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CONTENTS

FOREWORD

This RECORD provides information on ways to improve natural soils so that they will perform better in roadways. The roadway capacity of natural soils can be very significantly improved by a rearrangement of particles into a more dense and stable mass.

Lee and Suedkamp studied the relationships that exist in various soils when their particles are rearranged with additions of moisture. As in all nature, these characteristics are neither simple nor easily understood, but this addition to the literature in defining relations will be of considerable benefit in the logical control of compaction effort and the establishment of designs based on predictable results.

In further efforts to improve the benefit-cost ratios for stabilized soils, Drake and Haliburton investigated the use of minor amounts of sodium chloride as an additive to lime to modify its reactions with soil so that higher earlier strengths could be obtained. To predict the efficacy of treatment and the strength obtained, they developed an accelerated curing test. These minor additions may force changes in construction practice, but they will bring about significant advantages.

The next 4 papers deal more directly with lime stabilization of soils and cover the range of heavy clays and organic soils through shales. Neubauer and Thompson studied the stability properties of uncured lime-modified soils as a particular aid to alleviating difficult construction conditions due to highly plastic cohesive soils in a high moisture state. The results are truly outstanding. Alexander, Smith, and Sherman studied various commercial limes, both hydrated and quick, that are used in stabilizing some California soils. The significant improvements in strength with the use of the quicklime are of high interest, although the authors did not discuss the question of safety when quicklime is used. Hoskins, Hammerquist, and Irby briefly describe their efforts to find economical additives that would control the expansion of Pierre shale. They recommend that continuing studies be made to assess the possibility that increased amounts of a less expensive material could be economically more advantageous than the optimum amount of a more expensive material. Handling organic soils and realizing significant strength improvements through their manipulation have traditionally been difficult. The improvements demonstrated by Arman and Munfakh are indeed significant and were brought about through the use of lime additives. At least 2 percent lime was necessary to satisfy the base-exchange capacity of the organic materials before effective modification or stabilization could take place.

The last paper reviews the literature relating to stabilization of soils using sodium silicate. This process has been used mainly in stabilization at depth, but nevertheless there are possibilities of application to areas closer to the surface.

CHARACTERISTICS OF IRREGULARLY SHAPED COMPACTION CURVES OF SOILS

Peter Y. Lee, South Dakota State University; and
Richard J. Suedkamp, Federal Highway Administration

Although the curve of dry density versus moisture content commonly yielded by the standard AASHO compaction tests contains a single maximum, some soils have produced more complicated curves. Because these soils have not been well studied, extensive work has been carried out to establish the existence of irregularly shaped curves. The primary effort was focused on examining, from both a macroscopic and a microscopic level, the characteristics that lead to such curves. The investigation has established 4 types of compaction curves: one with a single peak, one with an irregular $1\frac{1}{2}$ peak, one with a double peak, and one that is almost a straight line with no distinct maximum dry density or optimum water content noted. There is a correspondence between the index properties on the modified Casagrande's chart and the kind of curve. The mineral constituents of the soil samples also affect the shape of the moisture-density compaction curve.

•SOILS with different physical properties react differently when exposed to the compaction process. Some clays and other highly cohesive soils are very unstable materials. As a result, use of such material is usually avoided in engineering construction. However, extensive areas of the world are covered by such soils, and they must often be used because it is economically unfeasible to bypass them or replace them with more suitable material.

As a consequence, typical soils have been subjected to standard compaction tests in which the maximum dry density achievable with given amounts of compaction is determined as a function of moisture content. The tests yield curves (Fig. 1) with a single peak for most soils and with irregular shapes for some soils (1, 2, 3, 4, 5, 6, 7). Because there are insufficient data on those soils, existing theories of compaction have not been extended to cover adequately the irregular curves.

The phenomena occurring during compaction are not completely understood. Nevertheless, 4 theories exist: Proctor's capillarity and lubrication theory (8), Hogentogler's viscous water theory (9), Lambe's physicochemical theory (10, 11), and Olson's effective stress theory (1). None of these adequately explains the irregularly shaped compaction curves found.

Olson obtained 2 peaks in the moisture-density curve for illitic clay. Lacking a diagnostic laboratory study, he gave only a tentative explanation of the results. He considered the concave portion of the curve between the 2 peaks to be governed by negative pore-water pressure. The formation of pore-water menisci between soil particles at their contact points caused an increase in effective stress from the surface tension of the pore fluid. The minimum dry density between 2 peaks occurred when all the menisci were fully developed. Beyond this point flattening of the menisci reduced the pressure differential across the surface, whereupon lubrication and double water layers helped to develop a more dispersed structure. The resulting increase in dry density continued until the second peak of optimum water was obtained. With only inconclusive data available, Olson suggested that a double-peak compaction curve could form only

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in soils containing a dominant percentage of plate-like colloidal particles. He also noted that a double-peak curve has not been observed with soils containing only a small amount of clay. However, Johnson and Sallberg (2) found that some sands exhibit very irregular compaction curves that are almost straight lines. They also noted that clay soils that are often highly structured may yield irregular compaction curves when tested by the standard AASHO Method T 99-57. Lambe (7) explained that the 1½-peak curve for sandy soils may be due to a capillary force phenomenon known as bulking. The existing theories could not be applied to these unusual compaction curves.

Soil to be used as engineering material must behave in a definite, explainable manner. However, present available data are incomplete, and the full range of soils exhibiting irregularly shaped curves is not known. Determining the optimum water content to achieve maximum dry density for soils exhibiting such characteristics could present a problem to the engineer. A system of classification aids the engineer in analyzing a soil so it can be used effectively as an engineering material. No one has attempted to group soils according to their compaction behavior. Such a system would further aid the engineer in evaluating a soil for earthwork construction.

An investigation was carried out to examine the existence of irregularly shaped moisture-density compaction curves. The experiments had 2 objectives: to establish the existence of irregularly shaped compaction curves and to examine from both a macroscopic and a microscopic level characteristics of soils that produce irregularly shaped compaction curves.

The results of this study confirm the existence of irregularly shaped compaction curves and also provide information on whether a correlation of compaction curve shapes and index properties exists. Knowing effects of various mineral constituents on the shape of the compaction curve increases one's understanding of soil behavior. In addition, the effect of varying the time between preparation of the sample and testing time was also investigated.

LABORATORY INVESTIGATION

Soil Test Specimens

To study more than one aspect of this investigation at one time, we followed a testing program that combined a number of variables (for the purpose of obtaining various shapes of compaction curves). Soil samples having different physical properties were tested. Because of the cost and time involved, it would have been quite difficult to obtain from different parts of the country a large number of soil specimens that would exhibit different physical properties. It is also difficult to determine exact mineral percentages of soil samples through X-ray diffraction analysis. Therefore, so that test samples would exhibit a wide range of physical properties and at the same time the exact mineral percentages of each specimen would be known, soil samples were combined from known minerals. The 5 most important and common minerals found in soils are kaolinite, montmorillonite, illite, quartz, and feldspar (12). It is known that more than 90 percent of all soils in the world are made up of silicate minerals. Kaolinite is the most common 2-layered silicate mineral encountered by engineers. The 2 most common 3-layered structures in soil are montmorillonite and illite. Feldspar and quartz make up a framework silicate structure. Because they are common rock-forming minerals, the frameworks are abundant in soils and also are found in pure deposits commonly known as sand. As a result 4 primary samples consisting of kaolinite, montmorillonite, illite, and sand, which was locally available, were obtained. The sand sample consisted primarily of feldspar. From these 4 samples an additional 24 samples were obtained by mixing arbitrary combinations of each (Table 1). In addition 7 natural samples, typically encountered in engineering construction from various locations in the country, were also tested as a basis of comparison (Table 2).

Test Procedure

Three types of compaction effort are frequently used in compaction tests: impact, kneading, and vibratory. Static compaction is also used on a limited scale in the preparation of test specimens. In addition to the differences in types of compaction effort,

Figure 1. Typical single-peak and irregularly shaped compaction curves.

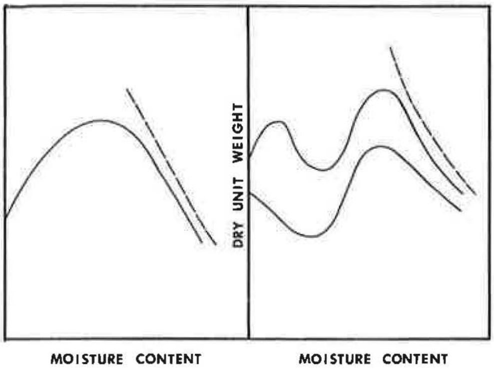


Table 1. Arbitrary-combination soil samples.

Sample	Sand[a] (percent)	Montmorillonite[b] (percent)	Illite[c] (percent)	Kaolinite[d] (percent)	Liquid Limit	Plasticity Index
1	0	0	0	100	35	5
2	0	0	100	0	51	21
3	0	100	0	0	548	500
4	100	0	0	0	0	NP
5	0	50	0	50	226	174
6	50	50	0	0	230	198
7	25	0	50	25	40	19
8	25	25	50	0	102	78
9	50	25	0	25	117	106
10	25	50	25	0	172	142
11	0	50	50	0	170	129
12	25	0	25	50	30	6
13	25	50	0	25	161	132
14	0	0	50	50	43	16
15	50	25	25	0	72	50
16	50	0	25	25	22	5
17	25	25	0	50	112	86
18	0	25	25	50	90	57
19	50	0	50	0	27	8
20	50	0	0	50	18	4
21	0	25	50	25	105	73
22	0	50	25	25	196	157
23	85	15	0	0	51	26
24	85	0	15	0	13	NP
25	85	0	0	15	11	NP
26	60	0	10	30	18	4
27	75	0	0	25	13	NP
28	75	0	25	0	17	NP

Note: All percentages identified by X-ray diffraction.

[a] Pure sand obtained locally.

[b] Bentonite obtained from Baroid Division of National Lead Company, Houston.

[c] Grundite obtained from Green Refractionery Company, Morris, Illinois.

[d] Clay hydrite-121 obtained from Thompson-Hayward Chemical Company, Kansas City, Kansas.

Table 2. Natural soil samples.

Sample	Soil	Liquid Limit	Plasticity Index
29	Pierre bentonite	310	244
30	Oregon clay, gray	70	32
31	Washington clay, red	30	8
32	Texas clay, light brown	82	47
33	Pierre shale, sample 1	68	29
34	Pierre shale, sample 2	110	66
35	New Orleans clay, gray	78	29

there are other variations that influence the moisture content-unit weight relations. These include size of mold, amount of compaction effort, maximum size of aggregates permitted, method of supporting the mold, and method of preparing the soil for testing. As a result, the standard methods have been set up for the various types of compaction. The most commonly and widely used is the standard AASHO method also identified as ASTM Designation D 698-70. In this method the soil fraction passing through the No. 4 sieve (4.76 mm) is compacted in 3 layers in a 4-in. diameter mold by dropping a 5.5-lb hammer 25 times per layer from a height of 12 in. In this investigation, we were interested only in the shape of moisture-density curves produced by this standard method. With the exception of the wetting period, other differences in apparatus and procedures are not covered within this investigation.

Because this investigation was concerned with the irregularly shaped compaction curves, much effort was focused on the low water content range. To diagnose the possible cause of such irregular curves, we varied the wetting period of the cohesive samples. The alteration of the procedure consisted of running the compaction test immediately after the preparation of the sample and then breaking each sample down and passing it through a No. 4 sieve. Thereafter the samples were kept in a control room for a period of 7 days, after which they were tested again. Because highly cohesive soils were used as the test specimens, the water content increments were varied by approximately 2 percent. The index properties consisting of plastic limit and liquid limit were determined for each test sample.

DISCUSSION OF TEST RESULTS

The extensive laboratory investigation involving 35 different soil samples and more than 700 standard compaction tests revealed many irregularly shaped compaction curves. Figures 2 through 7 show examples of the results obtained in this laboratory investigation. The existence of irregularly shaped compaction curves is clearly shown by the results of this study. Some of the curves (Fig. 2) clearly reveal the existence of double peaks, which were noted by Olson. A number of the irregular shapes resemble the 1½-peak curve (Fig. 3) that was found to be characteristic of some sandy soil specimens. Some of the test samples that had a large percentage of montmorillonite exhibit an oddly shaped curve with no distinct maximum optimum water content (Fig. 4).

A correlation was performed of the index properties (Table 1) and the compaction curve shapes. The results, shown in Figure 5, indicate an approximate range of soils that will produce irregularly shaped compaction curves. The soil samples tested with a liquid limit between 30 and 70 usually yield the typical single-peak compaction curve. However, the results indicated an exception to this range. Sample 23 (Fig. 3) exhibits a curve that is irregularly shaped. Soil test samples with liquid limit less than 30 and greater than 70 usually produce irregularly shaped curves. However, this range also had a discrepancy. Sample 20 located in the lower range near the boundary of irregularly shaped curves resulted in a typical single peak. It must be remembered that the liquid limit of 30 and 70 enclose only an approximate range of typical single-peak curves.

The discrepancies might be caused by the various mineral constituents of the test samples. Irregularly shaped curves resulted in the arbitrary combinations whenever montmorillonite was present. It is evident from the data that 50 percent or more of sand also caused irregularly shaped compaction curves. The only exception to this was sample 20, which had 50 percent sand and 50 percent kaolinite. The resulting index properties of this sample (LL = 18 and PL = 4) places it in the lower region of irregularly shaped curves. The compaction test, however, resulted in a curve that was typical.

Sample 23 consisted of 85 percent sand and 15 percent montmorillonite. It would be expected that this curve would be in the lower region of irregularly shaped compaction curves. The small amount of montmorillonite present yielded a liquid limit of 51. At the same time, however, the compaction test resulted in a distinct, irregularly shaped compaction curve if it is present in amounts of 15 percent or greater. Montmorillonite is an extremely plastic soil, and the small amount of only 15 percent combined with 85 percent sand, which is extremely nonplastic, yielded a liquid limit of 51 and plastic limit of 26. This is quite significant because in the Triangle Textural classification of

Figure 2. Double-peak compaction curves.

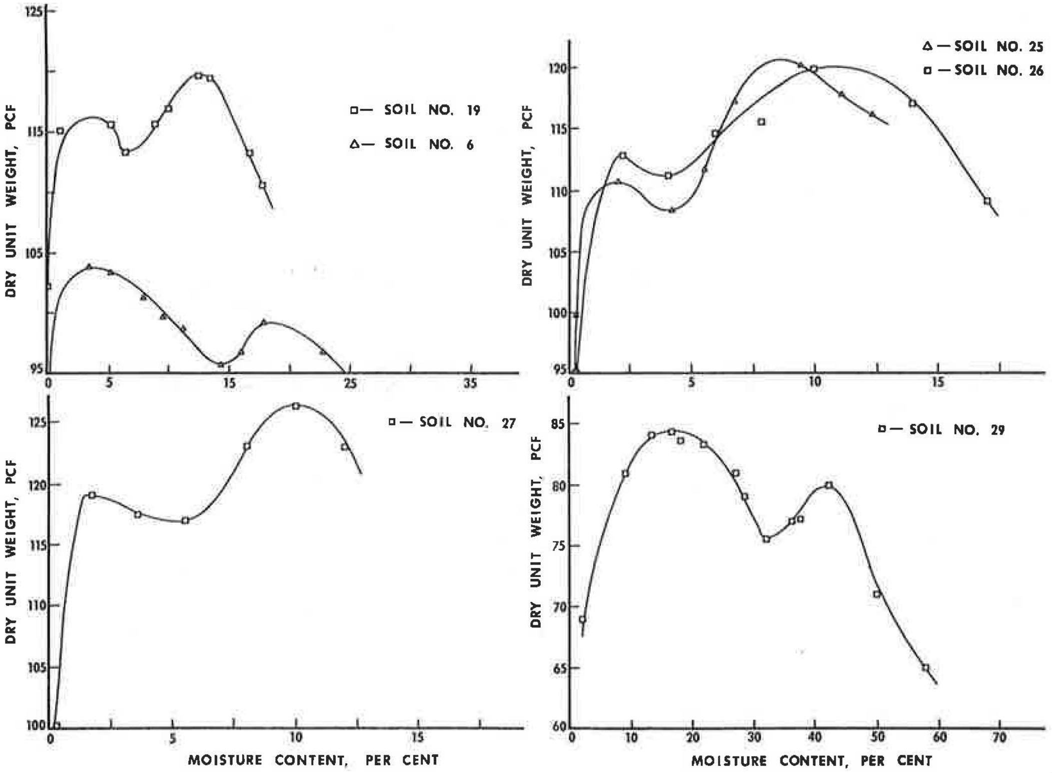
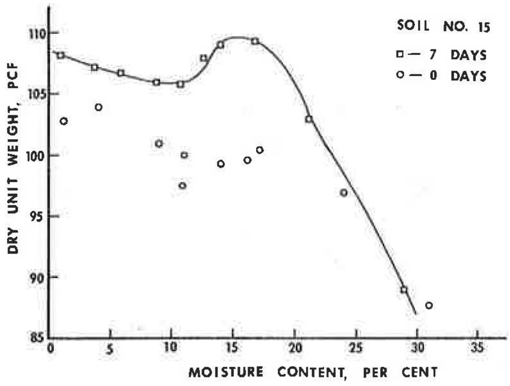


Figure 3. One and one-half peak compaction curve.



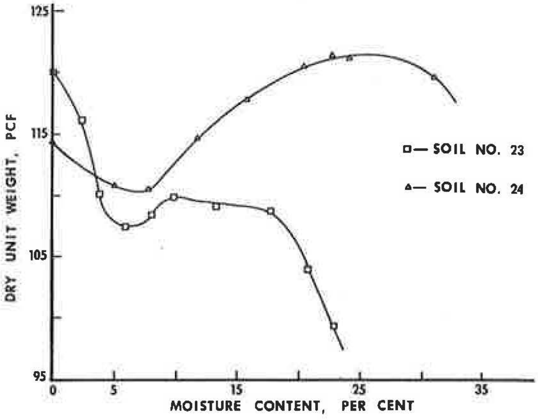


Figure 4. Oddly shaped compaction curves.

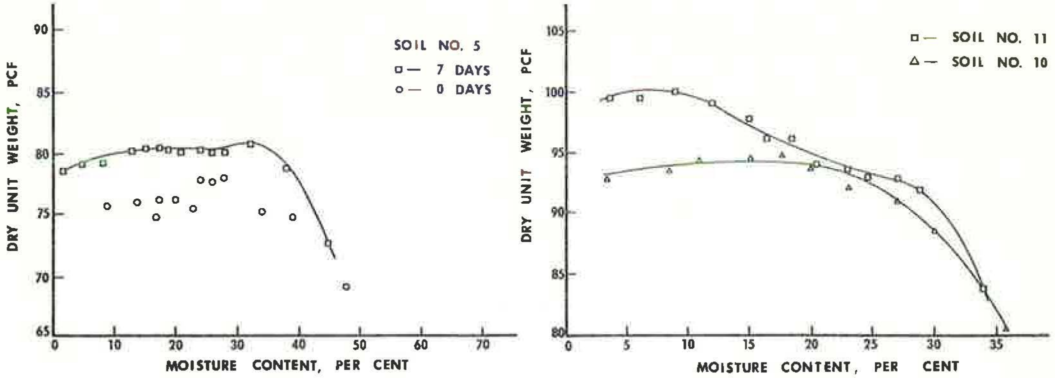


Figure 5. Correlation of laboratory results using modified Casagrande's classification.

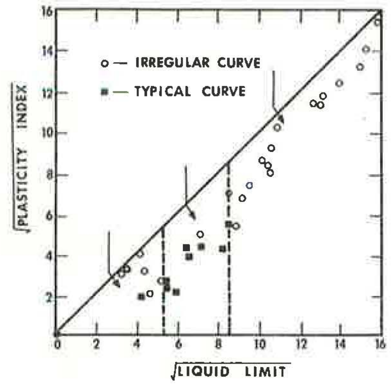


Figure 6. Compaction curves of kaolinite and illite.

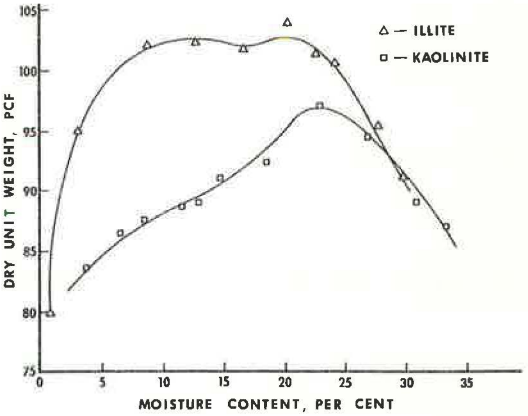


Figure 7. Four types of compaction curves found from laboratory investigation.

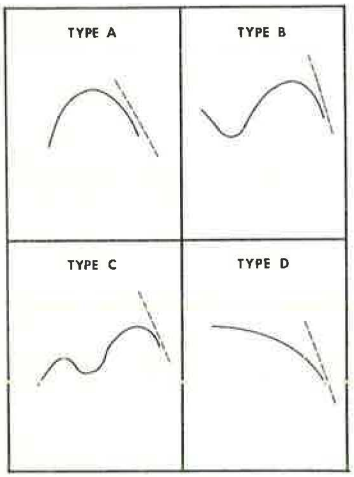
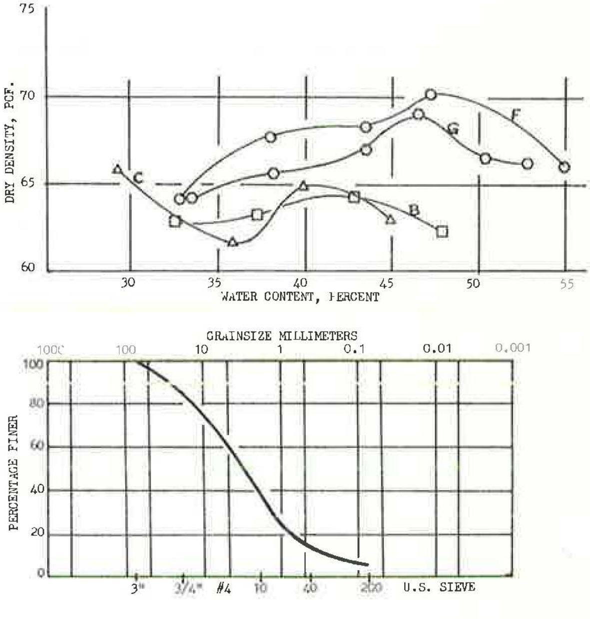


Figure 8. Moisture-density relations and grain-size distribution in Pennsylvania tests.



soils, which is based on the relative percentages of sand, silt, and clay, a soil sample containing 80 percent or more of sand and 20 percent or less of clay or silt is classified as sand. The compaction curve shape of this sample (Fig. 3) is characteristic of sand specimens, but the large liquid limit places it in the category of clays according to Casagrande's classification system.

The curve that is characteristic of soil containing 100 percent or a dominant percentage of sand and the remaining portion containing a certain amount of illite and montmorillonite is the 1½-peak curve (Fig. 3). In these curves a very high dry density is obtained at zero water content. This was a unique characteristic of some of the sandy samples tested. Under those circumstances water acts like an antilubricant. Observations by Horn (13) have shown that water is indeed an antilubricant for 3-dimensional network silicates such as quartz and feldspar. Negative pore-water pressure can also play a part in the compaction of sandy samples. At low water content some menisci begin to form in sandy samples because only a small portion of the water is absorbed by the particles themselves. These menisci tend to hold the particles together, thus increasing the shear strength and decreasing the density. Upon the addition of more water, the negative pore-water pressure and the antilubrication effect come in balance. Then if more water is added lubrication proceeds and the particles slide over one another, thus increasing the dry density. This phenomenon progresses until enough water is added so that it begins to displace the soil particles; this occurs at maximum dry density. Then the dry density decreases as the water displaces the soil particles.

Kaolinite yields typical single-peak compaction curve. The compaction curves produced by illite show the shape somewhat between single peak and slightly double peak (Fig. 6). When small amounts of kaolinite were combined with a large percentage of sand such as in samples 25, 26, and 27, the resulting curves were distinct double peaks (Fig. 2). Kaolinite is a bulky material, and evidently particular structure and size play an important part in the compaction process. Kaolinite has a particle thickness from ⅓ d to $^1/_{10}$ d where d varies from 0.3 to 3 μm. Kaolinite is also a 2-layered silicate sheet structure. Illite and montmorillonite are common 3-layered silicate sheet structures. Illite has a thickness of $^1/_{10}$ d where d varies from 0.1 to 2 μm. In the lower region, it is interesting to note the variation in the compaction curves with the differences in mineral structures present. The 1½ peaks have illite or montmorillonite present with a large percentage of sand, and the double-peak curves have a small percentage of kaolinite and a dominant percentage of sand present.

It is difficult to incorporate water into highly cohesive, heavy-textured clayey material. Because water plays an important part in the compaction process, the effects of the wetting period were also investigated. The wetting period is defined as the time that a sample is allowed to set at a controlled temperature and the water is permitted to become evenly distributed throughout the sample. This problem has been known for some time; as a result, the ASTM specifications require a minimum wetting period of 12 hours for highly cohesive materials. In this test, the samples were run at time = 0 or immediately after preparation and then again at time = 7 days. Some results showed a marked increase in the maximum dry density for most of the soil test samples (sample 15, Fig. 3, and sample 5, Fig. 4). A 7-day wetting period also resulted in a smoother curve. Highly cohesive soils tend to form small packets in which the water is concentrated. This phenomenon causes the water content of the sample to be unevenly distributed, and as a result the compaction curve may be very uneven. Therefore, this confirms the necessity of a wetting period for complete distribution of water. It also emphasizes the lubrication effect of water, which was recognized earlier by Proctor.

SUMMARY AND CONCLUSIONS

Seven hundred compaction tests on 35 soil samples exhibiting the physical properties given in Tables 1 and 2 did confirm the existence of irregularly shaped compaction curves. Certain characteristics of such curves were also established.

When the various compaction curves were correlated with index properties of the test samples following the modified Casagrande's classification scheme, 2 kinds of soils exhibiting irregularly shaped compaction curves were indicated. One consisted of highly cohesive soils with liquid limits greater than 70; the other consisted of soils with liquid limits less than 30. There were some exceptions to this macroscopic correlation. Although the index properties attempt to give a quantitative measure of the composite effects of all basic properties of a soil, they are not wholly sufficient. Nevertheless, the simple index tests usually identify soils exhibiting irregularly shaped compaction curves. If possible, however, they should be supplemented by a mineralogical investigation.

Properties of a clay are determined fundamentally by the physicochemical characteristics of the various mineral constituents present. This investigation revealed that the mineral constituents of the soil samples definitely affect the shape of the moisture-density compaction curve. Indeed, the clay mineral montmorillonite will affect the shape of the compaction curve when it is present in a soil in amounts of 15 percent or more. When the samples in the lower range containing a dominant percentage of framework minerals are combined with the 3-layered silicates, illite or montmorillonite, the result is $1\frac{1}{2}$-peak curve. When more than 50 percent of the framework minerals are combined with the 2-layered silicate, kaolinite, the double-peak curve resulted. The impact of the various minerals on the compaction curve shape is evident. Soils with more than 50 percent of montmorillonite, highly cohesive soils (LL > 100), usually yield oddly shaped compaction curves.

According to this investigation, 4 types of curves exist (Fig. 7). In addition to the typical single-peak compaction curve (type A), the results indicated a $1\frac{1}{2}$-peak curve (type B), a double-peak curve (type C), and a curve with no distinct optimum water content or oddly shaped curve (type D). According to the modified Casagrande's scheme, typical single-peak compaction curves appear between the approximate liquid limit boundaries of 30 and 70. In soils with a liquid limit greater than 70 both double-peak curves and oddly shaped curves are present. Soils with a liquid limit of less than 30 usually produce both double-peak curves and $1\frac{1}{2}$-peak curves.

In highly cohesive soils the wetting period influences the distribution of water throughout a sample and as a result also affects the maximum dry density. When water was first added to a dry sample, it tended to form "packets" consisting of small amounts of soil and a large concentration of water. This uneven distribution of water resulted in scattered compaction data and an uneven curve. However, after a period of time (7 days) the water became evenly distributed. The result was an increase in dry density and a smoother curve. Results have shown that the water becomes evenly distributed after 12 hours. The effect of a wetting period was not so pronounced in soils with low liquid limits.

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DISCUSSION

Umakant Dash, Bureau of Materials, Testing and Research,
Pennsylvania Department of Transportation

The authors have presented an interesting paper on a very practical subject. Although irregularity in the shape of compaction curves for some soils is well known, it is often viewed with suspicion in actual practice and even rejected without much justification. This paper will undoubtedly aid in a better understanding of the irregularly shaped compaction curves. The paper primarily deals with compaction characteristics of soils and presents a correlation of compaction curve shapes with index properties.

Recently, the Pennsylvania Department of Transportation made a study on the compaction characteristics of Pulaski granulated slag. Irregularity in the shapes of the compaction curves for the slag was observed. The shape of the compaction curve for the slag has one of the following characteristics: 1½ peak (type B) or double peak (type C). The slag falls in the category of soils having liquid limit less than 30 and thus confirms the correlation between liquid limit and shape of compaction curve presented by the authors.

The grain-size distribution curve and compaction curves (determined by Pennsylvania test method 106, equivalent to AASHO T-99) for a few samples that were tested are shown in Figure 8.

