle.

## SUMMARY DISCUSSION

### Evaluation of Chemicals by Groups

Classification of the chemical additives as in Table 1 permits some generalized statements concerning the effects of each group on the lime-flyash (pozzolanic) reaction. The bases and the basic salts, except sodium borate, greatly improved early strength, but did not improve long-term strength to a proportionate degree. Thus most chemicals in these two groups appear to be good activators (accelerators) of the pozzolanic reaction. One of the basic salts, sodium carbonate (soda ash), is considered the most promising trace chemical evaluated.

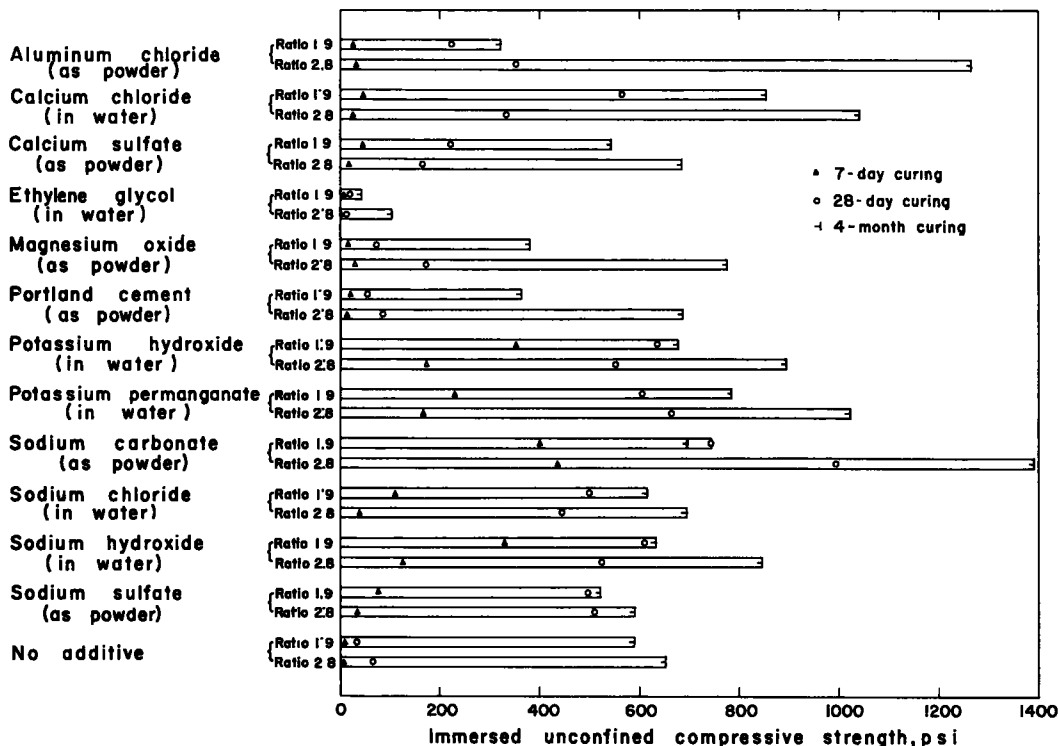


Figure 7. Effect of lime to flyash ratio on strength benefits from 0.5 percent chemical treatments. The mortars were composed of 75 percent Ottawa sand and 25 percent lime and flyash.

The neutral salts produced very good 28-day strengths, which were further improved after 4 months curing. With only two exceptions 4-month strengths were close to or higher than the untreated mortar strength. Seven-day strengths with neutral salt additives were higher than the corresponding untreated mortar strength, but the improvement was not always exceptional. The most promising activators in this group are potassium permanganate, sodium permanganate, lithium sulfate, lithium fluoride, lithium nitrate, sodium nitrate and sodium chloride (because it is cheap).

The acid salts did not appreciably improve 7-day strength, but some gave marked improvement to 28-day and 4-month strengths. The most promising chemicals in this

