HIGHWAY RESEARCH BOARD

Bulletin 98

# *Stabilization of Soils*

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## Bulletin 98

# *Stabilization of Soils*

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# Contents

# Effect of Petrographic Variations of Southwestern Iowa Loess on Stabilization with Portland Cement

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● LOESS, the dominant surface deposit in Iowa and in areas of adjoining states, is the first soil material to be studied for stabilization purposes by the Iowa Engineering Experiment Station. In general, loess areas are areas where shortages of aggregate exist or are expected, and a method of loess stabilization is highly desirable. From a scientific point of view loess is an excellent material for study because preliminary studies show that its properties vary gradually over wide areas, and the effects of these variations on stabilization problems may be evaluated.

The largest deposits of loess in North America are Wisconsin in age and are located in the southwestern Iowa area shown in Figure 1. The detailed property and stabilization studies are being made on loess from this area. By means of similar, somewhat-less-detailed studies, the loesses in other areas of Iowa will be compared to the southwestern Iowa loess so that their behavior may be predicted to some degree.

Properties of the southwestern Iowa loess vary in general, from west to east: The loess becomes finer textured and contains more clay, the carbonate content decreases, and the thickness of the deposit diminishes. Figure 2 is a tentative map showing the areal variations in clay content. On the basis of data from the loess-property-variation studies (1, 2, 3) four loess samples were chosen as representative of variations in the Wisconsin loess of southwestern Iowa. The sample locations are shown in Figure 1, and a brief description of each sample is given in Table 1.

## PETROGRAPHIC METHODS

### Particle Sizes

Mechanical analyses were performed on the four loess samples by hydrometer and pipette methods which have been described (4, 5). Sodium metaphosphate was used as the dispersing agent.

### Mineralogical Composition

Silt Fraction. Each loess sample was dispersed in water and separated into size fractions with a rising-current elutriator described in Appendix A. Samples from each size fraction above 5 microns were then separated into light and heavy minerals by allowing the grains to float or sink on bromoform (sp. gr. 2.87). Because the particle sizes are so small, centrifuging with a special centrifuge tube, described in Appendix B, was used to speed the separations. Light minerals were mounted in Canada balsam, heavy minerals in piperine, and identifications made under a petrographic microscope. Mineral percentages were determined by a statistical measurement of grains made with a Leitz integrating stage.

Clay Fraction. Minus-2-micron material was separated from each loess sample by removing a layer from a suspension and drying (6). The clays were then subjected to a number of determinative tests, including differential thermal analysis, X-ray diffraction, clay mineral staining tests (7), cation exchange determinations, particle-size measurements, and measurements of optical properties. The test methods and data on the loess samples have been recently summarized (8).

### Grain Shape and Roundness

Sphericity is an expression for grain shape and was estimated for individual grains by visual comparison with charts showing grains of known sphericity (9). The charts are based on Krumbein's (10) formula that sphericity is equal to grain width divided by grain length. Averages for a number of grains in each loess fraction were calculated.

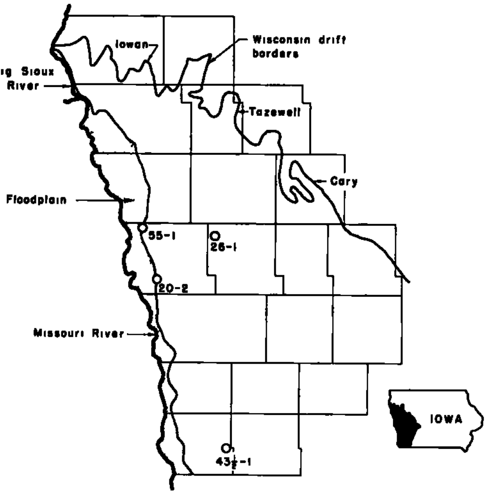

Figure 1. The Wisconsin loess area in southwestern Iowa, showing locations of the four samples selected for detailed studies.

Roundness refers only to the relative sharpness of grain corners and was noted qualitatively for a number of grains.

## Chemical Tests

Chemical tests performed on the four loess samples include the following: (1) cation exchange capacity, by an ammonium acetate method (11, 12); (2) exchangeable cations, determined with a flame photometer; (3) pH, by an electrometric method (15g soil in 30 ml water); (4) carbonate content expressed as percent $CaCO_3$, by a versenate method; (5) sulfate content, by a water-soluble sulfate determination (13); (6) chloride content, by Mohr's method; (7) free iron oxide content, by Jeffries' method (14); (8) total iron content (15); and (9) organic matter content, by a dichromate oxidation method (13).

## PETROGRAPHIC DATA

### Particle Sizes

Particle-size accumulation curves for the four loess samples are presented in Figure 3. Curves for other Wisconsin loess samples from southwestern Iowa are similar in shape, and most would fall between the limits shown.

### Mineralogical Composition

Non-clay minerals. Mineralogical compositions of the four loess samples are shown graphically by means of histograms in Figure 4. In these histograms the areas are proportional to relative percents by volume of the various minerals in each size fraction. The mineralogical compositions are summarized in Table 2.

TABLE 1

BRIEF DESCRIPTION OF THE FOUR LOESS SAMPLES SELECTED FOR DETAILED STUDY

Increasing Clay ↓

| Sample No. | Description | Geological Age | Engineering Classification |
|---|---|---|---|
| 55-1 | A very friable loess (low clay and high carbonate contents). Sampling depth 2½-3½ ft | Upper Wisconsin (?) | A-4 (8) |
| 20-2 | A typical friable loess, higher in clay than in the above sample Total loess thickness over 100 ft. Sampling depth 39-40 ft. | Wisconsin (Undifferentiated) | A-4 (8) |
| 26-1 | A typical medium-textured loess It is leached from the surface, but the underlying calcareous loess is not so calcareous as the friable samples listed above. Total loess thickness 30-40 ft. Sampling depth 4-5 ft. | Wisconsin (Undifferentiated) | A-6 (9) |
| 43½ -1 | A typical moderately plastic loess The entire loess section is leached Total loess thickness 15-20 ft Sampling depth 5½-6½ ft | Wisconsin (Undifferentiated) | A-7-6 (18) |

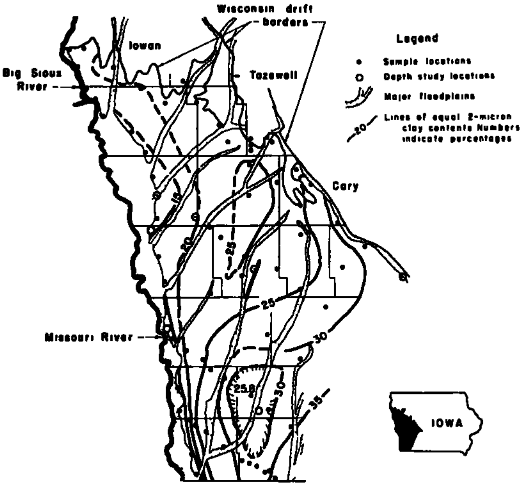

Figure 2. Tentative map showing areal distribution of clay contents in the Wisconsin loess of southwestern Iowa.

In general, as shown in Figure 5, the percentages of heavy minerals increase with decreasing grain size, probably due to sorting action during deposition of the loess. (Heavy mineral grains are more difficult to transport than light mineral grains of the same size.) The exception is in the sand fraction (74-115 microns), which forms a minor part of each sample and contains much secondary iron.

Clay Minerals. Clay minerals occur in the minus-5-micron material and are much-more difficult to determine quantitatively. The clay minerals are thought to occur mainly as minus-2-micron material, and determinations were made on this fraction. The results and interpretations of the various tests are presented by Davidson and Handy (8), and are summarized in Table 3. Differential thermal curves are shown in Figure 6; the large initial peaks indicate montmorillonite, but the 700-deg. montmorillonite peak is very much subdued compared to the 550-deg. illite peak. This may indicate a randomly interstratified mixture of the two types of minerals.

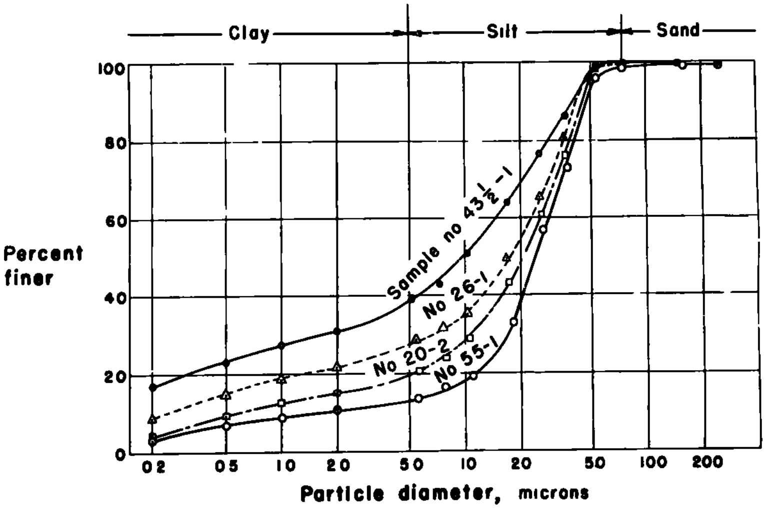

Figure 3. Particle-size accumulation curves for the four selected loess samples.

Nontronite, a montmorillonite mineral high in iron, gives a 550-deg. endotheric reaction similar to that of illite. Although the iron content determined chemically in the clay is high, as discussed later, it is probably not high enough to suggest other than an iron-rich beidellite as the montmorillonite mineral.

Only the X-ray-diffraction data show any systematic variations in clay mineral composition. Nor could any correlation be found between these variations and the particle-size data: a further indication of an interstratified relationship. The X-ray data indicate an increase in the montmorillonite: illite ratio with increasing clay content.

## Clay-Silt Relationships, Grain Coatings

Microscopic examination of the loess revealed that clay occurs mainly as coatings on larger grains, which tends to increase the effectiveness of the clay. The coatings are of two types: minute greenish specks adhering either individually or in clusters to the host grain (Figure 7) and as continuous, greenish, birefringent coatings partially covering the surfaces of larger grains (Figure 8). The lower photomicrograph of Figure 8 shows the birefringence of the clay coating. Since clay mineral grains are highly birefringent only when viewed on edge, they are apparently oriented flat against the host grain.

Grain coatings on the four samples are summarized in Table 4. The estimated extent of coatings on the host grains is given but is, of course, only an indication of the differences between samples, since the estimates apply only to coatings visible under the microscope.

## Sphericity and Roundness of Silt Grains

Sphericity is an index of grain shape; roundness is a measurement of the rounding of corners. The sphericity is probably most important for engineering applications, and as measured here it indicates the intermediate particle diameter divided by the maximum particle diameter (10). As seen in Figure 9, there is a range in

TABLE 2

MINERALOGICAL COMPOSITION OF MATERIAL LARGER THAN 5 MICRONS IN THE FOUR LOESS SAMPLES
(Percent by Volume of the Whole Sample)

| Sample No | 55-1 | 20-2 | 26-1 | 43½-1 |
|---|---|---|---|---|
| Total quartz | 41 | 45 | 48 | 45 |
| Undifferentiated qtz | 37 | 38 | 40 | 40 |
| Iron-oxide coated[a] | 1 | 2 | 4 | 3 |
| Clay coated[a] | 1 | 1 | 2 | 1 |
| Chert | 2 | 4 | 1 | 2 |
| Total feldspar | 25 | 17 | 16 | 15 |
| Undifferentiated feld. | 19 | 9 | 10 | 10 |
| Plagioclase | Trace | 2 | Trace | Trace |
| Microcline | 1 | Trace | Trace | 0 |
| Altered feldspar | 6 | 5 | 6 | 5 |
| Calcite | 8 | 11 | 1 | Trace |
| Volcanic glass | 2 | 1 | 1 | Trace |
| Muscovite | 1 | Trace | 1 | 1 |
| Biotite | Trace | Trace | Trace | Trace |
| Clay minerals (aggregates) | 1 | Trace | Trace | 1 |
| Total heavy minerals | 5.6 | 4.9 | 4.0 | 3.1 |
| Amphiboles | 2.0 | 1.8 | 1.4 | 1.2 |
| Pyroxenes | 0.8 | 0.6 | 0.4 | 0.5 |
| Dolomite | 1.3 | 1.1 | 0.0 | 0.0 |
| Iron oxides | 1.1 | 1.0 | 1.1 | 0.9 |
| Others | 0.4 | 0.4 | 1.1 | 0.5 |
| Minus 5 micron material | 13.0 | 20.0 | 27.8 | 39.0 |

[a] After dispersion, elutriation, and bromoform separation

sphericities in any sample, and these may be averaged. Average sphericities for the four loess samples are the following.

| Sample No. | Sphericity |
|---|---|
| 55-1 | 0.76 |
| 20-2 | 0.76 |
| 26-1 | 0.76 |
| 43½-1 | 0.77 |

The sphericity was also found to be constant in the various particle size ranges above 5 microns. From this it may be concluded that grain sphericity of the loess is not a variable, and its effect on engineering properties cannot be evaluated in this study.

Roundness of the grain corners was estimated, and no variations among the four samples were found. Angular or subangular grains predominate.

## Chemical Data

Chemical data which may affect the performance of loess as a construction material are summarized in Table 5. The cation exchange capacity is directly related to the clay content, and carbonates contribute slightly to the exchange capacity. Calcium is the principle adsorbed cation. Calcium and magnesium were not separated in the analysis, but a spot test failed to reveal any magnesium.

The pH of the loess samples is related to the carbonate content, the calcareous samples being slightly alkaline and the leached samples being near neutral. The carbonate contents from chemical tests show close agreement with microscopic data and serve as a check. It should be noted that the samples identified in the field as leached (26-1 and 43½-1) contain slight amounts of carbonates. Although calcium and magnesium carbonate were not separated chemically, the microscopic data indicate that dolomite (calcium-magnesium carbonate) constitutes some 10 percent of the carbonates present.

TABLE 3

RESULTS AND INTERPRETATIONS OF CLAY MINERAL TESTS ON MINUS 2 MICRON MATERIAL FROM THE LOESS

| Sample No. | Amount of Minus 2 Micron Clay, % | Staining Tests | X-Ray Analyses[a] | | | | Cat. Exch. Cap., m e./100g | | Differential Thermal Analyses | Optical Tests |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | Montmorillonite | Illite | Kaolinite | Quartz | | | | |
| 55-1 | 10.5 | Montmorillonite | Abundant | Almost equal to montmorillonite | Small amount | About 10% | 59.3 | All indicate that montmorillonite predominates | All indicate montmorillonite and illite The 700°C montmorillonite reaction is largely masked, and no variations between samples are shown | All indicate mixture of minerals |
| 20-2 | 15.4 | " | Abundant | About ⅓ the abundance of montmorillonite | Trace? | About 10% | 58.6 | | | |
| 26-1 | 21.5 | " | Abundant | Slightly less than in Sample 20-2 | Small amount | About 10% | 62.8 | | | |
| 43½-1 | 31.4 | " | Dominant | Contains very little | Absent | About 10% | 59.9 | | | |

[a] Analyses and interpretations by R E Grim

Soluble sulfates and chlorides are important, because of their possible deleterious effects on some stabilizing agents. They are absent or occur only as traces in the four loess samples.

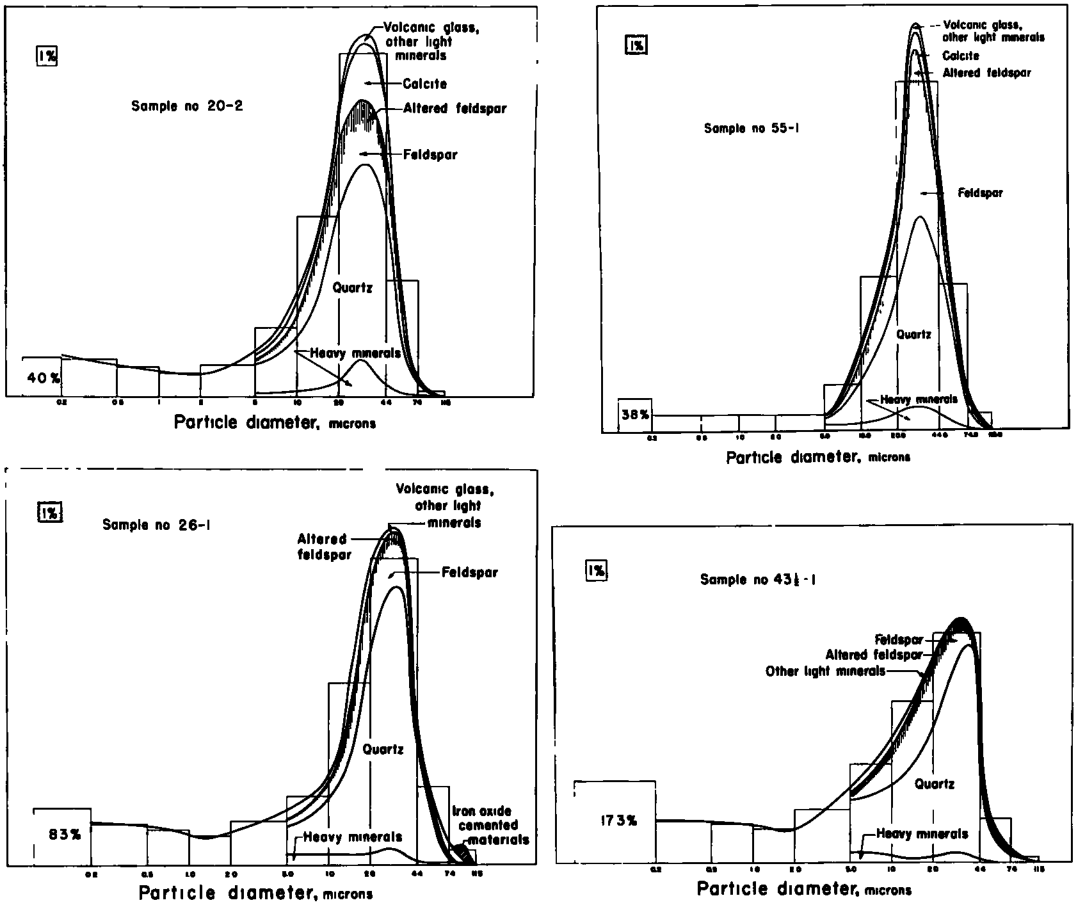

Figure 4. Mineral composition diagram for samples. Areas in the diagram represent mineral percentages by volume.

Free iron-oxide contents determined chemically correlate well with the microscopic data for iron-oxide-coated quartz, as is shown in Figure 10. The correlation between free iron and total iron is not close, and they are apparently unrelated. The free-iron content is greatest in the loess where there is a source of iron from

TABLE 4

GRAIN COATINGS IN THE FOUR LOESS SAMPLES

| Sample No. | Occurrence of clay particles | Est. % host grains are coated | Kinds of host grains | Other coatings |
|---|---|---|---|---|
| 55-1 | As individual particles and clusters of particles adhering to large grains. There are also a few continuous clay coatings. | 0 to 80, averaging 17.2. | Quartz, feldspars (altered and fresh), volcanic glass Clay coatings are not common on calcite or heavy mineral grains. | -- |
| 20-2 | Same as in sample 55-1. | 0 to 80, averaging 18 5. | Same as in sample 55-1 Some slightly coated calcite grains were noted | -- |
| 26-1 | Same as in sample 55-1, but with a greater number of continuous clay coatings completely surrounding the host grains | 0 to 100, averaging 23.2 | Same as in sample 55-1. | Red-brown Fe oxide coatings Host grains 0 to 100% coated, av. about 5% |
| 43½-1 | Same as above, also dispersed as individual particles and in large aggregates which are dominantly clay. | 0 to 100, averaging 38.0. | Same as in sample 55-1. | Red-brown Fe oxide coatings. Host grains av about 5% coated. |

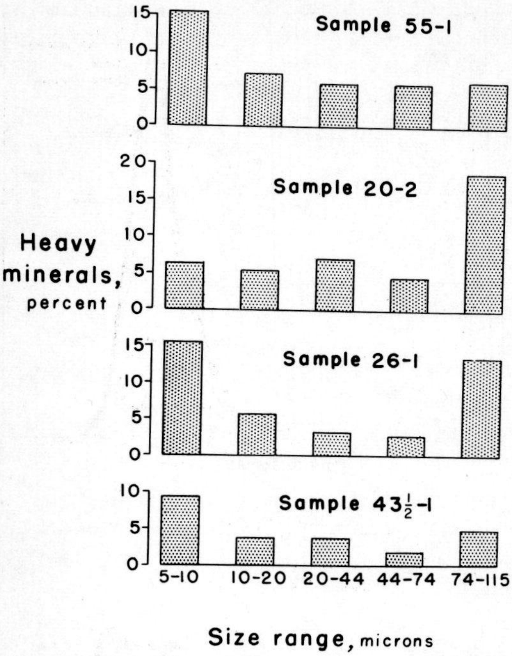

Figure 5. Percent by volume of heavy minerals in the various size fractions of each sample.

weathering in overlying soil horizons. Furthermore, secondary iron tends to become concentrated near the water table, accounting for the high free-iron content in Sample 26-1.

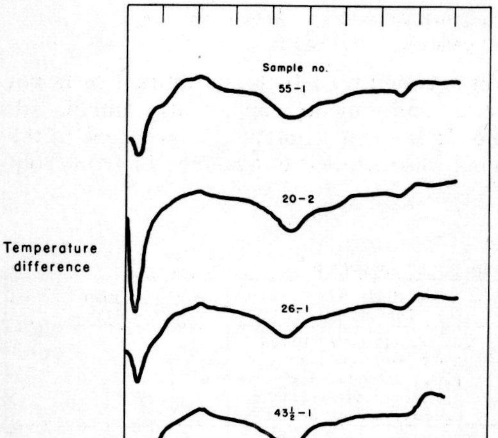

Figure 6. Differential thermal curves for minus 2 micron clay from the four loess samples.

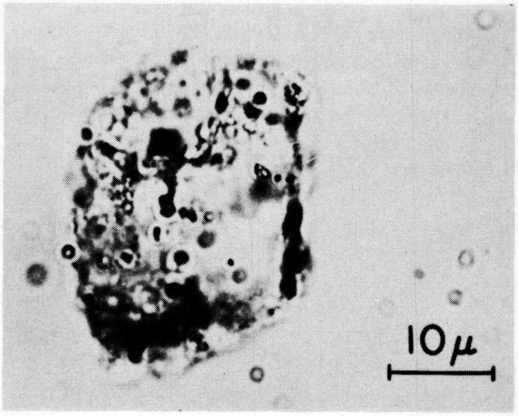

Figure 7. Photomicrograph of loess grains with particles of clay adhering.