ACCELERATED CURING OF SALT-TREATED AND LIME-TREATED COHESIVE SOILS

John A. Drake, Dames and Moore, Inc., Chicago; and
T. Allan Haliburton, School of Civil Engineering, Oklahoma State University

This paper describes an investigation to determine elevated temperature and reduced time that would be required for accelerated curing of 2 cohesive Oklahoma soils modified and stabilized with salt and lime and that would produce strength gain and chemical products similar to those obtained by 28-day cure at 80 F and 100 percent humidity. Lime-only treatment data were also collected for comparative purposes. A pilot study at 95 percent humidity and temperatures of 120, 110, 105, and 100 F revealed that a temperature of 105 F best simulated conventional strength-time curing conditions. For both soils, at salt-lime modification and stabilization treatments, 30 hours of 105-F curing produced equivalent 28-day strength. For lime-only treatment, accelerated curing times varied from 30 hours at modification optimum to 72 hours at stabilization optimum. Differential thermal analysis of conventional-cured and accelerated-cured samples indicated similar mineralogical characteristics existed when equivalent strengths were obtained. A tentative-strength mix design procedure using the technique for accelerated curing is suggested.

•RAPID DESIGN of highway base and subbase courses of lime- and salt-lime-treated cohesive soil is hindered by the length of time required to obtain "strength" values for the treated material. Strength gain with time is not very fast, and the strength after 28 days of curing is often taken as a design value; that is the conventional design procedure used at Oklahoma State University. Thus, at least a month is required to develop an adequate design. It would be advantageous to devise a procedure whereby 28-day strengths could be accurately predicted on the basis of tests completed in a much shorter time.

Lime and salt-lime reactions with cohesive soil are chemical in nature, and their reaction rate should be increased by curing at elevated temperatures. However, if the curing temperature is too high, reactions different from those obtained by conventional curing may possibly occur. If the curing temperature is too low, no great decrease in required curing time will occur. The problem in accelerated curing is, then, to obtain a short-time cure that gives both strength gain and chemical reactions equivalent to those obtained by conventional curing procedures.

This paper describes an investigation to determine an accelerated curing procedure that produces results equivalent to those obtained by 28-day curing in a moist room at 80 F and 100 percent humidity. Two cohesive Oklahoma soils treated with salt and lime admixtures were used in the study.

PREVIOUS RESEARCH

A minimum of published material exists concerning accelerated curing of lime-treated soil. Research that has been performed deals with comparisons between strengths of oven-cured and field-cured samples.

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Anday (5) used 2 soils, a clay gravel and a micaceous silty soil, and compared the unconfined compressive strength of field-cured specimens to specimens oven-cured in the laboratory at 140 and 120 F, both treated with 5 percent lime. Field-curing times were 30, 45, and 60 days, and laboratory curing times were 0.5, 1, 3, and 5 days. He concluded from the study that the unconfined compressive strength of samples field-cured for 45 days at summer temperatures could be predicted by an accelerated laboratory curing of 18 hours at 140 F or 2 days at 120 F. However, Anday recommended the use of 120 F in the laboratory curing because the lower temperature caused less moisture loss during curing, was more realistic, created a more convenient curing time and eased handling of the samples, and increased the accuracy obtained with small slopes of strength curves.

In 1961, Anday (6) expanded his research to compare field and accelerated curing of 6 different lime-treated soils native to Virginia. Based on his previous work, 120 F was used exclusively for laboratory curing. Field-curing times remained at 30, 45, and 60 days, and laboratory curing times were changed to 1, 2, and 3 days. Anday concluded that soil-lime specimens cured under field conditions would show an increase in unconfined compressive strength. However, the amount and rate of strength gain would be functions of soil type and climatic effects. Basing his field cure on 3,000 deg-days, or 40 to 45 days if 0 F is taken as datum, he concluded that laboratory specimens cured for 2 days at 120 F should predict the field-cured unconfined compressive strength.

Anday's purpose in both studies was to develop the basis for a quick laboratory method method of determining the suitability of a soil for lime stabilization under standard conditions.

Thompson (7), while assisting in development of the Flexible Pavement Design Manual for the Illinois Division of Highways, performed a study to determine design coefficients for lime-stabilized soils used as highway base and subbase courses. His research indicated that laboratory curing of samples at 120 F for 48 hours produced unconfined compressive strengths approximately equivalent to those obtained on samples cured for 30 days at 70 F and recommended that minimum design strength requirements be based on those results.

Lime and salt-lime treatment of cohesive soil has as a primary objective the improvement of engineering properties by reduction of plasticity or increase in strength or both. Small percentages of lime are usually required to modify or reduce the plasticity of cohesive soils, and little strength gain is attributable to this addition. The "lime fixation point" or "modification optimum" is the minimum lime content at which maximum plasticity reduction occurs. Lime stabilization of cohesive soils is the addition of lime to obtain substantial strength gain. Because obtained strength gains are relatively long term, a standard curing time of 28 days in a moist room is often used as a basis for design strength evaluations. A procedure to accelerate the rate of long-term strength gain is needed, for both actual field use and rapid laboratory mix design. The use of salt (NaCl) in conjunction with lime as a catalyst was evaluated by Marks and Haliburton (1). However, a curing time of 28 days in a moist room was still used to determine strength behavior.

Use of increased temperature to accelerate chemical reactions and thus decrease required curing time for strength development is the basic concept behind any rapid-curing procedure. Problems that arise are control of the rate of strength gain and maximum strength obtained. If the rapid-curing procedure increases the strength too quickly, both the rate of strength gain and the obtained strengths will be different from those obtained by conventional curing procedures. It is also necessary, for efficiency, to choose one temperature for accelerated curing of different soils treated with various percentages of lime and salt plus lime. Thus, the prime consideration of the research was to cure different soils with different chemical treatments at the same elevated temperature and achieve approximately the same mineralogical compactions and strengths as when cured under standard moist room conditions.

MATERIALS

Permian red clay (PRC) of medium plasticity was one of the cohesive soils used in this study. PRC is one of the predominant cohesive soil types of central and western

Oklahoma, originating from Permian marine deposits. The second Oklahoma soil, Roger Mills gray clay (RMGC), was chosen because its physical characteristics and geographical origin are different from those of PRC. RMGC is highly plastic clay, has a distinctive steel gray color, and is much younger in age than PRC. Research on salt, lime, and salt-lime treatment of both soils was done previously by Marks and Haliburton (1, 2, 3, 4).

Table 1 (1) gives the physical properties of PRC and RMGC. The texture of the 2 soils is quite different; the more plastic RMGC contains much lower percentages of the fine clay fraction than does PRC, and PRC contains much lower percentages of the coarse clay fraction.

Lime, in the form of pelletized quicklime supplied by the St. Clair Lime Company, Sallisaw, Oklahoma, was used throughout the study and contained at least 97.5 percent CaO. Rock salt (sodium chloride) was also used as a chemical admixture in the study. The rock salt passed the U.S. No. 40 sieve and contained not less than 99.0 percent sodium chloride.

TESTING PROCEDURE

The amounts of lime and salt necessary to modify and stabilize both PRC and RMGC were previously determined by Marks and Haliburton (1) in their feasibility study of salt-lime stabilization and are given in Table 2. These data were used with previously developed procedures (1) in preparing samples for unconfined compression testing and differential thermal analysis. A modified Harvard miniature (impact compaction) procedure was used to compact unconfined compression samples at optimum moisture and density to values given in Table 3. Impact energy was reduced in scale proportionally from the standard Proctor hammer, and equivalent densities were produced. All samples were then sealed with plastic wrap and dipped in melted wax to prevent moisture loss or gain during curing.

Two sets of samples for each soil type were compacted for unconfined compression testing; one set was cured in the moist room at 80 F and 100 percent humidity for 7, 14, 21, and 28 days. The moist-room-curing temperature of 80 F is similar to average temperatures measured in base, subbase, and subgrade material under Oklahoma highways during the spring and summer construction season (8) and thus may be taken as a reasonable approximation of field-curing temperature. The second set of samples was cured in a Blue-M Vapor-Temp humidity chamber at 105 F and 95 percent humidity for 12, 24, 36, 48, 60, and 72 hours. Unconfined compression tests were performed on at least 3 samples after each of the previously mentioned curing times at a loading rate of 0.02 in./min, equivalent to approximately 5 percent strain in 10 min. The entire sample was saved, and both moisture content and dry density were checked. The average strength of the 3 samples was used unless 1 sample gave results much higher or lower than the other 2; then, its value was disregarded, and the remaining 2 values were averaged.

Differential thermal analysis (DTA) of both PRC and RMGC samples was conducted according to procedures developed in the Soil Mechanics Laboratory (9). Samples for DTA were compacted and cured by using failed samples from strength testing. Differential thermal analysis curves were obtained from a Fisher Model 260 Thermalyzer connected to 1-mv Texas Instruments Serva/Riter II strip chart recorder. Plantinel differential thermocouples were used with a heating rate of 10 C/min from room temperature to 1,200 C and a recorder chart speed of 4 in./hr.

RESULTS OF STRENGTH CORRELATION

Preliminary studies were made of PRC and RMGC samples with 4 levels of lime and salt-lime treatment cured in the humidity chamber at 95 percent humidity and 120, 110, 105 and 100 F to determine the temperature that would most nearly approximate the strength-time behavior of samples cured in the moist room. The data shown in Figure 1 are typical of the effect of temperature on rate of strength gain for the 2 lime- and salt-lime-treated soils. A decrease in accelerated-curing temperature decreases the rate of strength gain. The 120-F humidity-chamber curves do not have a

Table 1. Physical properties of PRC and RMGC.

Properties	PRC	RMGC
Specific gravity	2.72	2.73
Liquid limit	38.6	60.5
Plastic limit	17.6	29.8
Plasticity index	21.0	30.7
Linear shrinkage, percent	18.4	17.8
Less than 10 μ	38.0	82.0
Less than 5 μ	32.0	73.0
Less than 1 μ	26.0	2.0

Table 2. Optimum modification and stabilization percentages for PRC and RMGC.

	PRC		RMGC	
Treatment	CaO	NaCl	CaO	NaCl
Lime modification	4		6	
Lime stabilization	8		11	
Salt-lime modification	4	1	6	2
Salt-lime stabilization	8	1	11	2

Table 3. Miniature standard Proctor compaction data for PRC and RMGC.

Clay	CaO (percent)	NaCl (percent)	Optimum Moisture Content (percent)	Maximum Dry Density (lb/ft^3)
PRC	4		20.0	95.5
	4	1	18.0	100.0
	8		25.0	90.0
	8	1	24.0	97.5
RMGC	6		25.0	92.0
	6	2	24.5	93.7
	11		27.0	92.0
	11	2	23.0	96.0

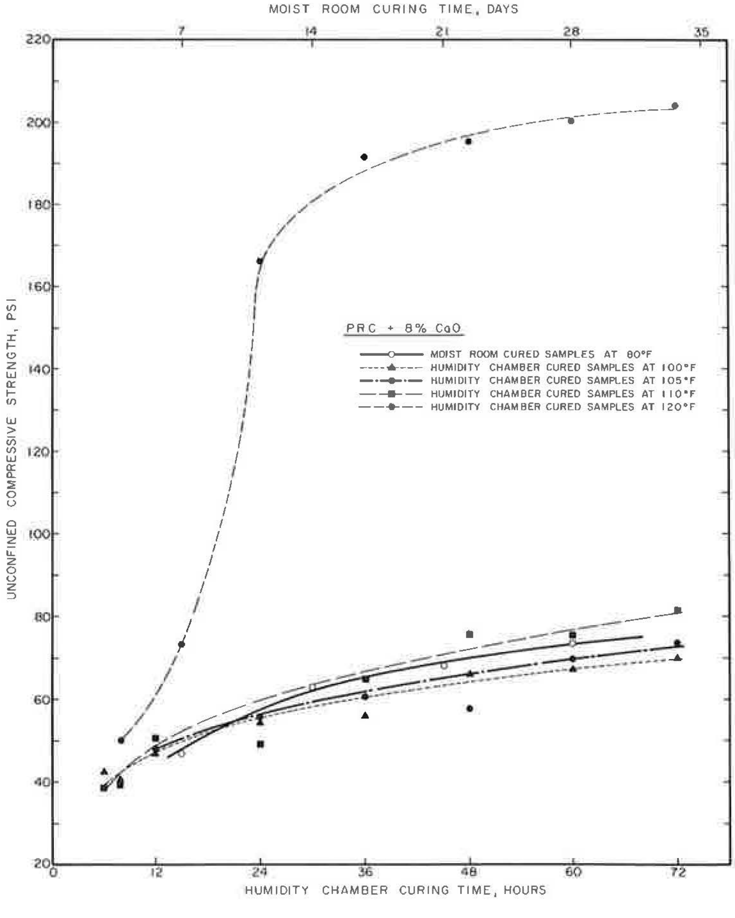


Figure 1. Typical strength-time behavior of lime-stabilized PRC at various accelerated curing temperatures.

shape or slope close to that of the moist-room curves, whereas the shape of the humidity-chamber curves at temperatures of 110, 105, and 100 F did approximate that of the moist-room curves. The 110-F curves, although similar in shape, did not appear to level off as the moist-room curves did after approximately 28 days of curing. The 100-F curves resemble both the shape and the slope of the moist-room curves but did not produce strengths equivalent to those obtained by 28-day moist-room curing, even after 96 hours of accelerated curing. To develop an expedient accelerated-curing procedure, one should use the highest curing temperature that gives acceptable results. For both soils and all treatments, the 105-F curves were found to best approximate the slopes and shapes of the moist-room curves and to produce strengths equivalent to 28-day moist-room curing; therefore, 105 F was selected as the accelerated-curing temperature. Once a single humidity-chamber temperature was found to approximate the effects of moist-room curing, a correlation between curing time and strength could be developed.

Figures 2 through 5 show unconfined compressive strength versus curing time for PRC and RMGC at lime and salt-lime modification and stabilization optimums cured in the moist room at 80 F and 100 percent humidity and in the humidity chamber at 105 F and 95 percent humidity. The strengths obtained at 28 days by moist-room curing were projected to equivalent strengths by humidity-chamber curing, and the curing times required to produce equivalent 28-day strengths were recorded.

The time scales used for plotting moist-room and humidity-chamber curing strengths are completely arbitrary. The criteria used for selecting a temperature for accelerated curing were as follows: The initial slope and general shape of the strength-time relation curves of humidity-chamber-cured were samples similar to those of moist-room-cured samples, and the rate of strength gain relation leveled off at a value close to that of the moist-room-cured, 28-day strength. Previous research at Oklahoma State University indicated that the strength-time relation for moist-room-cured samples usually levels off after 28 days. The OSU procedure of compacting samples at optimum moisture and then sealing them before curing probably causes this effect. However, this particular curing procedure was developed to simulate measured moisture conditions under Oklahoma highways during the first year after construction (10).

In any case, for the various soils and treatments, only the 120-F temperature samples did not produce a strength-time curve shape similar to that of the moist-room samples, and only the 105-F curve leveled off at approximately the correct strength. The typical data shown in Figure 1 also indicate that curing temperature was more critical than initially believed by the authors.

Figure 2 shows PRC and RMGC treated with their respective lime-modification optimum percentages. Although the 28-day moist-room-cured samples have a marked difference in strength, 62 psi for PRC and 42 psi for RMGC, the required accelerated-curing time to produce this equivalent strength was approximately equal for both samples. PRC needed an accelerated-curing time of 30 hours, whereas RMGC required 31.8 hours. The error that would result in rounding the curing to 30 hours for lime modification of RMGC would be 1.2 percent or 0.5 psi, on the conservative side. Therefore, the approximate accelerated-curing time for lime-modified samples cured at 105 F and 95 percent humidity could be taken as 30 hours.

Figure 3 shows PRC and RMGC at lime-stabilization optimum. The percentage of lime necessary for optimum stabilization by the OSU sample preparation and curing technique is generally unknown but, for Oklahoma cohesive soils, has been found to be approximately twice the lime-modification optimum (1). To obtain equivalent 28-day strengths at the lime-stabilization optimum required approximately 72 hours of rapid curing for both PRC and RMGC.

Figure 4 shows PRC and RMGC at their respective salt-lime modifications optimums. The strength of both PRC and RMGC has increased over that obtained by lime modification alone. However, the time required for accelerated curing has not changed markedly, as is to be expected because there is little free lime available for pozzolanic reaction at modification optimum. The required time for PRC has actually decreased to 28 hours, and the time for RMGC has increased to 38 hours. The error that would result from taking an accelerated-curing time of 30 hours for both soils is 1.5 percent or 1 psi for PRC and 11.6 percent or 6 psi for RMGC, on the conservative side.

Figure 2. Accelerated and conventional strength-time behavior at optimum lime modification.

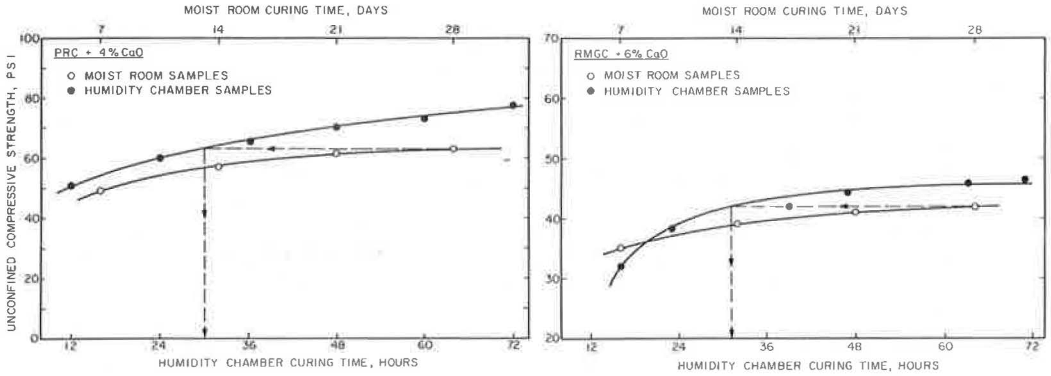


Figure 3. Accelerated and conventional strength-time behavior at optimum lime stabilization.

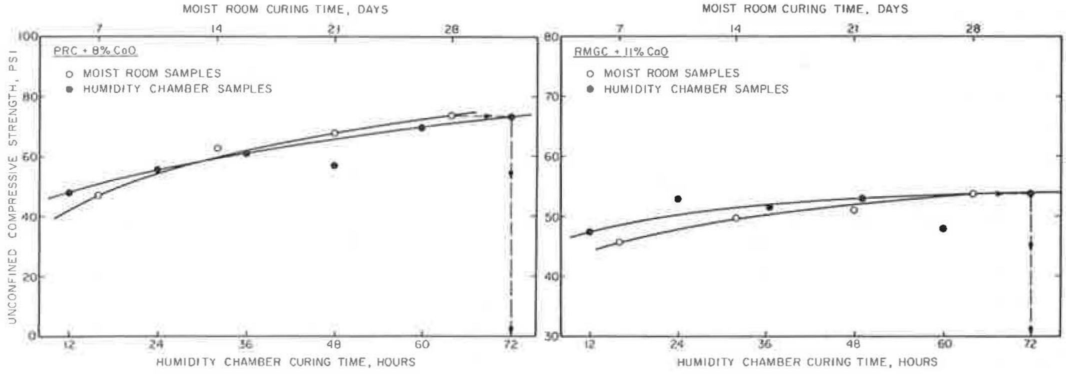


Figure 4. Accelerated and conventional strength-time behavior at optimum salt-lime modification.

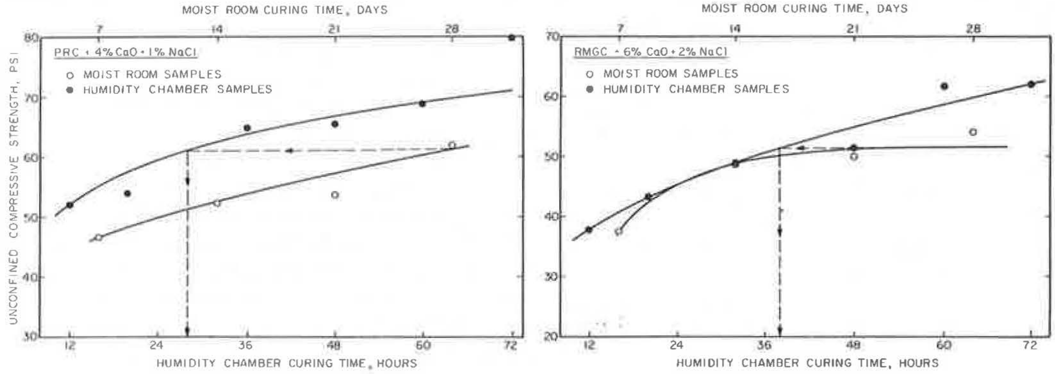


Figure 5. Accelerated and conventional strength-time behavior at optimum salt-lime stabilization.

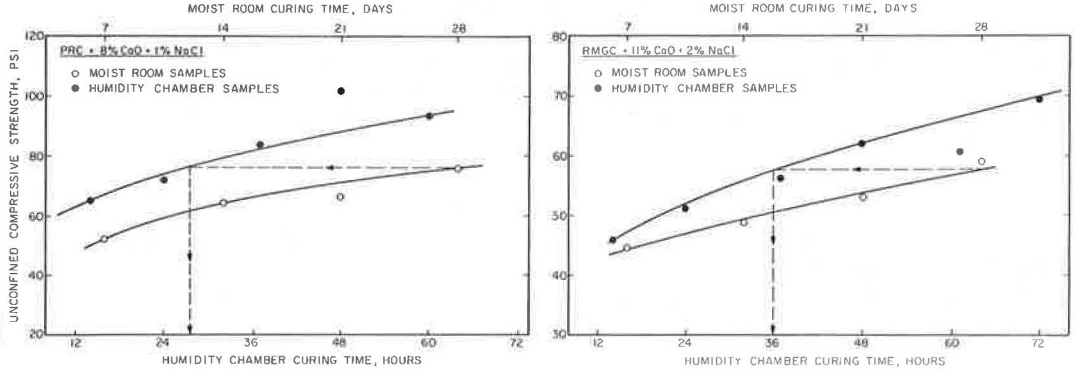


Figure 5 shows PRC and RMGC at the salt-lime stabilization optimum. Again this percentage is generally not known but, for the OSU procedure, is approximately twice the modification optimum. Although the strengths of both soils have increased, the accelerated-curing time required to produce equivalent 28-day moist-room strength has decreased markedly. Marks and Haliburton (1) reasoned that salt (NaCl) added to lime-treated soils would act as a catalyst and allow achievement of long-term strength gains more rapidly. This decrease from 72 hours of accelerated curing for lime alone substantiates their hypothesis. The error resulting from rounding the curing times required to 30 hours would be 2.6 percent or 2 psi for PRC and 5.3 percent or 2.5 psi for RMGC. Both salt-lime modification and salt-lime stabilization are approximately the same, further proof that salt increases the rate of lime-soil reaction.

Table 4 gives the times required to obtain equivalent 28-day strength by accelerated curing at 105 F for PRC and RMGC and standard times the authors think acceptable in developing an accelerated mix design procedure for lime and salt-lime modified and stabilized cohesive soils. The suggested curing time of 30 hours allows samples to be made during normal working hours on 1 day and tested during normal working hours the next day; the 72-hour samples may be cured over the weekend. Although such considerations are perhaps scientifically irrelevant, they are definitely pertinent in an engineering sense because of the wage-hour and overtime regulations of many governmental agencies.

DTA ANALYSIS OF CURED SAMPLES

Because lime and salt-lime reactions with cohesive soils are chemical in nature and thus temperature-dependent, the use of some method to determine the effects of accelerated curing on the treated soil's mineralogical composition was thought necessary. To produce an equivalent 28-day moist-room curing strength is not enough if that is done by changing mineralogical composition and reaction products rather than by simply accelerating the change that takes place naturally during moist-room or field curing.

Differential thermal analysis was used not to determine the exact chemical composition of the treated soil samples but merely to fingerprint mineralogically the moist-room and humidity-chamber samples to determine whether their mineralogical characteristics after respective curing procedures were similar. Differential thermograms for the 2 raw soils used throughout the study are shown with pure samples of illite and chlorite in Figure 6. Analysis of these curves indicates that both soils are composed mainly of illite and chlorite. RMGC appears to contain more chlorite than PRC because double peaks around 600 C are more pronounced in this material.

Differential thermograms were made for PRC and RMGC at lime and salt-lime modification and stabilization optimums by using failed strength samples after 7, 14, 21 and 28 days of moist-room curing and samples rapidly cured the numbers of hours equivalent to 28-day strength and also ±12 hours. DTA traces were then compared to see whether similar behavior occurred for moist-room and accelerated curing. Figure 7 shows comparative thermograms of 28-day strength, moist-room-cured samples and those of equivalent accelerated-cured samples. Although some variation in DTA peak size exists between accelerated-cured and moist-room-cured samples, the thermograms are remarkably similar, at least if the degree of accuracy and repeatability present in DTA are considered. The DTA behavior of both types of samples with curing time also appeared similar. If one considers the different clay mineralogy of the 2 soils and the various chemical treatments used, the DTA thermograms show reasonable agreement between mineralogical characteristics of accelerated-cured and moist-room-cured samples. With this agreement of DTA thermograms for the 2 types of curing to reinforce the previous strength correlation, it is believed the 2 curing procedures are, for all practical purposes, equivalent.

PROPOSED ACCELERATED MIX-DESIGN PROCEDURE

The accelerated-curing correlation obtained in this study is valid for samples cured in the moist room of the OSU Soil Mechanics Laboratory. It may be extended to other sample preparation and curing conditions by following the procedure used to determine

Table 4. Curing times in hours for equivalent 28-day strength at 105 F.

Treatment	PRC	RMGC	Standard
Lime modification	30	31.8	30
Lime stabilization	72	72	72
Salt-lime modification	28	38	30
Salt-lime stabilization	28	36	30

Figure 6. Differential thermograms of PRC, RMGC, illite, and chlorite.

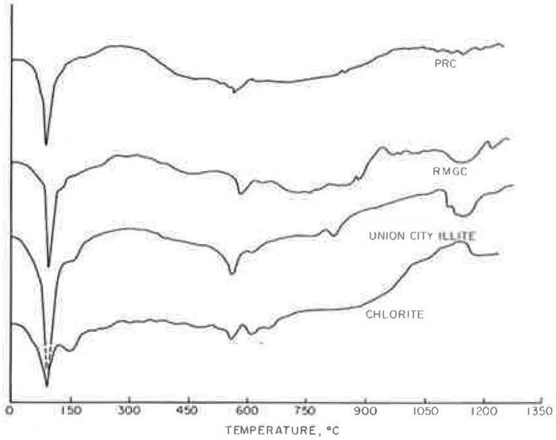
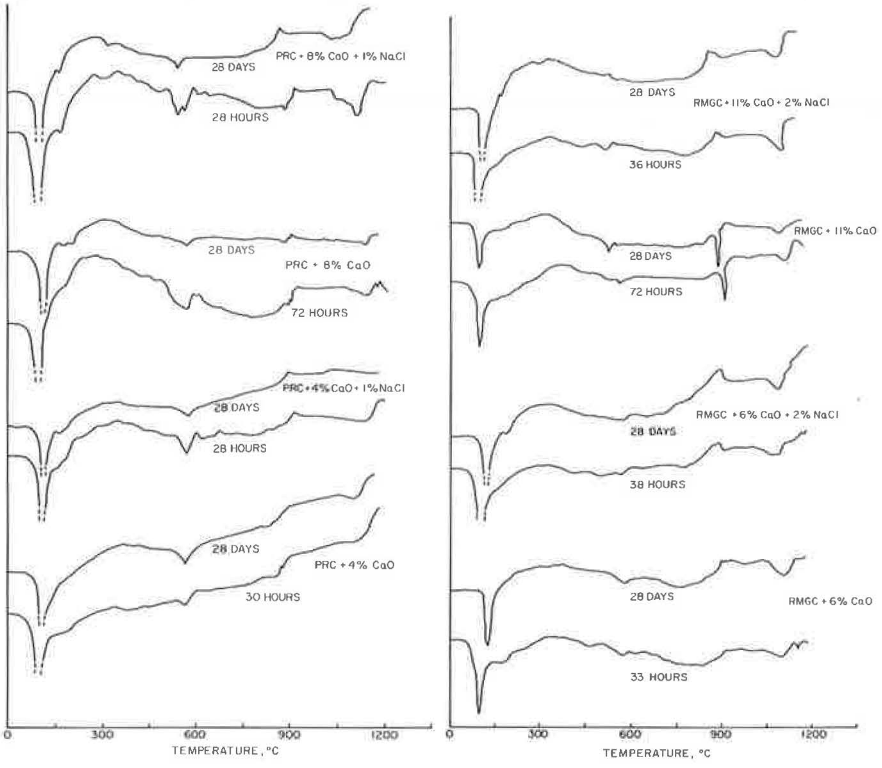


Figure 7. Summary of differential thermograms of 28-day moist-room and 28-day equivalent humidity-chamber strengths for lime- and salt-lime-treated PRC and RMGC.



the "conventional" strength correlation and then checking obtained mineralogy by DTA. Although the basic intent of this study was to study the feasibility of an accelerated-curing process for use at Oklahoma State University, the procedures may be used by other agencies to establish valid accelerated-curing procedures that simulate particular conventional-curing conditions. The study has also enabled the authors to propose a mix-design procedure to obtain equivalent 28-day strength properties of lime- and salt-lime-treated soils for use as base and subbase materials in Oklahoma. With proper equipment and adequately trained personnel, the following design procedure should take from 5 to 7 working days after the raw soil samples are received:

1. Using the sample preparation techniques described by Marks and Haliburton (1), run miniature standard Proctor compaction tests with 0, 0.5, 1.0, 1.5, and 2.0 percent NaCl to determine the optimum salt content (usually between 1 to 2 percent).
2. Run the pH test for lime modification optimum as described by Eades and Grim (11) by using increments of 1 percent CaO or $Ca(OH)_2$ until the pH peaks.
3. Using the sample preparation techniques described elsewhere (1), run miniature standard Proctor compaction tests at optimum salt content and lime contents from modification optimum to twice modification optimum, generally considered as the upper bound for stabilized optimum, in 1 percent lime-content increments, and compact 3 samples at each lime percentage to standard Proctor compaction maximum density at optimum moisture.
4. For lime treatment alone, omit the steps pertaining to salt treatment. However, salt-lime treatment is preferred to lime treatment alone for reasons described elsewhere (1, 2) and also in this paper.
5. Cure salt-lime samples by rapid-curing procedure at 105 F and 95 percent relative humidity for 30 hours to determine equivalent 28-day strengths. To determine equivalent 28-day strengths for lime-only treated samples at lime percentages above modification optimum, assume twice modification optimum as stabilization optimum and equally divide the number of whole percentages of lime between the difference of 42 hours in rapid-curing time. For example, the rapid-curing times for PRC with varying percentages of CaO are as follows: 4 percent, modification optimum, 30 hours; 5 percent, 40.5 hours; 6 percent, 51.0 hours; 7 percent, 61.5 hours; and 8 percent, stabilization optimum, 72 hours.
6. Run unconfined compression tests on rapid-cured samples by using the procedure described previously. For subbase, use the minimum lime content that will produce q_u = 50 psi. This material should be tentatively considered equivalent to typical "select" material subbase (12). For base material, use the minimum lime content that will produce q_u = 100 psi. This material should be tentatively considered equivalent to the "equivalent base" used in the Oklahoma subgrade index design procedure (12, 13).
7. Add 1 percent CaO or $Ca(OH)_2$ and 0.5 percent NaCl to design values to compensate for field-mixing procedures.
8. Place additives in field wet or dry, in any order, mix together, and compact between 12 and 24 hours later to at least 95 percent maximum density for original design values. It appears most feasible to place the NaCl in the mixing and compaction water used in conventional lime treatment. If reasonable inspection of field-mixing procedure and mixing moisture content is done, only field tests for compacted density will be needed. Failure to obtain required density after reasonable rolling time will be indicative (usually) of insufficient mixing, for field-compacted density values cannot usually be obtained without proper mixing of the NaCl.