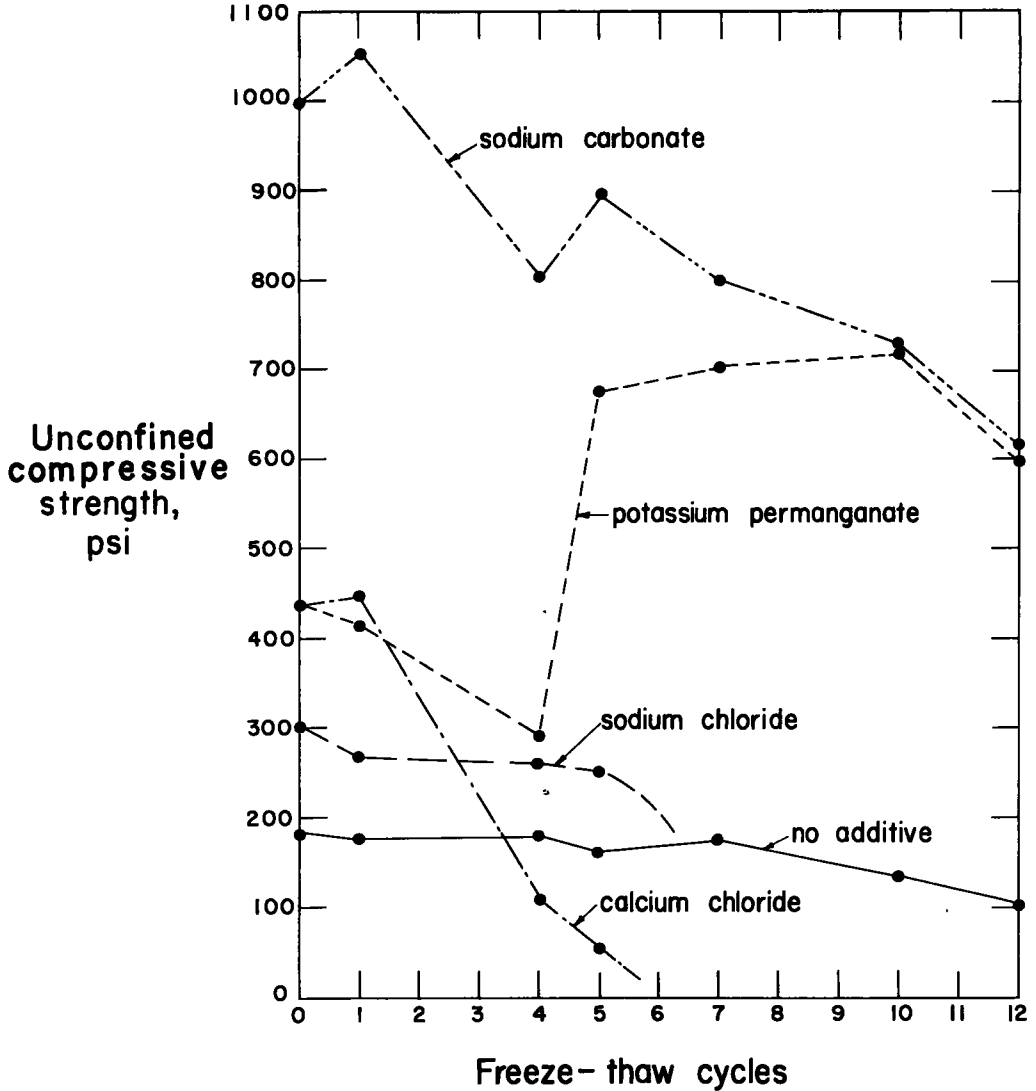


Figure 8. Effect of freezing and thawing on strength of 28-day cured 2 in. diameter by 2 in. specimens of 75:2.5:22.5 Ottawa sand-lime-flyash mortar treated with 0.5 percent each of sodium carbonate, potassium permanganate, calcium chloride and sodium chloride. Sodium carbonate was added to the mortar in powdered form; the other chemicals were added in the mix water.

group are the chlorides, particularly calcium chloride and manganous chloride. Calcium hypochlorite gave very good 4-month strength but 7- and 28-day strengths were low with this chemical.

In the miscellaneous group of chemicals, Quadrafos was most beneficial to 7-day strength; magnesium oxide was beneficial to 28-day and 4-month strengths.

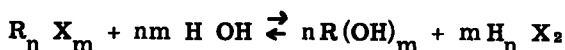
### Mechanism of Benefits

An explanation of the mechanism of the beneficial effects obtained with the different chemicals is difficult because each chemical or narrow range of chemicals may act in a completely different way. A chemical may act as a catalyst or as an inhibitor or as a component of the pozzolanic reaction, and also may go into side reactions other than the pozzolanic reaction and produce either cementing or inert materials.

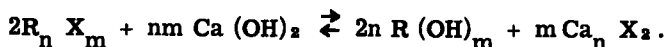
**Bases and Basic Salts.**—Alkaline additives increase the amount of available hydroxyl ions in the moistened Ottawa sand-lime-flyash system, and as a result the pozzolanic reaction may be accelerated by the increased solubility of the siliceous material (4). For example sodium hydroxide may act as a catalyst in which: (a) it first reacts with the siliceous material to produce intermediate sodium silicates, (b) the over-all reaction goes to completion when the intermediate sodium silicates subsequently react with lime (calcium hydroxide) to form sodium hydroxide and cementitious insoluble calcium silicates, (c) the sodium hydroxide is then free for further reaction with unreacted siliceous material.

The most promising of the alkaline additives, sodium carbonate, very likely reacts with lime in the moist Ottawa sand-lime-flyash mixture to form calcium carbonate and sodium hydroxide. The precipitated calcium carbonate contributes cementation to the system, and, as hypothesized in the preceding paragraph, the sodium hydroxide acts as a catalyst. The formation of calcium carbonate from the lime obviously decreases the amount of lime available for the pozzolanic reaction. The hypothesis is apparently consistent with the experimental data: with a 1:9 ratio of lime to flyash better strength results were obtained with 0.5 percent sodium carbonate than with 1 percent, whereas the opposite results were obtained with a 2:8 ratio. This indicates the possibility of an optimum amount of sodium carbonate for a given amount of lime.

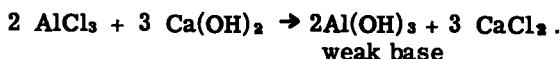
**Acid Salts.**—Acid salts undergo a hydrolysis reaction with the precipitation of weak bases (hydroxides). This may be summarized as follows:



With calcium hydroxide this reaction proceeds as follows:



With aluminum chloride as the acid salt, a weak base is precipitated and an equivalent amount of lime is withdrawn from the reaction,



The removal of lime results in a reduction of the lime to flyash ratio. Thus acid salts may impede the development of strength when the ratio of lime to flyash is 1:9. For example when aluminum chloride was used in mixtures having a 1:9 ratio of lime to flyash, strengths were much lower than when the ratio was 2:8. In mixtures with a 1:9 ratio 1 percent aluminum chloride gave lower strengths than 0.5 percent aluminum chloride.

Some of the weak bases formed, such as  $Al(OH)_3$  and  $Fe(OH)_3$ , have some cementing and water-proofing properties which may be beneficial. Such weak bases may also affect the long-term formation of hydrated calcium silicates and thus increase long-term strength.

Although calcium chloride is an acid salt, the principal long-term strength benefits

obtained with this chemical are thought due to a different type of chemical mechanism than discussed above. Calcium chloride being highly hygroscopic and deliquescent ensures a relatively high concentration of calcium ions over a long period of time by providing moisture for a solution. Since lime has a lower ionization constant than calcium chloride, the concentration of calcium ions from lime is lower than that from calcium chloride. Also lime is subject to conversion to calcium carbonate during long curing periods; when this takes place pozzolanic action ceases. The experimental data tend to support this line of reasoning: calcium chloride was found to be very beneficial to 4-month strength, whereas it only slightly improved 7-day strength.

Sodium chloride may act similarly to calcium chloride but there appears to be less benefit to long-term strength, perhaps because sodium chloride is less hygroscopic and deliquescent than calcium chloride. Another difference is that some sodium hydroxide is probably formed, and thus a small amount of catalysis would be expected. This may explain why sodium chloride gave slightly higher 7-day strength than calcium chloride.