The relationship between total iron content and heavy-mineral percentages from petrographic analysis is shown in

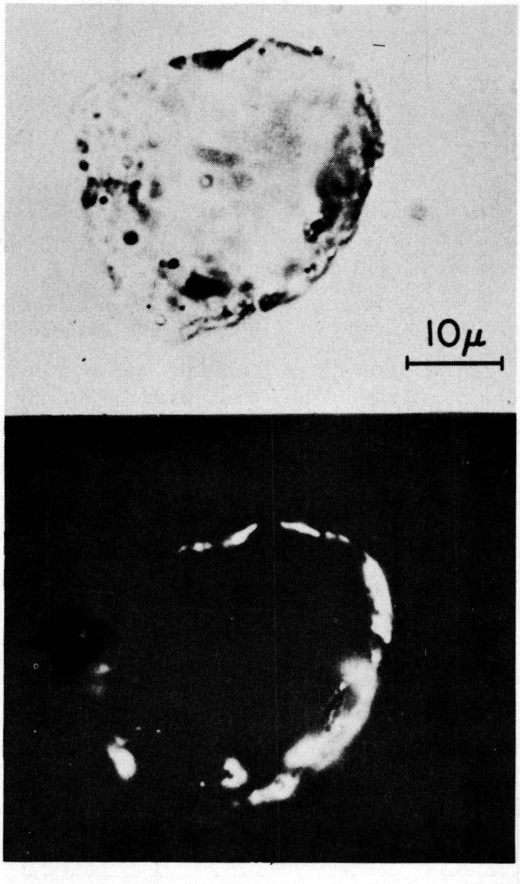

Figure 8. Continuous clay coatings on a loess grain. Above, under plane polarized light; below; under cross Nicols.

Figure 11. The correlation is almost negative, and it appears much better when the total iron is compared with the clay contents. Iron is common in the montmorillonite minerals and in the hydrous micas (illite and mixed-layer minerals).

Organic matter is present in small amounts in all samples. The content is somewhat higher in Sample 55-1, due to the presence of plant roots.

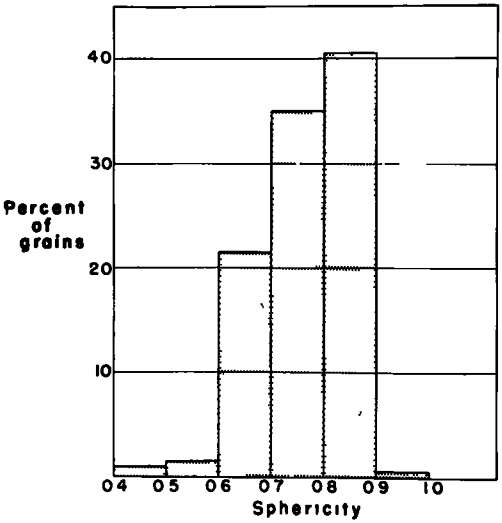

Figure 9. Histogram showing the range in sphericities of grains in a loess sample.

TABLE 5

CHEMICAL DATA ON THE FOUR LOESS SAMPLES

| Sample No. | Cat Exch. Cap., m.e./100g | Exch. Cations, m.e./100g | | | pH | CaCO$_3$, % | SO$_3$, % | Cl$_2$, % | Free Fe$_2$O$_3$, % | Total Fe$_2$O$_3$, % | Organic Matter, % |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Na | K | Ca | | | | | | | |
| 55-1 | 11.2 | 1.4 | 2.0 | 7.8 | 8.4 | 10.3 | Trace | 0.01 | 0.356 | 2.70 | 0.24 |
| 20-2 | 13.4 | 1.5 | 1.6 | 10.3 | 8.7 | 9.4 | 0 | 0 | 0.403 | 2.42 | 0.17 |
| 26-1 | 18.2 | 0.9 | 1.2 | 16.1 | 7.0 | 1.9 | Trace | 0 | 0.495 | 3.02 | 0.18 |
| 43½-1 | 24.4 | 1.3 | 1.3 | 21.8 | 6.7 | 1.6 | 0 | 0 | 0.429 | 4.82 | 0.37 |

## BEHAVIOR CHARACTERISTICS

A number of soil-mechanics tests were performed which give information on some of the behavior characteristics of the loess. These characteristics theoretically depend on such of the more-basic petrographic factors as the composition and particle sizes and shapes mentioned above. The tests now completed are listed in Table 6, together with the test methods and the data obtained. The sample numbers are arranged in the table from left to right to show the effects of increasing clay contents. All of the data show trends related to clay contents which are readily explained by the resulting increased cohesion and ability to hold water. Also, according to the X-ray data, as the total clay content in the loess increases, the percentage of montmorillonite in the clay also increases, amplifying the effect of the increase in clay content. For example, according to X-ray analysis Sample 55-1, with 10.5 percent of minus-2-micron clay, is about 5 percent montmorillonite; while Sample 43½-1, with 31.4 percent clay, contains about 30 percent montmorillonite.

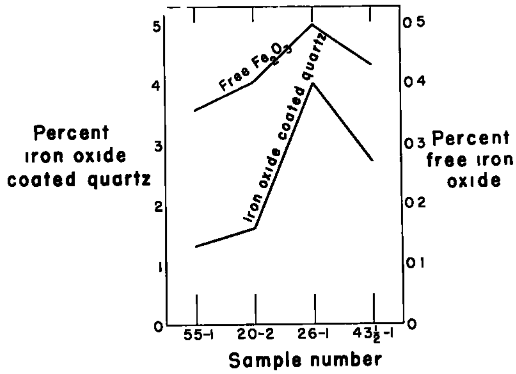

Figure 10. Comparison of data for free iron oxide determined chemically and iron oxide coated quartz grains measured microscopically.

The test for capillary rise gave some results which do not correlate directly with clay content. There is an increase in indicated capillary rise from Sample 55-1 to Sample 20-2, as expected with an increase in clay content and decrease in pore size. However, with successively higher clay contents the indicated rise is lower.

The capillary-rise test is essentially a measurement of the maximum head of water a column of initially loose, wet soil will support before air breaks through. Loess samples low in clay (55-1 and 20-2) apparently became puddled, or structureless, when put into the tube and soaked. Therefore, the finer the loess the smaller the pores and the higher the head which could be supported. Samples higher in

TABLE 6

BEHAVIOR CHARACTERISTICS OF THE FOUR LOESS SAMPLES

| Behavior Characteristics | | Sample Number 55-1 | 20-2 | 26-1 | 43½-1 | Test Method |
|---|---|---|---|---|---|---|
| Liquid limit, percent | | 29.6 | 30.8 | 39.4 | 51.9 | AASHO Method T89-49 |
| Plastic limit, percent | | 27.3 | 24.6 | 26.9 | 18.5 | AASHO Method T90-49 |
| Plasticity index | | 2.3 | 6.2 | 12.5 | 33.4 | AASHO Method T91-49 |
| Shrinkage limit, percent | | 27.2 | 22.3 | 23.3 | 19.1 | AASHO Method T92-42 |
| Standard Proctor density test | Max dry density, lb./cu. ft. | 110.0 | 108.4 | 107.0 | 104.3 | AASHO Method T99-49 |
| | Optimum moisture content, % | 17.8 | 18.0 | 17.7 | 19.1 | |
| Modified Proctor density test | Max. dry density, lb./cu. ft. | -- | 120.2 | 116.2 | 113.2 | Same as above except specimens are compacted in five layers by a 10-lb. hammer dropping from a height of 18 in |
| | Optimum moisture content, % | -- | 13.9 | 15.7 | 17.6 | |
| California Bearing Ratio of soil at standard Proctor density, % | At optimum moisture | 20.4 | 17.9 | 15.0 | 8.8 | Essentially the same as the "Suggested Method of Test for California Bearing Ratio of Soils" submitted by Corps of Engineers, U S Army to ASTM (16) except that specimens prepared are at maximum density at optimum moisture content determined by standard Proctor density tests |
| | After 4-day soaking | 15.1 | 8.6 | 4.5 | 1.5 | |
| Capillary rise, in. | | 54 | 67 | 66 | 40 | Essentially the same as the "Suggested Method of Test for Capillary Rise of Soil" submitted by Herman to ASTM. (16) |
| Engineering classification | | A-4 (8) | A-4 (8) | A-6 (9) | A-7-6 (18) | AASHO Method M145-49 |

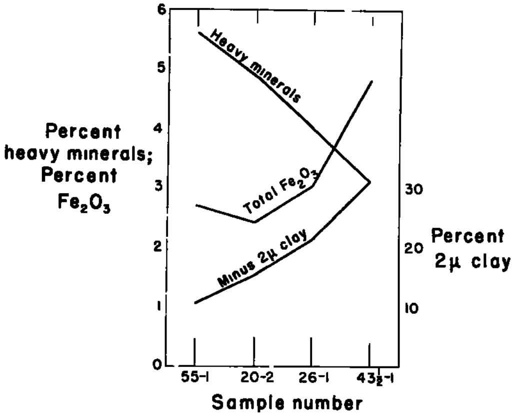

Figure 11. Relationship of total iron content to clay content in the four loess samples. The contribution of the heavy minerals to the total iron content is noticeable only in samples low in clay (55-1 and 20-2).

clay (26-1 and 43½-1) upon preliminary breaking up and sieving contained numerous granules or clusters of grains. These probably remained as granules throughout the test. The pores and air spaces between granules would be large, and less water head could be supported by such a sample. Higher clay contents would result in greater stability of the granules, due in part to lower permeability, and there would be a lower indicated capillary rise. The method of test used, therefore, succeeds at least qualitatively, probably as a result of some nonscientific good luck. It might be more reliable to measure the more basic responses, capillary potential and capillary conductivity, but these are more difficult to evaluate.

## SOIL CEMENT STABILIZATION

### Cement Requirements

Cement requirements for the four loess samples to meet Portland Cement Association criteria for soil cement were determined by Ally (17), using standard methods (ASTM D558-44, D559-44, and D560-44). The results are shown in Table 7, and the relationship between cement requirements and clay contents is shown in Figure 12.

TABLE 7

CEMENT REQUIREMENTS FOR THE FOUR LOESS SAMPLES TO MEET P C A. CRITERIA FOR SOIL CEMENT

| Sample No | BPR Soil group | P C A criteria | | | Cement requirements to meet criteria, % by volume | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Maximum soil cement loess, % | Maximum volume change, % | Maximum moisture absorption, % | Soil-cement loss | | Volume change | | Moisture absorption | |
| | | | | | Wet-dry | Freeze-thaw | Wet-dry | Freeze-thaw | Wet-dry | Freeze-thaw |
| 55-1 | A-4 | 10 | 2 | Not more than that necessary to fill voids at molding | <8.0 | 9.8[a] | 8.5 | 8.4 | <8.0 | <8.0 |
| 20-2 | A-4 | 10 | 2 | " | 8.0 | 13.5 | 10.5 | 12.1 | 10.0 | 11.0 |
| 26-1 | A-6 | 7 | 2 | " | <14.0 | 14.6 | 14.2 | 15.6 | <14.0 | <14.0 |
| 43½-1 | A-7-6 | 7 | 2 | " | 14.3 | 16.0 | 16.1 | 17.0 | 15.0 | 15.2 |

[a] Minimum cement contents which satisfy all criteria are underlined

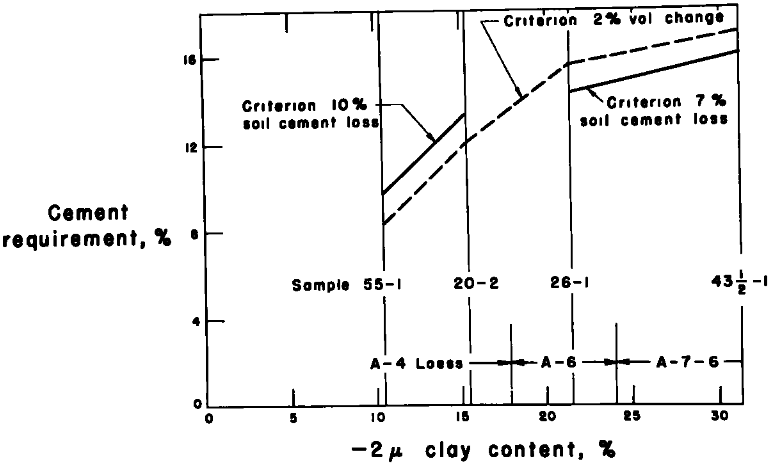

Figure 12. Relationship between cement requirement and clay content of southwestern Iowa loess. Cement requirements meet P.C.A. criteria for soil cement.

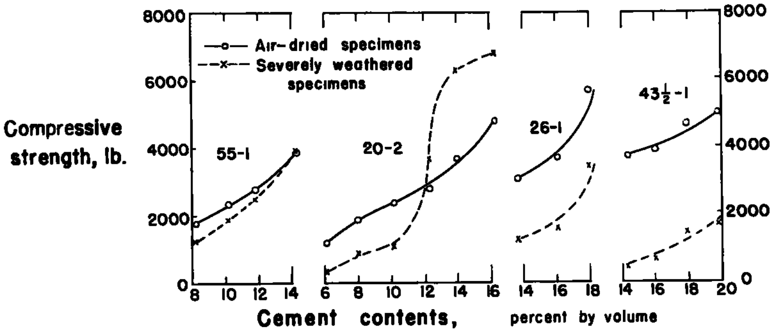

Figure 13. Compressive strengths of air-dry and severely weathered 2 inch by 2 inch soil cement cylinders containing varying amounts of cement. Air-dried specimens were moist-cured 7 days, air-dried 38 days, dried 1 day at 160 F., and tested. Severely weathered specimens were tested immediately after removal from the water bath.

## Weathering of Soil Cement

Method. Compressive strengths of 2-by-2-inch-diameter cylinders of soil cement were measured by Ally (17) after severe weathering by a procedure developed by the Civil Aeronautics Administration (18). The specimens were first moist-cured for 7 days, then subjected to the following treatments: (1) three days of capillary absorption in a moist cabinet; (2) four days of total immersion in water; (3) twelve cycles of alternate freezing at -10F. for 8 hours and thawing by total immersion in water at room temperature for 16 hours; (4) twelve cycles of alternate wetting by total immersion in water at room temperature for 8 hours and oven-drying at 160 F. for 16 hours; and (5) seven days of total immersion. The specimens were tested while wet.

Results. Figure 13 shows compressive strengths of severely weathered 2-by-2-inch cylinders compared with strengths of unweathered air-dried cylinders. Compressive strengths of both air-dried and severely weathered specimens increased with increasing cement contents. Not all specimens, however, lost strength as a result of severe weathering, and some showed a large gain. Since curves for the weathered specimens are on the basis of wet strength, and the other curves are on the basis of oven-dry strength, the gain is even greater than is apparent from the graph.

To determine if the weathering-strength gain is due to a curing effect in the weathering treatment, specimens were air-dried for periods of up to 45 days. Cement contents used were those which meet PCA minimum requirements. The strength gains with age are shown in Figure 14. Gains are greatest for Sample 20-2. This agrees with data from Figure 13 which indicate that only with Sample 20-2 was the cement content high enough to give a considerable strength gain after severe weathering. The conclusion is that the effect of severe weathering may be completely cancelled by a simultaneous cure.

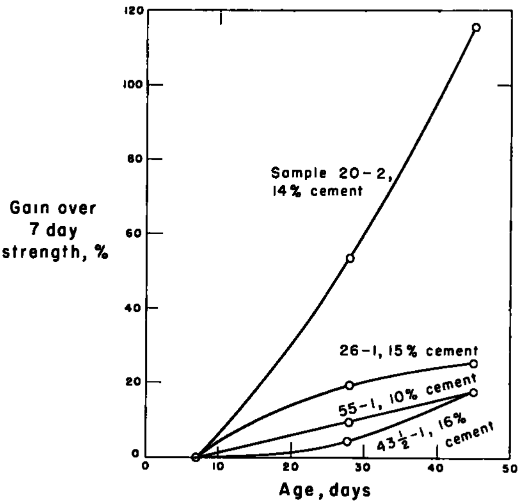

Figure 14. Gain in compressive strength of 2 inch by 2 inch soil cement cylinders by aging. The indicated age equals 7 days of moist curing plus days of air drying. Specimens were tested after drying 1 day at 160 F.

The strength gain is odd in that it is most pronounced in the sample second lowest in clay (20-2). The same behavior was shown by the sample lowest in clay (55-1), but a higher cement content was necessary. Petrographic studies showed nothing out of the ordinary in any of the samples—only the fairly continuous composition change between samples according to clay content. The possible importance of the weathering strength gain justifies further study to find reasons for the gain. Knowing these, it should be possible to determine if and how the gain may be encouraged with lower cement contents or with less-susceptible loesses.

In the present paper the reason for the gain can not yet be stated with certainty because in the loess there are several variables which may be responsible. These variables will be pointed out and discussed, and have been used as a guide in the planning of testing programs now under way. Results of these tests should show if the possibilities pointed out in this paper are correct.

# DISCUSSION

## Mineral Surface Areas

In applying petrographic data to stabilization problems, it was reasoned that fine material in a soil has a greater effect on chemical stabilization, including soil cement, than a like amount of coarse material with the same composition, since with most methods of stabilization the grain surfaces are to be coated. Therefore, in this study mineral content was expressed on the basis of surface areas in addition

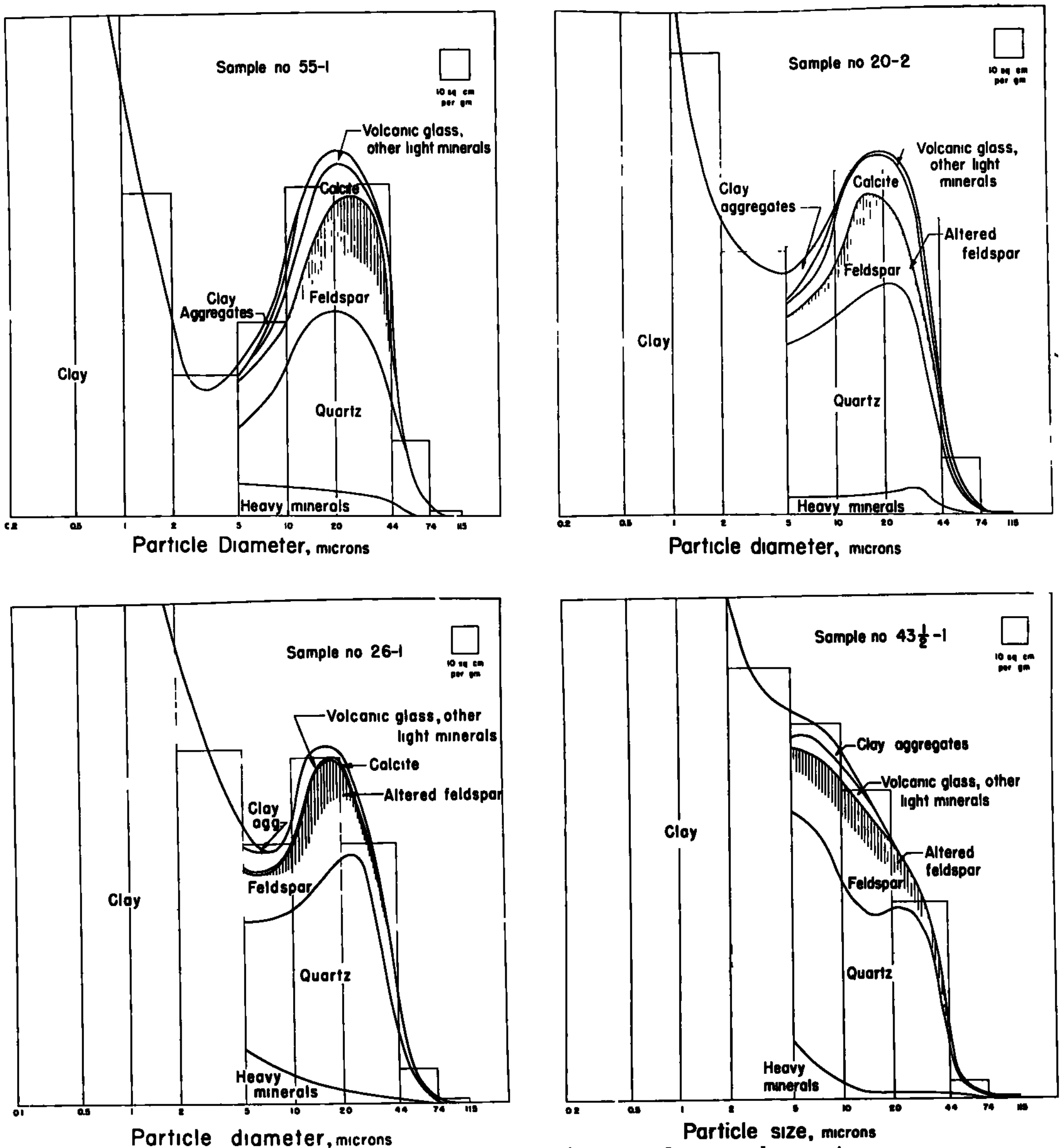

Figure 15. Mineral surface area diagram for samples. Areas in the diagram represent grain surface areas contributed by each mineral.

to the usual weight or volume percentages. Figure 15 shows histograms modified to show surface areas of grains of various minerals in each size fraction. By comparison with the standard histograms showing volume percentages (Figure 4). the new emphasis on the finer fractions may be noted. Since the composition of each sample varies in various size fractions, it is necessary to calculate and add together surface areas of the various minerals in each size fraction, thus obtaining the total area of each mineral in a sample. The areas may then be reduced to percentages, as given in Table 8. A discussion of the surface area calculations and errors involved is given in Appendix C.

## Application to Southwestern Iowa loess

It may be noted in Table 8 that the contribution of quartz to the surface area increases with increasing clay content.

Figure 16 shows the silt surface areas in the four loess samples, and an increase with increasing clay content may be noted. This is due to decreasing particle size of the silt.

TABLE 8

MINERAL SURFACE AREAS IN THE PLUS 5 MICRON FRACTION OF EACH LOESS SAMPLE

(Values given are percentages of the total sand-silt surface area)

| Sample No | 55-1 | 20-2 | 26-1 | 43½-1 |
|---|---|---|---|---|
| Quartz | 49.3 | 58.3 | 64.0 | 66.0 |
| Feldspar | 29.0 | 20.5 | 23.0 | 24.0 |
| Calcite | 9.7 | 12.8 | 1.2 | 0.8 |
| Volcanic glass | 2.5 | 1.4 | 1.1 | 0.4 |
| Heavy minerals | 8.3 | 6.3 | 7.6 | 6.3 |
| Other minerals | 1.2 | 0.8 | 2.8 | 2.5 |

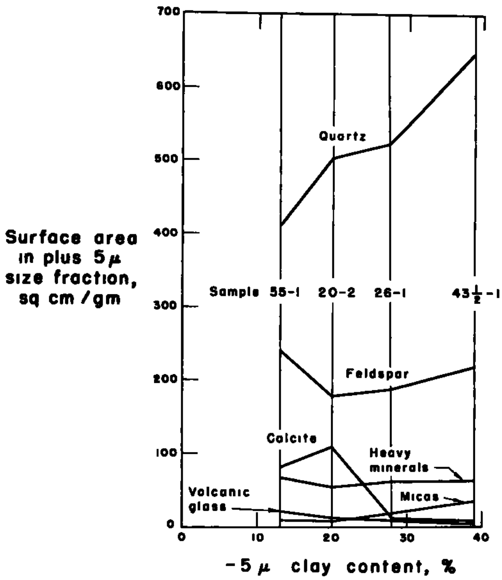

Figure 16. Mineral surface areas in the plus-5-micron-size fraction as related to the minus-5-micron clay content.