It should be possible to use on-site cohesive materials for all portions of the highway structure beneath the wearing surface, and that will often reduce construction cost and allow better job-site control of material properties than if "transported" materials are used. It is suggested that this procedure be evaluated for routine use in design of low-traffic highways and as a design option in high-traffic highways.

CONCLUSION

This study determined the temperature and time required to produce with accelerated curing of 2 cohesive Oklahoma soils modified and stabilized with lime and salt-lime additives to a strength equivalent to that obtained by curing in a moist room for 28 days at 80 F and 100 percent humidity. The following conclusions are indicated by or inferred from analysis of data collected throughout the study:

1. An accelerated curing process can be used to achieve 28-day unconfined compressive strengths of moist-room-cured samples;
2. A humidity-chamber curing temperature of 105 F produced the closest approximation of unconfined compressive strength-curing time behavior for moist-room-cured samples of PRC and RMGC;
3. Comparison of DTA thermograms for the accelerated-cured and moist-room-cured samples indicated that both types of curing produced similar mineralogical conditions;
4. The humidity-chamber curing times required to simulate 28-day unconfined compressive strength of moist-room-cured samples of PRC and RMGC are given in Table 4;
5. At stabilization lime contents, the reduction in curing time caused by addition of salt confirms the catalytic effect of NaCl on lime-soil reactions; and
6. A design procedure for salt-lime treatment of cohesive soil is proposed that reduces the time required to obtain design values from about 1 month to between 5 and 7 working days after raw soil samples have been received.

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STABILITY PROPERTIES OF UNCURED LIME-TREATED FINE-GRAINED SOILS

C. H. Neubauer, Jr., U.S. Air Force Weapons Laboratory, Kirtland, New Mexico; and
M. R. Thompson, University of Illinois

The early strength and deformation properties of uncured lime-soil mixtures and untreated soils were investigated. A substantial gain in stability is realized by the additon of lime to a fine-grained cohesive soil and is reflected by the combined immediate gain in modulus of deformation and strength (shear strength, CBR, and cone index) of the uncured lime-soil mixtures. The gain in stability can be attributed to the immediate cation exchange and to the flocculation and agglomeration reactions that occur when lime is mixed with a fine-grained cohesive soil. The laboratory results were further considered in developing linear regression equations relating cone penetrometer test results to shear strength, CBR, and modulus of deformation. The significance of the study findings is examined relative to problems such as expediting construction, increasing in situ subgrade support, and constructing temporary roads.

•IN CONSTRUCTION, there are many instances when soft, highly plastic, cohesive, in situ soil conditions present mobility problems for wheeled vehicles, are difficult to compact, and may not provide satisfactory subgrade support for subsequent pavement construction operations. These problems result from the combined effect of insufficient shear strength of the soil and the development of excessive deformations under the applied wheel loads. In general, soft unstable subgrade soil conditions are associated with high in situ moisture contents. Experience has shown that lime treatment of the in situ soil is a highly effective method for alleviating the problems and, thus, expediting construction.

LIME-SOIL STABILIZATION

Lime mixed with fine-grained cohesive soils causes cation exchange, flocculation, agglomeration of the soil, or all of these, and thus a lime-modified soil layer is created. These immediate reactions produce an improvement in workability and an increase in stability. Additional immediate benefits of treatment with lime are the reduction in the plasticity of the in situ soil and the creation of a more water- and swell-resistant material. Treatment with lime followed by compaction creates an uncured lime-soil layer (a "working table") overlying the in situ cohesive subgrade. The lime-modified soil layer may provide adequate stability to minimize deformations and increase shear strength to the extent that vehicle mobility and subgrade support problems are eliminated.

In some cases lime reacts with cohesive soils to produce a pozzolanic reaction (1, 2) in addition to the reactions just described. In such cases, adequate curing time is required to allow the pozzolanic reaction to occur. The results of that reaction are large increases in the compressive, tensile, flexural, and shear strengths of the stabilized lime-soil mixture. Only certain cohesive soils will react with lime to produce a pozzolanic reaction. Because adequate curing time is required to develop this pozzolanic reaction, the procedure cannot be classified as a construction expedient method. That

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means not that expedient lime-soil stabilization methods cannot be adapted to those cohesive soils that react with lime to produce a pozzolanic reaction but that, because of the time delay required for the pozzolanic reaction to develop, only the immediate "uncured" increase of stability can be realized relative to expedient operations.

Although lime has been successfully used for years as a construction expedient, few data and research findings are available concerning the strength and deformation properties of uncured lime-soil mixtures. Most of the research to date has considered the long-term, or cured strength, effects of lime stabilization. To make rational decisions concerning the amount of lime required to successfully modify the soil and to determine the required thickness of stabilization appropriate for a particular site require that techniques be developed for establishing engineering property data for uncured lime-soil mixtures.

EXPERIMENTAL PROGRAM

The purpose of the laboratory phase of this study was to investigate the effect of uncured lime-soil stabilization on the engineering characteristics of fine-grained soils. The laboratory investigation included 4 typical cohesive soils: Vicksburg buckshot clay, clay, Roxana loess, Hurst B, and Ava B. The properties of the soils investigated are given in Table 1.

A commercial high-calcium hydrated lime manufactured by the Mississippi Lime Company, Ste. Genevieve, Missouri, was used in the investigation. A typical analysis provided by the lime company indicated that the lime contained 96.2 percent $Ca(OH)_2$ and approximately 95 percent passed the No. 325 sieve.

Because weak subgrade conditions are normally associated with high water contents, the engineering properties of the natural soils and the lime-soil mixtures were evaluated at moisture contents above the optimum moisture content of the natural soils. The moisture-density relations were determined according to ASTM D 698, Method A. Figures 1, 2, 3, and 4 show the moisture-density relations for the natural soils and the lime-soil mixtures with 4 percent lime treatment. For each soil, the molding moisture content for the test specimens is shown. Molding moisture contents were selected on an arbitrary basis. It was assumed that for natural soil conditions, where the field moisture content of the subgrade soil is at or below optimum, weak subgrade conditions and vehicle mobility generally will not be severe problems.

The 4 soils were treated on a soil dry-weight basis with 0, 2, 4, and 6 percent lime. All samples were prepared at the moisture contents and densities shown in Figures 1, 2, 3, and 4. The lime-soil mixtures were allowed to mellow 1 hour prior to the preparation of specimens. Specimens of the natural and lime-treated soils were prepared for the triaxial and unconfined compressive strength tests (dynamic compaction) by using 4-in. molds, 2 in. in diameter, and for the cone penetrometer and CBR tests by following ASTM D 698, Method B. The natural and lime-treated soils were tested immediately after compaction.

The natural and lime-treated soils were evaluated by use of the CBR, cone penetrameter, triaxial (unconsolidated-undrained, confining pressures of 0, 5, and 25 psi), and unconfined compressive strength tests. The CBR tests were performed in accordance with ASTM D 1883. The rate of deformation used for the unconfined compression and triaxial tests was 0.05 in./min, and deformation measurements were taken during the test. The cone penetrometer laboratory test is not a standard laboratory testing procedure. The method adopted was to utilize a Riehle hydraulic testing machine to penetrate the specimen at a constant rate of 2 in./min. The cone penetrometer conformed to the dimensions of the cone penetrometer of the U.S. Army Engineer Waterways Experiment Station (3). Cone index values (CI) were obtained by dividing the recorded load by the cross-sectional area of the cone head, $\frac{1}{2}$ sq in. Thus, CI is the penetrometer pressure, in psi, required to effect penetration. The load readings were taken when the base of the cone was at 1- and 2-in. depths beneath the surface of the sample.

Average data values reported in this paper are based on 4 unconfined compressive strength specimens, 2 CBR tests, 2 cone penetrometer tests, or 2 triaxial test specimens.

Liquid and plastic limits (ASTM D 423 and ASTM D 424 respectively) were determined for the natural soils and the soils treated with 2, 4, and 6 percent lime. The lime-treated

Figure 1. Moisture-density relations for Hurst B.

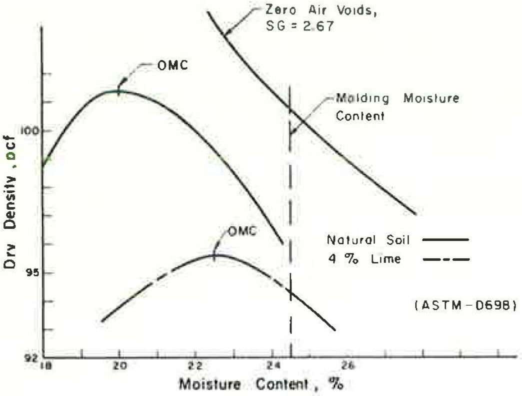


Figure 2. Moisture-density relations for Ava B.

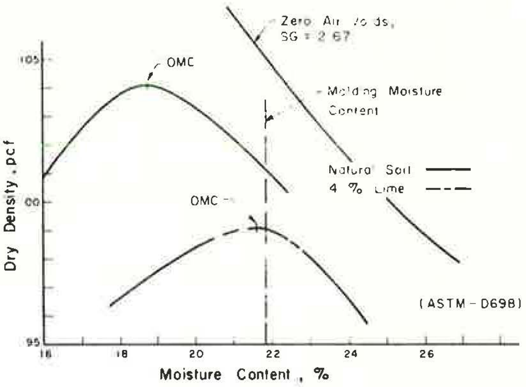


Figure 3. Moisture-density relations for Vicksburg.

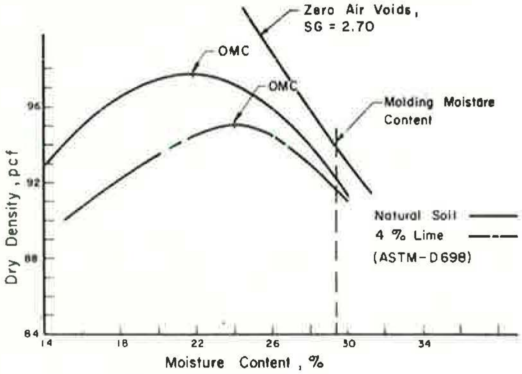


Figure 4. Moisture-density relations for Roxana.

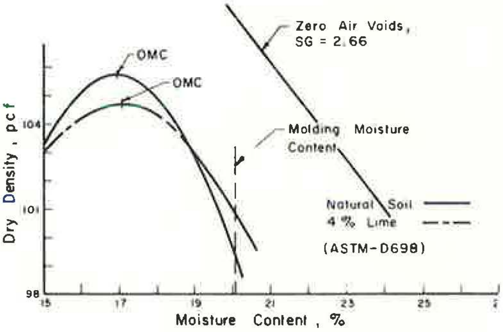


Table 1. Properties of soils included in study.

Soil	<2 μm Clay (percent)	Liquid Limit	Plasticity Index	AASHO Class	pH	Predominant Clay Minerals
Vicksburg	46	59	30	A-7-6(20)	6.0	Montmorillonite
Roxana	13	23	4	A-4(8)	8.0	Montmorillonite and mixed layer
Hurst B	24	35	15	A-6(10)	5.9	Illite and montmorillonite
Ava B	28	36	17	A-6(11)	5.2	Mixed layer

Figure 5. Immediate effects of lime treatment on cone index strength.

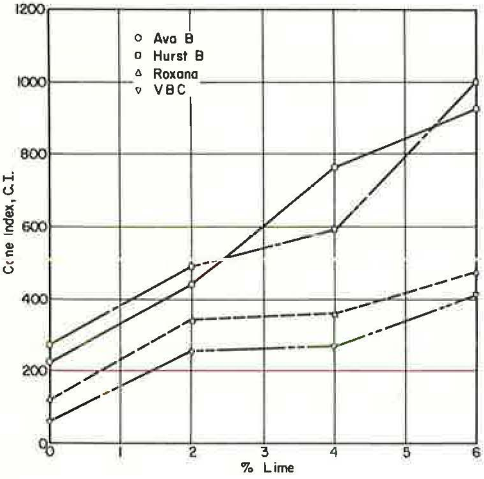
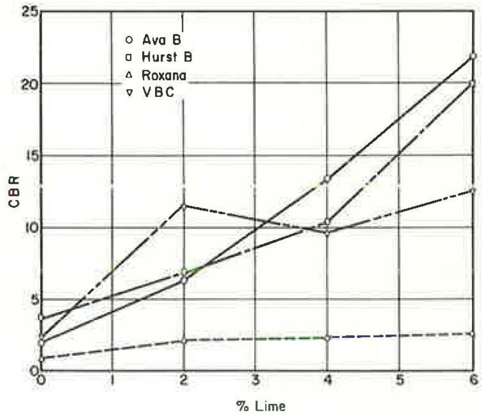


Figure 6. Immediate effects of lime treatment on CBR strength.



soils were allowed to mellow 1 hour before the tests were conducted.

TEST DATA

Test data illustrating the immediate effects of lime treatment on CI, CBR, and unconfined compressive strength are shown in Figures 5, 6, and 7 respectively. Figures 8, 9, 10, and 11 show the improved stress-strain characteristics of each soil treated with 6 percent lime as compared to the natural soils.

Thompson (4) has suggested the use of an initial tangent modulus for evaluating the modulus of elasticity of cured lime-soil mixtures. For high strength lime-soil mixtures ("reactive" soils), the use of the initial tangent modulus is acceptable. For natural soils and uncured lime-soil mixtures, for which the stress-strain curves are not linear, the use of an initial tangent modulus leads to higher, unrealistic E-values. For the case in which the uncured lime-soil mixture is to be used as a construction expedient (no curing), the mixture stress level will be high, near the ultimate strength of the mixture perhaps, and thus the secant modulus provides a more realistic evaluation of E. For this study the secant modulus at 75 percent of the maximum deviator stress and a confining pressure, σ_3, of 5 psi were considered most appropriate. The effect of lime treatment on the modulus of deformation is shown in Figure 12.

Plasticity properties of the natural and lime-treated soils are given in Table 2. Triaxial test results (confining pressures of 0, 5, and 25 psi) were analyzed to determine the shear strength parameters C and ϕ by employing a statistical linear regression procedure suggested by Herrin (5). C and ϕ data for the natural and lime-treated soils are given in Table 3.

DATA ANALYSIS

The cone index, CBR, and unconfined compressive strength data demonstrate the beneficial effect of lime treatment, without curing, on the strength properties of the soils. Strength increases of several hundred percent were obtained in many cases. The general trend was for strength to increase with increased lime contents, within the range of lime contents investigated.

A general examination of the shear strength data suggests that the natural and lime-modified soils essentially behaved as $\phi = 0$ materials. The ϕ values for the lime-soil mixtures ranged from 0 to approximately 6 deg with an average value of approximately $2\frac{1}{2}$ deg. Considering the low degrees of confinement that can be expected under typical construction expedient operations, we can conservatively characterize the uncured lime-soil mixtures as $\phi = 0$ materials.

The typical stress-strain curves shown in Figure 8, 9, 10, and 11 indicate the general nature of the modification attained from the lime treatment. Failure strains are substantially reduced, values were in the 1 to 2 percent range for the lime-soil mixtures, and maximum deviator stresses are increased. That is a particularly significant effect because high shear strengths can be developed in the lime-soil mixture without excessive deformation.

Modulus of deformation values were improved by the lime additions. E-values for the natural soils ranged from 195 to 720 psi. Values as large as 8,400 psi (Ava B, 6 percent lime) were obtained for the lime-treated soils.

With the exception of Hurst B, all of the soils were nonplastic at 2 percent lime treatment. Six percent lime was required to render Hurst B nonplastic. Even though the 2 percent treatments effectively reduced the plasticity properties, higher lime contents were beneficial based on considerations of increased strength and higher E-values.

Of the 3 procedures used to evaluate strength in this investigation, only one, the cone penetrometer method, is readily adaptable to rapid field testing. Because of the extreme soil variability and the urgency that may exist in relation to field construction problems associated with "weak" subgrade, the cone penetrometer appears appropriate for use in expedient field engineering operations. More fundamental engineering properties (shear strength, E, CBR) were estimated from cone penetrometer test data by establishing linear regression equations relating cone index and shear strength (unconfined strength/2), cone index and E, and cone index and CBR. The regression equations and their correlation coefficients are as follows (for all equations, R is significant at $\alpha = 0.01$):

Table 2. Plasticity test results for natural and lime-treated soils.

Soil	Natural Soil		2 Percent Lime		4 Percent Lime		6 Percent Lime	
	Liquid Limit	Plasticity Index	Liquid Limit	Plasticity Index	Liquid Limit	Plasticity Index	Liquid Limit	Plasticity Index
Vicksburg	59	30	NP[a]	NP	NP	NP	NP	NP
Hurst B	35	15	39	10	35	6	NP	NP
Ava B	36	17	NP	NP	NP	NP	NP	NP
Roxana	23	4	NP	NP	NP	NP	NP	NP

[a] Nonplastic.

Figure 7. Immediate effects of lime treatment on unconfined compressive strength.

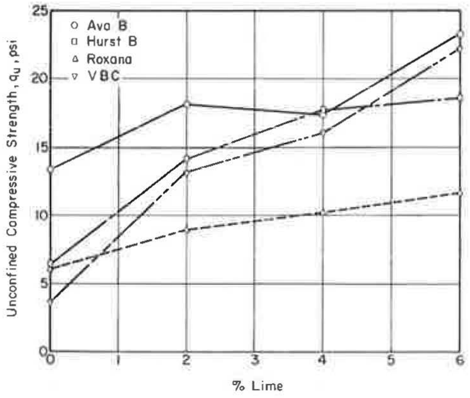


Table 3. Summary of C and ϕ data for natural and lime-treated soils.

Soil	Lime (percent)	C (psi)	ϕ	
			Deg	Min
Vicksburg	0	3.5	1	56
	2	12.2	3	15
	4	18.6	3	25
	6	19.9	3	2
Roxana	0	6.9	2	25
	2	9.8	0	
	4	11.4	2	4
	6	13.0	0	37
Hurst B	0	9.1	1	15
	2	14.9	1	4
	4	14.6	2	44
	6	17.4	5	58
Ava B	0	12.0	2	0
	2	16.3	5	25
	4	18.5	0	
	6	23.4	1	7

Figure 8. Immediate effects on stress-strain characteristics of lime-treated Vicksburg.

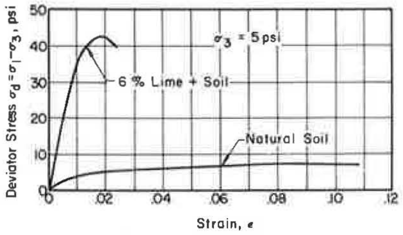


Figure 9. Immediate effects on stress-strain characteristics of lime-treated Roxana.

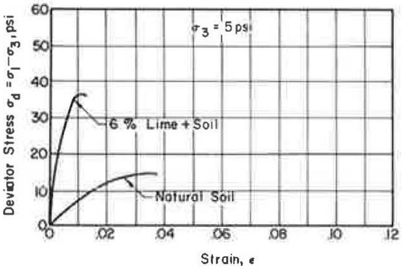


Figure 10. Immediate effects on stress-strain characteristics of lime-treated Ava B.

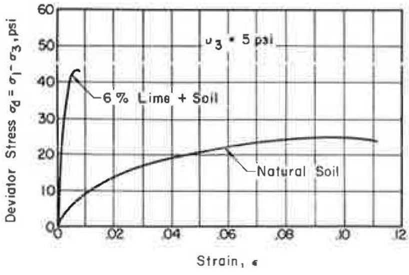
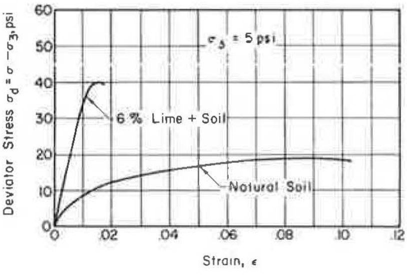


Figure 11. Immediate effects on stress-strain characteristics of lime-treated Hurst B.



$$S_s = 7.0 + 0.02\ CI \qquad R = 0.73$$
$$E = -22 + 6.2\ CI \qquad R = 0.79$$
$$CBR = -0.8 + 0.02\ CI \qquad R = 0.82$$

where

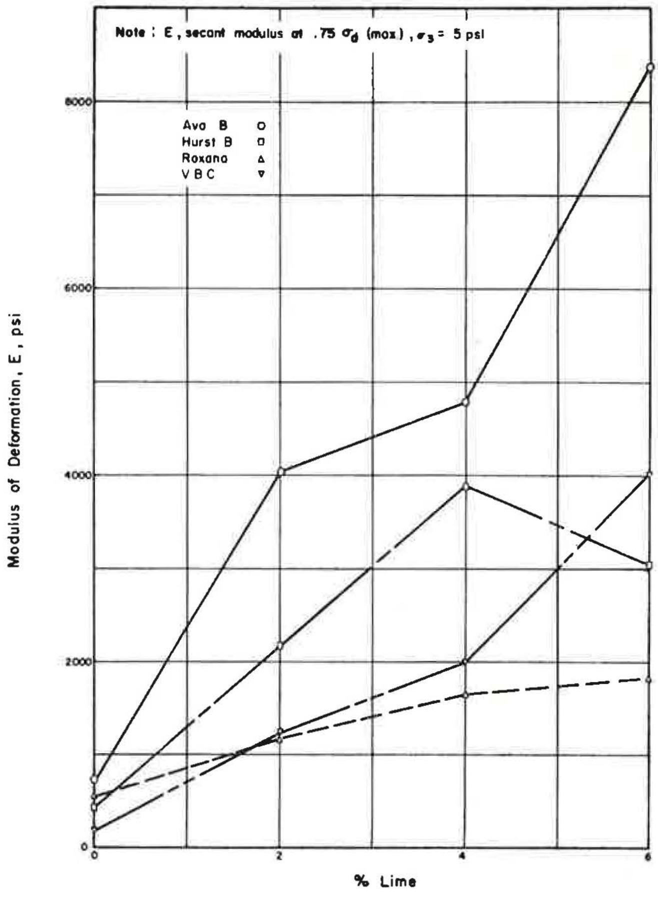
S_s = shear strength (unconfined compressive strength/2), spi;
CI = cone index value, psi;
E = secant modulus of elasticity at 75 percent of maximum deviator stress and 5-psi confining pressure, psi; and
CBR = California bearing ratio as determined by ASTM D 1883.

DISCUSSION OF RESULTS

Results from this study conclusively illustrate that substantial improvements in the strength and deformation properties of wet fine-grained soils (water contents in excess of optimum) can be effected by lime treatment. Such improvements are sufficient to warrant serious consideration relative to the effective utilization of uncured lime-soil stabilization for applications such as expediting construction and constructing temporary roads. The 2 basic factors that must be established in using uncured lime-soil stabilization are lime-treatment level and thickness of stabilization.

The data presented in this paper show that increased lime-treatment levels, up to 6 percent in this study, generally are beneficial based on considerations of strength and

Figure 12. Immediate effects of lime treatment on modulus of deformation.



moduli of deformation. Even though 3 of the soils were nonplastic at the 2 percent treatment level, additional lime effected further improvement. This finding indicates that those responsible for mix design for expedient construction operations should consider the strength and moduli properties of the lime-soil mixture in establishing final field lime contents.

It is possible, based on current construction capabilities, to lime-stabilize to depths of 24 in. (deep plow) in 1 lift (6). With "lift-construction" procedures, substantially thicker layers could conceivably be lime-treated. With such a wide range of potential thicknesses, it is apparent that the development of a general thickness design approach for uncured lime-soil stabilization is desirable.

Neubauer (7) has developed a procedure for estimating the thickness of lime-soil stabilization required to provide vehicle mobility. The procedure considers the strengths and moduli of deformation of the in situ subgrade and the lime-treated soil, wheel-loading conditions, and traffic volume (coverages). Strengths and moduli of deformation properties are estimated by the regression equations presented in this paper; the equations relate cone index to CBR, E, and shear strength. The proposed procedure is primarily directed to field engineering operations. Neubauer utilized his proposed procedure in analyzing 2 documented field test conditions and was able to satisfactorily predict field behavior.

The procedure proposed by Neubauer is capable of considering the value of trade-offs between the quality (strength and moduli of deformation properties) of the uncured lime-soil mixture and the required thickness. For example, alternate solutions may be either a high quality mixture (higher lime content) and moderate thickness or a lower quality mixture (reduced lime content) and increased thickness.

Although lime-soil mixtures at optimum water content or dry of optimum were not considered in this study, the same general type of improvement in strength and deformation properties would be obtained for such mixtures. For example, Thompson (4) has previously reported data indicating that substantial immediate CBR increases are achieved with lime treatment of a wide range of typical fine-grained Illinois soils compacted at AASHO T-99 optimum water content. The fact that decreased densities (relative to the natural soil) for a given compactive effort and water content are generally obtained for lime-treated soils (Figs. 1, 2, 3, and 4) is evidence that increased immediate stability is achieved over a wide range of water contents.

SUMMARY AND CONCLUSIONS

An investigation of the uncured effects of lime-soil stabilization was conducted. The early strength and deformation properties (cone index, CBR, shear strength, and moduli of deformation) of uncured lime-soil mixtures compacted wet of optimum were substantially improved relative to the untreated natural soils. The effects of the immediate lime-soil reactions of cation exchange, flocculation, and agglomeration are sufficient to greatly improve adverse in situ soil conditions for purposes such as expediting construction, providing increased subgrade support, and constructing temporary roads.

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RELATIVE STABILIZING EFFECT OF VARIOUS LIMES ON CLAYEY SOILS

Max L. Alexander, Robert E. Smith, and George B. Sherman,
Materials and Research Department, California Division of Highways

The relative stabilizing effects of 2 hydrated limes and 3 quicklimes from various commercial producers are discussed. The effects of variations in lime gradation and calcium hydroxide content on the unconfined compressive strength of 3 different soils are presented. The strengths of specimens treated with quicklime generally exceeded those of specimens treated with hydrated lime, even in comparisons based on equivalent $Ca(OH)_2$ content. It was also found that the coarser limes tested were more effective than the fine. Correlations are established between lime gradation and compactability and between specimen density and compressive strength.

•THE USE of hydrated lime to stabilize soils for highway construction has increased rapidly in California during the last decade. This has been accompanied by the progressive development of specifications by the California Division of Highways to control the major phases of work. However, very little change has been made in the requirements on the lime itself. In 1959, California specified a minimum calcium hydroxide content of 85 percent for hydrated lime used in stabilization work. This requirement was reduced to 75 percent in 1961 and then increased again to 85 percent in 1968. The fineness specifications, requiring that not more than 3 percent of the lime be coarser than the No. 30 sieve and not more than 14 percent coarser than the No. 200 sieve, were adopted in 1968.

Several lime products, other than finely ground hydrated lime, are commercially available in California today. These include the coarser grinds of hydrated lime as well as quicklime. The latter is not currently used by the California Division of Highways but has been successfully employed by various cities and counties within the state. Quicklime may be obtained for soil stabilization in a variety of forms, including pulverized and granular. It is apparent that there could be substantial savings if these other limes could be used by California to stabilize for road construction. Also, it would increase the availability of lime during periods of peak demand.

The object of this research, therefore, was to evaluate the relative stabilizing effect of commercially available limes. The soils used were representative of local materials that would be considered for use in the structural section as base or subbase if they were improved sufficiently by the treatment.

It is generally accepted that the reaction between a lime and a soil takes place in 2 distinct phases. The first is an immediate reduction in plasticity, presumably the result of a base exchange and flocculation that occurs when the strong calcium cations of the lime replace the weaker metallic ions on the surface of the clay particles. Also, the high pH in the soil after lime treatment causes disruption in the silica portion of the clay structure (1). There is a further pozzolanic, or cementing reaction, that may occur with some soils after lime treatment. Because the resulting strengths may approach 1,000 psi, the unconfined compressive strength test was selected for the evaluation of the various limes.

Sponsored by Committee on Lime and Lime-Fly Ash Stabilization.

The test procedure adopted for the research included the preparation of specimens of treated soils at densities and moisture contents similar to those expected in the field. An accelerated high-temperature curing technique was employed.