**Neutral Salts.**—The reactions of neutral salts with the Ottawa sand-lime-flyash mixtures are somewhat more complicated than those of the other groups. The most promising neutral salts, potassium permanganate and sodium permanganate, are strong oxidizing agents. It is believed that these chemicals oxidize the carbon in the flyash with the consequent production of potassium carbonate or sodium carbonate, and the precipitation of manganese dioxide. These carbonates, as discussed previously, then give rise to further reactions which are beneficial to strength. It is also believed that the permanganates, and other strong oxidizing agents, benefit strength by reacting with grains of flyash, cleaning the surfaces and making them more available for chemical reactions with lime. As already mentioned, several of the better neutral and acid salts were strong oxidizing agents.

**Miscellaneous Chemicals.**—Quadrafos (sodium tetraphosphate), the only chemical in the miscellaneous group that greatly benefited early strength, may react with lime to produce complex phosphate cementation products which supplement those produced by the pozzolanic reaction. The availability of sodium ions to form sodium hydroxide may improve the pozzolanic reaction as previously discussed under bases and basic salts.

The beneficial effect of magnesium oxide, another miscellaneous chemical, is in agreement with the findings of previous research that dolomitic monohydrate (Type N) lime gives greater strengths than high-calcium lime (6). The present data show that magnesium oxide was most effective when added as a powder, when the amount was 1 percent, and when the lime to flyash ratio was 2:8. The mechanism of the benefit can only be guessed at. Perhaps cementation by calcium magnesium silicates is better than by calcium silicates?

**Powdered vs Liquid Application of Chemicals.**—All chemicals tried gave best results when used in powdered form, rather than when added in the mix water as a solution or suspension. This may be due to consumption of the chemical by side reactions that take place more rapidly in solution or suspension than in a semi-dry system. Another possibility is that lesser amounts of chemicals than were studied are needed for optimum benefits when the chemicals are added in water.

The greatest benefit from use of a powdered chemical was with sodium carbonate. The reaction of this chemical with lime in the mortar is responsible for the previously discussed precipitation of calcium carbonate cement. Calcium carbonate is thought to be more effective as a cement when precipitated after the mortar has been compacted, because then the carbonate crystals are formed on the sand (and flyash) grains. This is more apt to occur when sodium carbonate is added in powdered form since when added in the mix water, the calcium carbonate may be prematurely precipitated before compaction is completed. Perhaps another advantage in using powdered sodium carbonate is that a slower production of sodium hydroxide may be more favorable to the sustained formation of pozzolanic cementing products.

## CONCLUSION

Several of the forty-seven chemicals evaluated in lime-flyash mortars greatly in-

crease early strength. Other chemicals benefit long-term strength more than early strength; calcium chloride is one of the most promising chemicals in this group.

All factors considered, sodium carbonate (soda ash) is considered the most promising trace chemical activator investigated. Best results are obtained when it is mixed in powdered form. The use of 0.5 percent powdered sodium carbonate in a mixture of 75 percent Ottawa sand; 5 percent lime; 20 percent flyash increased 7-day strength about sixty times, and 28-day and 4-month strengths about two times.

#### ACKNOWLEDGMENTS

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#### REFERENCES

1. Alexander, K. M., "Activation of Pozzolans by Treatment with Acid." Australian Journal of Applied Science, 6:327-333 (Sept. 1955).
2. \_\_\_\_\_, "Activation of Pozzolanic Material by Alkali." Australian Journal of Applied Science, 6:224-229 (June 1955).
3. Goecker, W. L., Moh, Z. C., Davidson, D. T., and Chu, T. Y., "Stabilization of Fine and Coarse-Grained Soils with Lime-Flyash Admixtures." HRB Bull. 129 (1956).
4. Handy, R. L., "Cementation of Soil Minerals with Portland Cement or Alkalis." HRB Bull. 198 (1958).
5. Lambe, T. W., and Moh, Z. C., "Improvement of Strength of Soil-Cement with Additives." HRB Bull. 183 (1957).
6. Lu, L. W., Davidson, D. T., Handy, R. L., and Laguros, J. G., "The Calcium-Magnesium Ratio in Soil-Lime Stabilization." HRB Proc. 36:794-805 (1957).

# Effect of Number of Test Specimens on Test Results on Slag-Lime-Flyash Mixtures

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This paper sets forth a scientific approach which may be used in evaluating new highway materials. This approach has been used in the evaluation of slag-lime-flyash mixtures at the University of Illinois in cooperation with the Illinois Division of Highways and the Bureau of Public Roads. Wet-bottom boiler slag and flyash are two by-products of the combustion of powdered bituminous coal. These materials have been combined with lime and water to produce a mixture that may be used as a base course for bituminous surfaces.

Numerous laboratory techniques and testing methods have been developed that will be of value to those who may be concerned with the analysis of this or similar mixtures in the future. This paper is concerned with the importance of relating the number of test specimens prepared to the variability of the laboratory techniques employed, in order that the significance of test results may be determined. If this approach is followed the laboratory test results may be interpreted with a known degree of reliability. Any conclusions drawn from the test data will therefore be of scientific value to all concerned.

## NATIONAL INTEREST

● **THE MAGNITUDE** of the present highway program makes it imperative that adequate scientific methods for evaluation, design, and construction of highway pavements be developed and put into use. The scientific approach must be used in the evaluation of new materials which are not currently being used in the highway field in order that these new products may be effectively incorporated in the production of strong, durable, and economical highways.

Highway engineers are continually confronted with the problem of improving the load carrying capacity of natural soil. Numerous methods of soil stabilization have been used in the past and new methods are continually being developed. The basic methods of stabilization may be classified as follows: (a) mechanical-improving the aggregate gradation; (b) compaction-increasing density; (c) bituminous-waterproofing and giving additional cohesion; (d) cementation; and (e) chemical. Methods of stabilization which satisfy more than one of the above functions are highly desirable. In order to determine whether a material will serve any or all of the above functions it is necessary to develop a laboratory test program which will give test results that are significant and that are truly measures of the desired properties. After the laboratory tests have been obtained it will no doubt be desirable to test the best materials under actual field conditions in order that information may be obtained regarding the workability of the material as well as the performance under field conditions.