Sample 20-2, which behaved so uniquely in the CAA weathering test, displays some major differences from the other samples: It has the highest calcite ($CaCO_3$) and lowest feldspar areas, and it has a rather high quartz area in relation to its clay content.

While it is possible that the variations in weathering strength gain for different loess samples are related to reactions which affect the setting of the cement paste, the strength of the cement bonds to the loess grains appears to be the major factor, especially since loess grains are mostly clay coated. One effect of high clay content is to reduce the effective nonclay surface areas available for cementing, and it is highly possible that the effective quartz surface area approaches an optimum in Sample 20-2. This is illustrated in Figure 17, where a cube function of clay content has been subtracted from the quartz surface areas. Such a relationship has the scientific status of a guess approximated, but it may point out the correct trend. Calcite is another major variable which will be investigated, perhaps by leaching some Sample 20-2 loess and testing for a strength gain on curing. Testing to determine accurate reference curing curves is now under way.

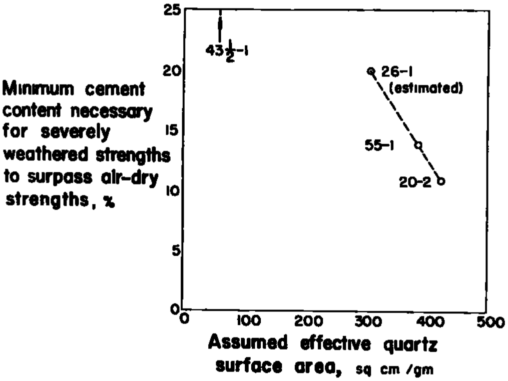

Figure 17. Cement contents at which severely weathered strengths equal air-dry strengths as related to an effective surface area calculated for quartz.

## STABILIZATION OF SOIL CEMENT WITH TREATMENTS OR ADMIXTURES

### Monoionic Treatments

Loess Sample 20-2 was treated by Nicholls (19) to obtain a monoionic soil material. The treatments used are those of Winterkorn et. al. (20): the loess is first treated with HCl and washed to obtain a hydrogen clay, and the various metallic hydroxides added either in solution or colloidal suspension and time allowed for the reaction to take place. The effects of these treatments on portland-cement stabilization of Sample 20-2 are summarized in Table 9. The most notable effect is that of iron. This may be due to replacement of the exchangeable cations by iron, or quite possibly by an excess of iron hydroxide, either coating the loess grains and making a poor bond, or coating the cement clinker grains and retarding the set (21, p. 479).

As pointed out by Grim (22, p. 143), the treatment to obtain a hydrogen clay may result in partial decomposition of the clay mineral and movement of aluminum ions to exchange positions. In such a case the aluminum would remain at the exchange position through the subsequent hydroxide treatment, and a monoionic clay would not be obtained.

## Admixtures

A number of additives to soil cement were investigated by Nicholls (19). The effects of these additives on portland-cement mixtures with Samples 20-2, 26-1, and 43½-1 are summarized in Table 10. Major changes were noted only with the addition of calcium chloride, which increased the rate of hardening, and with sodium methyl siliconate (a silicone), powdered Vinsol resin, and an asphalt emulsion sold commercially as a waterproofing agent for concrete. The silicone, the resin, and the asphalt emulsion all resulted in higher strengths of immersed specimens, probably due to waterproofing. The waterproofing effect may be considerably aided by the adsorptive power of clay mineral coatings for organic complexes.

The effects of calcium chloride are illustrated by data from Ally (17) on Samples 20-2 and 43½-1. As shown in Figure 18, with Sample 43½-1 the addition of 2 percent of calcium chloride increased 7-day and 45-day strengths about proportionally, indicating an overall strength gain instead of an acceleration effect. The same proportional increase is also shown in specimens subjected to the CAA weathering test. Whether the strength gain is due to an improvement in the character of the clay minerals, of the cement gel, or of the bonding action, is not yet known.

TABLE 9

EFFECTS OF SOME MONOIONIC TREATMENTS ON PORTLAND CEMENT STABILIZED LOESS

(Based on data from exploratory tests (19))

| Cation Used in Treating Natural Soil | Effect of Treatment on Properties of Soil Cement Mixtures |
|---|---|
| Aluminum | Effect not obvious |
| Iron | Large decrease in strength |
| Potassium | Slight decrease in strength |
| Sodium | Slight decrease in strength |

The addition of calcium chloride to Sample 20-2 with portland cement results in somewhat different trends than those mentioned above. The addition of 4 percent calcium chloride increased 7-day strengths but there was no further gain in

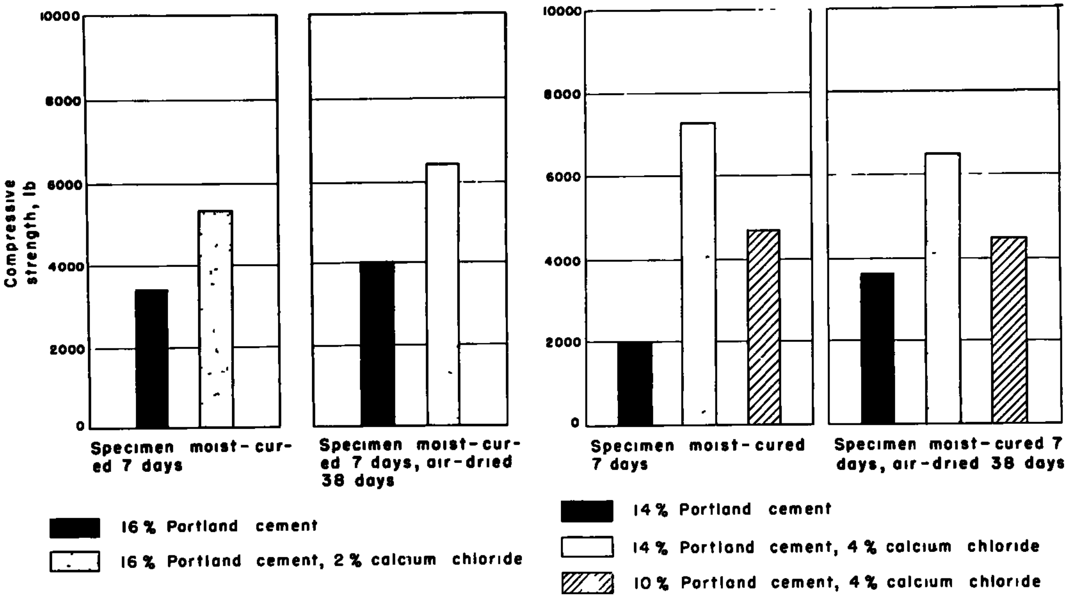

Figure 18. Effect of calcium chloride on the compressive strength of soil-cement with Sample 43½-1 (left) and Sample 20-2 (right). Specimens were dried 1 day at 160 deg. F. before testing.

TABLE 10

EFFECTS OF VARIOUS ADMIXTURES ON PORTLAND CEMENT STABILIZED LOESS

(Based on data from exploratory tests (19) ).

| Admixture[a] Added to Soil-Cement Mixture | Range of Admixture Tested, Percent[b] | Effect of Admixture on Soil-Cement Mixture |
|---|---|---|
| Calcium chloride | 0.1-6.0 | Increase in rate of hardening |
| Aluminum chloride | 0.2-1.0 | Effect not obvious |
| Ferric chloride | 0.2-1.0 | Effect not obvious |
| Lime, hydrated | 0.1-2.0 | Effect not obvious |
| Fly ash | 0.2-7.0 | Effect not obvious |
| A calcium salt of vinyl acetate-maleic acid polymer | 0.02-0.05 | Effect not obvious |
| Silicone A | 0.05-0.25 | Effect not obvious |
| Silicone B | 0.1-0.5 | Effect not obvious |
| Sodium methyl siliconate | 0.5-2.5 | Increase in immersed strength |
| Ethyl sodium silanolate | 0.05-0.5 | Effect not obvious |
| A fatty amine acetate | 0.2-3.0 | Effect not obvious |
| Vinsol resin, pulverized | 02-1.00 | Increase in immersed strength |
| Paraffin | 1 | Effect not obvious |
| A commercial waterproofing agent | 1-4 | Increase in immersed strength |
| Commercial admixture A | 0.02-0.30 | Effect not obvious |
| Commercial admixture B | 0.01-0.15 | Effect not obvious |
| Commercial admixture C | 0.04-0.15 | Effect not obvious |
| Commercial admixture D | 0.04-0.60 | Effect not obvious |
| Bentonite | 0.2-2.0 | Effect not obvious |

[a] Source and commercial name of admixtures are given in Appendix D

[b] Percentage of admixtures tested are expressed as percent of oven-dried weight of soil

strength with 38 days of additional curing. This would indicate that the major effect with Sample 20-2 is one of acceleration. Specimens without calcium chloride and subjected to the CAA weathering test showed a high strength gain, as has been discussed. With calcium chloride added, the specimens gave much lower strengths after severe weathering, indicating that the acceleration in cure may have been at the expense of the high strengths eventually obtained with cement alone. The effects of additions of smaller amounts of calcium chloride are now being studied in this relation.

## SUMMARY

Petrographic information on a soil material is essential for an understanding of stabilization involving cementing reactions, and methods have been presented by which petrography may be applied to stabilization problems. Possible explanations for various questions involving stabilization of loess with portland cement alone and with admixtures have been suggested. The validity of these interpretations will be determined by experimental work now in progress. When reactions between cement and the ingredients of soil are thus more fully understood, further work to find improvements may be carried on more efficiently. In other words, the approach might be labeled Big Think and Little Do. This can grow into Big Do. It is preferable to the ever-popular Big Do and Little Think, which is equivalent to the proverbial shot in the dark and may result in a large expenditure of ammunition for benefits obtained.

## ACKNOWLEDGEMENT

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# *Appendix A*

## A RISING-CURRENT ELUTRIATOR FOR FRACTIONATING SILTS

Separation of soils and sediments into size fractions prior to analysis is an important petrographic technique resulting in more-accurate and more-complete mineral-composition data. Coarse materials are usually separated by sieving, and materials too fine to sieve are separated by sedimentation procedures.

A rising-current elutriator is a separation device consisting of a vertical tube in which the sample is placed. Through this tube a fluid, usually water or air, flows upward at a measured velocity. Particles in the sample which are large enough and heavy enough to settle faster than the velocity of the current remain in the elutriator; smaller particles are washed or blown out at the top. The particle size at which the separation occurs may be varied by regulating the velocity and viscosity of the suspending medium.

A multiple-tube water elutriator (Figure A) was used to fractionate silts in the loess. Successive tubes have double diameters, decreasing the water velocity in each case by a factor of one fourth. According to Stokes' Law, the settling velocity of a sphere is proportional to the square of its radius: $V = Kr^2$. Since at the critical particle size in each tube the up-current equals the settling velocity, decreasing the settling velocity by one fourth means

$$V' = \frac{V}{4} = Kr'^2 = \frac{Kr^2}{4}$$

$$r' = \frac{r}{2}.$$

That is, doubling the tube diameter results in halving the particle size at which separation will occur. In the present study three tubes were used, and the water velocity was regulated so that material coarser than 20 microns diameter was retained in the first tube. This automatically resulted in material 10 to 20 microns being retained in the second tube, material 5 to 10 microns being retained in the third tube, and minus-5-micron clay being carried out in the overflow.

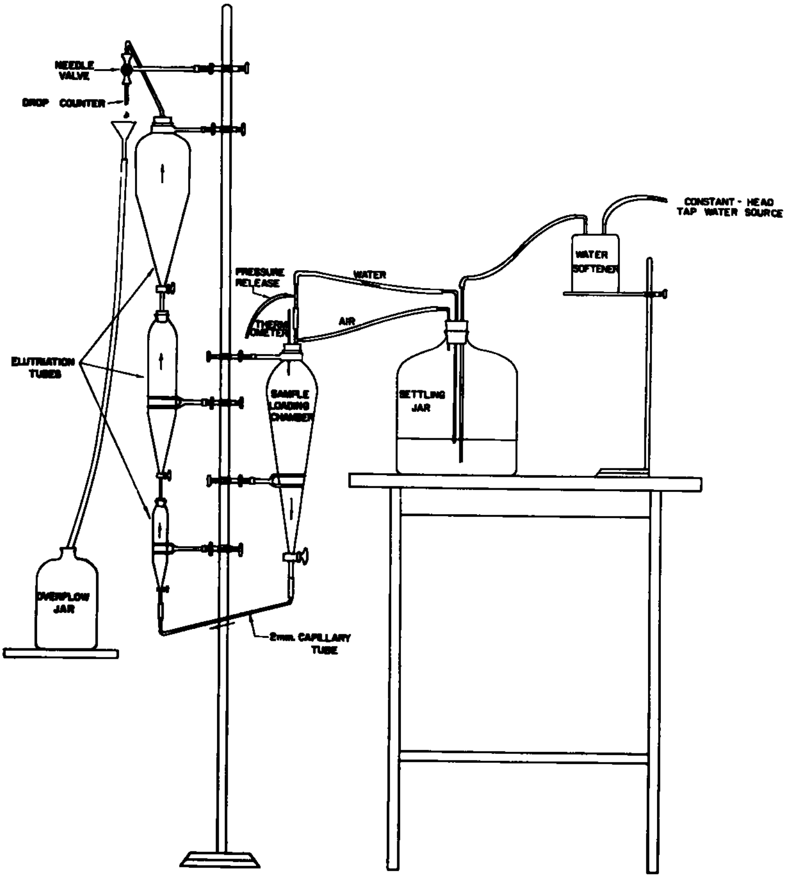

Figure A. Rising-current elutriator for the size separation of fine materials.

Dispersion of Sample

Because of flocculation, water elutriation has previously been reported as unsuccessful for separating materials finer than 10 microns, and various stirring devices have been built into the elutriation tubes (23). In the present study the loess was first dispersed as for a mechanical analysis, using a high-speed stirrer and sodium metaphosphate. Because the first tube in the elutriator is small in diameter and would have to be disproportionately long to accommodate the dispersed sample, a sample loading chamber (Figure A) was added to the apparatus.

The use of tap water in elutriation caused flocculation to occur in the third chamber and presented complete separation of the minus-5-micron clay. Distilled water also caused flocculation, probably because of dilution of the dispersing agent. The use of tap water with a small zeolite water softener was successful, the sodium in the softened water being sufficient to maintain dispersion. A settling jar was then necessary to prevent solid impurities from the water softener from entering the sample.

## Apparatus

The three elutriation tubes are made from glass stock 15/16 inches, 1⅞ inches, and 3¾ inches inside diameter, and were patterned somewhat after those of Kopecky and Anderson. The length of the taper on the tubes is about twice the inside diameter to prevent turbulence and to promote an even cross-sectional velocity. The tubes are arranged in a vertical column to prevent plugging of the connecting tubing. The sample charger is a 1000-ml. separatory funnel. It is connected to the first chamber by an inclined tube of capillary size to hold the velocity high and discourage plugging.

A 5-foot or 6-foot constant pressure head measured above the overflow is sufficient for elutriation. Flow is regulated with a needle valve located in the overflow line to maintain pressure throughout the system and to hold air in the water in solution. Otherwise air bubbles may restrict flow and cause a stoppage.

## Procedure

1. About 20 grams of soil free from material larger than 1 mm. diameter and preferably all passing the no. 325 sieve is dispersed as for a mechanical analysis, using a proportional amount of dispersing agent. If necessary, the material may be first sieved to prevent clogging the elutriator.
2. The water softener is recharged if necessary, the elutriator filled with water, and the system checked for leaks or pressure failure.
3. The input head is blocked with a hose clamp and the sample flushed into the loading chamber. Pressure is again applied and hose clamps are so arranged that air from the settling bottle is forced into the sample-loading chamber. When the chamber is almost empty, the hose clamps are changed to force water from the settling bottle into the loading chamber. The loading chamber now serves only as a passage for water into the elutriator.
4. The temperature of the inflowing water is noted on the thermometer mounted in the sample-loading chamber. The rate of flow, which is regulated by means of the needle valve, may be calibrated in number of drops in a given time.
5. Elutriation is continued until the overflow runs clear. The three stopcocks on the three elutriation tubes are then turned off simultaneously. The pressure is turned off, and the tubes are disassembled to remove the samples.
6. A complete elutriation requires about 16 hours and about 3 gallons of overflow. If at any time a pressure failure occurs, the separated materials in the tubes will settle into the lower tubes, and a longer time will be required. The process may be interrupted at any time by turning the three tube stopcocks, but it is usually more satisfactory to continue the process uninterrupted to avoid difficulties in restarting. The elutriator tends to slow down rather than to speed up after adjustment, due to air coming out of solution at the needle valve. The velocity must therefore be checked occasionally.

## Calculations

The velocity of flow may be calculated as in the following example:
From Stokes' Law,

$$V = \frac{2(d_1-d_2)\,gr^2}{9\eta} = \frac{L}{T},$$

where V is the settling velocity of the particles in cm. per sec., g is the acceleration of gravity, r is the particle radius in cm., $\eta$ is the viscosity of the settling medium, $d_1$ and $d_2$ are the specific gravities of the particles and the medium, respectively, and L is the settling distance in time T. In elutriation the time is measured with a stopwatch for a given volume of overflow. If the overflow is 10 ml., the settling distance in the large tube with radius $r_e$ = 1⅞ inches is

$$L = \frac{10}{r_e^2} = \frac{10}{71.1} = 0.1408 \text{ cm.}$$

The time T for a 10 ml. overflow is then

$$T = \frac{9\eta\,(0.1408)}{2(d_1-d_2)\,980r^2}.$$

If the separation in the large tube is to be made at 5 microns, the particle radius r = 0.0025 mm. At 68 F., $\eta$ = 0.01009 poise.

If $d_1 = 2.65$, $T = \frac{9(0.01009)(0.1408)}{2(2.65-1.00)(980)(0.0025)}$

= 63.1 seconds for 10 ml. overflow.

For convenience in timing, the rate of overflow may be converted to time necessary for 5 drops, and suitable graphs relating overflow to temperature prepared.

# *Appendix B*

## CENTRIFUGAL SEPARATION OF HEAVY FROM LIGHT MINERALS

The separation of minerals into light and heavy fractions by immersing grains in bromoform (sp. gr. 2.87) is a useful technique in mineral identification. Bromoform separation of silts presents special problems, such as the necessity for speeding separations by centrifuging.

### Apparatus

The inner and outer centrifuge tubes (Figure B) are described by Jeffries and Jackson (24). The outer tube is a 40-ml. double-thickness Pyrex centrifuge tube to withstand breakage.

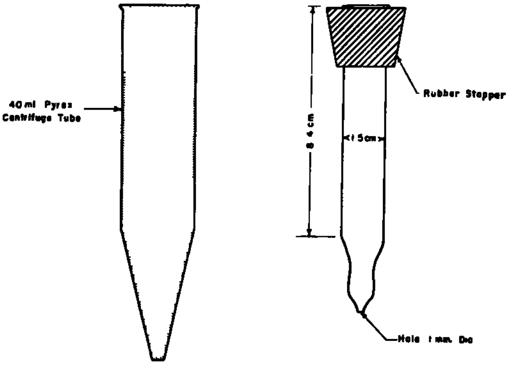

Figure B. Centrifuge tube apparatus for the bromoform separation of silts into heavy and light minerals.

### Procedure

1. A thoroughly oven-dried (110 C.) size fraction obtained by sieving or elutriation is quartered to obtain a representative sample. The fine fractions require further prolonged drying at 150 to 200 C. to prevent flocculation in the bromoform.
2. The outer centrifuge tube is filled about half full with bromoform, and the inner tube is inserted. About 1 gram of sample is placed in the inner tube and stirred. The assembled apparatus is stoppered and centrifuged at speeds not to exceed 1500 rpm. to lessen breakage. After sufficient centrifuging heavy minerals will have settled through the hole in the inner tube into the outer tube. The sample in the inner tube is stirred and centrifuged three times, or until no more heavy particles settle out.
3. The mineral separates are rinsed into filtering crucibles with bromoform, and the bromoform is filtered off with the aid of a partial vacuum. The bromoform is saved for future separation.
4. Grains in the filtering crucibles are rinsed with acetone, oven-dried at 110 C., brushed out and weighed on an analytical balance. Heavy mineral percentages may then be calculated. As the separation is seldom perfect, the percentages may be corrected after microscopic examination of the light fraction.

# *Appendix C*

## DERIVATION OF AN EXPRESSION FOR THE SURFACE AREA OF GRAINS IN A SIZE FRACTION

Surface areas of various minerals in a soil material may be calculated to aid in interpretation of stabilization data, since this emphasizes the finer, more active material.

The basic assumption used in the derivation of Stokes' Law is also applied here with the particles assumed to be spherical. Abrading is not common in silt-size material, so sphericities do not vary greatly between size fractions or between samples, and the error is more or less constant.

The mass of a spherical particle is

$$M = \frac{\pi D^3 \rho}{6}$$

where D is its diameter in cm. and $\rho$ its density in grams per cubic cm. In one gram there would be $\frac{6}{\pi D 3\rho}$ particles, each with a surface area of $\pi D^2$. The surface area in one gram of particles is therefore

$$S = \frac{6}{\pi D^3 \rho} \times \pi D^2$$

$$= \frac{6}{D\rho}$$

If d is in microns, S will be in square meters per gram.

In standard histograms the distribution of particles may be considered to be uniform throughout the size range of each bar. This is illustrated in Figure C; the narrower the bars of the histogram, the more accurate the assumption. The relationship between specific surface and particle diameter is shown in Figure D. If $D_1$ and $D_2$ are the limits of the size fraction, Sm is the mean specific surface of the fraction. In Figure D,

$$Sm\ (D_2 - D_1) = \int_{D_1}^{D_2} S\,dD$$

$$= \frac{6}{P} \int_{D_1}^{D_2} \frac{1}{D}\,dD$$

$$= \frac{6}{P} \ln \frac{D_2}{D_1}.$$

$$Sm = \frac{6 \ln D_2/D_1}{P\,(D_2 - D_1)}.$$

Example: With a 10-to-20-micron fraction of loess with a density of 2.70 gm. per cu. cm.,

$$Sm = \frac{6 \ln\left(\frac{20}{10}\right)}{2.70\ (20-10)}$$

$$= 0.154 \text{ sq. m./gm.}$$

$$= 1540 \text{ sq. cm./gm.}$$

The surface areas thus calculated for the various size fractions separated in the loess are given in Table A.

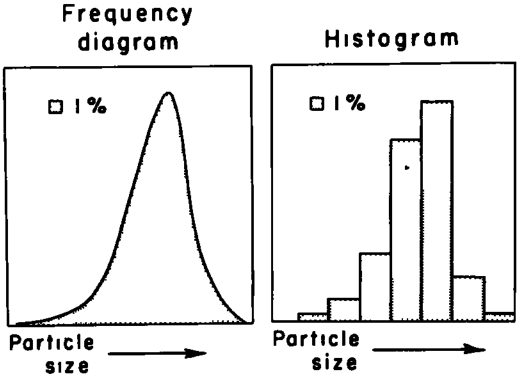

Figure C. Assumption of a uniform distribution of grain sizes in a size fraction. At left is the true representation of a soil; the graph at the right shows the assumption made.