MATERIALS

Three soils, representing areas within the Sacramento and San Joaquin Valleys where extensive lime-stabilization work has been done or is being planned, were selected for this study. Soil 1 was obtained from the southwest portion of Tulare County along Route 43 south of Corcoran. Soil 2 was obtained from the embankment material used on Interstate 5 through the city of Stockton. Soil 3 was obtained from the vicinity of the Sacramento Metropolitan Airport where extensive lime stabilization work has been done on Interstate routes 5 and 880. The essential classification data for those soils are given in Table 1.

Five limes were obtained from the 3 major lime producers in California. Limes A and B were hydrated limes representing that currently being used in construction on California highways. Limes C, D, and E were quicklimes representing various types that are commercially available. Sieve analyses were determined for the limes and are given in Table 2. The sieve analyses of the hydrated limes were determined in accordance with AASHO Designation T 219-661. Hydrated lime A had 92 percent passing the No. 200 sieve, and hydrated lime B had only 76 percent passing. The sieve analyses of the quicklimes were determined by dry sieving 200 samples through nested 8-in. diameter screens with a mechanical sieve shaker and a 10-min shaking time. The particle-size differences between the 3 quicklimes are due to the characteristics of the particular stone and the manufacturing processes used. The granular texture of quicklime C is attributed to the properties of the particular limerock and the processing used by the producer. Quicklime D was pulverized as a part of the manufacturing process in order to produce a finely ground product. Quicklime E is referred to as "mill run" as it was obtained from the processing plant before final pulverization.

Calcium hydroxide, $Ca(OH)_2$, contents were determined for each lime by using California test method 414. That method is a modification of ASTM Designation C 25. Calcium oxide, CaO, equivalents were determined from the calcium hydroxide contents by using a conversion ratio of 1.32:1.00, which is in accordance with definitions in ASTM Designation C 25. Both the calcium hydroxide and the calcium oxide contents indicate the amount of lime available for reaction in the treatment of a soil. The percentages of $Ca(OH)_2$ and CaO are relative based on initial equal amounts of lime by weight. They are as follows:

Lime	CaO	$Ca(OH)_2$
Hydrated A, fine	65	86
Hydrated B, coarse	64	85
Quicklime C, granular	95	125
Quicklime D, fine	93	123
Quicklime E, mill run	93	123

For limes C, D, and E, the percentages of $Ca(OH)_2$ are after hydration.

The quicklimes have a greater purity than the hydrated limes studied because of less carbonation, inert or foreign materials, or free water. The net effect is that, based on equal initial weights of these limes, the quicklimes yielded about 1.5 times as much $Ca(OH)_2$ as the hydrates. Therefore, in subsequent comparisons of the effects of the 2 types of limes on soils, they will be presented in forms of equivalent $Ca(OH)_2$ content. This is done by multiplying the lime content of the specimens (in percent by weight) by the $Ca(OH)_2$ factors given in the preceding tabulation.

TESTING

The soils were dried and processed until all material, other than rock, passed the No. 4 sieve. Measured amounts, by weight, of soil, lime, and water were thoroughly blended together by using a mixing bowl, trowel, and water-metering burette. The soil

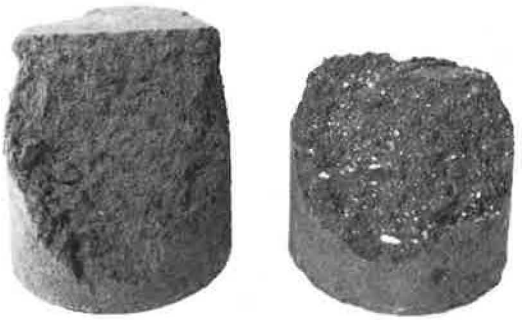
Table 1. Physical properties of soils.

Property	Soil 1	Soil 2	Soil 3
SE	2	2	2
Liquid limit	38	39	49
Plasticity index	18	20	19
Minus No. 4 sieve	99	96	98
Minus No. 200 sieve	75	85	94
Classification			
AASHO	A-6(11)	A-6(12)	A-7-5(13)
Unified	Sandy clay-CL	Lean clay-CL	Lean clay-CL-ML

Table 2. Particle-size analyses of limes.

Percent Passing Sieve	Hydrated Lime		Quicklime		
	A	B	C	D	E
No. 4					100
No. 8			100		95
No. 16			83		82
No. 30	99	98	49	100	62
No. 50			12	99	43
No. 100			3	90	28
No. 200	92	76	1	34	12

Figure 1. Distribution of fine and coarse lime particles.


Fine Lime

Coarse Lime

Table 3. Effect of lime on soils.

Lime		Soil 1			Soil 2			Soil 3		
Designation	Percent[a]	H_2O[b] (percent)	Dry Density (lb/ft³)	Strength (psi)	H_2O[b] (percent)	Dry Density (lb/ft³)	Strength (psi)	H_2O[b] (percent)	Dry Density (lb/ft³)	Strength (psi)
A	1	16.5	109	95	16.6	106	30	19.9	99	32
	2	16.3	106	312	16.5	103	87	19.7	96	102
	3	16.1	105	507	16.4	101	120	19.6	95	131
	4	16.0	105	497	16.3	101	128	19.4	94	118
B	1	16.5	110	126	16.6	105	23	19.9	99	37
	2	16.3	107	360	16.5	102	79	19.7	96	114
	3	16.1	105	508	16.4	101	100	19.6	95	131
	4	16.0	103	450	16.3	100	117	19.4	94	118
C	1	18.2	111	233	17.6	108	94	20.0	99	60
	2	18.1	110	458	17.5	105	222	19.8	98	181
	3	17.9	109	531	17.3	102	227	19.6	96	194
	4	17.8	106	486	17.2	100	219	19.5	93	177
D	1	18.2	109	181	16.7	106	65	20.0	97	54
	2	18.1	104	504	16.6	102	122	19.8	94	122
	3	17.9	101	480	16.5	100	159	19.6	94	114
	4	17.8	102	425	16.4	99	178	19.5	92	113
E	1	18.2	111	142	—	—	—	20.0	99	59
	2	18.1	107	477	15.8	104	165	19.8	96	157
	3	17.9	103	514	17.3	101	175	19.6	94	157
	4	17.8	101	463	17.2	99	178	19.5	92	137

[a] Based on dry weight of soil.

[b] Based on dry weight of soil and lime.

and lime were combined first to aid in distribution of the lime, and then the entire predetermined volume of water was blended in. Care was taken during mixing to avoid crushing or breaking up lumps of lime that would normally not be broken up by construction procedures (Fig. 1).

The amount of water to be added to the test specimens was estimated in advance on the basis of moisture-density curves established in accordance with the California impact-compaction test 216 (2). Because the kneading compactor produces densities in the order of 90 to 95 percent of those required by test 216, the specimen moistures were usually higher than impact optimum. For a given lime, the same approximate moisture was used for all lime contents. It is acknowledged that the actual optimum moisture content for a soil would vary with lime content, particularly with the quicklimes. However, it was established in a supplemental investigation that this factor would increase rather than decrease the relative effects found by using the arbitrary water contents.

After the soil, lime, and water were blended, the mixture was placed in an airtight container and allowed to loose cure for approximately 24 hours. This time corresponds to a practical minimum field-curing period for most projects. After the loose-curing period, the mixture was compacted into 4-in. diameter by 4 in. high tin liners with the kneading compactor. The soil was introduced into the mold in 20 equal increments accompanied by an application of the compactor ram at a foot pressure of 250 psi. This was then continued for 100 load applications at a foot pressure of 350 psi after all of the material was introduced. A final static pressure of 350 psi was applied over the surface area to level and smooth the surface. This compactive procedure resulted in densities equal to approximately 95 percent of the maximum density obtained by test 216.

Curing the compacted test specimens was accomplished in 2 phases. While the compacted specimens were still in the tin liners, the ends were capped and sealed with tape and then placed in a 110-F oven for 7 days. A review of work by other researchers (3, 4) indicated that 7-day curing at 110 F would be approximately equal to 3 months in the road. Also, because roadways are normally subjected to saturation after a period of time, the curing cycle was completed by removing the tin liners and burying the specimens in a confined, saturated sand for 21 days.

After the curing phase, the ends of the specimens were capped with plaster of paris to provide smooth bearing surfaces and then loaded to failure in the compression testing machine. The compressive load was applied at the rate of 0.05 in./min.

DATA EVALUATION

The data accumulated from the testing program are given in Table 3. The values shown for dry density and unconfined compressive strength are averages of 3 tests, but, because of an insufficient quantity of soil 2, the values shown for treatment with quicklime D represent single specimen tests.

The compressive strengths attained and their relative $Ca(OH)_2$ contents are shown in Figure 2. One phenomenon that became immediately evident was the reductions in strengths as the lime contents were increased above an optimum amount. This was particularly evident in tests on soil 1. Evidence will be presented later that indicates that this may be primarily due to decreases in the compacted dry density of the test specimens. However, it is acknowledged that the apparent optimum lime content increases with longer curing periods before testing (5).

An overall difference was shown in the response of the soils to the limes; soil 1 gained compressive strengths of 2 to 3 times those of soils 2 and 3. Also, soil 1 responded comparably to each of the limes. Both the maximum strengths attained and rate of increase were essentially the same regardless of the lime used.

The most significant thing to consider in evaluation of data shown in Figure 2 is the effect of the limes on the less responsive soils 2 and 3. The highest strengths were attained with the granular quicklime C and then the mill-run quicklime E; fine quicklime D and hydrates A and B had about equal effect. Also, coarser hydrate B was at least as effective as fine hydrate A in terms of compressive strengths attained. For those soils tested, therefore, the coarser the grading of lime was, the higher the compressive strength obtained.

This is contrary to the intuitive conclusion that higher strengths should result from the more intimate mixing achieved with finely ground lime (Fig. 1). The possible cause of the strength variations appeared to be associated with the density of the compacted test specimens. Although the data are not presented, it was found that the compaction densities (California test 216) for the soils treated with the mill-run quicklime were higher than those obtained from the soils with hydrated lime. Similarly, review of the data given in Table 3 shows that both the amount and the type of lime affected the density of the compacted specimens. In every instance the densities of the compacted test specimens decreased as the amount of lime increased. The decrease in density averaged about 2 lb/ft^3 with each 1 percent increase in lime content. The density of soil 1 dropped from 111 to 101 lb/ft^3 when treated with 1 and 4 percent of quicklime E.

At a constant quicklime content, the different gradations caused variations in density of as much as 8 lb/ft^3. The general trend was for the specimens treated with granular quicklime C to have the highest density and those treated with finely ground quicklime D to have the lowest. This corresponds to the trend of compressive strengths noted previously.

To test the hypothesis that the fineness of the lime can account for these variations, an experiment using coarse and fine lime from the same source was prepared. This was accomplished by thoroughly pulverizing a portion of the granular quicklime C and then treating duplicate specimens with equal amounts of the pulverized and granular quicklime. A soil similar to soil 3 was used in this series of tests. The densities and compressive strengths of the specimens are given in Table 4.

The densities of the specimens treated with the pulverized quicklime ranged from 1 to 3 lb/ft^3 lower than those treated with the granular quicklime. Also, for amounts of lime greater than 2 percent, the compressive strengths were less for specimens treated with pulverized quicklime than for those treated with granular quicklime.

Because an arbitrary moisture content was used in this testing, it was necessary to verify that the effects of fineness on density and compressive strength were not due to a difference in water demand between the 2 quicklime treatments. Duplicate sets of soil specimens were treated with 3 percent of granular quicklime, and the same quicklime was pulverized. They were then compacted by the kneading compactor with varying amounts of water to develop the complete moisture-density curves for the 2 treatments. The soil with the granular quicklime had a higher density at all moisture contents.

It is concluded that the coarser quicklime would be at least as effective as that finely ground if used to stabilize soils for roadways. Two to 4 percent of lime is typically used for this purpose.

The effect of particle size on density and compressive strength, for both quicklime and hydrate, may be observed by comparisons based on an equivalent $Ca(OH)_2$ content. Figure 3 shows the density of the treated soils and the percent passing the No. 30 sieve at an equivalent $Ca(OH)_2$ content of 2.5 percent. A decrease in density is observed with increasing fineness of lime. Figure 4 shows a positive correlation between density and compressive strengths for soils 2 and 3.

Supplemental testing after the scheduled work was completed showed that specimen densities would have continued to increase with moisture contents up to 6 percent above the optimum established by test method 216. This is due to the lower effective compactive effort of the kneading compactor as compared to the California impact test. However, it was found that the increased moisture contents made no substantial change in the relative stabilizing effects of the different limes. Treatment with the granular quicklime C still resulted in higher densities and higher compressive strengths than treatments with the finer quicklimes and hydrated limes.

To summarize, it has been established that the fineness of lime affects the compactibility of a treated soil. At lime contents above 2 percent, the resulting reduction in density from use of the finer limes causes a strength loss greater than the benefits that would otherwise be expected from use of the finely ground products.

RELATED FINDINGS

After the compressive strength tests were completed, many of the test specimens were broken open for observation and, in several instances, a distinct stair-step effect

Table 4. Average density and strength of granular and pulverized quicklime C.

	Granular		Pulverized	
Lime (percent)	Density (lb/ft³)	Strength (lb/in.²)	Density (lb/ft³)	Strength (lb/in.²)
1	97	47	95	55
2	94	105	93	122
3	93	150	90	129
4	91	155	88	120

Figure 2. Percentage of calcium hydroxide and unconfined compressive strength of soils.

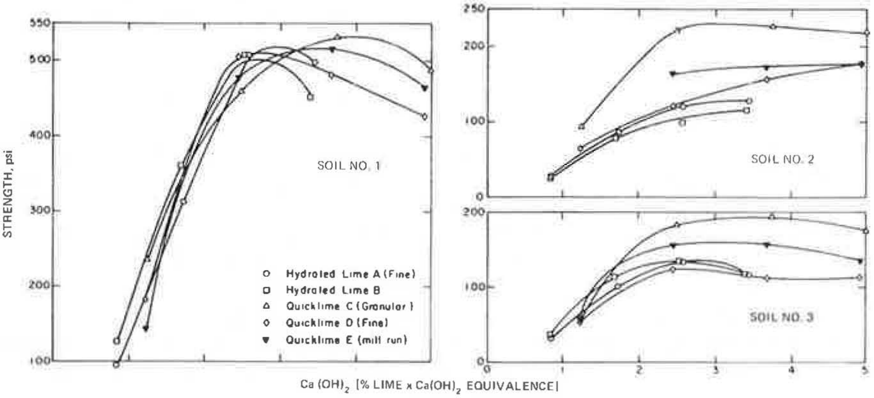


Figure 3. Compacted density of treated soil and fineness of lime.

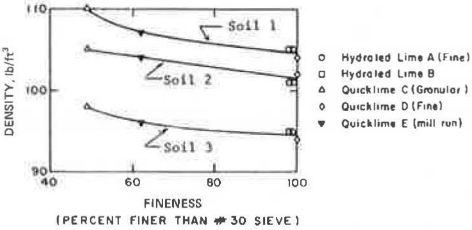
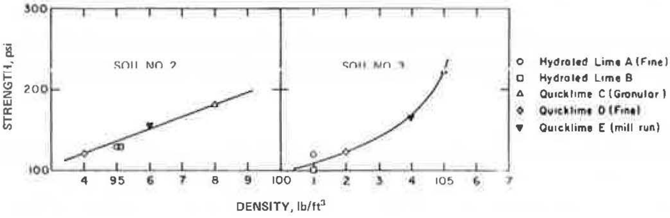


Figure 4. Density and unconfined compressive strength of soils.



was observed where the material separated on the smooth surfaces caused by the compactor foot. Because these separation planes were caused by only a single application of the compactor foot, it would be assumed that normal compaction procedures during construction of a roadway could also create a similar effect when the treated material is placed in more than 1 layer. This is verified by observation of separation of field cores along compaction planes. It is concluded that lime-stabilized soils should be compacted in as thick a lift as practicable, as long as the desired compaction is attained.

It was also found that quicklime may cause expansion of the compacted soil under certain conditions. A series of test specimens, treated with the granular quicklime C, was compacted immediately after preparation, that is, without the 24-hour loose-curing period. Excessive expansion and pop-outs on the surface of the specimens became evident almost immediately after compaction. This phenomenon was apparently the result of expansion of the quicklime particles as they reacted with water during the hydration process. As a result, it was concluded that a minimum loose-curing period is necessary when granular quicklime is used.

CONCLUSIONS

1. When compared on an equal percentage by weight basis, quicklime is generally more effective than hydrated lime for improving the compressive strength of soils. The primary reason is the greater calcium hydroxide potential of quicklime. (The quicklimes included in this study had an equivalence of approximately 50 percent more calcium hydroxide than the hydrated limes of the same weight.) When compared on the basis of equivalent $Ca(OH)_2$ content, all forms of quicklime studied equaled or exceeded the hydrated limes in improving the compressive strength of soils.
2. At lime contents above 2 percent, higher compressive strengths are obtained with the coarser quicklime products than with those ground excessively fine. This is apparently due to the effect the fine limes have of reducing the compactibility of the treated soil. The resulting lower densities of test specimens, and lesser strengths, offset the benefits that would otherwise be expected from use of the fine limes.
3. A fineness of hydrated lime of more than 75 percent passing the No. 200 sieve does not appear to increase its effectiveness in soil stabilization.
4. A loose-curing period for soils treated with quicklime is required to prevent expansion and pop-outs caused by hydration of the lime. A 24-hour period is satisfactory, as determined by laboratory tests.

ACKNOWLEDGMENTS

This work was done in cooperation with the Federal Highway Administration. The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Administration.

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INVESTIGATION OF SEVERAL ADDITIVES FOR CONTROLLING THE EXPANSION OF PIERRE SHALE

E. R. Hoskins and D. W. Hammerquist,
South Dakota School of Mines and Technology; and
D. Irby, U. S. Bureau of Mines

Soil expansion and the accompanying pavement distress in roadways constructed on the Pierre formation have been a serious problem for the South Dakota Department of Highways for many years. Because of the distribution of the Pierre formation in South Dakota, most highways in the western half of the state have been affected. In the last decade, research has been carried out by the highway department and other agencies to find for primary roadways a suitable design that would eliminate or at least minimize roadway heave and the subsequent breakup of the pavement. The investigation summarized in this report is a continuation of that major research effort.

•ALONG Interstate 90 from Wasta west to Rapid City, there are many sections of portland cement concrete pavement that has buckled and faulted. In the New Underwood area, after the pavement had started to fail, 8-in. diameter by 5-ft deep holes were drilled adjacent to the slab, and a water-lime-sand mixture was poured into the holes. A full description of this work has been given by McDonald (1). Selected slabs in this area were also cored and direct water induction applied to the subgrade. This work was reported by Whyte (2). Those techniques, while encouraging, were apparently not sufficient to completely control the problem, and the alternative was to resurface the most critical areas of pavement distress and to patch and repatch many of the other "cut" areas. The patching and repatching have continued to this time and in all likelihood will have to be continued for the rest of the life of the highway.

In 1968 and 1969 an investigation of pavement distress in roadways in western South Dakota was carried out by Hammerquist and Hoskins (3). This investigation attempted to correlate expansive soil properties, soil moisture, and faulting and jointing in the underlying Pierre shale with areas of severe pavement distress. The investigators found that, in sections of primary roadways in which the subrade had been treated prior to construction, pavement heave was still possible. Moisture apparently could get down through the subgrade (or come up if the water table rose) into the faulted and jointed material under the subgrade and cause swelling along the strike of the faults. When the strike of the fault crossed the roadway at approximately a right angle, a prominent sharp bump in the surface resulted. Because swelling is apparently still possible, even after extensive subgrade treatment during construction, it seems obvious that the currently used lime treatment is not totally effective.

The purpose of the investigation we now report on was to find out whether some of the locally available materials with known swell-inhibiting properties might be used to more effectively and more economically control swelling in roadways built in the Pierre shale. The principal materials we chose to investigate were gypsum, lignite, and byproduct ash from the state cement plant. We also ran tests on lime, an inert grit (carborundum powder), and slime tailings from the Homestake Mining Company's mill.

Sponsored by Committee on Lime and Lime-Fly Ash Stabilization.

PROCEDURE

Several barrels of Pierre shale were collected during the summer months of 1969. Samples were chosen to obtain "typically difficult" samples of the Virgin Creek member. Shale samples were collected from the Pierre, Fort Thompson, and Draper areas. The samples from the Draper area were obtained from a construction site on Interstate 90.

The samples were placed in plastic bags in order to retain as much of the field moisture as possible. The samples were then mixed together and sieved through a No. 4 sieve and placed again in plastic bags. A standard Proctor test (AASHO Designation T-99) was performed, and maximum dry density and optimum moisture content were found. The results of the Proctor test established the maximum dry density at 81.5 lb/ft^3 and the optimum moisture content at 36 percent. The expansion tests were run at 95 percent of Proctor density and at 36 percent of moisture content.

The cured sample was placed in a Rowe consolidation cell at the required density. The material was compacted in the cell in a single lift in a hydraulic testing machine. The Rowe cell used in this investigation is a hydraulic consolidation cell. It is 10 in. in diameter and 5 in. high. A 3-in. thick sample was used in the cell to allow for 2 in. of expansion during the test.

Tests took from 10 to 30 days to complete. The sample, after being placed in the cell, was allowed to rebound. When the sample had stabilized, as evidenced by constant readings on the dial gauge, the saturation water was introduced into the bottom of the sample at approximately 0.5 psi. Confining or overburden pressure was introduced at the top of the sample, inside a convoluted rubber bellows, at 2 psi or approximately 300 psf. The purpose of the confining pressure was to simulate the weight of the pavement and subgrade on the soil. A dial gauge attached to the bridge on the cover of the cell read expansion of the sample to 0.0001 in.

RESULTS AND CONCLUSIONS

The results of this investigation are shown in Figure 1. The results have been computed in terms of percentage of volume increase of the soil sample versus elapsed time. These results are given quantitatively in Table 1.

The conclusions to this investigation depend on the criteria one chooses to evaluate the results. If reduction of the swelling ability of the subgrade is the principal aim of using the additive, then from our tests lime appears to be slightly better than the other additives at the same concentration. All of the additives we investigated, however, including inert grit reduced swelling by at least one-third compared to the raw soil.

Permanence of the treatment is another criterion that must be considered. Lime treatment has now been shown to still be effective after 8 years (1). None of the other materials has yet been evaluated for long-term effectiveness.

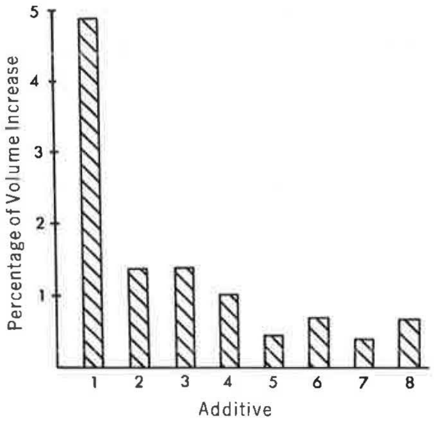
The effect of the various additives on the strength or bearing capacity of the subgrade is also extremely important. Part of the economy of using lime as a stabilizing agent is in the reduced thickness of pavement that is required because of the increased strength of the subgrade. The effect of the other additives on a measure of strength, such as a California bearing ratio test, has not been investigated. The increased bearing capacity of treated subgrades also aids in the construction of the highway by giving the contractors an all-weather road for their construction equipment. This relatively stable temporary surface reduces the amount of construction time lost because of bad weather.

Finally, the relative costs of the various additives must be considered. The cost of producing lime is probably much higher than the cost of producing any of the other additives. It is quite probable, for example, that the cost of buying and shipping a ton of mill tailings from the Homestake mine to a highway construction site anywhere in western South Dakota would be much lower than the cost of getting a ton of lime to the same site. If so, a higher percentage of mill tailings might be used in the subgrade to secure a treatment equal to or perhaps better than that the lime is currently giving. A greater thickness of subgrade might also be treated. Lignite and waste products from the state cement plant may have similar cost-effectiveness characteristics. The only way to substantiate any of these suppositions is to make a detailed cost study of the various materials, and such a study was not included as a part of this investigation.

Table 1. Percentage of volume increase.

Elapsed Time (min)	Type of Additive 1	2	3	4	5	6	7	8
10,000	4.30	0.92	1.13	0.92	0.35	0.52	0.31	0.36
15,000	4.69	1.32	1.32	0.95	0.37	0.63	0.35	0.54
20,000	4.91	1.39	1.39	1.02	0.44	0.71	0.43	0.70

Figure 1. Percentage of volume increase ater 20,000 min for 8 additives.



Note: The additives, given by number in Figure 1 and Table 1, are as follows:

Number	Additive	Percent
1	Raw soil	0
2	Gypsum	5
3	Inert grit	3
4	Waste product from state cement plant	5
5	Lime, dry	5
6	Lignite	5
7	Lime, slurry	5
8	Slimes	5

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LIME STABILIZATION OF ORGANIC SOILS

Ara Arman and George A. Munfakh, Louisiana State University

The potentials of improving the engineering characteristics of organic soils found in Louisiana were studied. A comprehensive literature survey on soil-lime reactions and the properties of organic soils was made. Laboratory studies of approximately 1,100 samples of inorganic and organic soils showed that the engineering characteristics, such as plasticity and strength, are improved by the addition of lime. At low organic content, lime effects a reduction in the plasticity index, with the minimum occurring at about 4 percent lime content. At higher organic content, the plasticity index continues to decrease with increasing lime content. The dry density and optimum moisture content of soil-lime mixtures vary with the amount of lime added to the soil, depending on the amount of fine clays. Changes in pH of soil-lime mixtures showed that increased levels of lime treatment were able to neutralize the acidity of organic matter in the soil and initiate the soil-lime reaction. Previous studies at the Louisiana State University have shown that the organic content of the majority of soils in Louisiana is approximately 20 percent. Based on this study it is concluded that the addition of lime improves the engineering characteristics of the soils studied.

•REACTIONS between lime and soils have been studied by several researchers (1, 2). Diamond and Kinter (3) detected 2 reaction stages: (a) a rapid process during which the plastic properties improve, but little permanent strength develops; and (b) a later slow process of strength development with the formation of cementitious products.

Other workers (4, 5, 6) have shown that the addition of lime to a plastic soil results in a reduction of the plasticity index (PI). Both the plastic limit (PL) and the liquid limit (LL) of the soil are affected. Usually, an immediate increase in the plastic limit is observed after the addition of lime. This increase varies directly with the lime content up to some limiting amount, referred to as the "lime fixation point" (7). However, lime has different effects on the liquid limit; some authors report decreases in this property after addition of lime (5, 6), and others report substantial increases (8). Other workers (4, 9) have found both increases and decreases that depend on the type of the soil. Anday (10) found that strength development in soil-lime systems is comparatively slow, characteristically requiring several weeks to many months at normal temperatures. Thompson, who studied the elastic properties of soil-lime mixtures, reported that lime treatment improved the shear strength (11).

Soils can be divided into 2 major groups: organic and inorganic. Organic soil is defined as "any soil containing a sufficient amount of organic matter to influence its engineering properties" (12). The organic matter is derived from the decay of plants.

Clare and Pollard (13) indicate that organic materials retard the hardening of soil-cement mixtures by retaining the calcium ions liberated during the hydrolysis of the calcium silicates and aluminates in the cement. On the basis of that mechanism, we should be able to relate the "activity" of organic material to its capacity to adsorb calcium ions from solutions of high pH. Thus, soil-lime mixtures should also be affected by the presence of organic matter.

Thompson has shown that organic matter adversely affects the pozzolanic reaction (14). That influence can logically be attributed to the retardation of the strength-producing

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pozzolanic reaction (15). Organic matter has a high base-exchange capacity (16). When lime is added to organic soil, some of the Ca^{++} ions are used to satisfy the exchange capacity of the organic matter, thus reducing the number of calcium ions available for the pozzolanic reaction. Maclean and Sherwood (17), in their study of the effect of organic materials on soil-cement reactions, indicated that some hydroxyquinine compounds were the active organic components that interfered with the reactions by forming complexes with the metal ions (such as calcium). Similarly, the formation of such complexes in soil-lime mixtures would reduce the quantity of available calcium and affect the long-term pozzolanic reaction.

A study of cementitious reactions between soil and lime and the effect of organic matter on these reactions was conducted by Ferrell (20) using the specimens formed from soils investigated in this study. His results show that organic soil-lime mixtures produce cementitious products similar to those reported for inorganic soil-lime reactions and that the organic matter does not appear to block the reaction that results in increase in the soil strength.

TESTING METHODS AND SPECIMENS

The testing schedule for this study was prepared for investigating the effects of soil-lime reactions on the engineering characteristics of the mixture and the effect of organic materials on these reactions. The short-term reaction was studied by its effect on the Atterberg limits and the long-term pozzolanic reaction was studied by the change in the compressive strength. Three types of cylindrical specimens were fabricated: mineral, nonorganic soils; organic soils obtained from natural deposits (without alteration of organic content); and natural organic and mineral soils (organic content predetermined through the varying of mineral content). Clays and silty clays (Table 1) were used because clay minerals are the major components largely controlling the physical properties of stabilized soil (18).

The organic soils were obtained from different areas in Louisiana. Loose samples were oven-dried and then crushed in a disk crusher to pass a No. 10 sieve (ASTM Designation D 421-58). Part of the same sample was pulverized to pass the No. 40 sieve for determining the Atterberg limits and the pH. The organic content was found from dry combustion, 450 C for 6 hours (12).

The plasticity of organic soil-lime mixtures was determined by placing samples with different lime and moisture contents in plastic bags (not airtight) and curing them in a 100 percent humidity room at 75 F for periods of 48 hours, 7 days, and 28 days. Liquid and plastic limit tests were then performed on the cured samples.

The effect of lime on the density and optimum moisture content was studied by use of the standard compaction test (AASHO T-99). After the mixtures of organic soils, lime, and water had been cured in the 100 percent humidity room for varying lengths of time, the standard compaction test was performed.