One method of stabilization, lime-pozzolan stabilization of natural soils, has received considerable attention during the past decade. A pozzolanic material is defined by ASTM as a silicious or alumino-silicious material which in itself possesses little or no cementitious value but which in finely divided form and in the presence of moisture will chemically react with alkali and alkaline earth hydroxides at ordinary temperatures

to form or to assist in forming compounds possessing cementitious properties.

Campaigns to clean up smoky cities have resulted in large quantities of flyash being precipitated from the smoke. Some of the flyashes have been found to possess physical and chemical properties that meet the above requirements for a pozzolanic material. Since large quantities of flyash are presently available at a nominal cost, it is logical that uses for this material be investigated.

The problem of developing a laboratory program which will give test results that are significant and that truly measure the desired properties of the material is a very challenging and interesting task.

### SLAG-LIME-FLYASH MIXTURE STUDY

In the state of Illinois there are numerous plants producing power by burning pulverized bituminous coal. Two by-products of this burning process are wet-bottom boiler slag and flyash. Wet-bottom boiler slag is a dense, hard, angular material and is not to be confused with blast furnace slag. Wet-bottom boiler slag is known also as black slag. These waste products are presenting large disposal problems in the state of Illinois, since over 1½ million tons of the two materials are produced annually.

In some Illinois counties the supply of aggregate for base courses is becoming limited, and suitable substitute materials that may be economically used are highly desirable. Since slag and flyash are primarily waste products, the economic consideration in localized areas where they are available and where the usual base-course aggregates are scarce will favor their use—if mixtures containing these materials have the required physical properties.

The original purpose of the slag-lime-flyash mixtures study was to evaluate the merits of the slag-lime-flyash mixture in regard to its possible use as a base course material with a bituminous surface. The general requirements were adequate workability, strength, durability, and ability to be produced at a reasonable cost. This paper is not a report on the research project in its entirety, but describes only a scientific approach that was used to obtain reliable and meaningful laboratory test results.

### EARLY TESTING PROBLEMS

Since this was a new material there were no standard ASTM tests which could be performed to determine whether the mixture was acceptable. Therefore, methods of evaluating the mixture had to be developed. This necessitated the development of a laboratory procedure that would reveal the true nature of the mixture. A preliminary study was required to answer some of the early testing problems in order that the variability of the mixture could be determined.

Compressive strength was chosen as the basic test to evaluate the variability of the mixture because this test was easy to run and it measured the combined effect of cohesion and internal friction of the mixture. Since the load carrying capacity of a material consists primarily of cohesion and internal friction, a comparison of the compressive strengths should indicate the most promising mixture. The remainder of this report presents the procedure that was used in limiting the variables involved in compressive strength tests within reasonable limits and in determining the number of test specimens required to provide the results within an acceptable degree of accuracy.

One of the early questions was the determination of the size of specimen to be used in testing. After evaluating research procedures used by others on similar materials a 2- by 4-in. cylindrical specimen was selected for use in the investigation. Until the effect of mixing time was investigated it was necessary to arbitrarily fix the mixing time. Densities of the 2- by 4-in. specimens were equated to maximum density by determining a compactive effort that would produce the same density as that produced by the standard compaction test (AASHTO Designation T99-57, Method A). The initial number of test specimens was determined by the capacity of the mixing equipment. Additional questions were raised in regard to capping, removing, curing and testing specimens. Initially obtained results, although indicating high compressive strength, were not consistent enough to enable the data to be of scientific use for evaluation purposes.

It was on the basis of this preliminary study that the research project was developed



in order that some of the above questions could either be resolved or eliminated by extreme care in the laboratory procedure. This large variation simply necessitated an analysis of all known variables and the determination of the effect of both the inherent variables of the materials and the variables produced by the testing procedures. This approach will enhance the development of standard procedures and will also enable the development of significant test results for a given degree of reliability.

## LABORATORY TECHNIQUES AND EFFECT ON TEST RESULTS

### Sampling and Preparation of Materials

In this test program numerous samples of the raw materials were obtained in accordance with the ASTM D75-48 Specification. Whenever possible, the size of sample was large enough to complete one phase of the testing. As the testing progressed, it was evident that different materials obtained from the same plant at different times had different characteristics.

In determining the size of the test specimen it was anticipated that several specimens would be required to give an average compressive strength value that would be significant. It was desirable to use as small a specimen as possible to prevent the handling of extremely large quantities of the materials. Since more than 95 percent of all the raw materials passed the No. 4 sieve, the 2- by 4-in. cylindrical mold was used. This size of specimen also eliminated any L/D corrections since the ratio was two. In the laboratory testing, all material larger than  $\frac{1}{2}$  in. was removed from the sample and discarded. In order to minimize physical variation in the flyash and slag, the total sample of each material was mixed separately in a pug mill mixer for 45 minutes after which it was placed directly in its storage container.

The lime was obtained in sealed metal drums and transferred to 1 gal paint cans which had an air-tight seal. Type No. 3 Permatex was used to coat the lids as an added precaution to assure a completely air-tight seal. It was found that lime must be handled with care in the laboratory to prevent it from carbonating. Carbonation is the formation of calcium carbonate due to the reaction between lime and carbon dioxide from the air. The calcium carbonate is actually inert and does not combine to form calcium silicates. In the preliminary study

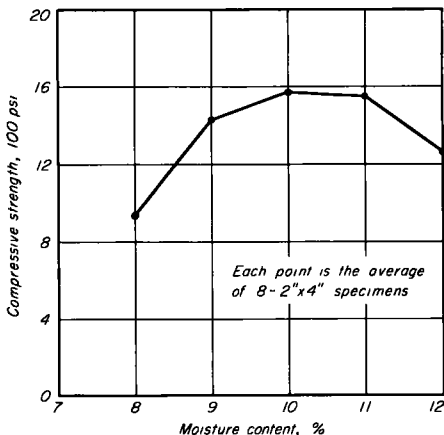
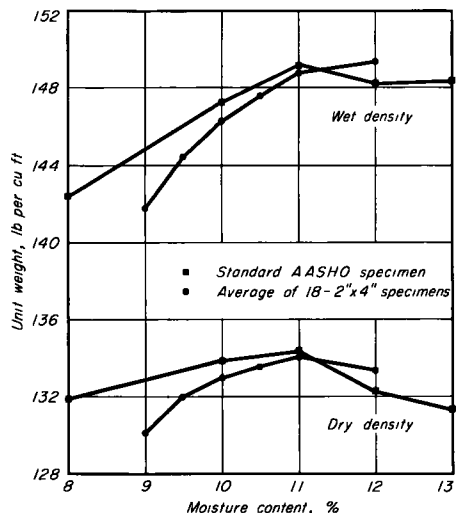


Figure 1. Relationship of compressive strength to moisture content for 72.1 percent slag, 24.0 percent flyash, and 3.9 percent lime mixture.