TABLE A

SURFACE AREAS OF GRAINS AS RELATED TO PARTICLE SIZE

| Particle Diameter, microns | Surface Area in 1 gm of Material, sq cm |
|---|---|
| 0 2-0 5 | 67,900 |
| 0. 5-1 0 | 30,800 |
| 1. 0-2. 0 | 15,400 |
| 2 0-5 0 | 6,790 |
| 5 0-10 | 3,080 |
| 10-20 | 1,540 |
| 20-44 | 714 |
| 44-74 | 384 |
| 74-115 | 237 |

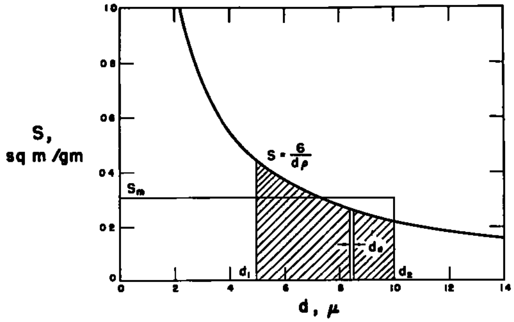

Figure D. Relationship between specific surface and particle diameter, showing integration to obtain the mean surface area of a size fraction.

# *Appendix D*

## SOURCE OF ADMIXTURES USED IN PORTLAND CEMENT STABILIZATION OF LOESS

| Material | Source |
|---|---|
| Portland cement, Type I | Hawkeye Portland Cement Company, Des Moines, Iowa |
| Lime, hydrated | Linwood Stone Products Company, Buffalo, Iowa |
| Flyash | Louisville Gas and Electric Company, Louisville, Kentucky |
| A calcium salt of vinyl acetate-maleic acid polymer, known as "Krilium No. 6 (CRD-186)" | Monsanto Chemical Company, St. Louis, Missouri |
| Silicone A, known as Linde silicone SF99 | Linde Air Products Company, Tonawanda Laboratory, Tonawanda, N. Y. |
| Silicone B, known as Linde silicone C25 | Same as above |
| Sodium methyl siliconate | General Electric Company, Chemical Division, Waterford, N. Y. |
| A fatty amine acetate, known as "Armac T" | Armour and Company, Chicago, Illinois |
| Vinsol resin, pulverized | Hercules Powder Company, Wilmington, Del |
| A commercial waterproofing agent known as "Hydropel" | American Bitumuls Company San Francisco, California |
| Commercial admixture A, known as "Pozzolith No. 2" | Master Builders Company Cleveland, Ohio |
| Commercial admixture B, known as "Pozzolith No. 8" | Same as above |
| Commercial admixture C, known as "Stearolith" | Same as above |
| Commercial admixture D, known as "Omicron Mortarproofing" | Same as above |

# Stabilization of Bank-Run Gravel by Calcium Chloride

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Specimens of different moisture contents without calcium chloride and with varying percentages of added calcium chloride were prepared. Each sample was tested for its density, its bearing capacity, and its ability to swell under saturation and shrink in open air. Two kinds of gravel specimens were prepared: one to represent the upper limits of fines, usually used in road construction; the other one was prepared by washing fines from these samples to represent the lower limit of fines. Calcium chloride was used in percentages of 0. 1, 0. 5, and 1. 0.

Four main types of experiments were conducted on washed and unwashed gravel: (1) modified Proctor tests at different moisture and calcium chloride contents; (2) California Bearing Ratio tests at different moisture and calcium chloride contents; (3) CBR-penetration tests of dried and soaked samples prepared with a constant compactive effort and with varying percentages of calcium chloride; and (4) pH tests on samples with different percentages of calcium chloride.

The addition of calcium chloride resulted in higher optimum dry densities both washed and unwashed bank-run gravel. The addition of calcium chloride will result in either a higher optimum density or an equal density at a lower compactive effort. If a high compactive effort was used, the washed material gave higher densities and higher CBR values than the unwashed material. The addition of calcium chloride had little effect under these conditions. If a low compactive effort was used, the unwashed material gave higher densities and the addition of calcium chloride had an appreciable effect. CBR values were lower for the washed sample regardless of whether calcium chloride was used. Base exchange occurs upon addition of calcium chloride, as shown by a drop in pH.

Addition of calcium chloride caused a loss in CBR values on immediate testing at low compactive efforts and at moisture contents above the optimum. However, calcium chloride caused a gain in CBR at high compactive effort and moisture contents below optimum. The addition of calcium chloride reduces the loss in CBR of soaked samples with a comparatively large percentage of fines. The CBR of the washed sample were about the same for all soaked samples regardless of calcium chloride content. Curing of specimens indicated that any loss in CBR caused by addition of calcium chloride is at least partially recovered by the curing process. Soaking of the samples lowered their CBR values. Drying the samples raised their CBR value. Drying followed by soaking gave higher CBR values than immediate soaking.

Calcium chloride is most effective on gravels containing an appreciable amount of material passing 200 mesh. The effect is minor when the fine content is below 5 percent.

● SOIL stabilization refers to the technique by which an increase in the bearing capacity of soils can be attained. Since there is no generally accepted engineering test for "stability" of soils, the magnitude of the dry density obtained by a standard procedure is often used as a measure of the degree of stabilization of the soil mass.

The moisture content and the type of soil are among the most-important factors affecting this technique. In this study, stabilization of a bank-run gravel with calcium chloride was studied, and by experimentation the effects of calcium chloride under different moisture conditions and compactive efforts were observed.

### Scope

The technique of soil stabilization has many applications in earth structures where a control of the physical properties of materials is desirable. This technique may be satisfactorily applied in the construction of bases and subbases of paved or unpaved roads and in treating the surfaces of unpaved roads.

## APPROACH TO THE PROBLEM

The nucleus of this study was directed toward the effects of calcium chloride on the clay and colloidal fractions of the gravel. Specimens of different moisture contents without calcium chloride and with different percentages of added calcium chloride were prepared. Under these conditions each sample was tested for its density, its bearing capacity and its tendency to swell under saturation and to shrink in open air.

Two kinds of gravel specimens were prepared: one to represent the upper limit of fines usually used in road construction; the other one was prepared by washing fines from this material to represent the lower limit of fines.

## THEORETICAL ASPECTS

Soils are dispersed systems which are composed of three phases: solid, liquid, and gaseous.

Solid elements can be of mineral or organic constitution and of various sizes and shapes. Their range is from coarser sizes of sands and gravels to the finer particles of silts, coarse clays, and fine clays. Particle size is related to the physical properties of soil elements. The surface activity in smaller fractions of soils is greater. This relationship establishes the clayey elements as active fractions and the coarser material as the skeleton of the soil system. The relationship of the amount of surface area per unit of mass is an important soil property which is known as the specific surface. The classification of various soil textures into groups on the basis of their effective diameters is an important measure of the chemical and physical behavior of soils. But, also, it is of special interest to know the specific surface of each of these groups. The relationship of particle size and the specific surface is important.

The surface of a colloid can be considered as "active mass" which constitutes the monomolecular surface layer. In colloids the mass ratio of total mass of a particle to its active mass is small and this mass is related to particle size and specific surface. A colloidal system is a heterogeneous system in which the surface molecules (active mass of the solid) are important and the inert mass is inactive and can be largely disregarded. The other active portion of this sytem is the dispersion medium which may be composed of water, or water plus air.

Specific surface increases as the size of the soil particles decreases. Therefore in smaller-sized particles the extent of the interfaces between particles, and their surface activity, is greater. This fact causes the plasticity of clays. Internal pores or channels increase the specific surface. Control of the surface phenomena of clays can control soil plasticity and help to stabilize the soil material.

Clay, from a chemical point of view, can be defined as a material formed from decomposition of feldspars and similar silicates. Clays are able to unite with water to form a partially gelatinous state and exert a binding action upon coarser particles in soil. Finer clays are colloidal materials in which particles smaller than 2 microns in size predominate. As colloids, clays are composed of dispersed formations of mineral or organic fragments. These fragments are essentially crystalline and rarely amorphous. Crystalline elements are built up of units of alumina and silica. The alumina unit contains two sheets of closely packed oxygen atoms or OH groups. Aluminum atoms hold three oxygen atoms or OH groups from each sheet. In the silica unit there is one sheet of oxygen atoms. Each silicon atom is surrounded by three oxygen atoms from one sheet, and one oxygen atom from the other sheet. The latter oxygen atom is also shared by the alumina sheet and helps to link the silica sheet to the alumina sheet. According to the arrangements of these sheets, there are two basic groups of clay minerals, the kaolin group and the montmorillonite group. The former type is built up of one silica sheet to one alumina sheet; the latter type of mineral consists of a crystal lattice which is composed of two silica sheets to one alumina sheet.

The binding ability of colloids makes them important cementing agents in the formation of aggregations of soil particles. Dehydrated colloidal oxides of iron and alumina are especially responsible for the stable aggregation of the lateritic soils. By dehydration, as the film of water which coats the clay particles becomes thinner, the particles approach to each other to form aggregations. Aggregation in the colloidal state is created by electric charges (potential) and by envelopes of bound water, which affect the stability of colloids.

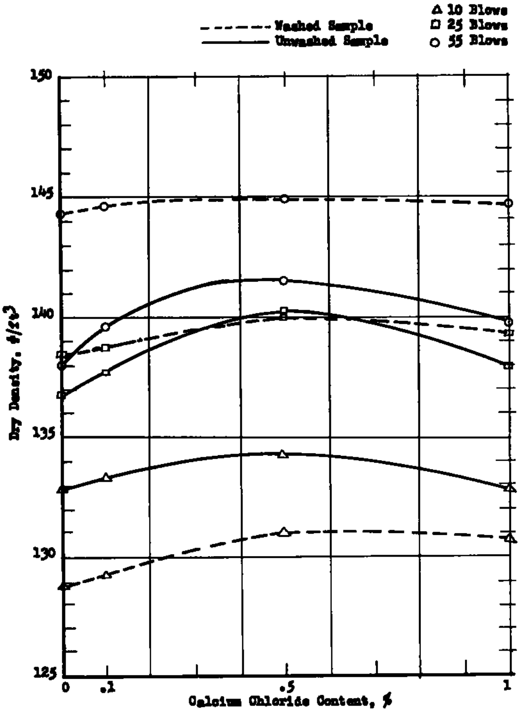

Figure 1. Optimum dry densities of bank-run gravel of various percentages of calcium chloride.

Most clay particles are negatively charged and have a tendency to migrate toward positively charged units. The migration velocity depends upon the strength of the charge. At higher potentials there is a repulsion among particles, at lower potentials the tendency toward repulsion diminishes, and after collision (resulting from Brownian movement) the particles coalesce and settle out as floccules.

The surface behavior of a solid particle in touch with water tends to create certain layers which are formed by the accumulation of differently charged particles. The solid (in the case of clay) develops a negatively charged surface formed by $OH^-$ ions adsorbed from the water, according to C. E. Marshall.

As a counterbalance, $H^+$, or other positively charged ions, will be attracted to the surface to form a positively charged layer. This arrangement creates a Helmholtz double layer, which leads to a difference of electrical potential between the solid and the $H^+$ layer. The $OH^-$ layer, being nearer to the surface, is fixed and the $H^+$ layer is weaker and movable. This original Helmholtz concept of a movable $H^+$ ion layer can be modified and two distinct types of behavior can be noticed in the $H^+$ layer. Ions nearer to the negative $OH^-$ layer can be considered fixed and others are dispersed throughout the liquid phase surrounding the solid and can be considered as a diffused layer. This allows a variation in electrical potential across the interface. The potential occurs not through the $OH^-$ layer but through the $H^+$ double layer, and it can be considered as consisting of two parts. The first part is between the surface ($OH^-$ layer) and the fixed (compensating) $H^+$ layer, the second part is between the fixed positive layer and the body of the solution. This second type of potential is the electrokinetic potential (or zeta potential) and is measured from the fixed layer to an infinite distance in the solution.

Zeta potential is largely responsible for the stability and the electrical effects observed in clay colloids. The balancing of the surface charges creates more nearly neutral surfaces; this lowers the zeta potential and the electrical effects. In the absence of the electrical charge, collisions from Brownian movement tend to lead to a greater binding action between the particles. Therefore, the isoelectric point (zero zeta potential) may be reached and coagulation between particles may set in. At this stage the clay is stabilized and will not exhibit extreme swelling and shrinkage, and the particles may be closely packed. The neutralization of the surface can be attained by introducing enough positive ions into the fixed positive layer to equal the charges between the fixed layers of $OH^-$ and $H^+$. This can be done by base exchange. A salt, such as calcium chloride, can be injected into the soil water and calcium ions will replace the $H^+$ ions in the fixed positive layer around the colloidal particle. This mechanism creates the base exchange concept in which, by replacing original surface ions by other introduced ions the surface, characteristics of a clay colloid may be altered. During this base exchange, because of the smaller ionic radii of the calcium ions as compared to hydrogen ions, it is possible to introduce more calcium than the number of H+ ions replaced. In addition, the charge of each ion is twice as great. After the critical (isoelectric) point is reached, a further accumulation of calcium ions may create an excess of positive charges in the fixed positive layer; hence a less-stable system may be created. It can be concluded that at the isoelectric point the zeta potential and the distance between particles will be a minimum. This results in more-densely packed particles in the soil and a maximum density, which is related to the bearing capacity of the soil.

TABLE 1

DENSITY VALUES AT DIFFERENT COMPACTIVE EFFORTS, WATER-CONTENTS, AND CALCIUM CHLORIDE CONTENTS

| % $CaCl_2$ | Moisture (%) | | | | | | | Compactive Efforts (Blows/layer) |
|---|---|---|---|---|---|---|---|---|
| | 3 | 4 | 5 | 6 | 7 | 8 | 9 | |
| Unwashed | | | | | | | | |
| 0.0 | | | | 131.7 | 132.9 | 130.6 | 129.6 | 10 |
| 0.1 | | | | 130.6 | 133.4 | 130.7 | 130.4 | 10 |
| 0.5 | | | | 132.4 | 134.2 | 133.9 | 129.8 | 10 |
| 1.0 | | | | 132.8 | 130.0 | 132.8 | 130.6 | 10 |
| 0.0 | | | 136.0 | 136.8 | 135.2 | 132.4 | | 25 |
| 0.1 | | | 137.5 | 137.7 | 135.8 | 132.9 | | 25 |
| 0.5 | | | 138.8 | 140.3 | 136.6 | 133.2 | | 25 |
| 1.0 | | | 137.6 | 137.9 | 136.2 | 133.6 | | 25 |
| 0.0 | 139.2 | 138.0 | 136.1 | 135.0 | | | | 55 |
| 0.1 | 139.3 | 139.5 | 136.4 | 136.2 | | | | 55 |
| 0.5 | 140.2 | 141.5 | 137.0 | 136.8 | | | | 55 |
| 1.0 | 139.7 | 139.8 | 136.8 | 136.9 | | | | 55 |
| Washed | | | | | | | | |
| 0.0 | | | 124.6 | 128.5 | 128.9 | 127.1 | | 10 |
| 0.1 | | | 124.5 | 128.6 | 129.2 | 127.3 | | 10 |
| 0.5 | | | 125.9 | 128.8 | 131.0 | 132.7 | | 10 |
| 1.0 | | | 125.0 | 128.6 | 130.6 | 129.1 | | 10 |
| 0.0 | | 131.5 | 135.1 | 138.5 | 136.8 | | | 25 |
| 0.1 | | 132.0 | 135.3 | 138.8 | 136.8 | | | 25 |
| 0.5 | | 135.3 | 136.4 | 140.0 | 137.2 | | | 25 |
| 1.0 | | 132.6 | 135.8 | 139.3 | 136.9 | | | 25 |
| 0.0 | 141.0 | 144.3 | 139.9 | 138.0 | | | | 55 |
| 0.1 | 139.9 | 144.6 | 139.9 | 137.8 | | | | 55 |
| 0.5 | 138.1 | 144.9 | 142.8 | 138.5 | | | | 55 |
| 1.0 | 140.2 | 144.7 | 141.6 | 138.0 | | | | 55 |

Soils, irrespective of their origin or source, have potential acidic properties; that is, they may contain hydrogen ions which can be replaced by other positive ions; these H+ ions can be neutralized with various alkalis to form salts. The concentration of its H+ ions in solution determines the strength of the acidity of soil. Therefore, the study of the pH value is important. The measurements can be done either by electrometric or by colorimetric methods.

## EXPERIMENTAL WORK

### Materials

The soil sample was taken from the working face of a gravel pit located about 2 miles south of Newfield, New York, just off the south side of Route 13. Material from the same pit was used by James W. Spencer of Cornell University for the construction of a test road near Newfield. This test road contained sections stabilized by calcium chloride; these sections performed well.

The gravel was screened through a $^3/_4$-inch screen; only the portion passing the $^3/_4$-inch screen (about two thirds of the entire material) was used in the experiments.

The approximate mineralogical content of the gravel portion between $^1/_4$ and $^3/_4$ inches was as follows:

| | |
|---|---|
| shale | 50% |
| sandstone | 40% |
| igneous | 5% |
| limestone and misc. | 5% |

The mineralogical composition of the clay fraction was approximately 50% illite and 50% montmorillinite, determined by x-ray diffraction.

The material as taken contained a high percentage of fines—near the upper limit of bank-run gravel for road construction. In order to provide a second sample low in fines, with all other variables remaining constant, half of the original sample was washed (stirred with water, followed by decantation). The gradation of the unwashed sample follows:

| Sieve No. | Unwashed Gravel % Passing |
|---|---|
| 4 | 62 |
| 8 | 49 |
| 14 | 39 |
| 28 | 34 |
| 48 | 24 |
| 100 | 18 |
| 200 | 12 |

After washing, the sample contained 5 percent material passing a 200-mesh sieve.

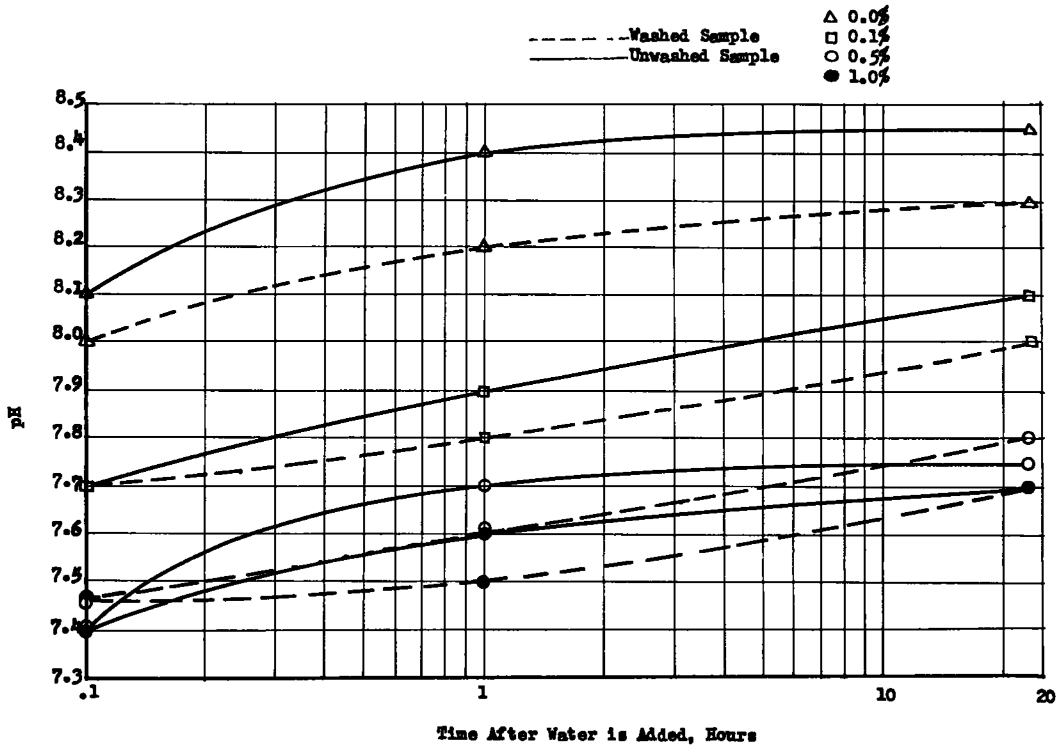

Figure 2. Changes of pH of bank-run gravel with various percentages of calcium chloride at various times after water is added.

### Equipment

(1) CBR molds, standard size: 6-inch diameter, 7-inch height; (2) spacer disc, standard size: 6-inch diameter, 2-inch height; (3) CBR mold collars, standard size: 2-inch height; (4) compacting hammer, standard modified AASHO type: 10 lb. weight, height of drop: 18 inches; (5) volume of each cylinder: 0.0818 cu. ft.; (6) hydraulic loading apparatus, 6000-lb. capacity, 3 sq. in. cross-section penetration piston; (7) surcharge weight: 10 lb.; (8) dial gauge, Ames 0.001-inch type; (9) oven, temperature controlled (50 C.); and (10) tripod for height measurements, equipped with a 0.001-inch Ames gauge.

### Laboratory Procedure

Four main types of experiments were conducted on washed and unwashed gravel: (1) modified Proctor tests at different moisture and calcium chloride contents; (2) (CBR) California Bearing Ratio penetration tests at different moisture and calcium chloride contents; (3) drying and soaking tests at approximately the optimum moisture content with a constant compactive effort and at different percentages of calcium chloride; and (4) pH tests on samples with different percentages of calcium chloride.

### Procedure for Preparing Washed Gravel

Material was placed in a tilted pan and water was constantly applied over it. By frequent stirring the suspended fines were washed away. The material was dried in open air.

### Modified Proctor Tests

The equipment consisted of the standard cylinders and the modified Proctor hammer. Each sample, at a particular moisture content, was compacted in five layers with a certain compactive effort. The compactive efforts applied were: 10, 25, and 55 blows per layer. Then the dry density at any particular moisture content was determined. The moisture contents were determined on a dry-weight basis and increments of 1.0% moisture were added until the optimum moisture of the sample was passed at any particular compactive effort. The resulting dry densities were determined.

## California Bearing Ratio (CBR) Penetration Tests

After weighings for the determination of dry densities, the sample was subjected to the CBR-penetration test and the loading intensity at each value of penetration was obtained. From the collected data CBR values for each test at each moisture content were established.