Changes in strength were determined from unconfined compression tests of mixtures without lime and with different lime contents. Two types of specimens were used: one with sufficient water to approximate the liquid limit and one with water content equal to the optimum moisture content (AASHO T-99). These 2 moisture contents were chosen for obtaining the strength of the soil both in the weakest and in field-compacted conditions (12). Also, approximating the liquid limit allowed the mixing of very homogeneous samples.

Immediately after mixing, the soil-lime-water mixture was placed in aluminum cylindrical molds (20) with a diameter of 2.5 in. (This diameter allows the soil samples to fit in the unconfined compression and triaxial equipment without being trimmed.) Then samples were allowed to consolidate in the mold under a 40-lb surcharge approximating the average natural overburden pressure of 8.34 psi (12). After the 48-hour curing and consolidation period, the samples were extruded from the chambers, placed in plastic bags, and kept in the moisture room for the rest of the curing period. Separate samples were cured for testing at 48 hours, 7 days, and 28 days.

Various mixtures of soil and lime were also tested triaxially. Undrained quick tests were performed on samples with optimum moisture content. After curing periods of 7 and 28 days, the samples were tested under confining pressures of 0, 5, 10, 15, and 20 psi.

To determine the amount of lime needed to stabilize a soil requires a determination of the amount that will satisfy the reactions between calcium and the organic and inorganic compounds and the calcium absorption of the hydroxyquinines in organic soils as reported by Maclean and Sherwood (17). (Soil stabilization, in general, is defined as the changing of any physical or chemical property or both of a soil to improve its engineering performance and thus render it suitable for a particular use.)

Hilt and Davidson (7) used the plastic limit to determine the "lime fixation point." This method is satisfactory but time-consuming. The pH test devised by Eades and Grim (19) was used in this study to determine the lime content for soil stabilization because it is quicker. Eades and Grim explain that "since lime is very alkaline (pH-12.4 at 25° C), the silicates, especially the clays will be attacked, freeing silica and alumina with which calcium reacts to form calcium silicates as long as a high alkaline condition is maintained" (19). Because a high alkaline condition is required for the formation of calcium silicates, the least amount of lime that will produce a pH of 12.4 is the amount required to stabilize the soil. The pH readings were made on the soil-lime mixtures after curing periods of 1 hour and 7 days.

All tests in the study were performed by using a minimum of 2 identical specimens. Three identical specimens were used for the unconfined compression, Atterberg limits, and pH determinations. Whenever the test results of a set of specimens varied more than ±3 percent, the tests were repeated.

TEST RESULTS

Atterberg Limits

The effect of lime on the Atterberg limits is illustrated by plots of the liquid and plastic limits of organic soil-lime mixtures and the $Ca(OH)_2$ content for different curing periods. The change in Atterberg limits of a typical soil (M-16, silty clay, with 22.00 percent organic content) is shown in Figure 1. The net effect was a decrease in the plasticity index (Table 2). [Within a period of 48 hours, the PI decreased from 22 to 7 because of the addition of 12 percent of $Ca(OH)_2$ to soil M-16, as shown in Fig. 1.] This change in the plasticity can be attributed to the base-exchange reaction of soil and lime (the divalent calcium ions Ca^{++} replace the weaker univalent ions, such as Na^+ and H^+, on the clay particle surface) and the crowding of the calcium ions on the surface of the clay particles, resulting in flocculation.

Figure 2 shows the plasticity index and the lime content. The plasticity decreases until a critical point (4 percent lime) is reached. The increase in plasticity after this point can be explained by the chemical satisfaction of the reaction between lime and soil. Excess lime acts as a lubricant to increase the plasticity.

Density and Optimum Moisture Content

The changes in the maximum density and optimum moisture content of a typical organic soil (M-1) are shown in Figure 3. For this soil, the density decreased and the optimum moisture content increased as the lime content was increased. For example, the addition of 4 percent of lime reduced the density of soil M-1 from 82.6 to 75.0 lb/ft^3 for a curing period of 7 days. However, for soil M-9 (Fig. 4), an increase in the density was observed after the addition of lime. The optimum moisture content decreased. These effects are attributed primarily to flocculation and the change in gradation due to pozzolanic reaction that results in cementing fine clay particles. Figures 3 and 4 also show that as the curing time is extended the density further changes, thus indicating that pozzolanic reaction is taking place and continuing.

Compressive Strength

Lime increases the strength of mineral and inorganic soils because of the pozzolanic reaction. This increase also occurred in all the organic and inorganic soils tested in this research program. A typical increase in the unconfined compressive strength is shown in Figure 5. The changes in the unconfined compressive strength of 3 organic soil-lime mixtures, M-1, M-9, and M-16, are given in Table 3. Generally, the strength

Table 1. Basic soils for soil-lime mixtures.

Soil Designation	Soil Type	Organic Content (percent)	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index	pH
T-5	Silty clay	0	38.6	17.1	21.5	7.40
T-8	Silty clay	0	38.9	17.0	21.9	7.10
L-4	Silty loam	0	34.3	12.9	21.4	6.99
M-0	Heavy clay	0	67.7	27.5	40.2	7.80
M-1	Natural organic	8.10	63.0	27.0	36.0	7.40
M-9	Natural organic	14.20	90.3	52.1	38.2	6.61
M-10	Heavy clay and organic material	6.60	64.9	ND[a]	ND	7.08
M-11	Heavy clay and organic material	9.08	70.6	ND	ND	6.93
M-12	Heavy clay and organic material	12.60	72.8	ND	ND	6.77
M-16	Silty clay and organic material	22.00	62.95	40.55	22.40	5.40
M-18	Silty clay and organic material	10.00	ND	ND	ND	6.88
M-20	Silty clay and organic material	20.00	ND	ND	ND	5.45

[a] Not determined.

Table 2. Atterberg limits of soil-lime mixtures M-1, M-9, and M-16.

		48 Hour			7 Day			28 Day		
Soil	$Ca(OH)_2$ Content (percent)	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index	Liquid Limit (percent)	Plastic Limit (percent)	Plasticity Index
M-1	0	63.0	27.0	36.0	63.0	27.0	36.0	63.0	27.0	36.0
	2	70.5	35.3	35.2	70.6	46.7	23.9	61.2	38.0	23.2
	4	63.2	39.7	23.6	65.3	44.6	20.7	61.0	45.0	16.0
	8	66.1	41.65	24.55	69.0	43.25	25.75	63.65	42.38	21.23
M-9	0	90.4	52.2	38.2	90.4	52.4	38.2	90.4	52.2	38.2
	4	95.6	58.0	37.6	84.8	64.7	20.1	89.5	53.3	36.2
	8	87.6	59.6	28.0	79.1	67.85	11.25	80.6	55.8	24.8
	12	77.4	66.5	10.9	77.95	72.7	5.25	80.2	61.8	18.4
M-16	0	63.0	40.6	22.4	63.0	40.6	22.4	63.0	40.6	22.4
	4	67.0	54.4	12.6	64.75	57.5	7.25	69.0	52.32	16.68
	8	65.0	55.35	9.65	65.05	61.80	3.25	67.0	58.9	8.1
	12	63.1	56.3	6.8	62.9	59.8	3.1	62.6	59.1	3.50

Figure 1. Atterberg limits of soil-lime mixture M-16.

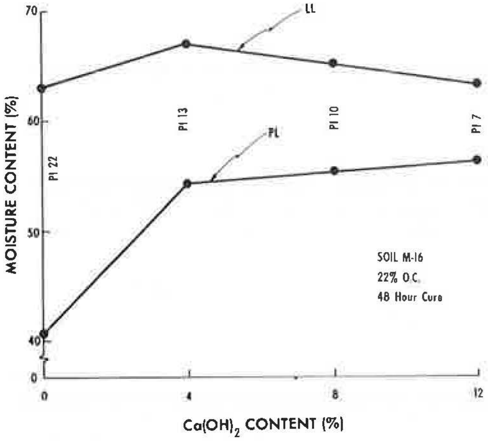


Figure 2. Plasticity index of soil-lime mixture M-1 at 3 curing times.

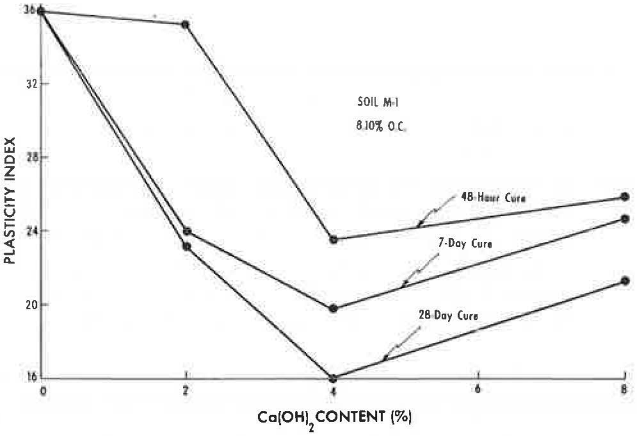


Figure 3. Density and optimum moisture content of soil-lime mixture M-1 at 2 curing times.

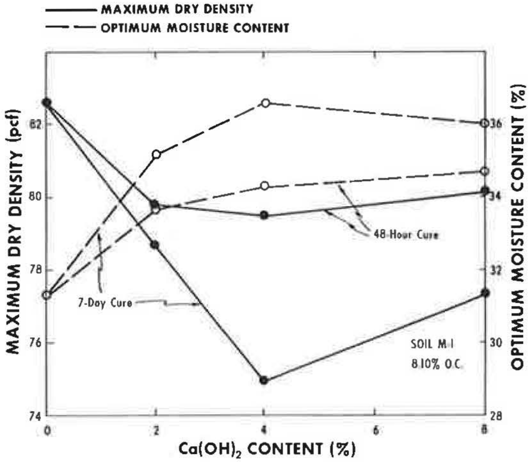


Figure 4. Density and optimum moisture content of soil-lime mixture M-9 at 3 curing times.

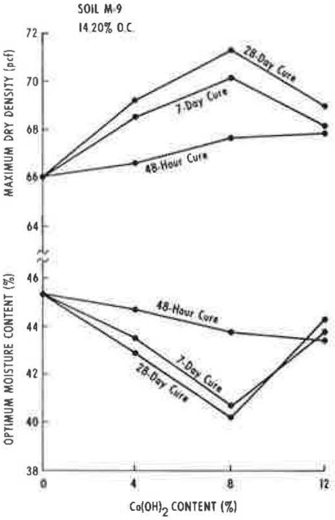
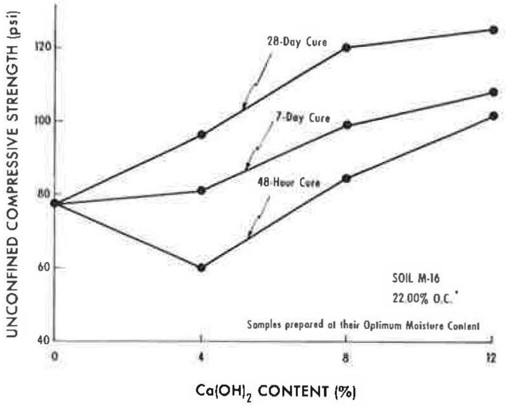


Figure 5. Strength of soil-lime mixture M-16 at varying lime contents and 3 curing times.



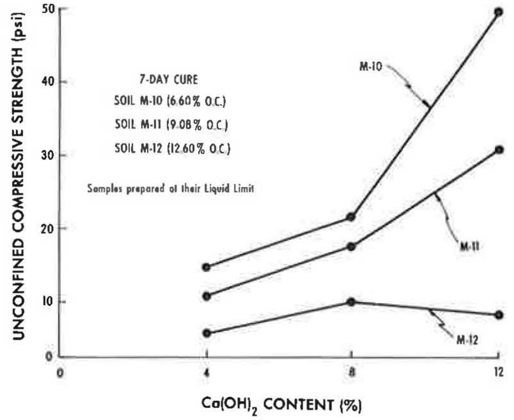
·Organic Content

Table 3. Unconfined compressive strength of soil-lime mixtures M-0, M-1, M-9, M-16, T-8, M-18, and M-20.

Soil	$Ca(OH)_2$ Content (percent)	Strength (psi)		
		48 Hour	7 Day	28 Day
M-0[a]	0	4	4	4
	2	7	5	9
	4	13	25	49
	6	13	30	59
M-1[a]	0	4	4	4
	2	3	3	4
	4	4	6	8
	8	4	5	7
M-9[a]	0	4	4	4
	4	3	5	6
	8	5	9	9
	12	7	11	11
M-16[b]	0	77	77	77
	4	60	81	96
	8	84	99	120
	12	102	108	125
T-8[b]	12	—	208	200
M-18[b]	12	—	105	160
M-20[b]	12	—	60	75

[a] Samples prepared with moisture content approximating the liquid limit.
[b] Samples prepared with optimum moisture content.

Figure 6. Strength of soil-lime mixtures M-10, M-11, and M-12 at varying lime contents and 1 curing time.



Note: 0% $Ca(OH)_2$ specimens were too weak and could not be tested.

Figure 7. Strength of soil-lime mixtures T-8, M-18, and M-2 at 12 percent lime content and 2 curing times.

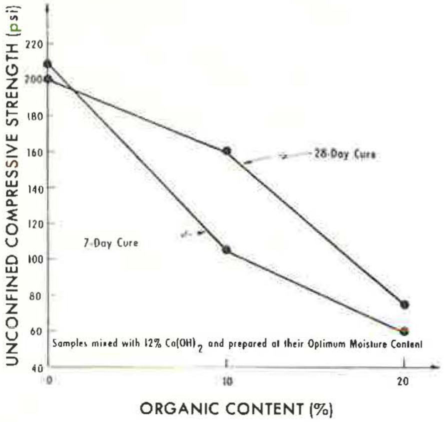


Figure 8. Failure envelopes of soil-lime mixtures M-0 and M-20 at varying lime contents and 1 curing time.

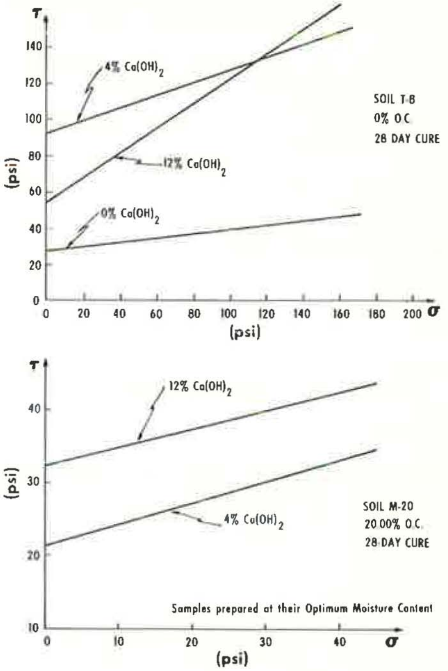


Figure 9. pH of soil-lime mixture M-16 for varying lime contents and 1 curing time.

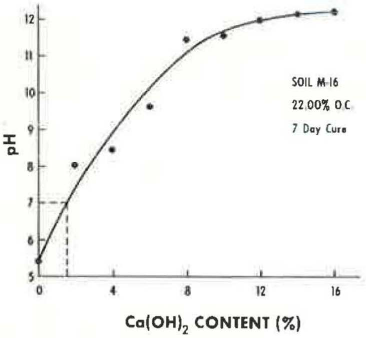


Table 4. pH of soil-lime mixtures M-0, M-10, M-11, M-12, M-9, M-1, and M-16.

Soil	Curing Time	pH With Percentages of $Ca(OH)_2$								
		0	2	4	6	8	10	12	14	16
M-0	1 hour	7.80	11.80	12.22	12.30	12.37	12.39	12.40	12.40	12.40
	7 day	7.80	11.00	11.54	11.69	11.99	12.06	12.22	12.40	12.40
M-10	1 hour	7.08	11.45	12.11	12.28	12.26	12.28	12.30	12.31	12.33
	7 day	7.08	10.33	11.45	11.92	12.11	12.19	12.21	12.22	12.24
M-11	1 hour	6.93	11.00	12.02	12.19	12.27	12.29	12.31	12.35	12.38
	7 day	6.93	9.72	11.12	11.66	12.00	12.15	12.20	12.21	12.22
M-12	1 hour	6.77	10.35	11.83	12.11	12.17	12.20	12.21	12.24	12.27
	7 day	6.77	8.34	10.80	11.61	11.99	12.10	12,19	12.21	12.22
M-9	1 hour	6.61	10.04	11.78	12.15	12.16	12.20	12.22	12.25	12.26
	7 day	6.61	8.34	9.29	11.40	11.98	12.04	12.13	12.17	12.20
M-1	1 hour	7.40	11.00	12.10	12.20	12.31	12.32	12.33	12.34	12.40
	7 day	7.40	9.65	11.52	11.95	12.15	12.20	12.22	12.23	12.25
M-16	1 hour	5.40	8.88	10.93	11.94	12.29	12.34	12.35	12.36	12.38
	7 day	5.40	8.04	8.47	9.62	11.46	11.56	11.99	12.15	12.21

increases with lime content. The maximum strength of soil M-1 occurred at an optimum lime content of 4 percent. However, the strength of the other 2 soils was still increasing at 12 percent, which indicates that the optimum lime content was not reached.

It must be emphasized that samples showing small increases in strength after the addition of lime were prepared at their liquid limits. At the time of testing they were fully saturated, they were assumed to represent the treatment of natural deposits, and they were used to determine whether any benefits could result from the addition of lime.

The strength changes due to the addition of lime in artificial mixtures of organic and mineral soils (M-10, M-11, and M-12) are shown in Figure 6. Three specimens were prepared from mixtures of a mineral soil (M-0) and a natural organic soil (M-9). These curves indicate that, although the strength increases with the addition of lime, the increase is affected by the presence of organic materials. This effect is also shown in Figure 7 for T-8, M-18, and M-2. When lime is added to the organic soil, some calcium ions are used to satisfy the base-exchange capacity of the organic matter. Fewer ions are available for the pozzolanic reaction. However, the results of pH readings for the soil-lime mixtures showed that a lime content of less than 2 percent was required to neutralize (pH = 7) the acidity of the tested organic soils. Thus, increased amounts of lime (above 2 percent) appear to satisfy the pozzolanic reaction and produce an increase in the strength (T-8, M-18, and M-20, Table 3).

Shearing Strength

Shearing strength was also studied for different soil-lime mixtures. The typical shift upward of the failure envelopes after the addition of lime is shown in Figure 8. The apparent cohesion of soil-lime mixtures increases with lime content because of the cementing action of the lime. For most organic soils, the evaluation of the apparent φ due to increasing lime content was difficult. However, for the mineral soil, the apparent angle of internal friction increased directly with lime content. This is most probably due to changes in particle size and shape.

pH and Optimum Lime Content

The pH of soil-lime mixtures (Table 4) increased directly with lime content because of the alkalinity of lime. When the pH of a mixture approaches 12.40 (pH of lime at 25 C), the soil-lime reaction is chemically satisfied as discussed earlier. For most of the soils, a pH of 12.40 was not reached for the lime contents studied. However, for lime contents of more than 12 to 14 percent, little increase in pH occurred for each content increment. Apparently, even at such high lime contents, some reaction between the soil and the lime was occurring.

Figure 9 shows that, in a typical soil (M-16) cured for 7 days, the pH does not fluctuate when the lime content is increased above 12 percent. Thus, 12 percent of lime was sufficient for stabilization. For the same lime content and curing period, the plasticity index of this soil dropped from 22.40 to 3.10 percent; the strength increased from 77 to 108 psi.

Figure 9 also shows that a lime content of less than 2 percent was needed to satisfy the base-exchange capacity of the organic matter. It was assumed that the base-exchange capacity of the organic matter was satisfied when the pH of the soil was raised to 7 by the addition of lime. Lime added beyond this point will not be used to satisfy the acidity of the organic matter, and it will be available for the pozzolanic reaction if the soil is reactive. The pH rose from 5.4 to 7.

CONCLUSIONS

The following conclusions and recommendations are made based on the results of this study.

1. Increases in strength with the addition of lime, changes in physical properties of organic soils, and increases in strength with time indicate that organic matter in soils does not block the pozzolanic reaction.

2. Changes in the pH of a soil containing 20 percent of organic matter indicate that the addition of less than 2 percent of lime is sufficient to safisfy the base-exchange capacity of organic matter in the soil (based on the assumption that a pH of 7 indicates satisfaction of the exchange capacity).

3. The plastic properties of mineral and organic soils are improved by the addition of lime. An immediate increase occurs in the plastic limit of the soil after the addition of lime. Both increases and decreases were observed in the liquid limit. The net effect of lime on the Atterberg limits of organic soils is a reduction in the plasticity index.

4. The moisture-density characteristics of mineral and organic soils are affected by the addition of lime. Both increases and decreases in the maximum density and optimum moisture content were observed for various soils. This fact indicates that a moisture-density test should be performed for every soil-lime mixture to determine the optimum moisture content to be used in the field.

5. The addition of lime increases the strength of all inorganic and organic soils tested, as measured by unconfined compression tests. The shear strength of mineral and organic soils is improved because of an increase in cohesion; small increases occur in the slope of the failure envelope. The increases shown in this study are significant in terms of original untreated strength of organic soil.

6. Organic soils tested in this study are suitable for stabilization with lime, provided the characteristics of the end product of these mixtures yield adequate strength and other engineering properties for a particular use.

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SODIUM SILICATE STABILIZATION OF SOILS: A REVIEW OF THE LITERATURE

Claude H. Hurley, Consulting Engineer; and
Thomas H. Thornburn, University of Illinois

This report consists of an annotated bibliography and summary review of the important literature on the use of sodium silicates in soil stabilization processes. Annotations are given for approximately 90 articles published between 1931 and 1965. On the basis of these articles the authors have summarized pertinent information on stabilizer properties, reaction mechanisms, injection methods of soil solidification, properties of stabilizer-soil mixtures, and use of sodium silicates as dustproofers and waterproofers and as secondary additives with other stabilizers. There is some evidence that sodium-silicate stabilization can increase the strength and durability of sandy soils or even relatively nonplastic fine-grained soils under mild climatic conditions; however, there is almost no field evidence to justify its use when the stabilized material is subjected to the freezing and thawing cycles typical of temperate-to-cold climates. Sodium silicates with or without the addition of precipitants are of little value in dustproofing or waterproofing fine-grained soils. On the other hand, laboratory tests seem to indicate that sodium silicate used as an additive can improve the strength and durability of soils stabilized with portland cement, lime, or lime-fly ash.

•THIS REPORT is the final one of a series on the chemical stabilization of soils. It is limited to a review of existing literature on soil stabilization with sodium silicate compounds. No original or laboratory investigations were conducted. Similar reports on lime (1), phosphoric acid (2), and inorganic salts and bases (3) have been completed and published. The prime objectives are to acquaint the highway engineer with the present state of the art and to suggest whether the subject appears to hold promise as an area of future research.

Within the limitations of available information, the report describes stabilizer properties, stabilization mechanisms, physical properties of stabilized mixtures, construction methods, and use of sodium silicate as a secondary additive. The opinions expressed were arrived at primarily from the study of the literature. We believe that the annotated bibliography includes all articles that deal directly with sodium silicate stabilization from 1930 to January 1965. Most of the articles listed were published since 1940.

Practically all of the early articles and many of the later ones refer to the use of sodium silicate as a chemical grout to stabilize sands in place or to reduce their permeability. Although such techniques are important, there is no direct relationship between the results obtained (often temporary) and the requirements for highway usage where the stabilized material is exposed to repetitive loads and climatic variations. When articles of this type are eliminated from consideration, the useful literature is meager indeed.

The literature also includes some publications on the use of sodium silicates in the manufacture of bonded sand molds and cores for casting molten metals. A review of several of those indicated that the temperatures employed are above the range commonly

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encountered in nature. Consequently, it was felt that those publications were not of particular value to the discussion of soil stabilization; thus, they were not thoroughly investigated.

SODIUM SILICATES

Sodium silicates constitute a group of chemicals that possess a wide range of physical and chemical properties. They are used in industry as adhesives, cements, detergents, deflocculants, protective coatings, rust inhibitors, catalyst bases, cleaning compounds, and bleaching agents. Silicates are produced at various alkali, Na_2O, to silica, SiO_2, ratios, water contents, and particle sizes depending on their proposed use. They are usually derived from the relatively abundant raw materials of silica, sodium salts, and water. Manufacturers have widely distributed outlets for the products; therefore, sodium silicates are readily available and are easily obtained in various packages for commercial use. Vail has prepared what appears to be the most comprehensive treatise on the manufacture, properties, and uses of the soluble silicates (4).

Manufacture

Sodium silicates may be prepared in a number of ways. Several so-called wet processes include the solution of infusorial or diatomaceous earth in the alkaline hydroxides with relatively poor quality and product control. The reaction between ferrosilicon, manganosilicon, or silico speigel and concentrated sodium hyroxide can be simply expressed as

$$2NaOH + Si + H_2O \rightarrow Na_2SiO_3 + 2H_2$$

and the reaction of sodium hydroxide and silicon carbide as

$$4NaOH + SiC + 2H_2O \rightarrow Na_2SiO_3 + Na_2CO_3 + 4H_2$$

Both of these processes involve relatively high cost. The silicates have also been prepared by reactions of sodium hydroxide with residues of the extraction of metals such as aluminum, beryllium, and tungsten from their respective silicate minerals.

Most of the sodium silicate made in the United States is produced by fusing a specially selected sand, SiO_2, and soda ash, Na_2CO_3, at high temperatures in open-hearth furnaces. The resulting product is a molten glass that can be allowed to solidify and then ground into various particle sizes. It most commonly has a composition (Na_2O:SiO_2) of 1:3.3. The molten glass may also be dissolved in steam under pressure to produce silicate solutions at various desired concentrations.

Other dry methods of preparing the silicates include the fusion of sodium nitrate, sodium hydroxide, or metallic carbonates with silica. In Europe the reaction of sodium sulfate, carbon, and silica when fused is most commonly employed.

Types

Theoretically, alkali and silica may be combined in all proportions, but present products do not exceed a silica to alkali ratio of about 4 to 1 by weight because of the very low solubility of fused silicates above this ratio. The soluble silicates range physically from readily and highly soluble crystals to lumps of transparent glass that are sparingly soluble even in boiling water and from slightly sticky fluids of the consistency of maple syrup to heavy viscous materials that barely flow. The available commercial products marketed by any one company usually range from 15 to 30 silicate solutions and 5 to 15 anhydrous silicates.

The silicates of greater alkalinity than SiO_2:Na_2O = 1.60 are definite crystalline salts of fixed composition and can be identified by specific chemical formulas, listed in order of decreasing alkalinity as follows:

$SiO_2:Na_2O$	Name	Formula
1:2	Sodium orthosilicate	Na_4SiO_4
2:3	Sodium sesquisilicate	$Na_6Si_2O_7$
1:1	Sodium metasilicate	Na_2SiO_3
2:1	Sodium disilicate	$Na_2Si_2O_5$

Silicates that are less alkaline than metasilicate form colloidal solutions. Because a molecule of Na_2O weighs about the same as a molecule of SiO_2, the molecular ratio and weight ratios are very nearly equal. Consequently, it has become standard practice to use weight ratios for sodium silicates that are more siliceous than metasilicate (1:1). The range in weight ratio, $SiO_2:Na_2O$, currently available is about 3.75 to 1.60. These solutions are obtainable in concentrations from 35 B to about 59 B at 68 F.

The silicate most commonly employed in the treatment of building materials is a solution known as water glass that has a silica:alkali ratio of about 3.22 and is sold at a density of about 41 B at 68 F or a specific gravity 1.394.

Properties

Sodium silicate solutions are clear, colorless, and odorless. They are nonflammable and, in terms of a health hazard, act like a strong soap. Silicates in the eye are painful and should be flushed immediately with water. Although considered nontoxic, accidental swallowing may cause stomach upset. These are the significant hazards to be guarded against when the silicates are being used.

If kept in an airtight container, silicates will not deteriorate during extended periods of storage. The containers must be made of steel or other nonreactive materials. Aluminum, galvanized iron, or zinc containers must not be used because of the possibility of forming hydrogen gas that upon expansion could possibly burst the containers. Silicates react slowly with glass, absorb carbon dioxide from the air, and lose water by evaporation. These factors cause a considerable change in voscosity with time; therefore, a discard date accompanies those samples stored in glass containers (5).

The colloidal nature of the common liquid silicates is indicated by the fact that their boiling point is only slightly higher and their freezing point slightly lower than those of water. The 41-B silicate freezes at only 1 or 2 deg below the point that water freezes. In freezing there is a rapid growth of long crystalline masses that contain a high content of water. These crystals tend to float when the solution is warmed, and it often happens that drums or tank cars of sodium silicate when frozen and then thawed will contain highly concentrated silicate at the bottom and a relatively dilute solution on the top. Vigorous agitation or pumping will provide enough mixing action so that the solution will be identical in all characteristics to the original.

The concentration of solids in the liquid silicates and the $SiO_2:Na_2O$ ratio greatly affect pertinent properties of the materials. As the solids concentration increases, the viscosity and the specific gravity of the solution increase. The more concentrated the solution is, the smaller the coefficient of expansion is. For a given total solids concentration, the solution with the greater alkali ratio has the higher specific gravity. The viscosity, however, shows a pronounced minimum at a ratio of about 1 Na_2O:2 SiO_2 and tends to increase rapidly as the alkalinity increases or decreases from this ratio. In general, the density of the solutions decreases with increasing temperature.

The alkali activity of sodium silicate solutions is affected by several factors. The pH value of the available silicates lies in the range of about 11 to 13. The pH decreases with increasing silica content. As the solids concentration of a given alkali:silica ratio is increased, the pH increases rapidly at low concentration but levels off at concentrations above about 50 percent. The high pH of the solutions is maintained as acids are added until the alkali is almost completely neutralized. The buffer capacity of the solutions increases with increasing proportions of soluble silica. Hence, within limits the silicate solutions will maintain a fairly constant pH despite the addition of acid.