Note 2 inch by 4 inch specimens compacted in 4 layers, 10 blows per layer, by a 4 pound hammer falling 12 inches

Figure 2. Relationship of standard AASHO density to 2- by 4-in. specimen density for 72.1 percent slag mixture.

mentioned previously, the lime was left exposed to air in an open bag, and it was determined by subsequent tests that carbonation had occurred. The degree of carbonation depended on the interval of time that the lime was exposed. The carbonation does not extend more than a fraction of an inch into the lime, but this is sufficient to cause the laboratory results to be inconsistent since a very small quantity of lime is used in making the cylinders.

### Proportioning of Materials

The major problem under investigation in this research was the determination of the effect of various proportions of slag, lime, and flyash on compressive strength. In order to evaluate the various combinations of slag, lime, and flyash, the moisture content had to be standardized. The effect of moisture content on compressive strength is shown in Figure 1. The relationship between unit weight and moisture content for both the AASHO compaction specimens (4-in. diameter) and 2- by 4-in. specimens is shown in Figure 2. Since the optimum moisture content from the standard AASHO compaction test corresponded so closely with the moisture content which gave the best compressive strengths, it was decided that the moisture content would be determined for each combination of slag, lime, and flyash by running a standard compaction test to obtain the optimum moisture content for each mixture.

### Mixing

The mixer used throughout this study was a Lancaster Mixer equipped with a muller. It was found that an increase in mixing time resulted in a higher compressive strength for a given mixture. This relationship is shown in Fig. 3. Since most present-day standards simply state that the material should be thoroughly mixed, it was necessary to analyse this data and determine an appropriate mixing time which would be comparable with average field conditions. Two minutes was determined to be a reasonable time, based on observations of field mixes and of laboratory-prepared mixtures, and the results of the laboratory mixing time study. These two minutes were divided into one minute of dry mixing, followed by the addition of the water, and one minute of wet mixing.

At one time it was thought that the additional mixing was actually changing the particle size and thus producing a higher compressive strength. A sieve analysis, before and after four minutes of mixing, revealed that the particle size was not changed. It was thus assumed that the increase in compressive strength was due to a better distribution of all ingredients.

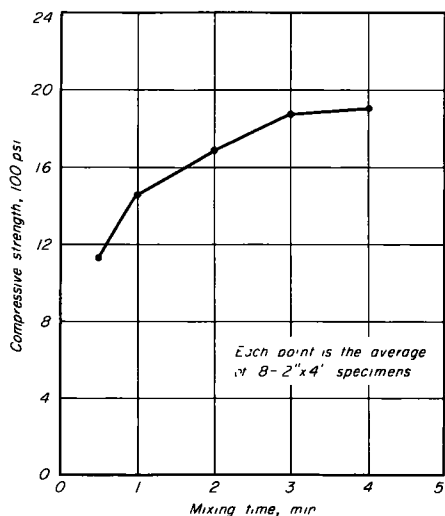


Figure 3. Relationship of compressive strength to mixing time for 72.1 percent slag, 24.0 percent flyash, and 3.9 percent lime mixture.

### Molding

The test specimens were molded immediately after each batch of material was mixed. The mixture was compacted in the molds in four equal layers with each layer receiving ten blows of a 4-lb hammer dropping 12 in. As shown in Fig. 2 this is equivalent to standard AASHO maximum density.

After each layer was compacted its surface was scarified in order to develop adequate bond between layers. In order that the final layer would receive the same amount of compaction as the preceding layers, a collar was made to fit over the mold so that it could be filled with loose

material to a height greater than the top of the mold. To assure the same amount of material in each layer, a scoop was constructed with a volume which approximated the volume of loose material required for a 1-in. compacted layer.

### Capping and Removal

Various removal times and capping materials were investigated. In order to determine the amount of variation in compressive strength of the test specimens which may be attributed to the capping and the removal procedures, the coefficient of variation for compressive strength was determined for various methods. The coefficient of variation is a convenient way to express the degree of variation as a percentage of the mean value. A certain amount of variation may be attributed to the testing technique but the increment between various coefficients of variation may be attributed to the capping and removal techniques. This test data is summarized as follows:

1. Extruded immediately and capped with Hydrocal, coefficient of variation, 10.5 percent.
2. Cured in the molds and capped with Hydrocal, coefficient of variation, 7.9 percent.
3. Cured in the molds overnight and capped with neat cement, coefficient of variation, 7.1 percent.

On the basis of the foregoing data, method 3 was used as the standard. Neat cement released the molds for continuous use following the initial curing period and the specimens were prepared for testing in one continuous operation.