TABLE 2

CBR VALUES AT DIFFERENT COMPACTIVE EFFORTS, WATER CONTENTS, AND CALCIUM CHLORIDE CONTENTS (PERFORMED IMMEDIATELY AFTER PROCTOR TESTS)

| %CaCl$_2$ | Moisture Contents (%) 3 | 4 | 5 | 6 | 7 | 8 | 9 | Compactive Effort, (Blows/ layer) |
|---|---|---|---|---|---|---|---|---|
| | | | | Unwashed | | | | |
| 0.0 | | | | 25 | 9 | 4 | 2 | 10 |
| 0.1 | | | | 26 | 7 | 2 | 2 | 10 |
| 0.5 | | | | 16 | 7 | 4 | 1 | 10 |
| 1.0 | | | | 15 | 5 | 4 | 2 | 10 |
| 0.0 | | | 67 | 28 | 9 | 4 | | 25 |
| 0.1 | | | 78 | 38 | 6 | 3 | | 25 |
| 0.5 | | | 56 | 13 | 5 | 2 | | 25 |
| 1.0 | | | 38 | 16 | 3 | 1 | | 25 |
| 0.0 | 98 | 40 | 15 | 5 | | | | 55 |
| 0.1 | 97 | 39 | 15 | 5 | | | | 55 |
| 0.5 | 107 | 42 | 16 | - | | | | 55 |
| 1.0 | 98 | 39 | 14 | 4 | | | | 55 |
| | | | | Washed | | | | |
| 0.0 | | | 8 | 21 | 16 | 17 | | 10 |
| 0.1 | | | 16 | 22 | 33 | 16 | | 10 |
| 0.5 | | | 20 | 29 | 40 | 4 | | 10 |
| 1.0 | | | 18 | 24 | 38 | 4 | | 10 |
| 0.0 | | 53 | 54 | 76 | 25 | | | 25 |
| 0.1 | | 64 | 65 | 39 | 16 | | | 25 |
| 0.5 | | 69 | 69 | 38 | 9 | | | 25 |
| 1.0 | | 67 | 68 | 35 | 5 | | | 25 |
| 0.0 | 129<br>(.16"p) | 129<br>(.16"p) | 22 | 7 | | | | 55 |
| 0.1 | 126 | 129<br>(.18"p) | 36 | 7 | | | | 55 |
| 0.5 | 96 | 193<br>(.10"p) | 50 | 8 | | | | 55 |
| 1.0 | 92 | 129<br>(.185"p) | 34 | 7 | | | | 55 |

Note 0.2-inch penetration was used except when otherwise indicated

## Drying and Soaking Tests

Soaked Samples. These samples were prepared with 55 blows per layer compactive effort in five layers using standard CBR molds. The cylinders thus prepared were subjected to soaking in water. The soaking period was 5 days on the average. A penetration test was run on each cylinder. During the entire period of soaking, periodic checks on weight of samples and height of samples were recorded. A 10-lb. surcharge weight was used throughout the soaking and testing procedure.

Dried Samples. They were prepared the same way as the soaked samples. The drying period was about 14 hours, at which time about 30 percent of the water had been evaporated at the constant oven temperature (50 C). Height and weight measurements were recorded as in the case of the soaked samples. After the drying period the samples were soaked in water as for the soaked samples. Height and weight measurements were periodically taken and recorded. A 10-lb. surcharge weight remained on the samples during the drying and soaking periods but not used during the CBR determinations (in order that more than one CBR could be run on one sample). Previous experiments showed that, for this material, nearly identical CBR's were obtained with and without the surcharge weight.

CBR Tests. Three different kinds of CBR tests were performed on the samples which had been dried and soaked as explained above. These tests and their details were as follows: (1) Immediate CBR tests were performed on the compacted samples with various percentages of calcium chloride before they were put in the oven. (2) Tests performed immediately after drying in the oven. (3) Tests performed immediately after the soaking period. (This type of testing applies to the samples which were soaked immediately, and to the ones which were soaked after a period of drying in the oven.)

TABLE 3

OPTIMUM DRY DENSITIES AT DIFFERENT PERCENTAGES OF CALCIUM CHLORIDE

Dry Densities pcf

| %CaCl$_2$ | 10 blows | 25 blows | 55 blows |
|---|---|---|---|
| | Unwashed | | |
| 0.0 | 132.9 | 136.8 | 138.0 |
| 0.1 | 133.4 | 137.7 | 139.5 |
| 0.5 | 134.2 | 140.3 | 141.5 |
| 1.0 | 132.8 | 137.9 | 139.8 |
| | Washed | | |
| 0.0 | 128.9 | 138.5 | 144.3 |
| 0.1 | 129.2 | 138.8 | 144.6 |
| 0.5 | 131.0 | 140.0 | 144.9 |
| 1.0 | 130.6 | 139.3 | 144.7 |

## Height Measurements

The equipment consisted of a standard tripod and an Ames 1/100-inch dial. Each cylinder was carefully oriented during the measuring and the tripod always was placed at the same marked points to avoid any reading error. Also, during the measurements a possibility of any jarring of the perforated cylinder plates made it necessary to record the height immediately and again after the cylinder was put in its resting place following weighing: the following height measurement was based on the latter reading.

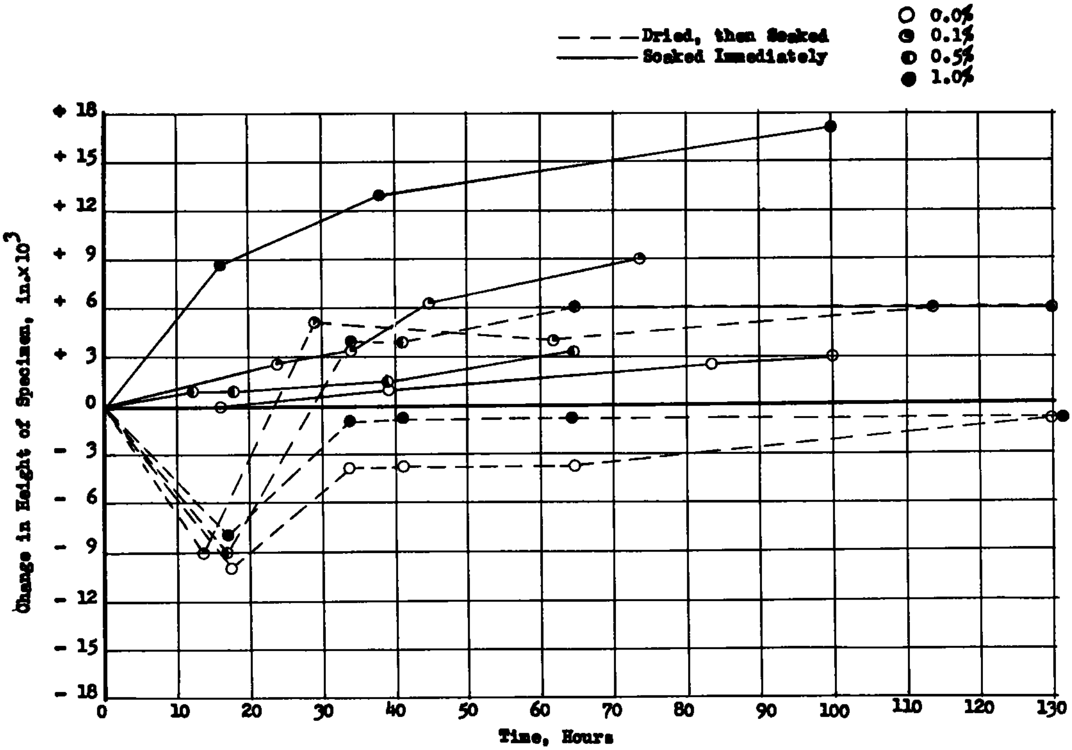

Figure 3. Shrinking and swelling of unwashed bank-run gravel with oven drying and soaking at various percentages of calcium chloride.

## Weight Measurements

Periodic determinations of weight were made during the drying and during the soaking periods.

The pH tests were conducted on the samples at different percentages of calcium chloride at various time intervals after addition of the calcium chloride. An electrometric type of pH meter (glass electrode) was used.

Measurements were made of changes in moisture content and height of specimens in all four concentrations of calcium chloride, for both washed and unwashed gravel, during the soaking period and during a cycle of drying and soaking. The results are shown in Figures 3, 4, 5, and 6.

## OBSERVATIONS

Density tests were conducted on the same sample at four different moisture contents, chosen according to the compactive effort used. Water was added after each test to make up the necessary moisture for the next test. The ideal approach, in the case of abundant soil material, would be to prepare a fresh sample for each different moisture con-

TABLE 4

CBR VALUES FOR SOAKED AND DRIED SAMPLES

| % CaCl₂ | Immediate | Oven-Dried | "Soaked" After Oven-Dried | "Soaked" |
|---|---|---|---|---|
| Unwashed | | | | |
| 0 0 | 16 | 113 | 39 | 5 |
| 0 1 | 14 | 111 | 40 | 8 |
| 0 5 | 36 | 111 | 47 | 8 |
| 1 0 | 9 | 44 | 20 | 22 |
| Washed | | | | |
| 0 0 | 81 | 129 ( 225"pen) | 68 | 35 |
| 0 1 | 73 | 129 ( 225"pen) | 67 | 34 |
| 0 5 | 71 | 127 | 67 | 34 |
| 1 0 | 53 | 119 | 64 | 26 |

Note All results are at 0 2-inch penetration unless otherwise indicated

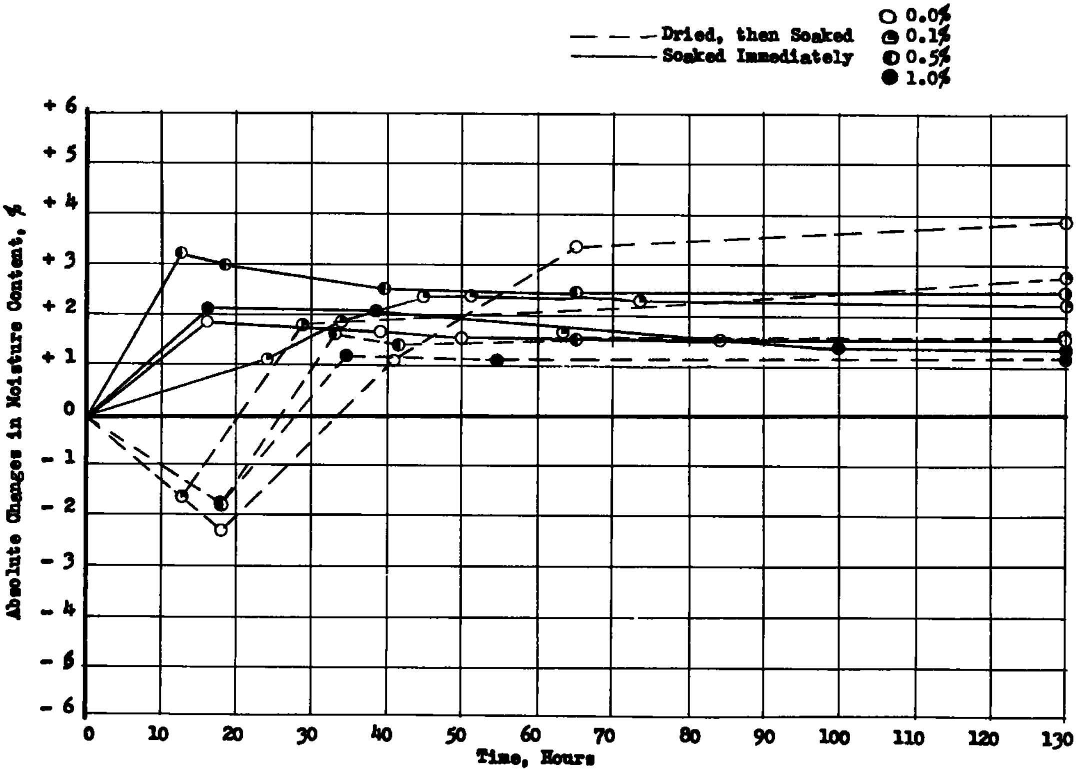

Figure 4. Change of moisture content of unwashed bank-run gravel with oven drying and soaking at various percentages of calcium chloride.

tent. This latter method would give more-accurate (comparable) CBR results. However, this would require four times the amount of soil material. The most serious shortcoming of the method used was that the gradation of the material changed considerably with compaction; the amount of the middle-range-size particles increased while the percentage of coarser material decreased. The results shown below were obtained with material from the same pit.

| Sieve No. | Percent Passing Fresh Sample | Percent Passing Used Sample* |
|---|---|---|
| 4 | 49 | 65 |
| 8 | 32 | 46 |
| 14 | 22 | 33 |
| 28 | 16 | 23 |
| 48 | 10 | 12 |
| 100 | 6 | 6 |

*The sample was subjected four times to a compaction test of 25 blows/layer in 5 layers (compaction with a standard modified Proctor hammer).

Also, the control of water content would be more accurate if the suggested latter method were used. Therefore, comparisons of the obtained densities and the CBR values have limitations. At the same moisture content and at the same compactive effort, the results from two different percentages of calcium chloride can be accurately compared, because for these cases any changes in gradation would be the same, resulting from similar previous compaction.

The addition of calcium chloride was done in solution and the water forming the salt solution was part of the initial moisture added to the soil. Four different percentages of calcium chloride were used: 0.0, 0.1, 0.5, and 1.0.

In the soaking and drying tests the most-important factors were the sensitivity of the measuring instruments (such as the gauge and the weighing apparatus) and the curing conditions for the samples. The humidity and temperature should be carefully controlled, and they should be nearly constant during the entire series of tests.

The CBR results were based on 0.2-inch penetration, since the values at this

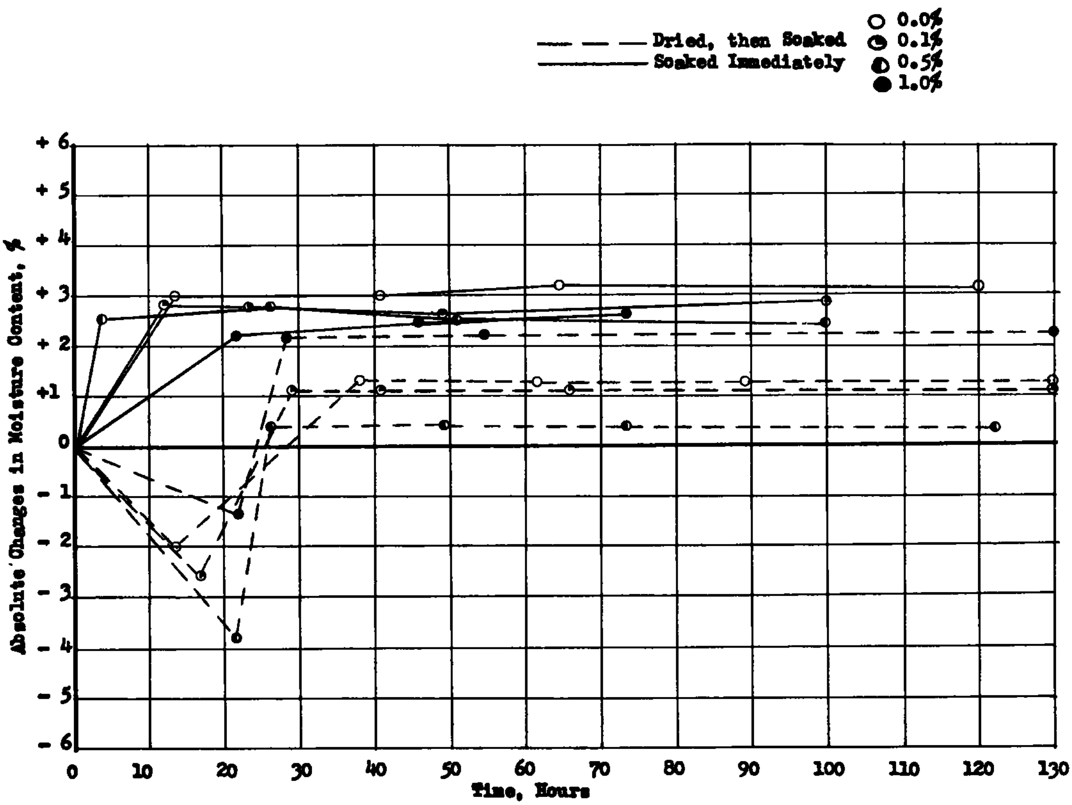

Figure 5. Change of moisture content of washed bank-run gravel with oven drying and soaking at various percentages of calcium chloride.

point were always higher than 0.1-inch penetration. As checks, some of the tests were repeated and nearly the same results were obtained. One factor affecting this (except for dried specimens) was that the top of a CBR sample was wetter and had a higher percentage of moisture; the percentage of moisture decreased from the top of the specimen downward. In case of any disturbances in the center of the top of the specimens occurring during the removal of the samples from the cylinders, the CBR was run on the undisturbed side of the top, without a surcharge weight. Tests showed that the presence or absence of the surcharge weight had little effect on CBR's for this material.

The fact that the addition of calcium chloride lowered the pH shows that base exchange occurred. Apparently 0.5 percent of calcium chloride gives nearly maximum base exchange—calcium for hydrogen.

CBR values (0.2-inch penetration) were calculated on the basis of the value of standard stress[1] which is based on crushed stone. Some values of CBR, especially in the cases with maximum compactive effort, were above 100 percent.

The tests involving drying, followed by soaking, of the specimens were run in an effort to determine any possible effect of curing on the CBR's of the material at various calcium chloride contents. This condition of the alternate dyring and wetting is one which must occur in the field. The one cycle used was extreme in rate of change of moisture, but should indicate the trend of results in the field.

The data presented in Figures 3, 4, 5, and 6 showed little trend as affected by percentage of calcium chloride. The changes in height shown are extremely small and within experimental error. The procedure involved lifting the specimens in and out of a water bath and tilting them to pour off excess water for a weight determination. This handling could dis-

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[1] $\text{CBR (\%)} = \frac{\text{stress from test}}{\text{standard stress}} \times 100$, standard stress = 1500 psi. for 0.2 inch penetration.

turb the specimens slightly, although great care was used. It should be noted, however, that each curve follows a specific trend (is not erratic), indicating that the data are reliable. There is a tendency for greater swelling, and immediate soaking, for larger percentages of clacium chloride. This tendency is eliminated, largely or completely, by a drying cycle before immersion.

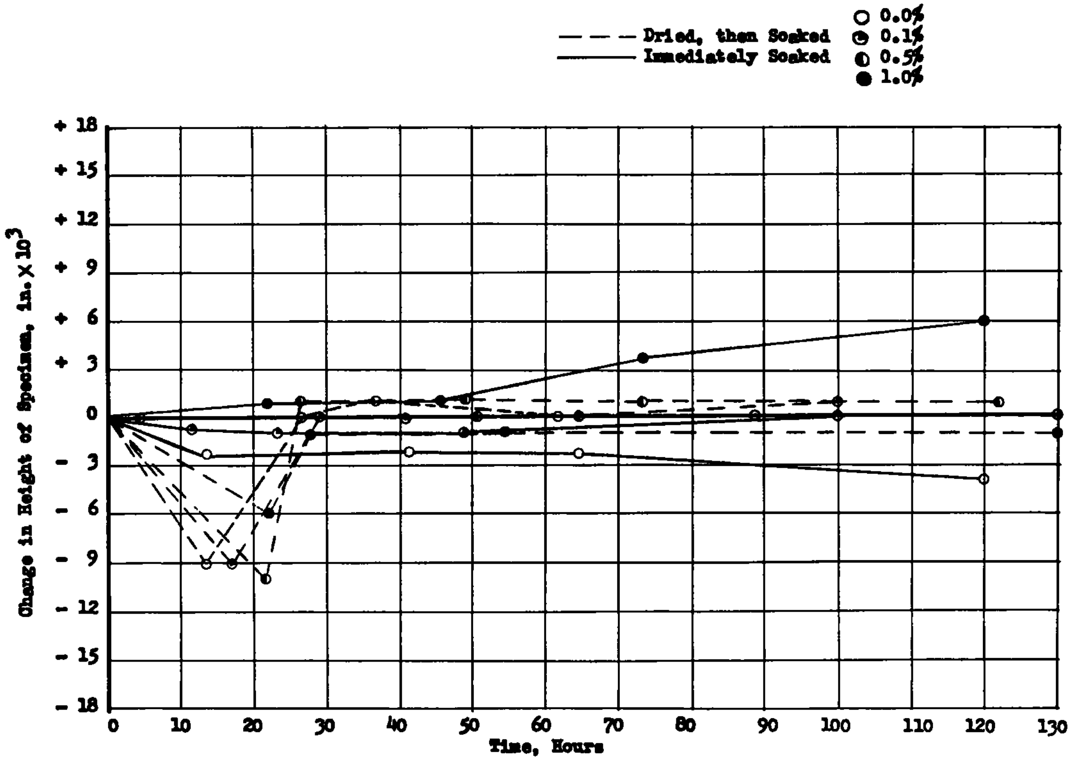

Figure 6. Shrinking and swelling of washed bank-run gravel with oven drying and soaking at various percentages of calcium chloride.

Calcium chloride is known to affect rate of drying of soils. In this experiment, the rate and extent of drying were extremely severe; they were chosen to measure the effect of exaggerated curing on CBR values. As a result, a good measurement of effect of calcium chloride on drying rate was not obtained. However, a general trend of reduction of drying rate with increasing percentage of calcium chloride can be noted by extrapolating all drying curves to the same drying time.

Further analysis of the fine portion (passing No. 200 mesh), showing types and amounts of the clay minerals and their type of base and base exchange capacity, would have been helpful in interpretation of the results.

The increase in density from 138 to 141.5 pcf. (for unwashed gravel, 55 blows, 0.5% calcium chloride) would reduce the void space by about 12% on a relative basis (from 160 down to 14% of the total volume). This might be a significant factor in resistance to freeze-thaw action.

TABLE 5

CHANGES OF pH WITH TIME AT VARIOUS PERCENTAGES OF CALCIUM CHLORIDE

| Unwashed Aggregate | | | | |
|---|---|---|---|---|
| %CaCl₂ | Time Interval | | | |
| | 5 min | 1 hr | 22.5 hours | 6 days |
| 0.0 | 8.1 | 8.4 | 8.45 | 8.3 |
| 0.1 | 7.7 | 7.9 | 8.1 | 7.9 |
| 0.5 | 7.4 | 7.7 | 7.75 | 7.7 |
| 1.0 | 7.4 | 7.6 | 7.7 | 7.6 |

| Washed Aggregate | | | |
|---|---|---|---|
| % CaCl₂ | Time Interval | | |
| | 5 min | 1 hr | 23.0 hours |
| 0.0 | 8.0 | 8.2 | 8.3 |
| 0.1 | 7.7 | 7.8 | 8.0 |
| 0.5 | 7.45 | 7.6 | 7.8 |
| 1.0 | 7.45 | 7.5 | 7.7 |

## CONCLUSIONS

1. The addition of calcium chloride resulted in higher optimum dry densities of both washed and unwashed bank-run gravel. The greatest increases were obtained with 0.5% of calcium chloride; amounts smaller or larger than this gave densities intermediate between those for no chemical and for 0.5% chemical. The amount of increase was appreciable, being a maximum of 3.5 pcf. for unwashed gravel at intermediate compactive effort (25 blows Proctor).

2. The addition of an optimum amount of calcium chloride will result in either a higher optimum density or an equal density at a lower compactive effort.

3. In the sample tested, if a high compactive effort was used, the washed material (few fines) gave higher densities and higher CBR values than the unwashed sample and the addition of calcium chloride had little effect. However, if a low compactive effort was used, the unwashed material (many fines) gave higher densities and the addition of calcium chloride had an appreciable effect. In the latter case, CBR values measured immediately (no curing period) were lower than for the washed sample, regardless of whether or not calcium chloride was used.

4. Base exchange occurs upon addition of calcium chloride, as shown by a drop in pH. The reaction is not complete in five minutes, but is substantially complete in one hour. A greater amount of base exchange occurred in the sample with more fines; this is correlated with the greater density obtained for this material. The amount of base exchange increases with increase in amount of calcium chloride added; however, 0.5% gave almost as much effect as 1%. This is in accord both with the theory of reduction of zeta potential by base exchange and with experimental density results.