REACTION MECHANISMS

In general, the physical properties of sodium silicates control their binding power. These properties may be altered with the use of physical or chemical or combined methods. The relative amounts of soda or silica in sodium silicate solutions may be changed by altering the basicity of the solution with additions of other chemicals. As a result a silica gel is usually formed. For instance, a sand-sodium silicate mixture is hardened in the ceramics industry as carbon dioxide gas is passed through it (6). Also, the viscosity of a solution may be increased by the application of heat or low pressure until gelation occurs and the silicate is essentially solid (7, 8).

The following discussion of the materials that cause precipitation of silica as silicic acid, the reactions that result in the formation of a complete gel, and the factors that control the character of the gel is based on the authors' interpretation of the information contained in numerous reports, especially those of Terzaghi (9), Vail (4), Iler (10), du Pont (11), and Philadelphia Quartz (5).

Formation of Silica Gel

All sodium silicate is alkaline in reaction. When a silicate solution is mixed with a solution of an acidic material, the mixture becomes opalescent; and, if the silica concentration is above 1 or 2 percent, silicic acid is precipitated and gelation occurs. Solutions with a lower silica concentration form silica sols whose highly hydrated colloidal particles are negatively charged except in moderately concentrated solutions.

Several classes of materials react with sodium silicate to form sols or gels. All acids or acid-forming materials such as chlorine, sulfur dioxide, ammonium salts, bisulfates, sodium silicofluoride, and bicarbonates are examples. Precipitation or coacervation may be produced by various liquids that tend to dehydrate the silicate solution. For instance, alcohol, glycerin, salt brine, and strong ammonia solutions will precipitate concentrated solutions of sodium silicate. Such precipitates are soluble. Organic materials such as casein, rubber latex, cane or beet sugars, and starch as well as certain organic synthetic resins are compatible with sodium silicate and are combined for certain uses such as on adhesives or coatings.

Sodium aluminate, zincate, and plumbate solutions also form gels with soluble silicates. Sodium silicate is precipitated by most salts of the heavy metals such as calcium, magnesium, aluminum, titanium, copper, and lead. These precipitates are believed to contain silicic acid along with metallic silicate. The insoluble metallic silicates have bonding power that has been investigated and employed most extensively in the treatment of building materials.

Character of Silica Gel

When a solution of a polyvalent metal salt is mixed with a solution of a soluble alkali metal silicate, such as sodium silicate, an insoluble metal silicate is usually precipitated as a gelatinous amorphous mass. Essentially all natural silicates are amorphous when precipitated from aqueous solution.

In a crystalline metal silicate, metal ions and silicate ions of uniform size and shape are arranged in a regular manner in a solid lattice or framework characteristic of the ionic sizes and charges. However, most solutions of soluble silicates do not contain silicate ions of uniform size. Instead they are a mixture of polysilicate ions. Even in the case of highly alkaline metasilicates, which are known to contain monomeric silicate ions, lowering of the pH upon addition of a solution of a metal salt causes rapid polymerization to polysilicate ions. Because the polysilicate ions are not of uniform size, they cannot arrange themselves along with the metal ions into a regular crystal lattice. Thus, when polysilicate ions combine with metal ions, the resulting insoluble precipitate is always amorphous.

Polymerization of basic metal ions is another factor that may interfere with crystallization. When the metal salt is mixed with silicate, the increase in pH in the environment of the metal ions at some point causes the formation of polymeric basic metal ions or colloidal metal hydroxide, which likewise cannot fit into the silicate lattice. Thus, precipitation of a metal silicate from aqueous solution at ordinary temperatures does not yield crystalline silicates but instead usually results in a colloidal mixture. This may be visualized as an adsorption of metal ions on gelatinous silica or as a mutual coagulation of the positively charged colloidal metal hydroxide and negatively charged colloidal silica. In dilute solutions colloidal metal silicates are precipitated at a slightly lower pH than metal hydroxide alone would be precipitated. This suggests that no definite compounds are formed and that the hydroxides are precipitated along with silica by mutual coagulation. Thus "colloidal silicates" may vary from homogeneous colloidal aggregates of extremely small units of polysilicic acid and metal-hydroxide to heterogeneous masses in which either silica or the metal hydroxide is present as discrete colloidal units, held together by the other component.

Precipitated metal silicates are difficult to characterize physically because they are usually not homogeneous in nature. When one solution is added to the other, at the point of mixing there is no control of the ratios of silicate and metal ions that are brought together; yet the composition of the precipitate as it is formed depends on the ratio of the components in the reaction medium. Homogeneous gels can be formed by keeping the pH low until all the metal salt and silicate solution are mixed. However, when the pH is raised to induce gelation, the gel is still not truly homogeneous. The silica is polymerizing continuously in such a mixture, and the proportion of metal ions bound to the silica particles will vary depending on the degree of polymerization.

Structurally, the silica gel consists of 3-dimensional networks made up of chains of micelles or particles that may range in size from polysilicic acid units each containing only a few silicon atoms to colloidal SiO_2 particles containing thousands of silicon atoms. The micelles are bonded together into chains, probably by hydrogen bonding. However, in view of the fact that the surface of particles of colloidal silica is composed of SiOH groups, it seems very likely that, when these particles come together, consolidation between these groups occurs and Si-O-Si bonds are formed between the particles.

From the preceding discussion it should be apparent that soluble sodium silicate is an unstable material and that mixing it with almost any other inorganic material is likely to cause the formation of a silica gel. However, the properties of the gel, such as strength, durability, and permeability, may vary greatly depending on factors such as concentration of the solution, $Na_2O:SiO_2$ ratio, temperature, and the kind of salts, acids, or bases with which it reacts. Further discussions of the reaction mechanisms will therefore be undertaken in conjunction with the sections relating to its use in soil stabilization.

USES OF SODIUM SILICATE IN SOIL TREATMENT

Sodium silicate has been used to improve building materials for more than a century (12). Johann van Fuchs, a pioneer in the commercial development of soluble silicates, proposed in the early 19th century that soluble silicates could be used as a hardening agent for artificial stone. He also recognized the reaction of silicates with calcium compounds. During the 19th century several patents were issued for the manufacture of artificial stones from solutions of silicates mixed with solutions of calcium compounds.

As early as 1910, sodium silicate is mentioned as being used as a dust palliative. The application of a solution of sodium silicate followed by an application of a soluble aluminum or calcium salt causes the precipitation of an aluminum or calcium silicate in voids of the road stone. This led to the development of macadam roads in which the crushed stone was held in place by a cementitious matrix formed by a slurry of finely divided limestone and sodium silicate.

Around 1915, Albert Francois, a French engineer, found that the effectiveness of cement grout could be increased by preceding the grout with an injection of alum followed by sodium silicate. Apparently the resulting slippery gel coats the granular surfaces and causes greater penetration of the cement grout.

Later, a Dutch engineer, Hugo Joosten, injected sodium silicate as a grout into deep foundations and followed that with an injection of calcium chloride. The combination of these 2 chemicals forms an insoluble gel that fills the voids and prevents seepage of water or the continuance of any settlement. This injection method (with modifications) has been extensively used in foundation work and in controlling seepage, but the method is not practical in solidifying the soil within 3 ft of the surface (13). Another injection method, in which the precipitation reaction is much slower, uses a solution containing sodium silicate and sodium bicarbonate. The gel formed is not the insoluble calcium silicate and gradually deteriorates; it does not have the strength of the calcium silicate gel.

The earliest American papers on the use of sodium silicates as soil stabilizing agents appeared in 1945-46 and indicated that the method showed promise (14, 15).

INJECTION METHODS OF SOIL SOLIDIFICATION

Of the nearly 100 references given in this paper, approximately 40 refer to the use of sodium silicate for the solidification of granular soils, usually at several tens of feet below the surface. Approximately 25 of these refer to the Joosten 2-step injection process or a modification of it and the remainder to the single injection process. One of the most useful bibliographies on chemical grouting was prepared by the ASCE Task Committee of the Soil Mechanics and Foundations Division (16).

The basic ingredient of nearly all silicate injection processes is a solution of sodium silicate in water, known as "water glass." This solution contains both free sodium hydroxide and colloidal silicic acid. The addition of salts or acids can cause the silicate solution to form a gel, for example,

$$\mathrm{Na}_{(2n-4)}(\mathrm{SiO}_n) + \mathrm{HCl} \rightarrow \mathrm{SiO}_n(\mathrm{OH})_{4-2n} + \mathrm{NaCl}$$

If the acid is replaced by an acid salt, such as calcium or magnesium chloride, a complex metal hydroxide silica gel is formed.

As previously discussed, very little is known about the structure of silica gel. It gives very little or no pattern when exposed to X-rays and is, therefore, considered amorphous. It has a high specific surface (600 m^2/gram, of the order of magnitude of montmorillonite) and is a very powerful and selective absorbent. On aging the gel shrinks (syneresis), becomes opalescent, and cracks; it dissolves readily in caustic. The life of silicate stabilization when the treated soil is exposed to air or to basic groundwater may be limited (16).

Numerous patents exist concerning variations of the silicate injection process, but the almost complete lack of published experimental and field data prevents even the beginning of the comparison of the many reactions. A single injection treatment has apparently a major advantage over the 2-shot, namely, the extent to which the separate solutions mix is never known. Also, the zone of mixing may in some cases be limited to the end of the injection pipe (16, 17).

On the other hand, the single injection silicate process is subject to many uncertainties. One of the most important of these is selecting the correct proportions in the mix to ensure that gel formation does not occur as the liquid is pumped into the soil but that it does set in a reasonable length of time after pumping is stopped. The appropriate setting time varies with the nature of the soil being injected. For highly pervious material containing groundwater in relatively rapid motion, a setting time as short as 15 min may be desirable. In moderately permeable sand, on the other hand, a longer setting time may be required in order to allow the solution to penetrate the ground to the desired distance from the injection hole (9).

Permanence

In general, 3 processes may lead to the deterioration of silica gel contained in the voids of a soil. These are shrinkage due to spontaneous expulsion of water (syneresis), shrinkage due to desiccation, and solution or erosion by groundwater. All types of silica gels undergo syneresis, and laboratory tests indicate the water loss may range from 20 to 60 percent by weight within 2 months. For some gels, shrinkage to nearly 25 percent of the original volume is possible depending on the type of agent and the concentration of silicate in the original solution. If a 1-shot sand-gel is subject to desiccation, it loses its cohesion and its permeability may increase almost to the original permeability of the sand (9).

Laboratory permeability tests on impregnated uniform sand were conducted with several types of gel under a high hydraulic gradient. All systems showed an initial permeability of 10^{-7} to 10^{-6} cm/sec. With some types of gel the rate of discharge increased several hundredfold in 24 hours, and in most cases the rate of discharge increased with time of percolation. Although admittedly the tests were much more rigorous than field conditions, it can be concluded that gel-impregnated sand subject to a hydraulic gradient (as in a grout curtain) would deteriorate with time because of solution or erosion by groundwater, unless some other process intervened to prevent percolation through the grouted zone (9).

Compressive Strength

Laboratory tests of impregnated sand indicated that acid gels with 16.5 percent water glass gave strengths 5 to 8 times that of an ultrabasic gel [produced with $Ca(OH)_2$] containing 45 percent water glass. Strength of the acid gel-sand ranged from about 1.5 to 2.5 ton/sq ft (approximately 25 to 28 psi) at times ranging up to 5 days (9). These values are only a fraction of those reported for samples solidified by the 2-step injection method, which gave strengths ranging from about 7 to 26 ton/sq ft (approximately 100 to 350 psi) shortly after injection (13).

Limitations

Injection processes whether 1- or 2-stage are limited to soils in which the voids are large enough to allow penetration of the mass under reasonable pressures. Generally, this means that the lower limit is a fine sand substantially free of silt and clay; however, gravels with appreciable fines could probably be successfully injected.

Other environmental factors may influence the setting time of the gel and thus the success of the treatment. When the solution is injected into saturated sand, some dilution is inevitable; and, other factors being equal, dilution may lead to setting times 3 to 20 times those for undiluted materials. On the other hand, injection into dry sand may shorten the setting time as much as 30 percent. Soils or groundwater with appreciable salt contents, even $CaCO_3$, may also significantly decrease the setting time for silica gels, whereas there is some evidence that the presence of organic matter may prevent completely the setting of some gels (9).

Mainfort (17) points out that when 2-step processes are used there is a tendency for the second chemical to push the first chemical ahead of it and to prevent adequate mixing or at least mixing in the proper proportions.

SODIUM SILICATE AS A SOIL STABILIZER

Since 1945, various researchers have reported on the effectiveness of sodium silicate as a soil-stabilizing admixture. Sometimes the silicate was used alone and sometimes in combination with various other chemicals. Sherwood concluded that sodium silicate, when used alone, is suitable only for the stabilization of sandy soils in a moderate climate (18). When the silicate is combined with soil and other chemicals, the results are mixed. Furthermore, there exist few or no field data to confirm the apparently beneficial results that have been obtained in the laboratory.

Soil-Sodium Silicate Reaction Mechanism

Because the process of silicate gel formation is not completely understood, it is impossible to state exactly how sodium silicate reacts with soils. Laws and Page (15) found that sodium silicate reacted with various clay minerals commonly present in soil, but they did not make a detailed study of the nature or possible mechanism of the reactions.

Sodium silicate can be used in soil stabilization mainly because it reacts with soluble calcium salts in water solutions to form insoluble gelatinous calcium silicates. That hydrated calcium silicates are cementing agents has been fairly well established. The exact nature of the bonding action of hydrated calcium silicates has been the subject of much investigation, but it has not been fully explained (12). Iler (10) believes that the silicate is in the form of polymeric chains of silicon-oxygen linkages, with sodium distributed along the chain on the oxygen. The addition of a polyvalent ion, such as Ca^{++} or Mg^{++}, results in the replacement of the Na^{+} by the divalent ion and permits bonding together of 2 chains to form an insoluble cross-linked gel structure. The effect of the addition of the silicate to appropriate soils is to increase water-stable aggregation and resistance to crushing and to decrease the plasticity index and swelling (15). On the basis of this evidence Wooltorton (19) postulates that the process involved is that of anionic exchange, resulting in the formation of insoluble silicate sheaths around the soil particles, accompanied by cementation effects. Murray (20) simply indicates that calcium silicate is precipitated as a continuous matrix in which a strong and rigid ionic polar bond is formed between the soil and stabilizer.

These concepts indicate that the important variables might be expected to be (a) the quantity of polymeric silica, which determines the amount of gel; (b) the frequency of sodium atoms along the chain, which determines the amount of cross-linking possible; and (c) the concentration of the precipitant, which determines the extent of cross-linking. In addition, it is believed that the elevation of the pH resulting from the presence of the sodium silicate solution would generate additional silicate by the solubilization of the already hydrated surface silica in the soil. Thus, the soil would be contributing additional stabilizing material to that added initially (21).

This might help to explain the highly favorable reaction that is usually obtained with silica sands. On the other hand, Laws and Page (15) found that, although clays of low activity, such as kaolinite, reacted favorably with sodium silicate, the stability of highly active clays, such as montmorillonite, might actually be impaired by the addition of the silicate. This lack of favorable reaction with many fine-grained soils was confirmed by Mainfort (14, 22) and to some extent by the studies at the Massachusetts Institute of Technology (21, 23).

Properties of Stabilized Mixtures

Foundry Molding Sands—For many years sodium silicate, often mixed with various other materials both organic and inorganic, has been used for the bonding of foundry sands (4). Such materials must retain strength at relatively high temperatures but must also be porous enough to permit the escape of air and other gases during the casting processes. Several references are included as representative of investigations that have been carried out on the factors that determine the properties of the molded sand. Three of those refer to the process of hardening the silicate by treatment with carbon dioxide (6, 7, 24). Because the molds are usually baked at temperatures of more than 400 F, the results of these investigations are not directly applicable to soil stabilization work. Morey and Lange (24), however, do note that an increase in the $Na_2O:SiO_2$ ratio caused a decrease in the strength of the material, and an increase in the percentage of sodium silicate (within limits) caused an increase in strength. (The amount of silicate usually ranges from approximately 2 to 10 percent of the weight of the sand.) It was also observed that an addition of 1 percent bentonite clay to the sodium silicate-sand mixture caused an increase in strength of about 10 percent more than that of the silicate mixture alone.

Jones and Kandic (8) discuss an investigation of a process for hardening silicate-bonded sand at low pressures. The silica sand was mixed with 2 to 6 percent sodium silicate and then hardened by pumping off 10 to 30 percent of the moisture at pressures below 10-mm Hg. Depending on the silicate composition and variations in time, pressure, and temperature, compressive strengths up to 800 psi were obtained. Although such a process could be impractical for field soil stabilization, it does indicate that under certain conditions high strengths may be obtained by treatments with silicate alone followed by dehydration.

Strength—Mainfort (14) reported on the results of a number of tests on sand, sandy loam, silt loam, and silty clay treated with various amounts of sodium silicate alone and in combination with other chemicals. The oven-dry strength of silicate-treated sand was greatly increased by 8 percent sodium silicate applied as a 40 percent solution. The results with the fine-grained soils were not so spectacular but were "satisfactory." The treated specimens, however, were not resistant to immersion for 24 hours, and various additive combinations were tried to improve the resistance to wetting. Calcium chloride solution appeared to give the best strength results when the silicate-stabilized specimens were submerged in it for 15 min, then removed, and air-dried. These specimens, however, also showed low resistance to submergence. Susbequently, treatment with 4 percent sodium silicate and 2 percent sodium aluminate gave relatively high strengths for sands and also improved resistance to wetting.

Laws and Page (15) in tests with pure clay minerals and various soils also noted improvements in strength when a fine sandy loam was treated with various amounts of sodium silicate up to 3.5 percent. They stress the effect of the silicate treatment in aggregating even some fine-grained soils but conclude that the most beneficial results were obtained with coarse-grained soils.

Various authors mention the improvement in strength to be achieved by treating soils with sodium silicate (with and without other additives) but present few or no experimental data (25, 20, 26, 19, 27, 28, 18).

Dogra and Uppal (29) report that amounts of sodium silicate as low as 2.5 percent with sands and sandy soils produced strengths (1,500 psi) that exceeded those produced with the same soils at 10 percent cement (1,300 psi). They also found that long curing periods (6 months) at room conditions increased compressive strengths of sand-clay mixtures.

The studies of chemical soil stabilization conducted at M. I. T. (21) initially indicated that magnesium carbonate in combination with sodium silicate was one of the most effective tried. A 5 percent silicate treatment of Massachusetts clayey silt increased its strength more than fourfold to 400 psi. The optimum concentration of $MgCO_3$ was 2 percent. Strength was found to increase with silica content, and the best results were found with sodium concentrations of 0.04 to 0.06 equivalents per 100 g of soil in combination with 0.75 equivalents of $MgCO_3$ per equivalent of sodium. Various grades of soluble silicate with $Na_2O:SiO_2$ ratios varying from 1:1.58 to 1:3.33 were used. Results obtained with solid silicates were generally not so satisfactory as with liquid silicates.

Later studies at M. I. T. (23) confirmed the effectiveness of the sodium silicate-magnesium carbonate combination with the Massachusetts clayey silt (L_W = 20 percent, PI = 6 percent) with wet strengths up to 600 psi at 6.4 percent total additive. The wet strength of New Hampshire silt (L_W = 28 percent, PI = 8 percent) was increased to 365 psi with 8.5 percent total additive and that of Vicksburg loess (L_W = 41 percent, PI = 15 percent) to 235 psi at 9.1 percent total additive. Attempts to stabilize Vicksburg buckshot clay (L_W = 65 percent, PI = 37 percent) were unsuccessful.

Winterkorn (30) reports on the field tests conducted by the U. S. Navy on beach sand stabilization with 1 to 3 percent sodium silicate by weight sprayed as a 40 percent sodium silicate solution. Although enough silicate was added to stabilize a 4- to 6-in. depth and the sand was compacted with rollers only, a hard brittle crust approximately ¼ in. thick was formed. This crust could not withstand the weight of a jeep or a ¾-ton truck used as test vehicles.

Moisture-Density Relations—Very little information is available on the effect of sodium silicate (with or without precipitants) on the density of compacted soils. The M. I. T. studies (21, 23) were performed on samples prepared in a Harvard miniature mold by 2-end static compaction at about 900-psi pressure. Initial tests on Massachusetts clayey silt stabilized with 5 percent sodium silicate showed variations in dry density of from about 116 to 134 lb/ft^3 depending on the type and amount of precipitant used; MgO gave the highest as-molded densities, with $MgCO_3$ about 3 lb/ft^3 less. Later tests with New Hampshire silt and Vicksburg loess indicated that the dry density after 1 day of humid curing varied with variations in silica, sodium, or magnesium content. The highest strengths were usually associated with the specimens having the highest densities. In the case of New Hampshire silt the densities varied from about 102 to 115 lb/ft^3, whereas the Vicksburg loess specimens varied from 100 to 107 lb/ft^3.

Ruff and Davidson (12) in their tests on Kansan till found that sodium sesquisilicate pentahydrate when added in the dry form had practically no effect on the as-molded maximum dry density in amounts varying from 2 to 6 percent. Increasing the silicate contents did, however, result in a significant increase in the optimum moisture content.

Durability—Although the preceding discussion has indicated that sodium silicate alone or in combination with certain precipitants may increase the stability of coarse-grained or relatively nonplastic fine-grained soils, there is considerable doubt about the permanence of the improvement when the soils are exposed to normal climatic variations.

Mainfort (31) states, "Good bonding of sandy soils has been attained with treatments of 6 percent or more of sodium silicate, but such treatments will not withstand attacks by moisture. Very little effectiveness was obtained by the application of sodium silicate to fine-grained soils. Earlier studies indicated that the addition of sodium aluminate greatly increased the resistance of the soil-silicate sample to moisture attack but later tests, using a variety of soils and more severe exposure tests, have clearly shown this combination to be unsuitable as a soil stabilizing agent." He continues, "Unless a very successful additive should be found, sodium silicate cannot be considered a suitable soil stabilizing agent for paving purposes."

Dogra and Uppal (29) found that sodium silicate-treated specimens of sandy soils were more resistant to abrasion than cement-sand. They also conducted durability tests in which the specimens were immersed in water for 5 hours and then dried at 70 C for 42 hours. Specimens containing 10 percent clay and 90 percent sand when treated with 1 percent sodium silicate withstood 12 cycles of such a test and showed weight losses of only 3.5 to 4.0 percent. Higher percentages of sodium silicate in this mixture led to rapid disintegration upon immersion. Also specimens with higher percentages of clay did not give encouraging results. When pure sand specimens were tested, some improvement was noted even with 1 percent sodium silicate. At a sodium silicate content of 6 percent the pure sand withstood 12 wet-dry cycles with a 6 percent weight loss. Smaller amounts of silicate led to failure after 6 cycles, and larger amounts gave higher weight losses per cycle. They also showed that when specimens were aged for long periods before testing they developed greater resistance to the softening action of water.

The studies at M. I. T. (21) with Massachusetts clayey silt indicated that the stabilization reaction was not sufficiently rapid to provide stability upon water immersion immediately after compaction. However, after 24 hours of curing at 100 percent relative humidity and then 24 hours of immersion, the specimens retained at least 65 percent of the 24-hour, humid-cure strength. The most severe test reported to be performed on the M. I. T. specimens involved 24 hours of humid curing, followed by 24 hours of air-drying, followed by 24 hours of immersion. In these tests the Massachusetts clayey silt treated with 5 percent sodium silicate and varying percentages of magnesium carbonate retained about 40 to 80 percent of its 24-hour, humid-cure strength after immersion.

Tests conducted at M. I. T. on New Hampshire silt and Vicksburg loess (23) showed that certain specimens treated with sodium silicate and magnesium carbonate exhibited considerable strength increases after 7 days of humid curing and 24 hours of immersion as compared with 24 hours of humid curing. Optimum strengths and additive levels were 365 psi at 8.5 percent total additive for the silt and 235 psi at 9.1 percent total additive for the loess. No further tests for durability were conducted. Although the durability tests cited are meager, it appears that they do justify the conclusion of Sherwood (18) that sodium silicate as a chemical, when used alone, is suitable only for the stabilization of sandy soils in a moderate climate.

The long-time effectiveness of sodium silicate in combination with other chemicals when used as a stabilizing agent with nonplastic soils is unknown, but the information currently available does not inspire confidence in such treatments. Possibly the best summation of the utility of soil stabilization with sodium silicate in combination with magnesium carbonate and magnesium oxide precipitants is given by Kozan and Fenwick (32). Although only Vicksburg loess and Vicksburg buckshot (alluvial) clay were tested, these 2 soils represent a rather wide range of fine-grained soils that would be encountered in engineering practice. The form of the curing and testing was similar to that of tests conducted in the M. I. T. laboratories. They conclude, "... these materials did not produce any major strength improvement." On this basis they state, "The formulations containing silicate compounds as the primary stabilizing agents were determined to be incapable of meeting the requirements established for the specified military road and airfield functions. On the basis of these findings, further investigation of these formulations for military use is not justified." Although the military requirements are more stringent than those for normal civilian highway use, it is apparent that no silicate-chemical combination has been found to successfully stabilize heavy clay soils. Furthermore, the amounts of additive required for slightly plastic, fine-grained soils approximate those of materials, such as portland cement, that are more readily available, easier to handle and mix (especially under field conditions), and are normally less expensive.

DUSTPROOFING AND WATERPROOFING

Soils

Although sodium silicate, with or without chemical precipitants, has not proved to be an ideal additive for soil stabilization, there is some evidence that it might have value as a dustproofer or waterproofer. The Waterways Experiment Station (33) reported on a test conducted by the U. S. Army Corps of Engineers in the arid southwestern part of the United States. A fine-grained clayey silt soil, ML-CL, was treated to a depth of 1½ in. by penetration with 30 percent solution of sodium silicate. The surface withstood 200 coverages by a 2½-ton military cargo truck without evidence of stress, cracking, or surface abrasion. An attempt to stabilize a well-graded sand, SW, by similar means was not satisfactory unless the loose sand was first compacted and the penetration treatment applied subsequently.

The apparent success of this classified project led to further research on sodium silicate and other materials at the Waterways Experiment Station (34). It is stated that the qualifications of a waterproofing and dustproofing material are (a) it should be effective in a relatively thin layer, preferably not exceeding 6 in.; (b) it should be effective when applied in quantities of 5 percent (by weight of dry soil) or less; (c) it should retain its effectiveness under the expected traffic for at least 1 full cycle of seasonal change without additional treatment; and (d) although advantageous but not essential, it should have the capability of being applied as a penetration treatment.

A total of 13 materials were evaluated on a test section of Vicksburg loess (L_w = 38, PI = 16, 97 percent < No. 200, 25 percent < 0.002 mm). Each material was incorporated with the soil by off-site mixing and placed in individual panels 12 by 6 ft to a 3-in. depth and compacted at optimum moisture content. A 16-in. untreated base (CBR = 36) was achieved by recompacting excavated loess in 4-in. lifts. Traffic tests were performed by a military truck loaded to produce 16,000 lb on each of 2 rear axles. Each test consisted of 40 passes at 5 to 7 mph with the truck wheels maintained on a path along the centerline of the panels. Seven such tests were conducted during a 13-month period. Data were collected to determine the resistance to traffic abrasion and amount of dust produced and the effectiveness of the surface in protecting the base from ingress of water.

For comparative purposes, the results of the sodium silicate treatment (14.5 percent of a 30 percent sodium silicate solution = 4.35 percent by dry weight of soil) will be compared to untreated control sections and a section treated with MC-O cutback asphalt (10 percent of 50 percent asphalt content = 5.0 percent by dry weight of soil). The results of the dustproofing tests expressed as weight in grams of dry material collected from an area 4 by 2.5 ft along the wheelpath in each panel are as follows:

Test	Control	NaSi	MC-O
1	1,148	905	185
2	3,481	1,237	284
3	2,783	2,012	262
4	3,058	2,730	368
5	1,969	2,242	606
6	7,824	5,479	722
7	3,970	3,164	516

The sodium silicate treatment produced some beneficial effect (as compared with the control) but was much less effective than MC-O, which rated 2 in effectiveness. Actually, the silicate treatment was rated 9 of the 13 materials tested.

The waterproofing characteristics of the treatments were based on the maximum moisture contents at 4- and 12-in. depths in the compacted base. The base was placed at an initial water content of 10.8 percent, and good waterproofers were presumed to maintain this value or less throughout the tests. The base under the sodium silicate panel approached 20 percent water content throughout the tests and approximated that of the untreated panels. Of the 13 materials tested sodium silicate rated 10 in effectiveness; the MC-O treatment rated 1. Laboratory capillary rise rewet tests confirmed the ineffectiveness of sodium silicate as a waterproofer with water contents (25 percent) after 4 days of soaking approximating those of untreated specimens.

Subsequent laboratory tests conducted by the Waterways Experiment Station (35) on various sodium silicate systems (silicate plus magnesium carbonate or oxide precipitants) showed that they were totally ineffective as waterproofers for Vicksburg loess and Vicksburg buckshot clay.

Concrete

It has been stated (36) that treatment of concrete with sodium silicate is the cheapest, simplest, and most effective way of overcoming the concrete dusting problem. The proper penetration of the silicate into a porous concrete surface is obtained by 3 applications that should be made at about 24-hour intervals. For best results a silicate with a Na_2O:SiO_2 ratio of about 1:3.3 and with a density of 41 to 43 B should be used. Various dilutions are suggested for each of the 3 applications with the estimate that 1 gal of silicate is sufficient to harden 200 to 350 sq ft of concrete surface. The article indicates a beneficial reaction between cement and sodium silicate, and that will be discussed more fully in the next section.