The molded specimens were weighed to determine the wet unit weight, after which a thin layer of material approximately  $\frac{1}{8}$  of an in. thick was removed from each end of the specimen. A mixture of cement and water was placed on each end of the specimen and metal plates were used to form a smooth surface. The specimens were then left undisturbed for  $20 \pm 4$  hr in the laboratory air. Following this initial curing period each specimen was removed from the mold by the use of a pedestal. Care was taken

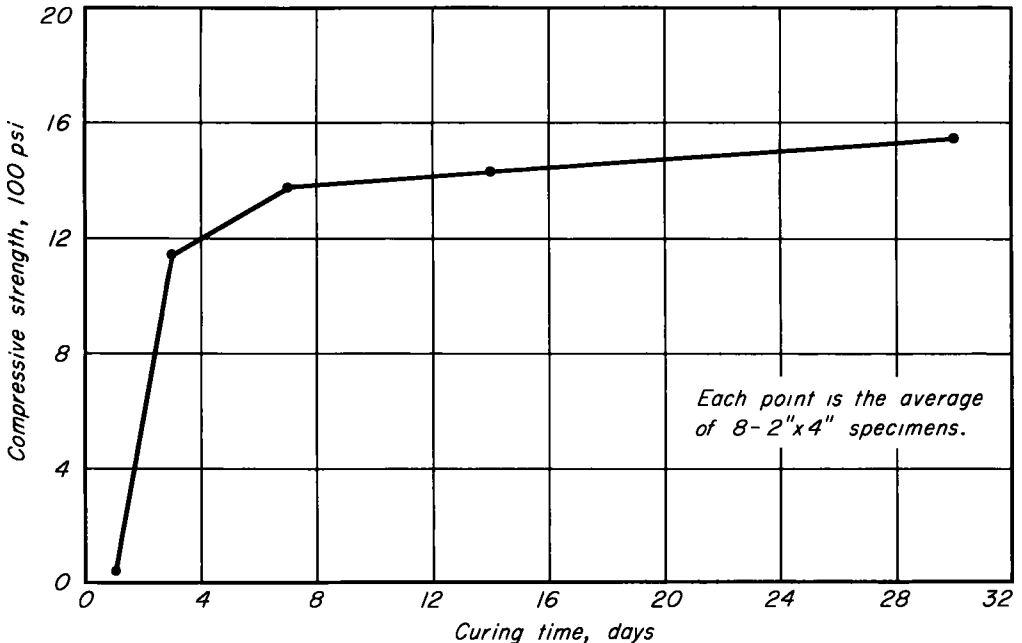


Figure 4. Relationship of compressive strength to curing time for 72.1 percent slag, 24.0 percent flyash, and 3.9 percent lime mixture.

to keep the specimens in compression since they are very weak in tension at this early age. The specimens were then given an appropriate identification number. Each series of specimens was gently placed and sealed in a 1 gal paint can.

### Curing

The sealed paint cans containing compressive strength specimens were placed in a constant temperature oven at 140 F and cured for an additional 13 days, thus giving a total curing period of 14 days. The relationship between the curing time in the oven and compressive strength is shown in Figure 4.

To compare field curing with oven curing, some specimens were placed in the field and companion specimens were placed in the oven. The field-cured specimens were buried in the soil in such a manner that the tops of the specimens were covered with 2 in. of soil. These specimens were cured and tested after various periods of time. The temperatures and relative humidities prevailing during the field curing are shown in Fig. 5. The compressive strength test results are shown in Fig. 6. It may be observed that for this mixture the 7-day oven-cured strength approximates the 1-yr field strength.

### Preparation for Testing

The average diameter of each cylinder was determined to the nearest 0.02 of an in. in order to accurately determine the area. After the specimens were measured they were allowed to reach room temperature, and then weighed to determine their net gain or loss in weight during the curing.

An investigation of the effect of saturation on the compressive strength of test specimens showed that saturated specimens had an average compressive strength of 100 psi less than comparable specimens tested in the dry state. This difference was not considered great enough to warrant the additional time and extra handling required for saturation, and all subsequent specimens were tested in the dry state.

### Testing

The rate of deformation used in all tests was 0.05 in. per min. All compressive

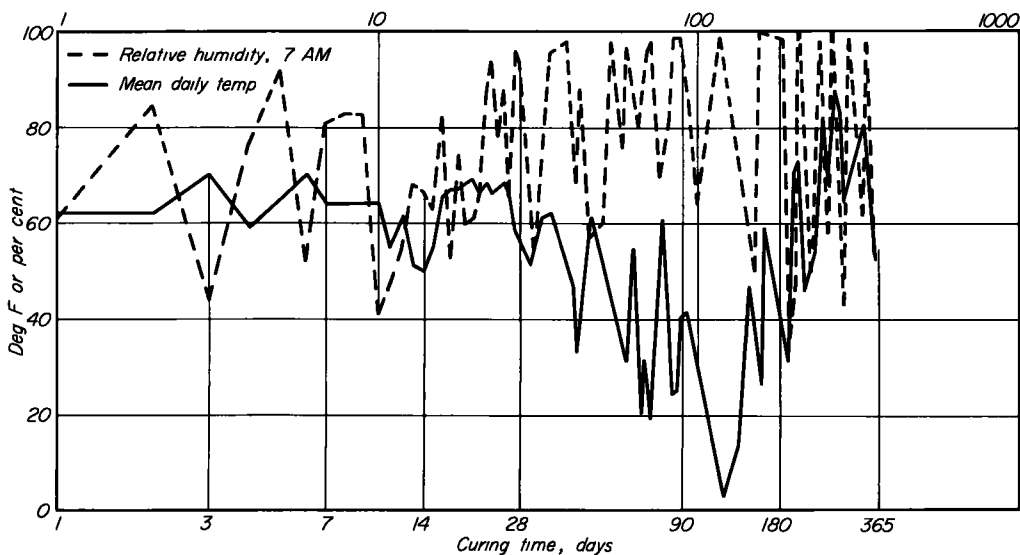


Figure 5. Temperature and relative humidity prevailing during field curing.

strength values were determined to the nearest 10 psi. There was no L/D correction applied to the standard 2- by 4- in. cylinder.

**DETERMINATION OF THE NUMBER OF TEST SPECIMENS IN RELATION TO THE TECHNIQUES USED AND THE RELIABILITY OF TEST RESULTS DESIRED**

Pilot Study to Determine Variations

After the laboratory techniques and known variables were standardized, a series of test specimens was made by the standardized techniques and tested to determine the effect of the remaining unknown variables on test results. The pilot study consisted of making 24 2- by 4-in. specimens with all of the known variables fixed. The compressive strength of the 24 specimens was analyzed statistically. The relationship between the variation in compressive strength and the number of test specimens used in the pilot study is shown by the dashed lines for two different probabilities in Fig. 7.

In order to understand the curves shown in Fig. 7, the reader is referred to ASTM Designation E122-56, which presents the recommended practice for "Choice of Sample Size to Estimate the Average Quality of a Lot or Process."