5. In general, addition of calcium chloride caused a loss in CBR values on immediate testing (no curing) at low compactive efforts and at moisture contents above the optimum. In general, addition of calcium chloride caused a gain in CBR values on immediate testing (no curing) at high compactive efforts and at moisture contents below the optimum. The increase was usually greatest for 0.5% calcium chloride, as it was for dry density.

6. The addition of calcium chloride reduces the loss in California Bearing Ratio of soaked samples with many fines. The bearing capacities of the washed gravel were about the same for all soaked samples, regardless of calcium chloride content (from 0.0% to 1%). The bearing capacities were about the same (very small reduction caused by calcium chloride and its retained water) for all samples after they were dried to approximately one half optimum moisture content and they were substantially equal after the dried samples were soaked.

7. Curing (that is, drying followed by soaking) of specimens indicated, on the basis of few tests, that any loss in CBR values (determined by immediate testing after molding) caused by addition of calcium chloride is at least partially recovered by the curing process.

8. Soaking of the samples lowered their CBR values. Drying of the samples raised their CBR values. Drying followed by soaking gave higher CBR values than immediate soaking, even though the final moisture contents in the two cases were about the same.

9. Calcium chloride is effective for increasing density, only on gravel containing an appreciable amount of material passing 200 mesh. The effect is minor when the fines content is below 5%.

## *Acknowledgment*

The authors wish to thank the following individuals and groups: J. W. Spencer, of Cornell University, for helping choose and obtain the material studied; Havens, Drake, and Gregg, of the Kentucky Department of Highways,, for analyzing the clay fraction; and the Highway Research Board Committee on Soils-Calcium - Chloride Roads, under the chairmanship of H. F. Clemmer, for sponsoring the study.

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# Strength of Soil-Cement as a Function of Degree of Mixing

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This paper describes a laboratory investigation of the following variables involved in mixing portland cement into a clayey silt: (1) uniformity of soil and cement mixture, (2) compressive strength of the resultant soil-cement, (3) time of mixing, and (4) accumulative energy required for mixing.

Mixing was performed in a pugmill-type mixer and the energy required for mixing was automatically recorded. Specimens were compacted in the Harvard miniature compaction mold and, after curing seven days, were tested in unconfined compression. Cement concentration throughout the soil-cement mixture was determined by means of a radioactive tracer that was incorporated uniformly in the cement. This method of measuring cement content proved to be rapid and reasonably reliable. Degree of mixing was measured by the mixing uniformity coefficient. It was concluded that:

1. The relationship between uniformity of mix and strength of soil-cement is a logarithmic one; i. e. a straight line is obtained on a plot of strength versus $\log_{10}$ of I. Thus, when the cement and soil are poorly mixed a small increase in mixing uniformity results in an even smaller increase in strength; whereas for a well mixed sample, a small increase in mixing uniformity results in a large increase in strength.
2. The strength of soil-cement varies directly with the $\log_{10}$ of the accumulative energy required for mixing. This relationship holds within the limits tested, but it is apparent that there is a point beyond which further increases in mixing energy would have little effect on the strength of the soil-cement.
3. The relationship between mixing energy and uniformity of mix (I) appears as a curve on a plot of $\log_{10}$ I versus mixing energy, so that a small decrease in I (increase in mixing uniformity) requires a very large increase in the amount of mixing energy.
4. Other things being equal, accumulative mixing energy is approximately proportional to mixing time.

●STABILIZING soils with portland cement has progressed from a modest beginning in the 1930's to an eminent position today as one of our most-common methods of soil stabilization. In the early days of soil-cement mixtures, farm equipment was used to mix the cement with the soil as there was no equipment designed specifically for that purpose. As soil-cement mixtures became more popular, the need for specially designed mixing equipment became increasingly apparent. This was particularly true with the clayey soils where it was necessary to pulverize the soil before mixing. It is to the credit of the mixing manufacturers that they met the need rapidly and effectively. Today we have powerful, highly mechanized mixing equipment that mix cement with soil far more speedily and efficiently than was once thought possible.

## The Mixing Problem

Many people firmly believe that, thanks to our modern equipment, the days of mixing problems are a thing of the past. There is truth in this belief, since under ideal conditions cement can be mixed to some extent with almost any soil, including the fat clays. There is still the question, however, of how well the cement and soil are mixed. Numerous tests have shown that field strengths of soil-cement seldom match laboratory strengths (1a, 2). There is growing evidence that this is due to insufficient mixing.

The British, in their investigations of field mixing, use as a measure of field mixing efficiency, the ratio of field strength to laboratory strength. They have found varying values for mixing efficiency depending on the mixer type and soil type, but a value of 60 percent is typical. They note that if the mixing efficiency could be increased to 80 percent, 30 percent less cement might be used for the same strength (1a).

Further evidence that a mixing problem does exist comes from the Engineer Research and Development Laboratories, Fort Belvoir, Virginia, where cement stabilization was attempted on a well-graded gravel, a silty sand, and a fat plastic clay. Results were termed satisfactory on the gravel and silty sand but a failure on the clay. In addition, the strengths of samples taken from the silty sand mixture varied as much as 400 percent, indicating insufficient mixing (1b).

TABLE 1

COMPOSITION AND PROPERTIES OF SOIL

| | Natural Soil | Soil-#40 | Soil-#10 |
|---|---|---|---|
| Percent by wt finer than #4 | 72 | | -- |
| " " " " " #10 | 58 | | 100 |
| " " " " " #40 | 43 | | 74 |
| " " " " " #200 | 27 | | 46 |
| Percent finer than 0.002 m m | 5 | | 8.6 |
| Percent clay mineral | 12 (Illite) | | 20 (Illite) |
| Predominant non-clay mineral | Quartz | | Quartz |
| Organic matter | trace | | trace |
| Liquid limit in % dry wt of soil | | 24 | |
| Plasticity Index | | 8 | |
| Optimum water content for compaction in percent of soil dry wt | | | 12.5 |
| Unconfined compressive strength in psi at optimum water content | | | 19 |

### Reasons for a Mixing Study

The foregoing was pointed out to show that, with many soils (especially those that are fine grained), a high degree of mixing is difficult to obtain. Logic tells us, and indeed there is ample proof, that the attainment of a high degree of mixing is advantageous, because the more uniform the soil and cement mixture, the higher the strength of the compacted soil-cement; that is, thorough mixing, by using the cement more effectively, produces higher strength at a lower cost than does incomplete mixing. Thus interest in the strength of soil-cement necessitates interest in the degree of mixing.

The following questions are therefore raised: (1) Is the relationship between strength and degree of mixing linear? (2) How does the strength of soil-cement vary with the energy expended in mixing? (3) At what water content (with respect to optimum for density) can the most efficient use of the cement be obtained for the least work? (4) How can the degree of mixing be quickly determined in the field? The answers to these and other questions lie in increasing our knowledge of the principles of mixing and of the relationships and interactions of the many variables involved in mixing soil and cement. The laboratory investigation described in this paper was aimed at contributing toward this goal.

## FACTORS INVOLVED IN THE SOIL-CEMENT MIXING STUDY

### Variables

The process of mixing cement with soil involves numerous variables, many of them interdependent. In this investigation, an attempt was made to isolate and study several of these variables.

The variables studied were: (1) degree of mixing; (2) compressive strength of resultant soil-cement; (3) time of mixing; (4) accumulative energy required for mixing; and (5) water content.

The variables held constant were: (1) Type of soil (the soil used was a glacial till, obtained from the Stockbridge Bypass area in Western Massachusetts, in which all material coarser than a No. 10 sieve had been removed; the soil properties and composition are listed in Table 1). (2) Portland-cement concentration (a 10 percent concentration was used since it is a typical one for soils of slight to moderate plasticity). (3) Type of mixer (the pugmill type mixer used is described in Section II-D). (4) Speed of mixing (the main shaft of the mixer was set to rotate at 60 r.p.m., see Section II-D). (5) Time from the start of mixing to compaction (if the times were not held constant the strength would differ due to varying amounts of cement hydration taking place before compaction). (6) Compactive effort used in molding strength samples. (7) Density of strength samples for a given water content.

### Determination of Cement Concentration

In any investigation of mixing soil and cement the most-difficult problem is that of determining the variation in cement concentration throughout the mixture. This may be done statistically by taking many small samples of uniform size from the mixture and finding the cement concentration in each. The normal ASTM T 144-49 titration procedure for determining cement content, while accurate and acceptable in most cases, takes an excessive amount of time. Since many hundreds of determinations had to be made, a more-rapid method was desired. Two other methods were investigated:

The Dye-Benzene Technique. In this procedure an oil-soluble, non-water-soluble dye is mixed uniformly with the cement. After the soil and dye-treated cement are mixed, many small samples of the mixture are taken, weighed, and a specified amount of benzene added to each. A colorimeter is used to determine the dye concentration and thus the cement concentration of each sample.

This procedure is satisfactory for certain soil-additive mixtures but proved unsatisfactory with cement. All of the dye could not be recovered, probably because the hydration of the cement trapped some of the dye.

The Radioactive-Tracer Technique. The determination of cement concentration by the radioactive-tracer technique is dependent on the generally accepted assumption that the rays emitted by radioactive material can be counted, and thus the quantity of radioactive material determined with a scintillation counter. If care is taken to insure that the geometry of each test is the same, i.e., position and size of the samples are the same, then the test is reasonably reliable. The accuracy with which the quantity of radioactive material present is determined, depends on the activity of the radiation, i.e., counts per second, and length of time of counting.

If the scintillation counter is in proper working order, the accuracy of counts per second can be figured statistically as follows: 67 percent of the time the counts per second for a given sample will be within one standard deviation of the mean, and 95 percent of the time within two standard deviations of the mean, where standard deviation in counts per second is equal to the square root of the total number of counts divided by the counting time in seconds (3). For example, if a sample were counted for 100 seconds and the total count was 10,000, the standard deviation in counts per second would $= \sqrt{\frac{10,000}{100}} = 1$ and 67 percent of the time the counts per second would fall within 1 percent of the mean.

The foregoing principles were employed to determine the cement content throughout a soil and cement mixture as follows: Cobalt 60 ($Co^{60}$) (a radioactive material) in powder form was mixed with cement in a ball mill until a uniform mixture was obtained. The uniformity of the mix was thoroughly checked by counting many small samples of the radioactive cement in the scintillation counter.

By the use of radioactive cement, determination of the cement concentration was a simple one. The small uniform size samples of soil-cement mix taken from each test were weighed, counted, and the cement content of each sample computed by comparison with a standard sample of radioactive cement.

## Determination of Degree of Mixing or Mixing Uniformity

Once the cement concentrations of many small-similar-size samples out of a large mixture are known, the uniformity of the mix can be computed. Before this is done, the scale to which mixing uniformity is related must be specified. In this paper, a perfect mixture exists when all the samples taken from the mixture have the same concentration of cement. This concentration should equal the concentration of cement in the entire mixture. However, both the definition of homogeneity and the values obtained for the mixing uniformity of the various mixes are dependent on the size of the samples taken from the mixture. Michaels and Puzinauskas have shown experimentally (8) that the mixing uniformity of a given mixture is inversely proportional to the square root of the volume of the samples taken from the mixture to determine the mixing uniformity.

There are several methods available for showing any variation from a homogeneous mixture of two materials. The simplest is to record the maximum range in variation of the samples taken; i.e., if ten small samples from a large mixture of soil and cement are analyzed, and the maximum and minimum cement concentrations determined are 15 percent and 5 percent respectively, then the range in variation is 10. This representation is unsatisfactory, however, since no indication of the variation within the maximum range is recorded.

A second method of representing variations in mixing uniformity consists of noting all variations from the mean concentration and taking the average as the mean deviation. In this method all the samples play a part in determining the final value used to represent the variation from homogeneity.

A third procedure that has more mathematical backing (4) than the above is the standard deviation or root-mean-square deviation; that is, the square root of the average of the sum of the squares of the deviations from the mean concentration. If C is the concentration of cement in the individual sample, $C^m$ the mean concentration and n the number of samples taken, then:

$$\text{Standard deviation} = \sqrt{\frac{\Sigma_0^n (C - C^m)^2}{n}} \qquad (1)$$

and

$$\text{Percent standard deviation} = \sqrt{\frac{\frac{\Sigma_0^n (C - C^m)^2}{n} \times 100}{C^m}}$$

Either of these values could be used as a measure of uniformity of mix as they both vary from a finite value at no mixing to zero at perfect mixing. Michaels (6) has presented a modification of the above, called the "uniformity index," which is used in this paper.

The uniformity index (I) is the ratio of the standard deviation at mixing time t to the standard deviation at zero mixing. The advantage of this method is that the values vary from unity at no mixing to zero for a perfect mixture.

The standard deviation at zero mixing is derived as follows:

Let $n_1$ = number of samples taken from cement

$n_2$ = number of samples taken from soil

$n = n_1 + n_2$

$\frac{n_1}{n} = C^m \quad ; \quad \frac{n_2}{n} = 1 - C^m$

then standard Deviation$_0$ =

$$\sqrt{\frac{\Sigma_0^n (1 - C^m)^2 + \Sigma_0^{n_2} (C^m - 0)^2}{n}}$$

$$= \sqrt{\frac{n_1 (1 - 2C^m + {C^m}^2) + n_2 {C^m}^2}{n}}$$

$$= \sqrt{C^m \left[1 - 2C^m + {C^m}^2\right] + {C^m}^2 (1 - C^m)}$$

$$= \sqrt{C^m (1 - C^m)}$$

Dividing (1) by (2) gives,
The uniformity index (I) = $\sqrt{\frac{\Sigma_0^n (C - C^m)^2}{n (1 - C^m) C^m}}$

The uniformity index (I), then, is a definite measure of the degree of mixing uniformity; the lower the value of I, the higher the degree of mixing, i.e., the more uniform is the mixture.

## Equipment

Mixing Apparatus. All mixing was performed in the "plastograph" pugmill-type mixer shown in Figures 1 and 2. The pugmill consists of two counter-rotating shafts 1.59 cm. in diameter one rotating at 1.5 times the speed of the other; each shaft has 10 perpendicular prongs with each prong 2.22 cm. long and 0.635 cm. in diameter. The inside dimensions of the mixing box are 7.2 cm. by 10 cm. by 10 cm. The torque required for mixing is recorded automatically.

Strength-Test Apparatus. The equipment for determining the strength of soil-cement samples consists of a Harvard miniature compaction apparatus (7) for compacting and molding the sample, and a motor driven, worm screw, strain controlled unconfined-compression apparatus (5) for testing the strength of the sample, in which the load is measured with a proving ring and the deflection of the sample with a strain gage.

Scintillation Counter (3). The apparatus used to determine presence of radioactive material is the electronically operated scintillation counter. It consists of a crystal of sodium iodide for interaction with the gamma rays emitted by the sample, a photoelectric surface for catching the light flashes caused by the gamma rays striking the crystal, a photomultiplier tube for catching the shower of electrons caused by the light flashes on the photoelectric surface, an amplifier to amplify the pulse from the photomultiplier tube, and a scaling circuit (pulse counter) to count the pulses.

## LABORATORY PROCEDURE

### General

1. Twenty-two grams of radioactive cement was weighed into each of forty-eight 2-ounce jars (enough jars for test series at two water contents). From each jar, one gram was removed and put in 1-dram vials. Each jar then contained the amount of cement that was used in each test. The 1-gram samples of cement, taken from each jar, served as standards for later determination of cement variation throughout the soil-cement mixture. These samples served the further purpose of checking the radioactive cement for uniformity of activity.
2. For each test, 210 grams of soil was weighed into a large dish. The required water was added to the soil and mixed by hand with a standard kitchen tablespoon to insure uniform water distribution throughout the soil. Only two water contents (based on the dry weight of soil and cement) were studied; 14 percent, which is slightly above the optimum water content for compaction, and 11 percent, which is slightly below the optimum.
3. The moist soil was put in the mixing box of the "plastograph" and mixed for 30 seconds to insure randomness of the location of the soil in the box. Cement was placed in the center of the box between the two counter-rotating shafts. The mixer was started and run for a specified length of time determined by a stopwatch. The mixing times studied varied from 2 seconds to 2 minutes.
4. The work required for mixing was recorded automatically on a graph and the accumulative energy per gram of soil mixed was computed by graphical integration.
5. Eleven small samples were taken from various sections of the mixer and compacted with a small rod in 1-dram vials. The dry weights of the samples were 1.8 grams ± 0.1 gram and the compacted heights were 0.9 cm. ± 0.1 cm. The samples were kept within these limits to insure statistical accuracy and similar geometry for later counting in the scintillation counter.
6. A strength sample was compacted in the Harvard minature mold from the soil-cement remaining in the mixing box. A constant compactive effort was employed by compacting in three layers, 25 tamps per layer, with 40 lb. of spring tension per tamp.

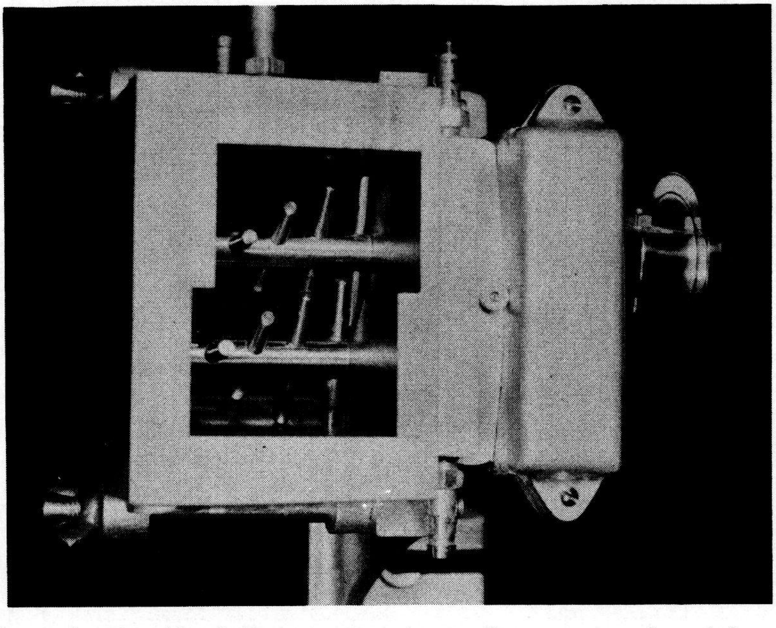

Figure 1. Detail of finger-prong mixer. Note staggered positions of prongs on each shaft.

7. The strength sample was cured for 7 days at greater than 90 percent relative humidity and then tested in unconfined compression.

8. The 11 small samples taken from the mix and placed in vials were weighed and counted in the scintillation counter. The weighing was done as soon after each test as possible to insure no water loss. The counting was done when convenient.

9. Cement concentration of each sample was computed by comparing the counts per second with that of the standard 1-gram cement sample. A measure of the variation of cement concentration through the soil-cement mixture was computed using the uniformity index. Ten percent was taken as the mean concentration from which the variation was measured. This was felt to be more realistic than using the mean of only 11 samples. In most cases the mean of the 11 samples was 10 ± 0.3 percent. The exceptions were for the very poorly mixed (short mixing time) tests.

## Exceptions

In order to have some tests in which a homogeneous mix was approached, several samples were made at each water content in which the cement was added to the dry

Figure 2. Mixing apparatus, the Brabender "plastograph."

## TABLE 2

### DATA SUMMARY

Water content w = 14 percent

| Test | I | Mean Conc.* | Mixing Time | Accumulative Mixing Energy | Compressive Strength psi. | Dry Density pcf. |
|---|---|---|---|---|---|---|
| | | % | sec. | meter grams per gram | | |
| 1 | 18.5 | 10.26 | 2 | 0.5 | 75 | 120 |
| 2 | 12.8 | 8.58 | 2 | 0.5 | 84 | 120 |
| 3 | 10.7 | 10.10 | 2 | 0.5 | 128 | 121 |
| 4 | 13.8 | 7.43 | 2 | 0.5 | 80 | 120 |
| 5 | 6.1 | 10.31 | 5 | 1.1 | 99 | 120 |
| 6 | 9.8 | 8.85 | 5 | 1.0 | 96.5 | 120 |
| 7 | 6.3 | 9.90 | 7.5 | 1.7 | 95 | 121 |
| 8 | 10.1 | 9.68 | 7.5 | 1.8 | 101 | 120 |
| 9 | 11.8 | 10.69 | 7.5 | 2.7 | - | |
| 10 | 13.1 | 9.55 | 7.5 | 2.5 | 88.5 | 120 |
| 11 | 8.9 | 8.78 | 7.5 | 2.0 | 109 | 120 |
| 12 | 3.0 | 10.06 | 15 | 3.3 | 131 | 121 |
| 13 | 7.3 | 10.39 | 15 | 3.3 | 120 | 122 |
| 14 | 7.5 | 8.95 | 15 | 4.8 | 88 | 121 |
| 15 | 2.7 | 9.85 | 30 | 7.9 | 139 | 121 |
| 16 | 3.7 | 9.25 | 30 | 7.9 | 140 | 121 |
| 17 | 2.5 | 10.03 | 30 | 8.1 | 136 | 121 |
| 18 | 3.2 | 9.96 | 30 | 8.1 | 125 | 121 |
| 19 | 1.7 | 9.63 | 120 | 30.3 | 205 | 121 |
| 20 | 3.4 | 9.67 | 120 | 36.6 | 145 | 121 |
| 21 | 2.1 | 10.10 | 120 | 38.0 | 173 | 121 |
| 22 | 2.6 | 9.91 | 120 | 31.4 | 152 | 121 |
| 23 | .34 | 10.04 | hand mix | - | 254 | 120 |
| 24 | .16 | 9.82 | hand mix | - | 303 | 120 |
| 25 | .60 | 9.94 | hand mix | - | 229 | 120 |
| 26 | .46 | 10.06 | hand mix | - | 243 | 120 |
| | | | Water Content w = 11 percent | | | |
| 1 | 6.1 | 9.91 | 5 | .95 | 157 | 121 |
| 2 | 8.6 | 0.01 | 5 | 1.0 | 160 | 121 |
| 3 | 4.3 | 10.10 | 15 | 3.3 | 232 | 122 |
| 4 | 2.6 | 9.63 | 15 | 3.6 | 231 | 122 |
| 5 | 2.8 | 10.25 | 30 | 8.1 | 224 | 122 |
| 6 | 4.4 | 9.35 | 30 | 6.8 | 186 | 120 |
| 7 | 3.4 | 10.31 | 120 | 38.0 | 237 | 121 |
| 8 | 3.6 | 9.87 | 120 | 31.6 | 246 | 122 |
| 9 | 0.37 | 9.85 | hand mix | - | 357 | 120 |
| 10 | 0.37 | 9.90 | hand mix | - | 368 | 120 |

* Average of samples taken out of mix.