SODIUM SILICATE AS A SECONDARY ADDITIVE

There are soils for which the requirements of common stabilizers, such as portland cement, lime, and lime-fly ash, are so high that it may be uneconomical to use them. Furthermore, these stabilizers when required in large quantities find limited military use because of the logistical problems involved. For these reasons some research has been carried out to determine whether minor amounts of other additives can reduce the requirements for the major stabilizer and still produce satisfactorily stabilized soil.

Soil-Cement

Mechanism of Reaction—Lambe, Michaels, and Moh (37) and Moh (38) present the following hypothesis to explain the beneficial effects obtained by the treatment of soil-cement with various sodium salts and NaOH. The hypothesis is based on the results of tests carried on in the M. I. T. Soil Stabilization Laboratory.

Most fine-grained soils contain large quantities of crystalline silica, feldspar, or colloidal clay minerals in a high degree of subdivision. These compounds are inherently acidic in nature and a small, but important, fraction of the silica and aluminosilicates present in soils is in a highly hydrous state on the particle surfaces. When exposed to strong alkalis, soils of this type function as powerful buffers. The surface silica and alumina react rapidly with the alkali to form silicate and aluminate salts, thereby reducing pH. The finer the particle size of the soil is, the greater will be the proportion of reactive silica and alumina available, and thus the more rapid and more extensive will be its buffering or alkali-consuming capacity.

When cement is admixed with soil and water, hydration of the cement leads initially to the formation of lime, calcium silicate, and calcium aluminate. However, reaction of the lime with reactive silica and alumina from the soil leads to formation of additional calcium silicate and aluminate, reduction of pH, and subsequent hydrolysis of the more basic calcium silicate and aluminate. The ultimate reaction products of soil and cement are summarized to be hydrous alumina and a calcium silicate gel of rather low

Ca:Si ratio. It is to be noted that, in soil-cement, nearly all the reactive calcium present initially in the cement is eventually available for production of cementitious silicate gel; hence, the quantity of cementitious material available for bonding in soil-cement is inherently greater than that in neat cement. Because, however, any lime that reacts with soil silica (or alumina) is immediately immobilized as insoluble silicate, and because cement particles are rather sparsely distributed between soil particles in typical soil-cement, the formation of cementitious silicate gel can be expected to be confined to a rather thin zone around each cement particle. Only when the cement concentration is high enough to permit these zones of gel formation to overlap will the soil become adequately stabilized.

When a sodium compound is added to soil-cement, the calcium ion concentration in solution is greatly depressed either by the common ion effect (with NaOH) or by the causticization reaction,

$$Na_2X + Ca(OH)_2 \rightarrow CaX + 2NaOH$$

Simultaneously, the pH is significantly elevated. Suppression of the calcium concentration retards precipitation of insoluble calcium silicate gel, while elevation of pH accelerates attack of soil silica or alumina or both and formation of soluble alkali silicate or aluminate or both. Because the alkali silicate is free to diffuse through the pore fluid, there is opportunity for rather uniform distribution of silicate throughout the soil mass. Eventually, however, neutralization of the free alkali by reaction with the soil reduces the pH and permits the calcium ion to go into solution. Reaction of calcium with the uniformly distributed alkali silicate or aluminate in the pore fluid results in formation of mixed calcium-sodium silicate and gelation. Formation of an increased volume of well-distributed, hydrous silicate gel can thus account for the generally beneficial influence of alkali metal compounds on soil-cement.

In terms of this mechanism, it becomes rather simple to explain why different sodium compounds differ so widely in their effects on various soils. In coarse-grained soils, the amount of reactive silica present is small, and thus the amount of cementitious silicate gel that can be contributed by the soil is limited. If gel-forming material can be provided from an external source, however, the reactive silica deficiency of the soil can be compensated for; hence, sodium silicate or aluminate is more effective in sands than other sodium compounds. With heavy clays, on the other hand, there is present a large amount of reactive silica—so much in fact that neutralization of alkali by such a soil occurs with great rapidity. When the generation of free caustic soda must result from reaction with lime from the cement, the rate of generation of caustic is likely to be no faster than its consumption by reaction with the soil so that the pH never becomes greatly elevated. Because of this, the calcium solubility is but little depressed, and calcium silicate formation occurs promptly; hence, uniform distribution of cementitious gel will not occur. If caustic soda is used as the additive, however, the pH is immediately raised to a high level and the calcium concentration correspondingly reduced; rapid formation of sodium silicate and delayed gelation by calcium will thus contribute to more extensive and uniform cementation.

With soils of intermediate fineness, such as silts, it is likely that the supply of reactive silica is adequate and that the rate of reaction of the soil with alkali is sufficiently slow that the reaction with lime can proceed virtually to completion before the free caustic is consumed. Under these circumstances, it is of relatively little importance which sodium compound is employed, at least in terms of ultimate strength development.

Strength—Several reports (37, 18, 38, 39) indicate that sodium silicate (especially metasilicate) is most effective in increasing the strength of cement-stabilized clean sandy soils. Tests by Lambe, Michaels, and Moh (37) on a Wisconsin sand (approximately 20 percent < No. 200) stabilized with 10 percent cement showed an increase in wet compressive strength of 40 to 290 psi at 1 day and 230 to 440 psi at 7 days when 1 percent sodium metasilicate was added to the mixture. Of all the sodium additives tested, the silicate was most effective. Similar results were reported from tests on New Hampshire silt (L_W = 28 percent, PI = 8 percent, pass 200 > 95 percent) and Vicksburg loess (L_w = 41 percent, PI = 15 percent, pass 200 > 90 percent) when stabilized with 5 percent cement and 1 percent sodium metasilicate. The 7-day strengths with the silicate additive exceeded those for cement alone by 150 to 250 psi (increases of 100 to 200 percent). Further tests with the 5 percent cement-stabilized New Hampshire silt indicated that increasing the amount of silicate additive from about 0.6 to 2.7 percent had little effect on wet compressive strengths up to 28 days. At the 1 percent additive concentration the sodium metasilicate plus 5 percent cement produced strengths comparable to those obtained with the silt stabilized by 10 percent cement alone. Several cement-stabilized clay soils tested showed little or no improvement with silicate additive. Although some clays having high carbonate contents did show some improvement with the metasilicate, other sodium additives were somewhat more effective. It was concluded, however, that salts of weak acids are not effective as additives to cement-stabilized heavy clay soils.

Moh, Lambe, and Michaels (39) report on a series of tests with 5 percent cement-stabilized New Hampshire silt in which various sodium silicates were tested in additive ratios of 0.54 to 2.80 percent. It was concluded that there is an optimum $Na_2O:SiO_2$ ratio of about 1:2 at the concentrations employed. At a given sodium concentration, higher ratios have higher early strength but lower ultimate strength. On the other hand, lower ratios (higher silica contents) tended to retard the cementing process. Generally, the other silicates tested gave even higher 28-day strengths than the metasilicate at equivalent additive concentrations.

Durability—Studies of the durability of soil-cement with silicate are few. In a discussion of some of the M. I. T. reports, Norling and Packard (40) briefly review some investigations carried out at the Portland Cement Association laboratories. The Massachusetts clayey silt was subjected to the standard soil-cement freeze-thaw test at cement contents of 6 and 9 percent, with and without 1 percent sodium metasilicate as an additive. Soil-cement loss at 6 percent cement content was 32 percent with no additive and 7 percent with silicate; at 9 percent cement content the losses were approximately equal. It was concluded that the cement requirements for an allowable loss, not exceeding 10 percent, would be 8.5 percent for cement alone and less than 6 percent with the silicate additive. Although no tests with the silicate were made with New Hampshire silt, the results with other sodium additives indicated that sodium metasilicate would probably reduce the cement requirement 3 to 4 percent.

Moh, Lambe, and Michaels (39) also studied the effect of various additives on soil-cement exposed to sulfate attack. The soil-cement specimens were prepared as for the standard strength tests and then immersed in saturated calcium sulfate solution. Comparisons were made with companion specimens immersed in distilled water. Tests on clean Wisconsin sand (with 7 percent cement) showed that the sodium metasilicate additive greatly delayed or reduced the deteriorating effect of the sulfate. After 1 year, the strength of the metasilicate specimens immersed in sulfate solution was still higher than the early strength of the untreated soil-cement and exceeded the strength of the untreated specimens subjected to equal treatment by about 200 percent.

Soil-Lime

Stabilization of soils with lime is thought to proceed by a mechanism similar in some respects to that of soil-cement. The high pH caused by the lime solubilizes part of the alumino-silicates in the soil, which in turn react with the calcium ions giving a cementitious calcium silicate (and perhaps aluminate) gel. Obviously, quicklime or hydrated lime will react with sodium silicate to produce gels. It might, therefore, be anticipated that the addition of sodium silicate to a lime-stabilized soil would provide more reactive silicate and thus speed up the cementing reaction.

Experience has shown that lime is especially attractive economically as a stabilizer for clay soils where cement requirements are high. Although sodium silicate alone or in combination with most precipitants is not useful for stabilizing clays, there is some reason to believe that as an additive to lime it could be beneficial.

Moisture-Density Relations—Practically all of the work on the use of silicates as additives to lime-stabilized soil was carried on at the Iowa State University Engineering

Experiment Station and has been reported by Ruff and Davidson (12). The soil tested was a plastic, calcareous, Kansan till ($L_w = 39$, PI = 23, < 0.005 mm = 40 percent). The predominant clay mineral is montmorillonite. Calcitic hydrated lime (about 70 percent available CaO) was used in all the tests in combination with 5 different powdered sodium silicates. These were 3 metasilicates in various stages of hydration (Na_2SiO_3, $Na_2SiO_3 \cdot 5H_2O$, and $Na_2SiO_3 \cdot 9H_2O$), a concentrated orthosilicate (Na_4SiO_4), and the sequisilicate pentahydrate ($Na_3HSiO_4 \cdot 5H_2O$).

Because it was recognized that in the field some time would be required for mixing, a study of the effect of aging prior to compaction was undertaken. Times used were 9 min and 1, 2, 3, and 5 hours between final mixing and specimen molding. It was found that the dry densities and optimum water contents of the mixtures varied depending on the type of silicate used and the aging times.

In all of the soil-lime-sodium silicate mixtures, the highest dry densities were obtained at aging times near zero. As the aging time increased, the dry density of the specimens molded from all the mixtures decreased. It was also observed that the molding moisture content influenced dry density, but that the molding moisture content that gave the highest dry density at the shortest aging time did not usually give the highest dry density at the longest aging time.

One explanation for the dependence of maximum dry density and optimum moisture content on aging time is related to the chemical reaction between lime and sodium silicate. As water comes in contact with lime and sodium silicate, the chemicals dissolve and then react to form an insoluble calcium silicate gel. Inasmuch as lime is comparatively slowly soluble, the quantity of gel formed gradually increases as the reaction between the lime and sodium silicate continues. The gel absorbs water and at the same time begins to hydrate. As the gel hydrates, it hardens. The longer this process continues, the more gel is converted into a hydrated calcium silicate.

The processes of gel formation and hydration remove free water from the mixture, and as the gel hydrates the character of the soil changes because the hydrated gel is cementing the soil particles together. The loss of the free water lowers that amount available to act as a lubricant during compaction. The effective gradation of the soil is changed and its resistance to the constant compactive force is increased as the soil particles are cemented together. Thus, both the optimum moisture content and the maximum dry density are affected by the introduction of the lime-sodium silicate gel formation and hydration processes, and this effect is greater with increasing aging time.

Density and the amount of decrease in density with aging time are also a function of the type of sodium silicate used. Highest dry densities (106 to 110 lb/ft^3) were obtained with anhydrous sodium metasilicate and sodium orthosilicate, but mixtures with these silicates had the greatest decrease in dry density (4 to 8 percent) with increase in aging time. Sodium metasilicate enneahydrate and sodium metasilicate pentahydrate gave the lowest dry densities (100 to 105 lb/ft^3) and they had a smaller decrease in dry density (1 to 6 percent) with increase in aging time. Intermediate dry densities (102 to 107 lb/ft^3) were obtained with sodium sesquisilicate pentahydrate mixtures, whereas the decrease in dry density with aging time was smallest (1 to 2 percent).

An explanation of this is related to the gel-forming ability of the different sodium silicates. In the previous explanation for the dependence of dry density on aging time, the controlling function of the amount of gel present was cited. A difference in the rate of gel formation by the different sodium silicates would produce different rates of change of dry density with aging time and might also account for the difference in dry densities obtained from the different sodium silicates.

Strength—A principal purpose of the Iowa tests was to determine the effect of aging time on the strength of the lime-stabilized soil when treated with various sodium silicates (12). Preliminary tests were made with the Kansan till combined with 6 percent calcitic lime and varying amounts of the silicates. Although the actual percentage of the silicates varied, the amounts were selected so that the anhydrous portion of each was equivalent to the anhydrous portion of 6 percent sodium metasilicate pentahydrate. The percentage reported, however, is on the basis of the weight of the sodium silicate in its normal state of hydration. The 6 percent metasilicate pentahydrate was selected on the basis of preliminary testing that showed this value to be optimum in stabilizing the Kansan till. For the initial set of tests, an aging time of 5 hours was selected. On the basis of these initial tests, the sodium sesquisilicate pentahydrate was chosen for further testing because it showed the least overall decrease in strength between aging times of from ¼ to 5 hours.

Table 1. Effect of silicates on strength of lime-stabilized Kansan till after 7 days of curing and 1 day of immersion.

Lime (percent)	Type of Silicate	Silicate (percent)	Aging Time (hour)	Strength (psi)
2	$Na_3HSiO_4 \cdot 5H_2O$	0	2	128
2	$Na_3HSiO_4 \cdot 5H_2O$	2	2	120
2	$Na_3HSiO_4 \cdot 5H_2O$	4	2	100
4	$Na_3HSiO_4 \cdot 5H_2O$	0	2	128
4	$Na_3HSiO_4 \cdot 5H_2O$	2	2	205
4	$Na_3HSiO_4 \cdot 5H_2O$	4	2	232
6	$Na_3HSiO_4 \cdot 5H_2O$	0	2	105
6	$Na_3HSiO_4 \cdot 5H_2O$	2	2	242
6	$Na_3HSiO_4 \cdot 5H_2O$	4	2	285
6	$Na_3HSiO_4 \cdot 5H_2O$	5.3	5	267
6	$Na_2SiO_3 \cdot 9H_2O$	8	5	231
6	Na_2SiO_3	3.4	5	225
6	$Na_2SiO_3 \cdot 5H_2O$	6.0	5	240
6	Na_4SiO_4	3.8	5	167

The data from the initial tests are given at the bottom of Table 1 and those from the second set at the top. All of the silicates except the orthosilicate produced 7-day wet strengths that were more than double those of the 6 percent lime alone even though the aging time for the silicate-treated specimens was 5 hours as compared to 2 hours for the lime-soil. The data given at the top of the table also show that except for the 2 percent lime mixture the addition of 2 and 4 percent sodium sesquisilicate resulted in significant increases in 7-day compressive strengths. On the basis of those tests, Ruff and Davidson (12) reached the following conclusions:

1. The immersed strength (25-hour immersion) of Kansan till-lime-sodium silicate specimens decreased with increasing aging time. That decrease was comparatively slight and varied considerably with molding moisture content and type of sodium silicate. One explanation of the decrease in strength with increase in aging time is given along the same line as the explanation for the decrease in dry density. The amount of gel formed by the lime-sodium silicate reaction increases with time; thus as the aging time increases the amount of gel present at the time of molding increases. But the structure of this gel can be broken by compaction, and even though it reforms it does not form as strong a cementing agent.
2. Results of tests on this same soil to determine the optimum mix design for maximum strength indicated a mixture with 6 percent lime and 4 percent sodium sesquisilicate pentahydrate produced optimum results. It was also determined that, for amounts of lime greater than 2 percent, immersed strength increased with increasing sodium silicate concentration to a value of 4 percent.
3. The effect of curing time on Kansan till-lime-sodium sesquisilicate pentahydrate mixtures was such that as curing time increased the strength of the specimens increased. The average rate of strength gain for all 4 mixtures (6 percent lime and 1, 2, 3, and 4 percent sodium sesquisilicate pentahydrate) was greatest in the period of 3 to 7 days. The more sodium silicate added, the higher was the average rate of strength gain for this period. For the period of 7 to 28 days, the addition of 4 percent sodium silicate increased the average rate of strength gain over the 1, 2, and 3 percent sodium silicate mixtures. Further investigation of the reaction products formed by these combinations of Kansan till, lime, and sodium sesquisilicate pentahydrate were indicated before an explanation for this change rate of strength increase could be provided.

Katti and Kulkarni (41) also studied the effect of sodium silicate and other trace chemicals on the strength of lime-stabilized expansive black clay soil. Various percentages of lime were used with 0.5 and 1 percent sodium silicate. The optimum lime content for curing times up to 28 days was found to be 8 percent. At lime concentrations below 8 percent, sodium silicate increased the 7-day strengths roughly 100 percent. Sodium silicate also reduced absorption of water but had no consistent effect on swelling. Regardless of the lime content the addition of the silicate had little effect on the 28-day strength.

Durability—In the Iowa tests the 6 percent lime-stabilized Kansan till specimens with varying amounts of sodium sesquisilicate pentahydrate were subjected to 14 cycles of a modified British freeze-thaw test. A tentative criterion was that the freeze-thaw specimens must have an unconfined compressive strength of at least 250 psi after testing and also have at least 80 percent of the strength of specimens that have been immersed during the same time but not subjected to freezing. The tests showed that increasing the silicate content from 1 to 4 percent resulted in continued increases in final strength. However, the 4 percent treatment gave an average specimen strength of only 218 psi, which amounted to about 50 percent of the strength of the immersed specimens. No basis is given for comparison with specimens containing no silicate.

Soil-Lime-Fly Ash

Studies of the effect of sodium metasilicate on soil-lime-fly ash mixtures were also conducted in Iowa (42). The 5 soils tested were Ottawa standard sand, dune sand, friable loess, alluvial clay, and Kansan gumbotil. Clay contents (< 0.002 mm) were about 0, 3, 14, 61, and 63 percent respectively. The predominant clay mineral in all soils was montmorillonite. The reaction of lime with soil and fly ash is similar to that with soil alone; however, the fly ash contains finely divided silicate and aluminate compounds that may be scarce or lacking in some soils, especially sands. The authors postulate that the effect of an alkaline additive, such as metasilicate, is probably as follows.

Alkaline additives increase the amount of available hydroxyl ions in the moistened Ottawa sand-lime-fly ash system, and as a result the pozzolanic reaction may be accelerated by the increased solubility of the siliceous material caused by the alkalinity. Sodium metasilicate, $Na_2SiO_3 \cdot 9H_2O$, very likely reacts with lime, $Ca(OH)_2$, or the moist Ottawa sand-lime-fly ash mixture to form highly cementitious calcium silicates and sodium hydroxide in the following way:

$$Na_2SiO_3 \cdot 9H_2O + Ca(OH)_2 \rightarrow CaSiO_3 + 2NaOH + 9H_2O$$

The sodium hydroxide presumably is then free for further reaction with unreacted siliceous material, thereby serving as a catalyst in the overall reaction processes.

Strength—The studies of Mateos and Davidson (42) illustrated the beneficial effects of small amounts of sodium metasilicate especially on sand-lime-fly ash mixtures.

Specimens prepared from a mixture of Ottawa sand, calcitic hydrated lime, and a medium quality fly ash (75:5:20 by weight) showed an increase in immersed compressive strength with increasing amounts of sodium metasilicate. In fact, the strength was increased greatly by the use of even small amounts of the chemical, and the increase was found to be more or less proportional to the amount of chemical used. The optimum was about 3 percent where a strength of 1,000 psi was found after 7 days of curing for specimens having that amount of sodium metasilicate. Sodium metasilicate in the amounts of 0.5 and 1.0 percent also increased the strength of the dune sand-lime-fly ash mixture.

Used in the amount of 1.0 percent, sodium metasilicate increased the strength of friable loess-lime-fly ash mixtures containing those fly ashes with relatively high percentages passing the No. 325 sieve (86 to 92) and possessing a relatively high specific surface (2,800 to 3,200 cm^2/g). It was suspected that the use of greater amounts of sodium metasilicate would improve greatly the strength of friable soils because of the previously mentioned 3 percent optimum concentration.

In general, the greater benefits were derived from the addition of chemicals, including sodium metasilicate, to the sandy soil-lime-fly ash mixtures. Benefits were seen to decrease with the increase in the amount of clay in the soil. At times the addition of chemicals to clayey soils was detrimental to the strength of the specimens. It was supposed that the decrease in strength in these soils was brought about by the excess sodium ions and high alkalinity present in the pore fluid of the soil-lime-fly ash mixtures. Both factors introduce disruptive forces in the clay structures that are not overcome by the cementitious bond of the pozzolanic reaction.

The strengths obtained with lime-fly ash mixtures depend greatly on curing temperatures. When soils are stabilized with lime and fly ash in the late part of the summer in temperate climates, they may not develop sufficient strength to withstand the imposed stresses of the colder seasons. A mixture of dune sand, calcitic hydrated lime, and a medium-quality fly ash without additive, cured for 7 days at 43 F, failed during a period of immersion in water. The addition of 1.0 percent sodium metasilicate gave 7-day immersed strengths of about 100 psi. After 28 days of curing, the mixture without additive showed an immersed strength of 41 psi. This strength was increased fivefold or sixfold by addition of 1.0 percent sodium metasilicate.

When a dolomitic monohydrate lime was used in place of the calcitic hydrated lime, stabilized dune sand showed about the same 7-day immersed strengths with or without the additive. However, at 28 days the untreated mixture had a strength of 111 psi as compared to more than 2 times that value for the mixture treated with sodium metasilicate. Thus, the beneficial effects of sodium metasilicate to the lime-fly ash pozzolanic reaction are important when low temperatures are expected during the curing period. These effects are seen to vary depending on the type of lime and probably also on the fly ashes used.

Addition of sodium metasilicate may lengthen the working season for stabilization of soils with lime and fly ash. Such salts also assist by lowering the freezing point of the free water in stabilized-soil mixtures. By depressing the temperature at which the soil water freezes, more time is allowed for strength gain; and the stabilized soil is exposed for shorter periods to the damaging effects caused by ice formation.

CONCLUSIONS

On the basis of the literature review the following general conclusions seem to be warranted.

1. Injection processes utilizing sodium silicates and precipitants, such as calcium chloride, in 1- or 2-step procedures have been used successfully to harden and reduce the permeability of coarse-grained soils. There remains some doubt about the permanency of the beneficial effects over long time periods, especially when the soil is subjected to high hydraulic gradients.
2. Sodium silicate alone or in combination with certain chemical precipitants can stabilize sandy soils and produce a base or subbase that seems likely to retain the beneficial effects in mild climates.
3. Although there is some evidence that sodium silicate with precipitating chemicals can increase the strength and durability of relatively nonplastic fine-grained soils, there is almost no field evidence to justify its use when soils are subjected to the freezing and thawing cycles typical of temperate climates. Furthermore, the total cost of additives necessary to achieve stability is probably greater than that of stabilizers such as portland cement with which much more experience is available.
4. Sodium silicates with or without precipitants are of little value in dustproofing or waterproofing fine-grained soils.
5. On the basis of laboratory tests, various sodium silicates used as additives appear to improve the strength and durability of soils stabilized with portland cement, lime, or lime-fly ash. Best results have been obtained with sands or relatively nonplastic fine-grained soils. The sodium silicates, as well as other sodium salts, appear to be especially useful in increasing the resistance of cement-stabilized soils to sulfate attack. The addition of a small amount of sodium silicate can significantly reduce the cement requirement for clean sands. The early strength of a lime-stabilized clay soil can be increased by small amounts of sodium silicates, and the resistance of the stabilized soil to cool temperature and freeze-thaw effects can be increased.

RESEARCH NEEDED

In view of the kinds of soils and the severity of climatic conditions that prevail in Illinois, it appears that further research on sodium silicates as soil stabilizers for highways is not warranted.

The only avenue that appears to hold promise for future research on silicates is in their use as minor additives to sand-cements in order to reduce overall costs by reducing cement requirements. However, in view of the additional difficulties of incorporating 2 admixtures when normal field stabilization techniques are used, there is some doubt as to whether research in this area can be justified.

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    This paper reviews the state of knowledge (through April 1961) concerning the application of chemical techniques to the modification of the properties of soil for road engineering. The general principles of soil stabilization are discussed, including the methods of stabilization, the mechanisms of stabilization, and the properties required of stabilizing agents. Sodium silicate is cited as a chemical suitable, when used alone, only for the stabilization of sandy soils in a moderate climate. It is noted, however, that the use of sodium silicate as a secondary additive in soil-cement reduced the amount of cement required for stabilization by as much as 50 percent. Forty-six references are listed.

19. Wooltorton, F. L. D. Engineering Pedology and Soil Stabilization, Use of Electrolytes in Stabilization. HRB Bull 108, 1955, pp. 55-56.
    Sodium silicate as one of the electrolytes important in soil stabilization is discussed. The effect of the addition of a suitable percentage of sodium silicate to appropriate soils is to increase water-stable aggregation and resistance to crushing and to decrease the plastic limit, plasticity index, and swelling. The process involved is apparently that of anionic exchange resulting in the formation of insoluble silicate sheaths around the soil particles and accompanied by cementation effects. Because silicates are electronegatively charged, they are not expected to stabilize active clays. The efficiency of the additive increases with decrease in clay activity, i.e., with decrease in the

silica:sesquioxide ratio. It is most likely to be most efficient with those soils of an electropositive character or with those active clays that have had imposed on them an electropositive nature by virtue of coatings of iron compounds.

20. Murray, G. E. Soil Stabilization by Chemical Means. Proc., Conf. on Soil Stabilization, M.I.T., Cambridge, 1952, pp. 107-121.

    The various aspects of soil stabilization by chemical means are explained by analyzing the forces operating in a soil system and noting how they can be altered by the introduction of a stabilizing agent. The possible mechanisms through which the stabilizing agents can act to alter soil properties and the effect of both the soil and stabilizer composition on the properties of the treated soil are discussed. The methods by which stabilizing agents form on or around soil particles, i.e., precipitation of calcium silicate from sodium silicate and calcium chloride, are analyzed. The modes of application of the basic concepts to present and potential stabilization methods are given. Discussion of the paper indicates the practical problems that must be solved if any stabilization method is to be employed successfully.

21. Chemical Modification of Soil-Cement. M.I.T., Cambridge, Soil Stabilization Res. Rept., Phase 11, 1959.

    This reports (1) the effect of sodium compounds as additives to cement-stabilized soils of widely different origin, fineness, and mineralogical composition; (2) investigation of reactions between cement, lime, sodium compounds, and soil minerals; (3) study of the effect of soda-to-silica ratio in sodium silicates as additives on strength; (4) study of the effect of additives on the resistance of soil-cement to sulfate attack. Results of durability tests (wet-dry, freeze-thaw cycles) indicate the strength test is a good measure of the effectiveness of sodium additives. Observation of test data indicates the effects of sodium additives in soil-lime are similar to those in soil-cement.

    Soil stabilization with sodium silicates and salts of divalent metals was investigated. Preliminary gelation tests run in the absence of soil indicated that magnesium carbonate, $MgCO_3$, calcium hydroxide, $Ca(OH)_2$, and calcium sulfate, $CaSO_4$, would be most promising for testing with silicates in soil. At 5 percent treatment by weight of Massachusetts clayey silt, sodium silicates with $MgCO_3$ as precipitant are effective stabilizers, increasing strength to more than 400 psi. The curing reaction is essentially complete within 5 hours. An optimum concentration of 0.75 equivalents of $MgCO_3$ (2 percent by dry soil weight) per equivalent of sodium was found. The stabilization reaction is not sufficiently rapid to give stability upon water immersion immediately after compaction. At least 65 percent of 24-hour, humid-curing strength is retained upon 24-hour curing followed by 24-hour immersion.

    Study was made of effect of major variables—silica, sodium, and magnesium content—on soil stabilization. Strength increases with silica content; however, for best results the sodium concentration is confined to a range between 0.04 and 0.06 equivalents per 100 g of dry soil. Below this, strength development is slow. Above 0.06 equivalents per 100 g of dry soil and a corresponding amount of precipitant, $MgCO_3$, crystallization of sodium carbonate, Na_2CO_3, with resultant expansion cause deterioration of the stabilized soil. At high sodium levels (high pH) and low magnesium concentration, removal of the magnesium prevents any significant strength development. Solid silicates are not so effective as the corresponding silicate solutions when applied in the same manner. Attempts to increase effectiveness of solid-silicates through heating or increasing particle fineness have been unsuccessful. Some stabilization was achieved with metasilicate and orthosilicate (more soluble silicates).

22. Mainfort, R. C. Soil Stabilization With Resins and Chemicals. HRB Bull. 108, 1955, pp. 112-120.

    Sodium silicate is effective as a stabilizer of sandy soils when used at a concentration of 6 percent by dry weight of soil but is ineffective when used with fine-grained materials. It develops a strong bond between soil particles but does not withstand action of water. The most effective form of sodium silicate is a 40 percent solution, 40 to 42 B, with a ratio of 1 part Na_2O to 3.25 parts SiO_2. The injection process employed in practice is discussed and projects are cited.

23. Soil Stabilization With Sodium Silicates. M. I. T., Cambridge, Soil Stabilization Res. Rept., Phase 12, 1960, pp. 29-46.