A definition of standard error and limit of accuracy as shown in Fig. 7 follows: The standard error implies that for a given number of specimens, n, the mean value of n specimens will be within plus or minus the value shown on the ordinate (Fig. 7) of the population mean 68.3 times out of 100. The population mean is obtained by testing all the samples in the universe. Similarly, limit of accuracy means that for a given number of specimens, n, the mean values of n specimens will be within plus or minus the value shown on the ordinate of the population mean 95.5 times out of 100.

It was believed that for the data to be meaningful the maximum allowable difference between the estimate of the true mean and the true mean, which would be the result of testing (by the same methods) all of the units in the universe, should be set at plus or minus 100 psi for a probability of 95.5 percent. This limiting condition was based solely on judgment.

From the pilot study curve it is seen that six 2-by 4-in. specimens fulfill the above requirement. The mixer used had a capacity such that eight specimens could be made in one batch. Based on a consideration of the above factors it was decided that eight

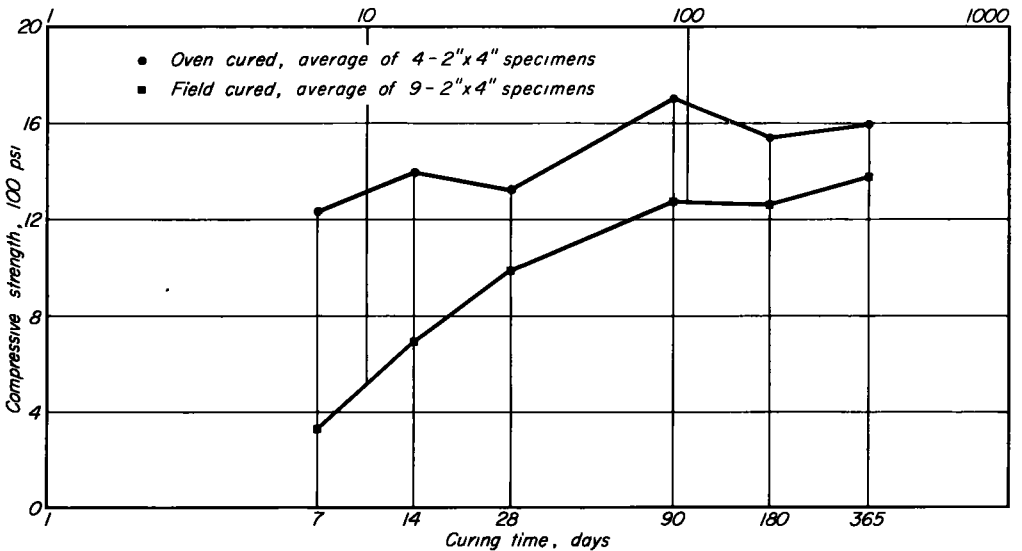


Figure 6. Relationship of compressive strength to curing time for 72.1 percent slag, 24.0 percent flyash, and 3.9 percent lime mixture.

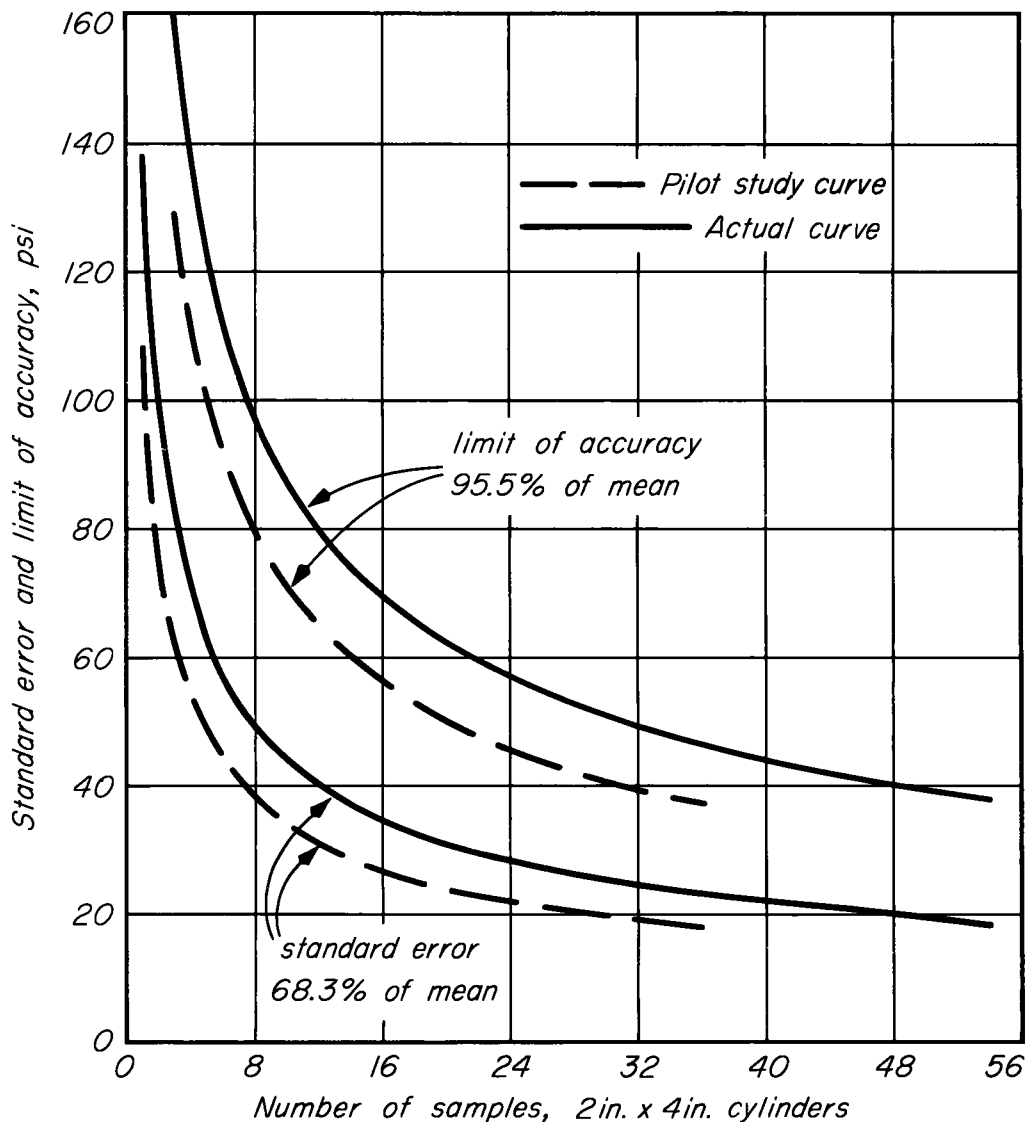


Figure 7. Standard error and limit of accuracy curves for compressive strength of 2- by 4-in. specimens.