** Measured to nearest pound per cubic foot.

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soil and mixed by hand. Water was added to the soil and cement and again mixed by hand. No attempt was made to obtain a value for the work required for mixing. Because of the high homogeneity of these samples, a correction was applied to the computed mixing uniformity index. The correction consisted of subtracting from the computed I the value that would be obtained (due to the counting error) from a completely homogeneous mixture. With the scintillation counter, radioactivity of cement, and time of counting that were used, this I amounts to 0.0044; i.e., if the same sample were counted many times, the I obtained would equal 0.0044.

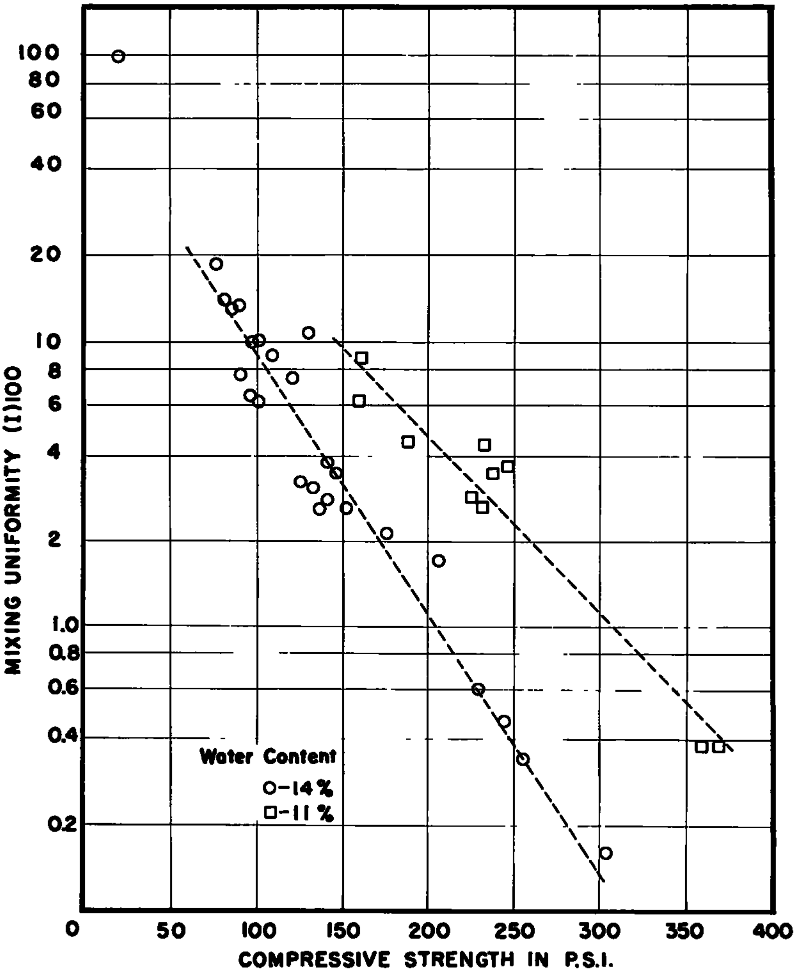

Figure 3. Unconfined compressive strength of soil-cement mixture versus the log of the mixing-uniformity index (I) of the mixture.

## RESULTS

A total of 80 tests were run. Of these, the first 44 were principally concerned with evolving a satisfactory procedure, and in checking the reliability of the radioactive tracer method for determining cement concentration. Of the remaining 36 tests, 26 were run at 14-percent water content and 10 at 11-percent water content. More tests were planned at the second water content, but due to time limitations, they were omitted. The above number of tests is sufficient to establish trends and to draw tentative conclusions. While many of the results found here will not apply quantitatively for other soils, the trends should be applicable. However, the number of tests at 11-percent water content is admittedly small and any conclusions drawn from them will need considerable additional verification. Results of all tests are shown in Table 2 and Figures 3 through 8.

With the degree of mixing measured by the mixing uniformity index, in which zero mixing is unity and a completely homogeneous mixture is zero, the following trends are observed:

1. The strength of soil-cement varies

as the log of the mixing uniformity. An increase in uniformity of the mix (measured by a decrease in I) results in an increase in strength. This effect becomes more pronounced as a homogeneous mixture is approached (see Figures 3 and 4).

2. The rate at which the strength of the soil-cement increases with increase in uniformity of the mixture (decrease in I), is greater for the soil mixed at slightly below the optimum water content, than for the soil mixed at slightly above optimum. In addition, the strength at any given I is higher for the samples mixed and compacted on the dry side of optimum than on the wet side (see Figures 3 and 4).

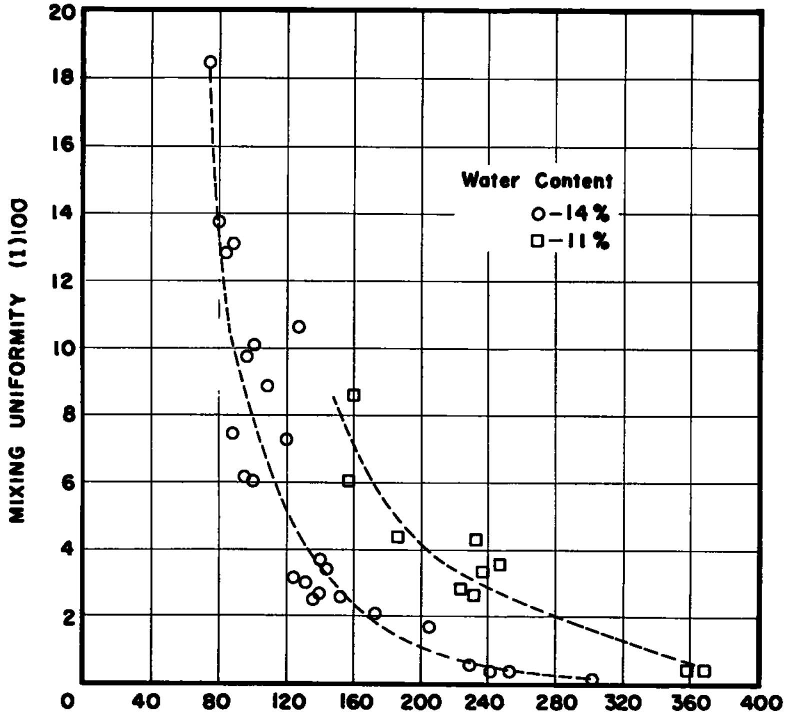

Figure 4. Unconfined compressive strength versus the mixing-uniformity index.

3. Mixing Uniformity (I) varies as some exponential function of mixing time; i.e. a plot of log I versus log mixing time approximates a straight line which can be expressed by the equation $I = kt^n$ where n is a negative number. This relationship holds, within the limits tested, for mixing at 14-percent water content. The data, while inconclusive, indicate that this relationship holds only for a relatively short time when mixing at 11-percent water content. Apparently the point is quickly reached beyond which I does not decrease with mixing time (see Figures 7 and 8).

4. Accumulative mixing energy is approximately proportional to mixing time; i.e., the torque required for mixing does not change significantly with time, after the first few seconds of mixing. Plots of strength versus mixing time are similar to those of strength versus accumulative mixing energy (see Figures 5 and 6).

5. The strength of soil-cement varies as the log of the accumulative mixing energy required and as the log of the mixing time (see Figures 5 and 6).

This relationship appears to hold within the limits tested for the 14-percent water content but not for the 11-percent water content. As previously indicated, Figure 7 shows that when the cement and soil are mixed at 11-percent water content maximum uniformity occurs very rapidly, after which there is no decrease in I with mixing time. Since strength is dependent on I, curves such as those drawn in Figures 5 and 6, for 11-percent water content, appear logical.

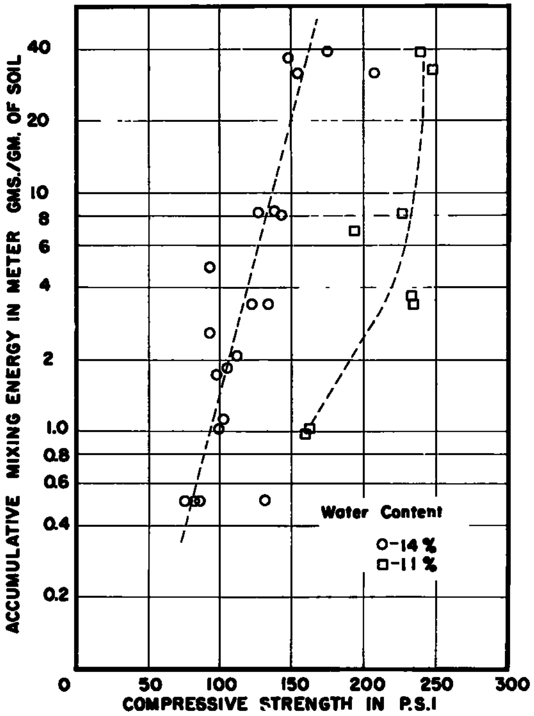

Figure 5. Unconfined compressive strength versus the accumulative energy required for mixing the cement with the soil.

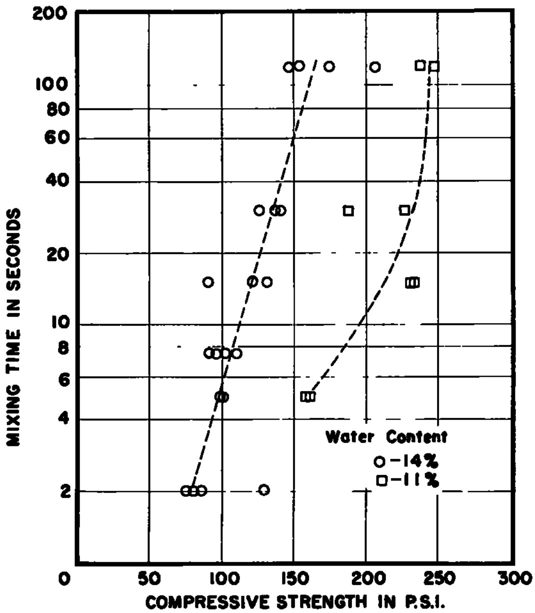

Figure 6. Unconfined compressive strength versus mixing time.

In addition, the one point that is considerably off the 11-percent water-content curves represents a sample whose density was about 2 pcf. less than most of the other samples. Since a small increase in density can result in a large increase in strength, the point would be much closer to the curves if a correction were made to account for the difference in density.

6. At any given mixing time or any given accumulative mixing energy, the strength of the soil-cement mixed and compacted slightly dry of optimum is greater than when mixed and compacted slightly wet of optimum. There is also some indication that the rate, at which strength increases with mixing time and with increasing accumulative mixing energy, is greater for the dry side than for the wet side (see Figures 5 and 6).

## PRACTICAL SIGNIFICANCE OF RESULTS

### Strength as a Function of Degree of Mixing and Mixing Time

One of the prime objectives of this investigation was to determine the relationship between the strength of soil-cement mix and the uniformity of the cement and soil mixture. The results, as previously stated, show that this relationship is not linear but logarithmic.

Since for a moderately well-mixed sample a small increase in the uniformity of the mixture results in a large increase in the strength, it is advantageous to obtain a uniform soil-cement mixture. Unfortunately, in order to get a small increase in uniformity, a large amount of energy is necessary, or, comparing strength and energy directly, Figure 5 shows that the increase in energy required is larger than the increase in strength obtained. The question of how long to mix with a given mixer is a matter of economics. Does the money saved from the reduced amount of cement required, due to improved mixing, more than offset the cost of the additional time and work needed to get the improved mixing?

There is obviously an optimum amount of field mixing for a specific mixer under a given set of mixing conditions. At present the optimum amount in number of passes or in forward speed of the mixing equipment cannot be predicted. This fact only serves to emphasize the need for a better understanding of the principles involved in mixing soil and cement.

## The Effect of Water Content

The water content at which the soil and cement are mixed greatly affects the results obtained. (Before examining these effects a short digression on water contents may avoid later confusion. All water contents are based on the dry weight of the soil plus cement. Thus, the 11-percent water content, which is somewhat dry of the optimum for compaction (12.5) of the soil and cement, is very near the optimum for compaction of the soil alone. In addition, the term "optimum water content," as used in this paper, always refers to the water content for maximum compaction of the soil and cement mixture.) A study of Figures 3 through 7 shows that greater efficiency is obtained by mixing slightly dry of optimum rather than slightly wet of optimum. For example:

1. Maximum mixing uniformity is approached more rapidly when mixed slightly dry of optimum than when mixed slightly wet of optimum.
2. The energy required for a given mixing time is about the same in both cases, but the energy required to approach ultimate mixing uniformity with the mixer is less when mixed on the dry side than when mixed on the wet side. In all likelihood the more-granular nature of the mixture on the dry side of optimum enables the maximum mixing uniformity to be approached more rapidly than on the wet side, where the additional water adds to the stickiness of the mix and slows down the attainment of maximum mixing uniformity.
3. The ultimate uniformity of mix is higher (lower I) when mixed on the wet side but not enough to compensate for the greater amount of time and energy required to obtain it.
4. The strength of the soil cement mixed and compacted on the dry side of optimum is higher than for the wet side. In addition, the rate at which the strength increases as the uniformity of the mix increases, is greater for the dry side mixtures. This observation is somewhat contrary to the recommendations of the Portland Cement Association (1) which specify that the soil-cement mixture should be compacted at optimum or slightly wet of optimum. Their reason for so specifying results from many years of field observations. They note that if the soil and cement are compacted dry of optimum, satisfactory curing of the surface of the soil-cement mixture is difficult to obtain. The results clearly show the great effect a small amount of water can make, as the difference in water contents between the two series of tests was only 3 percent. While the importance of close field control of mixing and compaction is recognized today, it should be reemphasized.

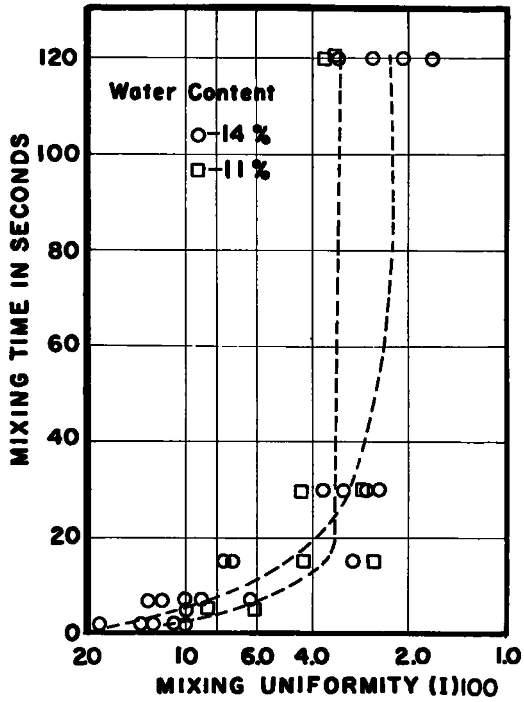

Figure 7. The log of mixing uniformity versus mixing time.

## Differences Between Field Strengths and Maximum Possible Strengths

The British in their investigation of mixing have found that, with many soils, field strengths fail to match laboratory strengths (1a, 2), where laboratory strengths are represented by the strengths of samples taken from the field and given additional mixing in a laboratory mixer before compaction. This is all the more significant when one considers that the strengths of samples mixed by hand are appreciably stronger than those mixed in a laboratory mixer (see Table 2 and Figure 3). Thus, field strengths may be only a small fraction of the ultimate that theoretically can be developed.

The reason for this difference in strengths is partially due to the lower degree of field control over the variables involved (water content, cement concentration, etc.) in comparison to the control over the variables when mixing in the laboratory. Usually, however, a more-important reason is the lower degree of mixing that is obtained in the field. As no method has been developed for determining field degree of mixing, most engineers, on soil-cement construction today, are quite satisfied that they are getting good mixing as long as the soil is readily pulverized and the soil and cement are visually mixed well. The term "good mixing," however, is strictly relative.

By the standards of 20 years ago, the term "good mixing" can justifiably be used to describe the quality of field mixing today, but on consideration of the improvement in mixing that is still available, there is less justification for the use of the term. Table 2 and Figure 3 show that the minimum I attained with the laboratory pugmill mixer (which must be considered as efficient or more efficient than any field mixer today) is about 0.025. The strength at this I, for example, is half of that at I = 0.001 so from a strength viewpoint, the importance of degree of mixing, and the room for improvement in the quality of mixing, are apparent.

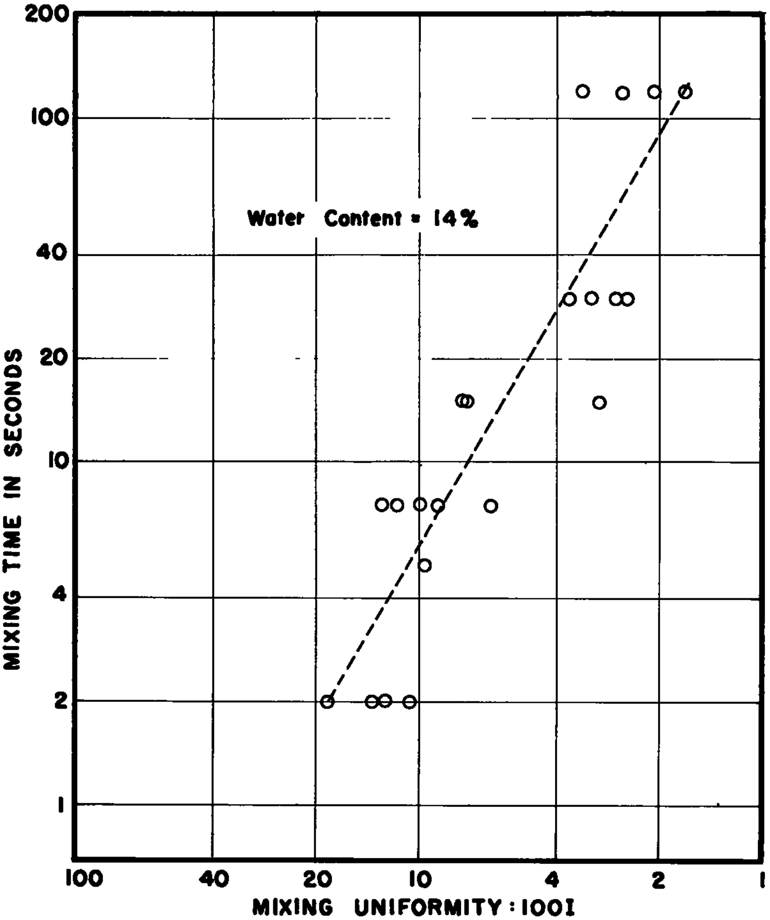

Figure 8. The log of mixing uniformity versus log of mixing time.

## Comparison of Results with Those of Other Experimentors

While comparatively little work has been done on the principles of mixing additives with soil, Michaels and Puzinauskas (6) have investigated the variables involved in mixing dextrose, kaolinite and water. Although such a system is quite different from a system of cement, silty soil, and water, there is significant similarity in some of the results.

For example, they observe that:

1. Except for the very short and very long mixing times, log I versus log mixing time plots as a straight line which can be expressed by $I = kt^n$ where t = mixing time and n is a negative number.
2. Mixing uniformity is approached more rapidly on the dry side of the plastic limit than on the wet side. Considering the plastic limit similar to optimum water content, the results are in qualitative agreement.

The work of the British has already been mentioned. Their work indicates, in agreement with this study, that there is still considerable room for improvement in the degree of mixing that is obtained in the field.

## Accuracy of Data and Sources of Error

To get statistical accuracy in a soil-cement study, many hundreds of tests are necessary, due to the large scatter of results obtained in various aspects of the test procedure. For example: The standard deviation of strengths of 10 soil-cement samples mixed by hand, prepared, cured and tested in the same way was 8 percent. Thus, the strength points on the various graphs are plotted with a probable accuracy of plus or minus 8 percent. In addition, the values obtained for accumulative mixing energy are accurate only to about plus or minus 5 percent. Since a slight change in water content, caused by evaporation or by a mistake in addition, can appreciably affect the energy requirements, the value of 5 percent is probably conservative.

The accuracy of the value obtained for mixing uniformity is greatly dependent on the uniformity of the mixture. Since only 11 samples are used to determine mixing uniformity in each test, the values obtained for the nonuniform mixtures are less accurate than those obtained for the uniform mixtures. However, an accuracy of plus or minus 3 percent may be taken as typical.

With the foregoing in mind, any interpretation of the results from relatively few tests must be made with appropriate reservations.

## RECOMMENDATIONS

The results obtained in the investigation prompt the following recommendations:

1. Soils similar in nature to the one used in this study should be mixed slightly on the dry side of optimum to obtain maximum uniformity for the least work. Whether the soil and cement mixture should then be compacted slightly dry of optimum, at optimum, or slightly wet of optimum appears to be dependent on such variables as: (1) amount of water required for complete cement hydration; (2) curing process to be used; (3) time from start of mixing to compaction; (4) strength desired; (5) flexibility desired; and (6) resistance to weathering desired. If a satisfactory curing procedure is available, the strongest soil-cement results from compacting slightly dry of optimum. On the other hand, higher compressibility, greater flexibility and greater resistance to weathering are obtained by compacting slightly wet of optimum. The procedure followed today is to compact at optimum, and perhaps this is the best compromise.
2. On large soil-cement jobs an investigation should be conducted to determine the amount of mixing required to obtain maximum strength and mixing uniformity for the least cost with the existing equipment. This would not slow up the construction process as the soil to be stabilized would be available for investigation enough ahead of time to allow for normal highway-construction procedures to be followed. The investigation would consist of mixing, at various speeds and number of passes, with the mixer, cement with some of the soil in question and making strength samples from the different mixes. The strengths at 24 hours (if time is short) or at some other time, could then be compared and the optimum amount of mixing, in terms of cost and desired strengths, determined. In this way, the actual probable field strength of the soil-cement mix could be predicted ahead of time, and in many cases, savings in cement quantities could then be obtained. Until an on-the-spot method of determining degree of mixing in the field is available, the above investigation would prove helpful on major soil-cement jobs.
3. More effort should be spent on increasing our knowledge of the principles of mixing, and specifically, on the principles involved in mixing additives to soil.
4. The mixing-equipment manufacturers should continue their efforts to develop better and more-efficient mixing equipment.

## CONCLUSIONS

The work described in this paper is only part of a continuing investigation on mixing that is being conducted at the M. I. T. Soil Stabilization Laboratory. Results to date indicate: (1) strength of soil-cement varies as the log of mixing uniformity (I); (2) strength varies as the log of mixing time, or as the log of accumulative mixing energy, up to the ultimate mixing uniformity of the mixer; (3) ultimate mixing uniformity and maximum resultant strengths are obtained more rapidly when mixing slightly on the dry side of optimum than on the wet side of optimum; (4) at any given mixing time or at any given I the strength of soil-cement mixed and compacted slightly dry of optimum is greater than when mixed and compacted slightly wet of optimum; and (5) there is appreciable room for improvement in mixing, especially for the clayey soils.

## ACKNOWLEDGMENTS

The writer is indebted to his thesis supervisor T. W. Lambe for his encouragement and constructive criticisms and to R. T. Martin, A. S. Michaels, and V. Puzinauskas for their helpful ideas. The writer is grateful to J. Irvine and J. Winchester of the Chemistry Department at the Massachusetts Institute of Technology for their assistance in the development of the radioactive-tracer technique for determining cement concentrations in soil-cement.