2- by 4- in. specimens would be made to evaluate each condition.

#### Correlation of the Reliability, Probability, and Number of Test Specimens for the Actual Test Program

After completing the test program, the reliability of the pilot study was checked against the reliability of a curve which was developed from an analysis of all the routine test specimens. Figure 7 shows both the original pilot study curve and the actual curve. It is interesting to note that the actual curve based on an analysis of 54 specimens has a slightly larger variation than the computed curve. For eight specimens the pilot study curve indicated that the mean compressive strength would be within plus or minus 80 psi of the population mean, whereas, the actual curve indicates that the average of eight specimens would be within plus or minus 100 psi of the popu-

lation mean. Somewhat less care used in the routine testing may be responsible for this variation. It was fortunate that the average of eight 2- by 4-in. specimens satisfied the initial criteria.

### SIGNIFICANCE OF TEST RESULTS

A statistical analysis of the actual data based on ASTM Designation E122-56T showed that the average compressive strength of eight 2- by 4-in. specimens was with  $\pm 100$  psi of the population mean 95.5 times out of 100. It follows that a significant difference for a probability of 95.5 percent of the time requires that the average compressive strength of eight specimens differ by more than 200 psi from the average of another set of eight 2- by 4-in. specimens. The reason for the significant difference being 200 psi is that it is possible for the average of eight 2- by 4-in. specimens to be 100 psi above the population mean and the average of another set of eight 2- by 4-in. specimens to be 100 psi below the population mean. This information makes the task of analyzing the test data a relatively simple matter.

### SUMMARY

This paper sets forth an approach which may be used in determining whether new materials possess qualities that will enable them to be used satisfactorily in the ever expanding highway field. First, it is necessary to evaluate all of the known variables that may affect the specific material under consideration. After this has been done and all of the variables have been standardized there will still be some variation in the test results due to uncontrollable variations or unknown variables. These variations can be reduced to a known value by means of a pilot study and a statistical analysis. The exact amount of the variation can be either raised or lowered by decreasing or increasing the number of test specimens used in the evaluation of each item. Thus, to obtain significant data within a given range for a given probability it will be necessary to test a given number of specimens.

By use of this technique researchers may avoid to a large extent the pitfall of drawing conclusions from data based on too few test results. This will also help eliminate the embarrassing situation of subsequently finding that the conclusions are erroneous because all of the variables were not considered.

### *Discussion*

**B. B. CHAMBLIN, JR.**, Highway Research Engineer, Virginia Council of Highway Investigation and Research, Charlottesville, Virginia—This application of statistical techniques to a practical investigation into the compressive strength of slag, lime, flyash mixtures illustrates the advantages gained in the interpretation of test data when these techniques are applied to experimental design.

The same principles are useful in many other fields of testing, some of which are: concrete beam and cylinder work; California Bearing Ratio soil comparisons; field density measurements; and routine control of research test specimens. Most laboratories are faced with a variety of possible applications of "statistics."

When a number of supposedly identical specimens are tested for some characteristic, the test results are seldom identical; they are distributed about a mean value, some values larger, some smaller. If this distribution can be assumed "normal" (bell shaped), and it usually can, two statistics can be computed. They are: (a) Mean of test values and (b) standard deviation of values. These two statistics, and the number of samples tested, completely determine the characteristics of the distribution, and should be reported with the data. From these, other useful measures can be computed. Some are: (a) Coefficient of variation and (b) confidence limits for true mean. The "standard error" and "limit of accuracy" used by Hollon and Danner are forms of the confidence limit.

The writer's use of confidence limits is best illustrated by an example. Assume this hypothetical data, normally distributed, of strength values from cylindrical specimens.

1. Number of samples tested  $N = 24$
2. Mean of results  $\bar{X} = 1,600$  psi
3. Standard deviation  $s = 50$  psi

It can be seen that this sample data can be utilized in making assumptions about the population (or true parameters). The relationship between sample mean,  $\bar{X}$ , and true mean,  $\mu$ , is given by this equation.

$$\mu = \bar{X} \pm \frac{ts}{\sqrt{N}} \quad (1)$$

where "t" is a standard score computed for various degrees of certainty. It is found in tabular form in most statistical references. At the 95 percent confidence level, for 24 samples equals 2.07. By Equation 1,

$$\mu = 1600 \pm \frac{(2.07)(50)}{4.90} = 1600 \pm 21.1$$

This suggests that the true mean value of cylinder strength lies between 1,580 and 1,620 psi. If this statement is made, the probability of its being correct is 95 in 100. This probability was controlled when t was selected from the tabled values.

In this same manner, by working backwards, the number of samples necessary to insure a certain width of confidence limits may be calculated. Solving Equation 1 for sample number,

$$N = \left( \frac{ts}{\mu \pm \bar{X}} \right)^2$$

( $\mu \pm \bar{X}$ ) is selected, t is determined from tables, and s is found by testing a number of nominally identical samples.

It is hoped that this discussion will amplify the presentation by Hollon and Danner, and that papers such as this will lead to the wider use of statistical techniques by testing engineers throughout the country. The assignment of definite numerical levels of confidence to conclusions drawn from testing program results is certainly a positive step in the advancement of experimental projects.

**CLOSURE, George W. Hollon and Ellis Danner**—We wish to thank Mr. Chamblin for his discussion of our paper and we appreciate his support of the concepts which we presented.



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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.

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