The research described in this paper was conducted for a thesis to be submitted in partial fulfillment of the requirements for the S. M. Degree at the Massachusetts Institute of Technology and was sponsored jointly by the Massachusetts Institute of Technology and the Department of Public Works of the Commonwealth of Massachusetts.

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## *Discussion*

**JAMES H. REYNOLDS, Jr., ERDL, Roads and Airfields Branch, Fort Belvoir, Virginia**—Congratulations are in order for this vigorous attack on a significant problem which only recently has received adequate attention. The ingenious techniques are particularly interesting, and emphasize as no other approach has done the remarkable benefits that could be derived from proper design of equipment and of field processing. Results obtained are conclusive in their trends and startling in some of their implications.

Although the increase of strength with decreasing $I_V$ is conclusive as shown in Figure 3, it is felt that other possible contributors to this effect should be discussed. (Here it might be noted that $I_V$, being analogous to the coefficient of variation, is, in fact, a direct index of non-uniformity.) Data for densities of the strength samples perhaps should be included if only for the peace of mind of the reviewer, since great strength increases are possible with relatively little density increase. Again, it might have been noted that there is a well defined (although small) increase of mean cement content with mixing time, strength, and (inversely) with the $I_V$. It is perhaps unrealistic to assign the very large strength increase to the small increase in cement content, but the undeniable relationship should be examined. It is of interest that Cornell, under contract to ERDL, has developed relationships for tensile strength on CL and CH soils which apparently verify Baker's conclusions.

As dramatic as Figure 3 is (strength versus $I_V$), enthusiasm must unfortunately be tempered by the practical consideration of Figure 5 which shows that to double the strength, the accumulative energy must increase over 200 fold—a consideration which offers considerable challenge to equipment manufacturers and field engineers who expect to extrapolate or exploit these or similar studies in their designs. (A plot of accumulative energy versus $I_V$ should perhaps be included in the paper to underline this point.) It might be noted that J. C. Smith of Cornell, under contract to ERDL, has adapted the Michaels-Puzinauskas technique to the evaluation of full scale commercial mixers with some encouraging results. For the first time, it is felt, the way has been opened to definitive comparisons of mixing equipment.

Some expanded discussion of the Cobalt 60 technique might be in order; particularly desirable would be the relationship between radioactivity of the mix versus cement content. This approach may receive wide application in laboratory studies and deserves separate treatment, particularly in view of the clumsy (although accurate) ASTM titration procedure. In this connection it may be pointed out that ERDL has analyzed a large number of field samples of soil-cement by means of an automatic titrator. Control samples show that with this expedient method an accuracy of 0.1 percent in cement content may be expected on a sample with 14 percent cement.

It might be noted that "tighter" strength results may perhaps have been possible had strengths been taken after 14 or even 28 days curing since with NPC the strength-time slope is quite steep at 7 days.

A plot of the mean coefficient of variation of cement content (taken from Baker's data) and of the $I_V$ versus time shows that no more accuracy is to be expected of the C. V., although the slightly flatter slope of the C. V.—time relationship (on a log-log plot) illustrates lower sensitivity of the C. V. to the time factor, and thus may allow easier analysis. Other than its convenience in plotting, little immediate advantage can be seen in the use of $I_V$, since it is, essentially, the standard deviation divided by a constant.

Again, to present some academic opposition to an academic concept, even the most "perfectly mixed" soil will, when sample size equals particle size ($<1\mu$) be poorly mixed.

One possible explanation for the wet-versus-dry side of optimum discrepancy (MIT versus PCA) may be, that in the field, the soil water is already present and thoroughly diffused and absorbed in the soil system, thus not being as readily available for the hydration of the additive. In Baker's experiments the water was added to air-dried soil and not being thoroughly diffused (despite the hand mixing) may be easily available to the cement. It should further be noted that Baker's conclusions are derived from initial strength, whereas the case for "optimum plus" mixing is largely based on the durability of cement under weathering conditions.

The next step is obviously to extend these and similar analyses to evaluation of other types of laboratory equipment, full-scale equipment, and to surfacing design principles, and then to saturate equipment manufacturers and design engineers with their significance.

**P. J. M. ROBINSON and J. F. CAPPS**—Baker's paper is a valuable contribution to the work initiated at M. I. T. on the problem of mixing, and in its grasp of the significance of the factors affecting the problem and the practical value of the applications suggested for its conclusions is worthy of study both from the academic and the engineering point of view.

As far as the contents of the paper itself are concerned, perhaps the salient point which occurs is that pointed out by the

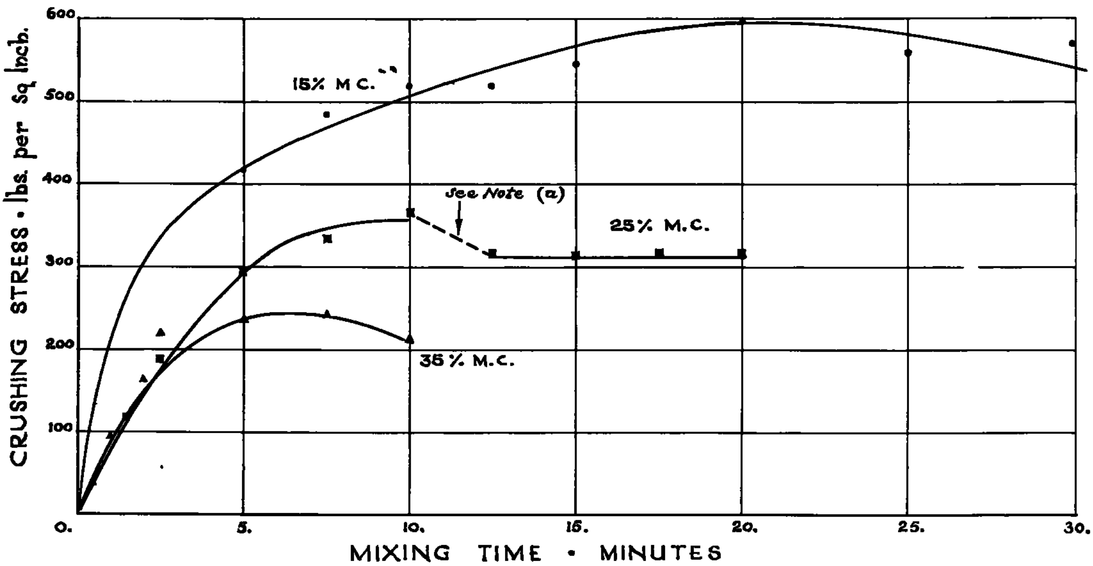

Figure A. Relationship between mixing time and compressive strength for Ferndown clay + 20 percent of 417 cement in Winkworth mixer.

author—the large variation in strength of soil-cement specimens. It appears that the author makes the assumption that as the standard deviation of 10 specimens is 8 percent of the mean, then each of his results, the compressive strength of one specimen, can be out by plus or minus 8 percent. In fact, if 10 results do have a standard deviation of 8 as Grubb (Reference A) has shown, the range over which the individual results can be assumed to lie is $\frac{8}{0.3249}$ percent or ± 12 percent.

This is based on samples mixed by hand. Machine mixing is anyway not as good from the point of view of consistency. In an experiment at M. E. X. E. 10 specimens of identical mixes, two sets mixed in different machines, the third set mixed by hand, gave coefficients of variation of 14. 5, 20. 1 and 10. 7 percent. While the value of 10. 7 percent agrees well with Baker's result on hand mixing, it does show that the strength errors from machine mixes are likely to be considerably higher than he estimates. The fact that in the early stages of machine mixing the consistency will be even less than the values quoted and one specimen will, therefore, have a greater possibility of not having a truly representative strength makes the likely error still greater. The point of this argument is that for indicating anything other than general trends in a series of results, one specimen is not sufficient, as Baker has pointed out, and British practice is to base results on ten specimens wherever possible. As Baker has further pointed out, this increases the magnitude of the work and makes the evaluation of a series of tests very laborious. Nevertheless, it is quite clear from his results that this large number of specimens is the only basis for accurate work. Incidentally the author uses on pages 9 and 19 the expression Standard Deviation, for a percentage. British practice is to use the term Coefficient of Variation for this figure.

A further point is the consideration of the performance of the cement itself. For example, in the above specimens where identical mixes were used, the cement being 417 a proprietary brand containing calcium chloride. While the coefficients of variation were as quoted, giving an indication that the hand mixing gave a marked improvement on the dispersion of the cement through the soil, yet the strengths achieved in the order quoted were 652, 594 and 484 lb. per square inch respectively. This may indicate that there is an optimum degree of mixing for the attaining of a high strength and that further dispersion of the cement may produce a lower strength. This is illustrated graphically in Figure A, which shows some results obtained in an experiment at M. E. X. E. It is notable that after reaching a peak there is a fall in crushing strength. This would lead to the conclusion that the two minutes in the machine in Baker's work is not the time required by that particular mixer to reach this optimum value. This is borne out by the curve in Figure B for a double paddle mixer with sand, where six minutes mixing has not achieved this point. The finger prong mixer used by Baker would not be regarded as likely to equal a double paddle mixer in performance, so that the optimum value would be reached even later.

(It is not clear whether this drop in strength is a disadvantage from the point of view of practical considerations of a constructional material, and investigations as to performance under freezing conditions and so on would be interesting.)

As regards the difference in strength due to the change in moisture content, this might be only due to the increase in the water-cement ratio rather than to any difference in the mixing itself. The similar power absorption figures for the two sets of results shown by Baker do not indicate that the properties of the soil have changed very much as far as the mixer is concerned. Tests at M. E. X. E. on sand, Figure C, over the range of moisture contents used by Baker, for which the power-moisture-content figures are given in Figure D show a difference of about 30 psi. This is a similar figure to Baker's results. Experiments with a clay where the power figures shown in Figure F do vary considerably with moisture content still show, as in Figure E that the increasing moisture content gives about this decrease in strength. From these, it might be inferred that even where moisture content is affecting the degree of mixing, increases give decreasing strength, which is what would be expected from experience in concrete practice.

There is a further factor which will cause a difference in Baker's results which may modify the above, and that is the difference in density which, since he has not

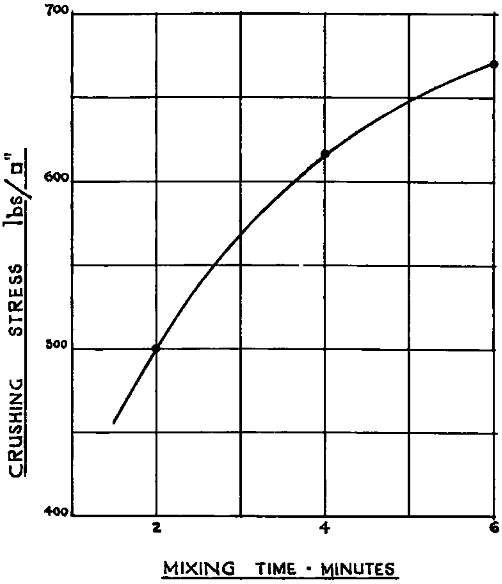

Figure B. Relationship between mixing time and crushing strength for Hurn sand + 14 percent of 417 cement at 13 percent moisture content and 106 pcf. dry density mixed in a double-paddle mixer.

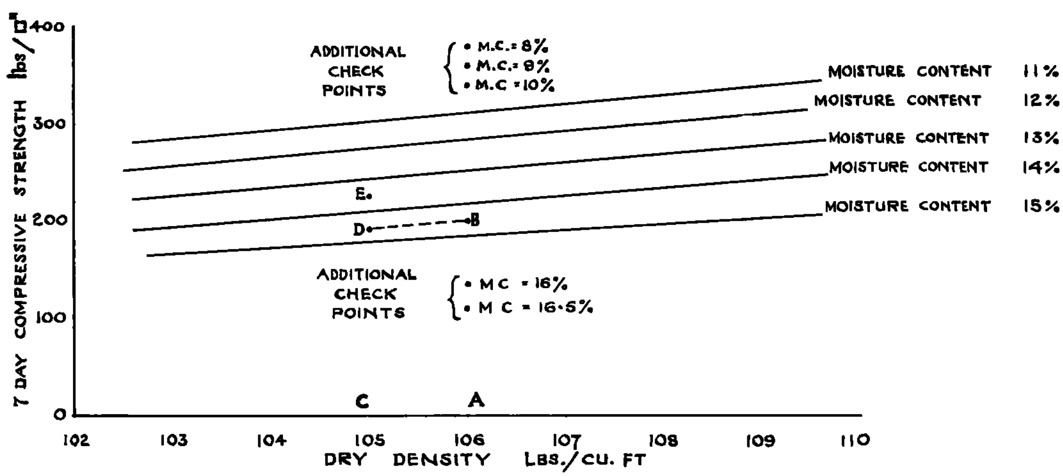

Figure C. Effect of moisture content and dry density on compressive strength of Hurn sand + 8 percent N.P. cement + 2 percent lime.

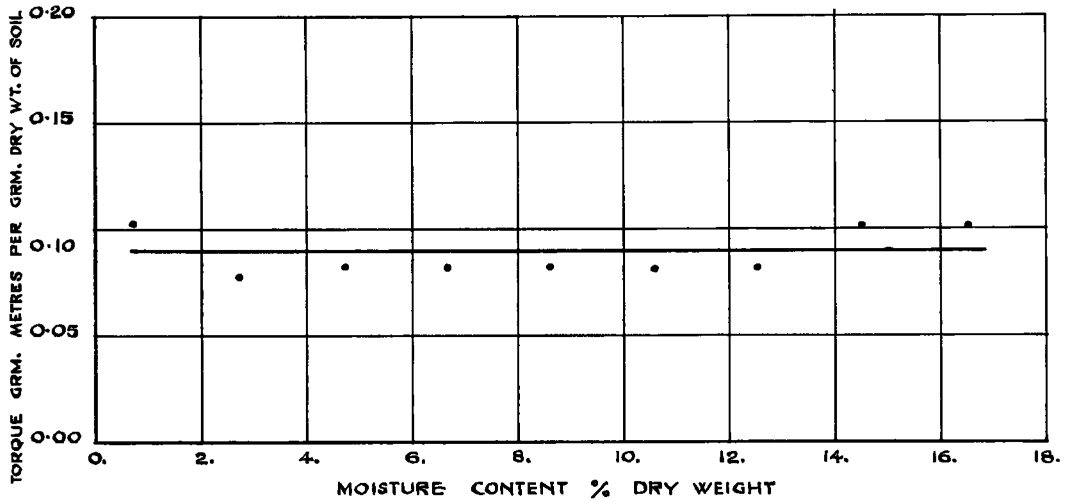

Figure D. Relationhhip between moisture content and mixing power for Hurn sand in Baker-Perkins double Z-paddle mixer.

given values, it has not been possible to compute.

The question then arises as to how valid are the conclusions which Baker has drawn. Those which are considered to necessitate further discussion are enumerated below.

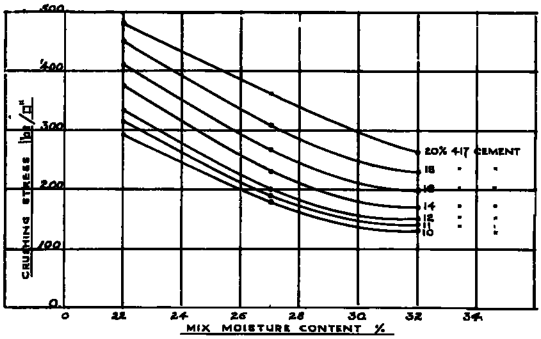

Figure E. Relationship between moisture content and crushing strength for Ferndown clay + 417 cement.

## Conclusion 2

Strength varies as the log of mixing time, or as the log of accumulative mixing energy up to the ultimate mixing uniformity of the mixer.

It may be that Baker has not reached the ultimate mixing uniformity of this mixer. While the linear log relationship seems to hold before the highest compressive strength is being approached, as shown in Figure G, it appears that a peak is reached and the relationship changes. That this peak is not due to a loss of strength due to setting phenomena occurring with the quick action '417' cement is shown by the relationship for normal Portland which indicates the same thing at about the same time. This diagram confirms rather clearly that Baker's mixer is less efficient than the double paddle type, and it even seems as though the subsequent handling and forming of the specimen in the early stages of mixing may confer a higher strength than the mixing alone.

## Conclusion 3

Ultimate mixing uniformity and maximum resultant strengths are obtained more rapidly when mixing slightly on the dry side of optimum than on the wet side of optimum.

It appears that Baker may have been experiencing what appears to be due to moisture along with sandy soils, though it is not clear what effect density may have had.

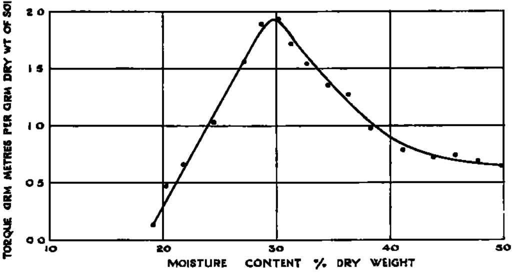

Figure F. Relationship between moisture content and mixing power for Ferndown clay in Baker-Perkins double Z-paddle mixer.

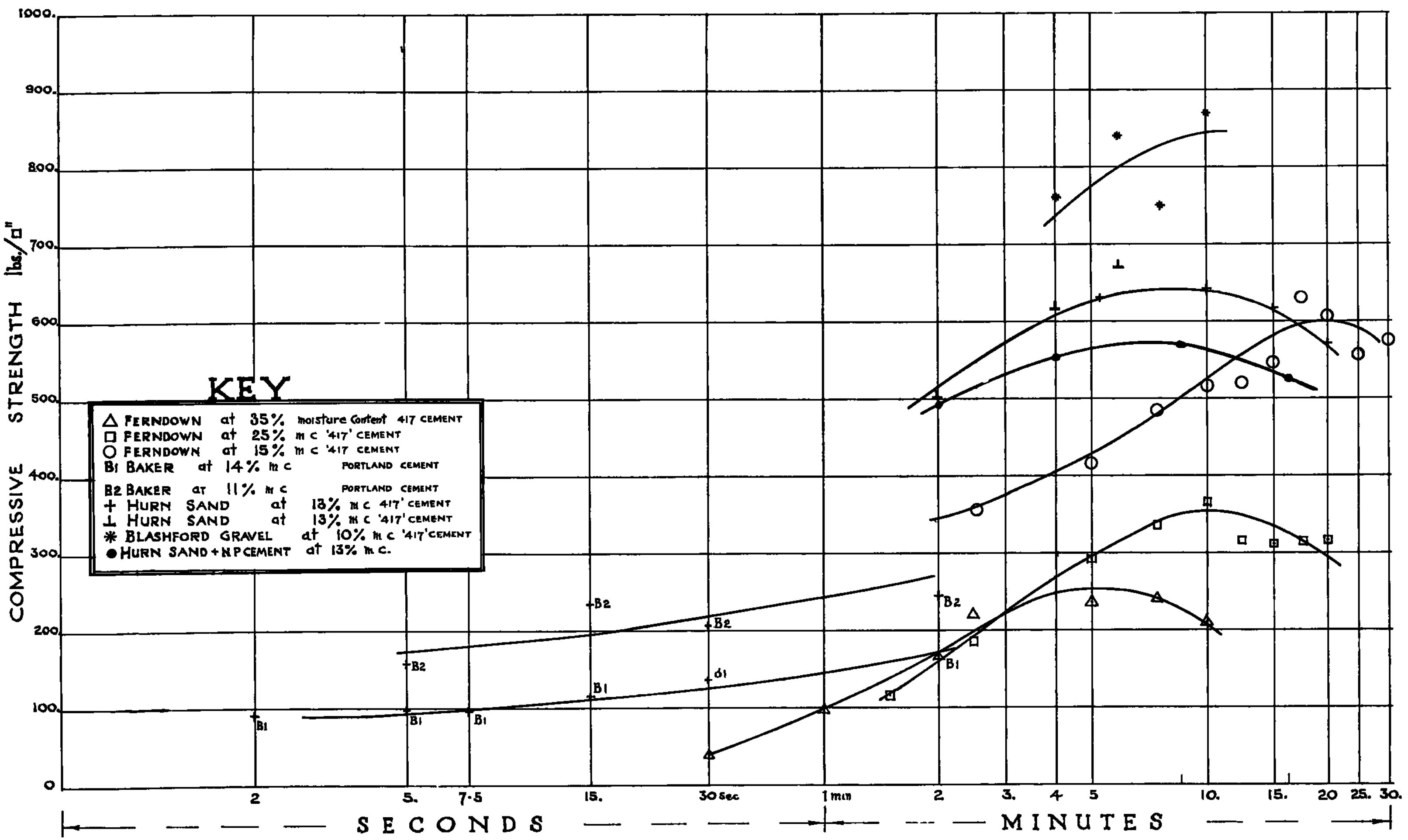

Figure G. Relationship between mixing time (logarithim scale) and compressive strength of various soil-cement mixes.

Certainly his conclusion does hold good for the clay soils but only as far as the magnitude of strength is concerned. Although the values of the maximum strengths are higher in the dry state, the wet soils achieve their maximum more quickly.

Finally some comment is necessary on the method of measuring the degree of mixing. Work is proceeding in Britain on methods based on the chemical determination of the cement content. So far, this has not proved a satisfactory system but nothing exhaustive has yet been planned with it. Baker's method certainly seems to offer a most satisfactory solution from the point of view of the ease with which it can be carried out. Certainly it is clear that compressive strength may be most misloading if used as a measure of the mix achieved. If this work continues, some consideration should shortly be given by those concerned with this problem in Britain and America to standardising both the methods used in carrying out this work and the presentation of the results obtained from it.

NOTE (a) In Figure A it will be noticed that there is a discontinuity in the graph of results for 25 percent moisture content. This is reproduced from the original where it was thought that this might be because the tests with 12½ - 20 minutes mixing time at this moisture content were carried out some time after the other tests were completed, and a different batch of cement was used for them. In fact, when these results are interpreted as on Figure G they do not seem to represent such a discontinuity.

Reference A.

"The Best Unbiassed Estimate of Population Standard Deviation Based on Group Ranges". Frank E. Grubbs, Chalmers L. Weaver, Ballistic Research Laboratory No. 596, Aberdeen Proving Ground, 1946.

**CLYDE N. BAKER, Jr.**, Closure—The author is grateful for the critical discussion of his paper by Reynolds. His suggestion that density data should be included in the paper is a point well taken. The addition has been made, and it is interesting to note that the densities are approximately the same for the test series at both water contents. The suggestion was offered that the inclusion in the paper of a plot of radioactivity versus cement content would be desirable. This relationship is linear, since the cement used in all the tests contained Cobalt 60 in the form of cobalt oxide such that one gram of cement had an activity of 0.05 micro curies. This activity was chosen as being about the minimum with which satisfactory counting results could be obtained in the scintillation counter.

The author also appreciates the commentary by Robinson and Capps as it significantly adds to the value of the paper. Their comments evidently result from an extensive amount of laboratory research.

It should be pointed out that the data discussed in this paper were only the first part of a series of tests. The complete results and conclusions from these tests can be found in a thesis by the author under the title "Strength of Soil-Cement as a Function of Degree of Mixing," M.I.T., June 1954.

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