    It was found that New Hampshire silt [L_w = 28 percent, PI = 8 percent, A-4(8)], Vicksburg loess [L_w = 41 percent, PI = 15 percent, A-7-6(10)], and Massachusetts clayey silt [L_w = 20 percent, PI = 6 percent, A-4(4)] can be stabilized with sodium silicates and basic magnesium carbonate. The best results obtained after 7 days of humid curing and 1 day of immersion are 365 psi wet strength in NHS at 8.5 percent total additive, 235 psi in VL at 9.1 percent, and 610 psi in MCS at 6.4 percent. Unsuccessful attempts were made to stabilize Vicksburg buckshot clay [L_w = 65 percent, PI = 37 percent, A-7-6(20)] with sodium silicate and magnesium carbonate. The effect of silica, magnesium, and sodium contents on each of the soils was studied. An explanation of the observations was advanced in terms of the formation of a strong, cementitious gel that results from the presence of polyvalent gel-forming cations in the inorganic salts and in the soils. Several inorganic salts were reacted with sodium silicate in the absence of soil to investigate their gel-forming characteristics. Magnesium oxide formed a gel that is as strong as that of the magnesium carbonate, but stabilization was considerably slower than stabilization with the carbonate. Calcium hydroxide was found to be a much less successful precipitating agent than magnesium carbonate.

24. Morey, R. E., and Lange, E. A. Factors Affecting Sodium Silicate-Bonded Sands. Foundry, April 1959, pp. 188-198.

    Tests conducted and reported on were a part of an investigation of the carbon dioxide process for hardening molds and cores. The temperature range (400+ F) in which the tests were performed is unrealistic with respect to soil stabilization in general, but several significant aspects of the results should be noted. An increase in the $Na_2O:SiO_2$ ratio caused a decrease in strength, and an increase in percentage of sodium silicate used caused an increase in strength. Also, it was observed that an addition of 1 percent bentonite clay to the sodium silicate-sand mixture caused an increase in strength (about 10 percent) over the sodium silicate-sand alone.

25. Buchanan, S. J. Military Application of Soil Stabilization. Proc., Conf. on Soil Stabilization, M. I. T., Cambridge, 1952, p. 295.

    The author outlines the methods of soil stabilization applicable to the difficult problems encountered in the development of air force installations in forward areas. The use of sodium silicate is indicated as a possible solution to some of these problems.

26. Murray, G. E. Chemical Soil Stabilization: Effect of Stabilizer Structure on Preparation and Properties of Stabilized Soil. HRB Proc., Vol. 34, 1955, pp. 602-613.

    The effect that the structure of a stabilization agent, i.e., magnesium, calcium, zinc, and sodium acrylates, has on the characteristics of a treated soil is discussed. The response of the treated mass to load depends on chemical structure of the admixture and distribution of the agent throughout the mass. Results indicate that a stabilizing agent that forms an initially rigid 3-dimensional matrix should be placed under optimum conditions because it cannot adjust to applied stresses to provide uniform distribution needed for effective stabilization. Cement and sodium silicate are cited as examples of such a nonviscous mixture. A stabilizing agent that can undergo viscous flow can be placed at less than maximum density because drying stresses cause it to densify. This type of stabilizer has lower initial strength.

27. Zhinako, N. M., Drozd, P. A., and Ioseleva, M. A. Resistance to Frost of Sands Stabilized by Chemical Methods. Sbornik Nauch. Rabot. Beloruss. Politekh. Inst., No. 54, 1966, pp. 57-62, in Russian.

    Aqueous solutions of mixtures of sodium silicate and of salts of bivalent and trivalent metals (e.g., $MgSO_4$) on introduction into sandy ground make the latter mechanically stable (temporary compressive strength of 115 psi).

28. Zhmako, N. M., and Ioseleva, M. A. The Consolidation of Sand by the Introduction of Mixed Salts in the Reaction With Sodium Silicate. Belorus. Politekh. Inst. Im. I. V. Staline. Sbornik Nauch. Trudov, No. 78, 1960, pp. 162-167, in Russian.

    Soil stabilization was studied. Sulfates, chlorides, and acetates of calcium, strontium, barium, magnesium, and cadmium were mixed singly with sand and the mixtures were treated with a solution of sodium silicate. The compressive strengths of the specimens were determined after time intervals up to 270 days. Sulfates and chlorides of calcium and magnesium were combined and, after being mixed with sand, were also treated with sodium silicate solutions. Mixtures containing calcium salts have the highest strength, up to 162 psi, because of the formation of hydrated calcium silicate.

29. Dogra, R. N., and Uppal, I. S. Chemical Stabilization of Sand and Sandy Soils, Laboratory Experiments With Sodium Silicate as Stabilizer. Jour., Indian Roads Congress, Vol. 23, No. 1, 1958, pp. 161-172.

    Investigations compared the properties of sodium silicate-sand and sodium silicate-sandy soils with the properties of cement-sand and cement-sandy soils. Strengths of pure sand and nonplastic soil specimens were greater at even low quantities of sodium silicate concentrations (1,500 psi at 2.5 percent on dry soil basis) than strengths of samples with higher concentrations of cement additive (1,300 psi at 10 percent). Sodium silicate-treated specimens of sandy soils were more resistant to abrasion than cement-sand. Durability test results (immersed 5 hours and dried 70 C for 42 hours) on specimens of 10 percent clay, 90 percent sand, and 1 percent sodium silicate showed loss of 3.5 to 4.0 percent in 12 cycles. However, larger additive concentration caused rapid disintegration of the specimens upon immersion. Use of sodium silicate with soils of higher clay content produced specimens with poor durability.

30. Winterkorn, H. F. Final Report on Beach Sand Stabilization Research. Bureau of Yards and Docks, Dept. of Navy NOy-15087, 1949, 564 pp.

    Pertinent statements are made concerning the use of sodium silicates as dust preventives and road binders, soil stabilizers, and soil solidifiers. Data are presented that show the results of field tests with aqueous solutions of sodium silicate used as a stabilizer of beach sand. Strips 10 ft long by 50 ft wide were stabilized to depths of 4 to 6 in. by loosening the sand, spraying an application of 40 percent solution sodium silicate at concentrations of 1 to 3 percent on dry weight basis of treated depth of sand, and compacting with pneumatic and flat rollers. Tests indicated the sections had a hard brittle crust about $\frac{1}{4}$ in. thick that did not withstand the weight of a jeep or $\frac{3}{4}$-ton truck used as test vehicles.

31. Soil Stabilization by the Use of Chemical Admixtures. Roads and Engineering Construction, Vol. 89, No. 10, 1951, p. 112; Mainfort, R. C. A Summary Report on Soil Stabilization by the Use of Chemical Admixtures. U. S. Civil Aeronautics Administration, Tech. Development Rept. 136, Feb. 1951.

    This article presents a summary of the results obtained from laboratory and field investigations of the soil-stabilizing properties of numerous chemicals with respect to their applicability for road and airport construction. Sodium silicate was among the most commonly available materials found to be unsuitable stabilizing materials. However, small quantities of sodium silicate were effective in reducing the plasticity of fine-grained soils.

32. Kozan, G. R., and Fenwick, W. B. Soil Stabilization. U. S. Army Engineer Waterways Exp. Station, Tech. Rept. 3-455-7, Feb. 1965.

    A laboratory investigation was made to examine the relative effectiveness of several primary additives in stabilizing Vicksburg loess and buckshot clay. Included as additives were sodium silicate solution and anhydrous orthosilicate and metasilicate. Magnesium carbonate and magnesium oxide were examined as precipitating agents. These materials produced very little improvement in strength (40 psi maximum). Although they were small, the strengths developed by using sodium orthosilicate in the loess (lean clay) during humid cure were effectively retained upon soaking. Specimens of the heavy clay treated with sodium orthosilicate disintegrated during soaking.

33. Review of Materials and Methods for Dustproofing and Waterproofing Soils. Corps of Engineers Research and Development Report, U. S. Army Engineer Waterways Exp. Station, Misc. Paper 3-176, July 1956, pp. 6-7.

    The review includes a discussion of applicability of sodium silicate for dustproofing and waterproofing. A fine-grained clayey silt (ML-CL) was satisfactorily dustproofed with treatment of 3 gal/sq yd of a 30 percent solution of sodium silicate in an arid, southwestern area of the United States. Depth of penetration was 1½ in. The surface withstood 200 coverages of 2½-ton cargo truck without evidence of stress, cracking, or surface abrasion. Attempt to stabilize a well-graded sand (SW) by similar means was unsuccessful because of the deeper penetration of sodium silicate in the more pervious material. However, satisfactory penetration was obtained by first compacting the loose sand then applying the sodium silicate.

34. Freitag, D. R., and Kozan, G. R. An Investigation of Soil Waterproofing and Dustproofing Materials. HRB Bull. 282, 1961, pp. 13-27.

    The capabilities of a number of materials to waterproof and dustproof a lean clay (L_w = 38 percent, PI = 16 percent, A-6) were evaluated by means of field and laboratory studies. Traffic was applied at intervals during a 13-month period of exposure to weather to a test section consisting of panels treated with 13 selected materials. The amount of material abraded by the traffic was measured to determine dustproofing effectiveness, and observations of moisture changes in the compacted soil under each surface were made to provide an indication of the degree of waterproofing supplied by the various treatments. A 30 percent solution of sodium silicate was used and was found to be a relatively ineffective water dustproofer.

35. Dustproofing and Waterproofing of Soils. Laboratory Studies of Soil Waterproofing Materials, U. S. Army Engineer Waterways Exp. Station, Tech. Rept. 3-530-2, July 1963, 40 pp.

    Results of tests on specimens, prepared by using the Harvard miniature compaction apparatus with a compacting effort of 20 tamps of a 40-lb spring tamper on each of 5 layers per specimen, indicated various sodium silicate systems were totally ineffective as waterproofers for either of the 2 test soils: Vicksburg loess (CL, PI = 15 percent, L_w = 39 percent) and buckshot clay (CH, PI = 40 percent, L_w = 63 percent). Anhydrous silicates (sodium orthosilicate, Na_4SiO_4, and sodium metasilicate, Na_2SiO_3) were premixed with a modifier, either magnesium carbonate or magnesium oxide, before being added to the soil; whereas the silicate solution, $Na_2O:SiO_2$, was incorporated after mixing of the soil and modifier.

36. The Dust-Proofing of Concrete With Sodium Silicate. Indian Concrete Journal, Vol. 29, No. 9, 1955, p. 291.

    The article discusses the relative merits of using sodium silicate as a dustproofer of concrete surfaces. It indicates that sodium silicate is cheaper than alternative dustproofing materials, can be stored indefinitely in a closed container without deterioration, is nontoxic and nonflammable, and is very effective in penetrating and hardening a relatively porous concrete surface.

37. Lambe, T. W., Michaels, A. S., and Moh, Z. C. Improvement of Soil-Cement With Alkali Metal Compounds. HRB Bull. 241, 1960, pp. 67-103.

    A large number of soils of widely different geographical origin, fineness, and mineralogical composition have been tested to determine their response to stabilization by cement in the absence and presence of a selected group of alkali metal compounds at low concentration. The soaked compressive strength of nearly all cement-soils tested was substantially improved (twofold to tenfold) by the use of relatively small quantities of sodium compounds that form insoluble compounds with calcium. Optimum additive concentration was found to correspond very nearly to a sodium ion concentration in the molding water of 1.0 N, that is, between 0.5 and

2.5 percent by weight, depending on soil and additive. Five percent cement and 1 percent sodium silicate-silt gave strengths comparable to those of 10 percent cement-silt, but sodium silicate was the most beneficial additive in clean sands where it doubled early and ultimate strength of sand-cement specimens. Economics of additive treatment is discussed, and it is shown that considerable financial saving can be realized by the use of the sodium additives. A hypothesis to explain the action of the additives is proposed. It attributes the beneficial action to enhanced participation of soil-derived silica in the cement-hydration process. Alkali metal compounds exert their major influence on cementation in soils at the earliest stages of the cure process by promoting silica solubilization and retarding calcium silicate precipitation; they have relatively little effect on the composition of the final cementing components.

38. Moh, Z. C. Soil Stabilization With Cement and Sodium Additives. American Society of Civil Engineers, Jour. Soil Mech. and Found. Div., Proc. ASCE, Vol. 88, SM6, Paper 3356, 1962, pp. 81-105.

This is a report of an investigation of improvement in the effectiveness of portland cement as a stabilizer by use of chemical additives. Tests showed that, with all types of soils studied, from nonplastic sands to heavy clays, cement stabilization can be substantially improved by incorporation of relatively small quantities of sodium compounds. Strength improvements varied from 15 to 250 percent. The degree of effectiveness of the sodium additives depends largely on the characteristics of the soil involved; sodium metasilicate is most effective in clean sands and least effective in plastic clays.

39. Moh, Z. C., Lambe, T. W., and Michaels, A. S. Improvement of Soil-Cement With Chemical Additives. HRB Bull. 309, 1962, pp. 57-76.

This paper summarizes the results obtained from detailed study on the use of alkali additives in soil-cement. Study of the long-term effects of immersion in sulfate solution of a cement-stabilized sand indicates that sodium metasilicate greatly delays or reduces the deteriorating effects of the sulfate, thereby prolonging the life of the soil-cement. Examination of the effect of the soda-to-silica ratio in sodium silicate showed a 1:2 ratio to be about optimum for improving strength of New Hampshire silt with 5 percent cement. A higher soda-to-silica ratio in sodium silicate tended to give higher early strength but lower ultimate strength, while a lower ratio retarded the cementing process.

40. Norling, L. T., and Packard, R. G. Discussion of Improvement of Soil-Cement With Alkali Metal Compounds. HRB Bull. 241, 1960, pp. 103-108.

The effect of alkali metal compounds on the durability of soil-cement was studied. The results of tests on 4 soils show that the effect of chemical additives on durability, as measured by freeze-thaw and wet-dry tests, is similar to that of the additive on compressive strength. The addition of 1.0 percent sodium metasilicate reduced the cement required to satisfy durability specifications of Massachusetts clayey silt [L_w = 20 percent, PI = 6 percent, A-4(4)] from 8.5 percent to 6.0 percent.

41. Katti, R. K., and Kulkarni, K. R. Effect of Trace Organic Chemicals on Lime Stabilization of an Expansive Soil. Jour., Indian Roads Congress, Vol. 27, 1962, pp. 215-230.

The effect of lime, CaO, and lime plus 0.5 and 1 percent sodium silicate, Na_2SiO_3, and other inorganic chemicals on strength, water absorption, and swelling of a black soil was studied. Maximum strength, after 7, 14, and 28 days of curing the mixtures, was obtained when 8 percent lime was used. Sodium silicate increased strength, about 100 percent after 7 days of curing, at lime concentrations below 8 percent, reduced absorption, and had no consistent effect on swelling. Mixtures containing sodium silicate did not have maximum strengths significantly different from those containing lime alone when tested at 28 days.

42. Mateos, M., and Davidson, D. T. Further Evaluation of Promising Chemical Additives for Accelerating Hardening of Soil-Lime-Fly Ash Mixtures. HRB Bull. 304, 1961, pp. 32-50.

    The results of an investigation on the effect of various amounts of 12 chemicals on the strength of a mixture of Ottawa sand-lime-fly ash are presented. The effects of 4 selected chemical additives, including sodium metasilicate, on the strength of soil-lime-fly ash mixes, including 4 natural soils (dune sand, friable loess, alluvial clay, and Kansan gumbotil), calcitic hydrated and dolomitic monohydrate lime, and 3 fly ashes, are also presented. The evaluation showed sodium metasilicate to be very beneficial to sandy soil-lime-fly ash and friable loess-lime-fly ash mixtures. In general the benefits decreased with increasing clay content (montmorillonitic mineral content) of the soils tested.

43. Jorgensen, L. R. Solidifying Gravel, Sand and Weak Rock. Western Construction News, Vol. 6, No. 21, 1931, pp. 591-593.

    The process invented by Joosten for binding loose material to obtain a solid watertight mass by successively injecting solutions of sodium silicate and salt into gravel, sand, weak rock, masonry, or concrete is discussed. Compressive strengths range from 300 to 1,200 psi. The magnitude of the strength depends on the relative quantities of siliceous material present in the materials and the relative grain size of the treated materials. Applications also include waterproofing masonry shaft linings by reducing water flow from 150 gpm to a negligible amount.

44. Pohl, K. A. The Chemical Solidification of Loose Soils. Engineering Progress, Vol. 13, No. 4, 1932, pp. 85-88.

    The author discusses the characteristics of the chemical solidifying process using successive injections of silicic acid and salty solutions. The procedures of solidifying practice are outlined. The principal soil property changes, including increased compressive strength, reduced permeability, and increased resistance to chemical attack, are enumerated. An example of the process application is given.

45. Japp, H. New Methods of Excavating Quicksands, No. 11. The Engineer, Vol. 156, No. 4060, 1933, pp. 422-425.

    This paper discusses the Joosten soil consolidation process and presents examples of practical applications to engineering projects. Included are examples of the successful uniform consolidation of sandy soils containing patches of fine-grained soils, the adherence of clay layers and concrete structures to chemically solidified coarse-grained water-bearing strata, and the development of a 15-tsf compressive strength in a ballast containing about 25 percent silt and clay.

46. Harding, H. J. B. Groundwater Lowering and Chemical Consolidation of Foundations. Concrete and Constructional Engineering, Vol. 30, No. 1, 1935, pp. 3-12.

    The Joosten consolidation process was employed to strengthen existing foundation materials prior to adjacent excavation for a rebuilding project in Kingston, England. Waterlogged ballast and sand foundation soils were solidified into a form of artificial sandstone with a crushing strength of 30 tsf.

47. Gutmann, I. The Moscow Subway. Engineering News-Record, Vol. 116, 1936, p. 515.

    Solidification of quicksand by injecting sodium silicate and calcium chloride solutions under 70- to 90-psi pressure is discussed. Compressive strength of the solidified material was from 280 to 850 psi and up to 3,600 psi in quartz-abundant material. About 90 gal of chemicals were required to solidify 1 yd^3 of material. Solidification occurred in 15 to 20 min. Poor pervious concrete lining was waterproofed by chemical grounting.

48. Joosten, H. Chemical Solidification of Loose Soils. The Constructor, Vol. 19, No. 8, 1937, pp. 23-24.

    The author discusses the method of sealing and reinforcing porous material underground by chemical solidification. A presentation of the techniques employed, the limitations of the process, and some of its possible uses is given.

49. Karghin, V. A. Interaction Between Concentrated Solutions of Sodium Silicate and Calcium Chloride. Prikladnoi Chimyl, 1937, pp. 93-98, in Russian.

50. Bodascher, I. O. Strengthening and Sealing in the Building of Rumanian Wells by Chemical Processes. Die Bautechnik, Book 10, March 10, 1959, pp. 139-140, in German.
    This describes the processes employed to reduce the flow of water into the Mihar well, drilled on the grounds of the salt mines of Vioara, Rumania. A sodium silicate-calcium chloride 2-step injection technique used with cement reduced the flow of water through a gravel layer connected to a nearby water-bearing strata.

51. Lewin, J. D. Grouting With Chemicals. Engineering News-Record, Vol. 123, No. 7, 1939, pp. 61-62.
    The use of sodium silicate with calcium chloride, aluminum sulfate, and other metallic salts in the consolidation of soils is discussed. Chemical grouting is said to be applicable to soils possessing permeabilities of 10^{-1} to 10^{-4} cm/sec. The most commonly used sodium silicate solution has a density not less than 42.25° B and a sodium oxide, Na_2O, to silica, SiO_2, ratio of 1:3.22. Examples of applications include the reduction of the permeability of sand from 40 to 0.15 gpm and the increase of the bearing capacity of sand to 1,300 psi.

52. Consolidating Loose Soils. Roads and Road Construction, Vol. 22, No. 256, 1944, pp. 103-104.
    The method of stabilizing granular soils by injecting solutions of sodium silicate and common salt into a deposit is discussed. The compressive strength of the treated soil can be increased by 140 to 570 psi. The major drawback of the process is stated as its limitation to application in cohesionless soils.

53. Riedel, C. M. Chemical Soil Solidification and Chemical Sealing of Leaking Concrete. Jour. of American Water Works Assn., Sept. 1945, pp. 849-862.
    The underlying principles of Joosten's injection method of chemical soil solidification and sealing of leaking concrete are outlined. The uses of the process in water works and other public utilities construction are discussed. Examples are listed.

54. Chemical Injections Solidify Soil to Stop Foundation Settlement. Construction Methods, Vol. 27, No. 12, Dec. 1945, p. 78.
    Silicate of soda and calcium chloride were used to solidify a wet, medium fine sand under the heavy foundation of a mill-engine and crusher at the Clewiston, Florida, sugar mill of the U. S. Sugar Corporation. Injection pressures used were about 400 psi; after solidification, injection pipes could be driven into the deposit only with great difficulty.

55. Injected Fluid Stops Running Sand. Western Construction News, Vol. 23, No. 9, 1948, pp. 89 and 101.
    Free-running sand in a cable tunnel was solidified by pumping a dilute solution of sodium silicate and sodium bicarbonate (1-shot system) at 200-psi gauge pressure to the face of the tunnel. The treatment made the sand into a soft sandstone (80- to 100-psi compressive strength), made it possible to place the liner plates without breast boards or piling and without danger of losing ground, and developed an arch of solid sand above the tunnel that diminished the pressure on the liner plates.

56. Bristow, T. Footings Dug Without Shoring in Chemically Solidified Sand. Engineering News-Record, Vol. 143, No. 19, 1949, p. 48.
    A sodium silicate solution was used to solidify a free-running sand foundation soil. Solidification was accomplished by introducing 500 gal of the solution under 200-psi pressure for each footing. Compressive strength of the solidified sand was about 75 psi.

57. Debecq, A. The Silicatization of Soils. Rev. Tech, Luxembourg, Vol. 41, 1949, pp. 29-30.

Methods of densifying soil and increasing its resistance to load are discussed. When the Gayard process is used, sodium silicate is injected with 3 to 15 percent sodium or potassium bicarbonate, 3 to 15 percent sodium chloride, and 0.3 to 1.0 percent sodium or potassium hypochlorite of the weight of sodium silicate, Na_2OSiO_3, used. The injection techniques are described.

58. Riedel, C. M. Latest Developments in Soil Solidification. Midwest Engineer, Vol. 1, No. 6, 1949, pp. 6-8.

This is a discussion of the chemical solidification of loose, sandy soils, based on the Joosten Process. The characteristics of the solidified material are presented, including 250- to 1,100-psi compressive strength, 160-psi tensile strength, and very low acid solubility. Limitations of the process are comparative economics and soil grain size. Examples of practical applications are listed.

59. Wright, R. E. Chemical Stabilization of Sand Speeds Driving of 10-ft Tunnel. Engineering News-Record, Vol. 143, No. 5, 1949, pp. 42-43.

A storm water tunnel in San Francisco, 2,700 ft long with a 10-ft horseshoe section, had to be driven through dry sand that came through cracks between lagging and breast boards more rapidly than it could be removed. Remedy was injection of sodium silicate with reacting agent in the form of sodium bicarbonate. The amount of combined solution required for stabilization varied from 30 to 45 gal/yd^3 of solidified soil.

60. Chemical Treatment Cures Sick Tunnel. Construction Methods and Equipment, Vol. 31, No. 9, 1949, pp. 66-73.

Injections of 8,400 gal of silicate of soda and 5,100 gal of calcium chloride were used to stop excessive flow (250 gpm) of water under a 50-ft hydraulic head through a 4-ft medium fine sand strata so that a tunnel could be advanced 62 ft through the strata.

61. Askalonov, V. V., Korablinov, A. M., and Koltunovskaya, B. M. Application of Silicate Treatment to Loess Soils. Stroitel. Prom., Vol. 28, No. 4, 1950, pp. 10-12.

Equipment and procedures are described for impregnation of loess soils with sodium silicate for purposes of mechanical fixation of the soil. The treated surfaces, bound apparently by cementation with partly hydrated SiO_2, are satisfactory in respect to strength, hardness, and water resistance.

62. Gershoy, E. C., and Mueller, C. Soil Stabilization for Transmission Tower Footings. Electric World, Vol. 135, No. 3, 1951, pp. 60-61.

Tower footings in loose sand can be strengthened and solidified with the aid of a mixture of sodium silicate, Na_2SiO_3, and sodium bicarbonate, $NaHCO_3$. The mixture forms a silica, SiO_2, gel that holds the grains of sand together in a stable semi-permanent mass that can be excavated with a conventional auger without sloughing or caving. Concrete can be poured directly into the bored hole without need of forms. A mixture of 12 parts by volume of sodium silicate (1:3.22, 41 B) 60 parts of water, and 28 parts of a solution containing 6.6 lb $NaHCO_3$ per 100 lb gels in 45 to 60 min.

63. Riedel, C. M. Chemicals Stop Cofferdam Leaks. Civil Engineering, April 1951, pp. 23-24.

The method of using sodium silicate-calcium chloride solutions to reduce the permeability and increase the strength of granular materials is illustrated. The strength developed depends on the silica content of the natural soil. The injection process is limited to soils containing less than 25 percent clay, silt, or minus No. 140 sieve (0.105 mm) sand. The bearing strength of the stabilized soil approaches 700 psi, and the strata is impervious to water even under considerable hydrostatic head.

64. Keil, K. The Densification and Strengthening of Bonding Soils (Adhering Loose Rocks). Bautechnik, Vol. 29, 1952, pp. 42-46, in German.

Binding soils can be stabilized by chemical or electrolytic treatment based on the physical change that the soil undergoes by elimination of the decisive influence of the water. This is made possible by the limitation of the exchangeability of ions and the capacity of water molecules to be adsorbed at the unsaturated border areas of the clay minerals. The action of this chemical strengthening is always limited to the chemical sphere of influence. Although sodium chloride was formerly used, especially in America, for the electrolytic treatment, Keil uses labile water glasses, Na_2SiO_3. The mutual actions between chemicals and soils are discussed.

65. Soil Stabilizing Permits Sand Augering. Electrical West, Vol. 110, No. 5, 1953, pp. 82-83.

To overcome the difficulty of caving holes in sandy areas, it was necessary to develop a process of stabilizing the sand long enough to permit augering the hole, setting the template and reinforcing steel, and pouring the concrete. The soil was stabilized by injection of a chemical solution into the alluvial deposit at 100-psi pressure.

66. Lambe, T. W., and Michaels, A. S. Altering Soil Properties With Chemicals. Chemical and Engineering News, Vol. 32, No. 6, 1954, pp. 488-492.

The authors present a thorough discussion of the soil-stabilization problem. The various factors that influence the type and amount of stabilization agent needed to produce a satisfactorily stabilized mass are noted. These include the soil properties; the use for which the agent is intended, i.e., a true additive, a trace chemical, a waterproofer, or an aggregation or flocculation agent; and the means by which the soil is chemically treated. The future of chemical stabilization is also discussed.

67. Gnaedinger, J. P. Soil Stabilization by Injection Techniques. Industrial and Engineering Chemistry, Vol. 47, Nov. 1955, pp. 2249-2253.

Problems pertaining to the injection process of stabilization of subsurface soils are discussed. A comparison is drawn between AM-955, a product of the American Cyanamid Company, and sodium silicate when used in stabilizing sands. It is noted that AM-955 generally provides more satisfactory results but is more expensive. Principal civil engineering problems that can be solved effectively by injecting chemicals into the soil are discussed. These include the prevention of seepage into underground structures, construction of caissons and tunnels, prevention of seepage beneath dam foundations, and sealing pervious strata in water and oil wells.

68. Golder, H. Q. Use of Chemical Injection Processes in Underpinning and Tunnelling. Der Ingenieur, Vol. 67, No. 49, 1955, pp. B173-176, in German, and discussion, pp. B176-177, in English.

This paper discusses the injection processes and lists types, limitations, and uses. Chemical processes are divided into 2 groups, the single-solution and the 2-solution processes. Single solution is cheaper when a large volume of material is treated, but placing of chemical is difficult to control. Therefore, it is used for impermeabilizing strata. The 2-solution is more adaptable to situations where strength and stability are prime considerations. Numerous applications in British construction are described.

69. Hauser, E. A. Composition and Structure of Siliceous Compounds. Silicic Compounds, 1955, pp. 7-20.

70. Hauser, E. A. Soil Stabilization. Silicic Science, 1955, p. 141.

This and the preceding report discuss the basic reactions that occur when sodium silicate is used with a calcium salt for stabilizing a soil deposit. It is stated that, soon after the soluble calcium salt is injected, an ion exchange reaction occurs that causes the formation of calcium silicate and releases sodium ions. The calcium silicate forms a gel that solidifies quite rapidly.

71. Lambe, T. W. Chemical Stabilization Can Make Construction Materials of Weak Soils. Engineering News-Record, Vol. 155, No. 3, 1955, pp. 41-42, 44, 46, 48.
By a minor change in physical characteristics, discarded soils can be converted into usable highway construction soils. The importance of and new uses for conventional stabilizers as construction materials are discussed. Aids to soil mixing, information on methods of obtaining greater density, and a discussion of antistripping agents, aggregants, and dispersants are presented.

72. Numata, M., Marugasu, T., and Kurosaki, T. A New Method of Chemical Grouting to Solidify Loose Ground. Trans., Japan Society of Civil Engineers, No. 12, 1956.
The use of sodium aluminate plus sodium silicate plus sodium hydroxide in grouting is discussed. With a coagulation period varying from 1 min to 1 hour, the compressive strength of treated sand approached 20 to 30 kg/cm^2. Field testing disclosed a reduction of permeability from 10^{-3} to 10^{-5} cm/sec.

73. Riedel, C. M. Chemicals Dry Up Pump Pit Job. Engineering News-Record, Vol. 157, No. 14, Oct. 4, 1956, pp. 49-50.
The 2-shot Joosten method of soil solidification was employed to alleviate a water problem in the modernizing of Chicago's old pumping stations. The process consisted of first injecting a solution of silicate of soda, followed immediately by a strong calcium chloride solution, into the wet fine sand present at the job sites. As the chemicals met in the ground, a rapidly formed silicic gel of high surface tension bound the loose sand particles together into an insoluble sandstone-like mass of hardness varying from 150 to 900 psi, depending on the character of the loose material. After being properly solidified, the soil was watertight and withstood groundwater and saltwater acidic action. Its hardness generally increased with time, and it possessed a bond to steel comparable to that of concrete.

74. Rozgaj, S. Chemical Injections Based on Waterglass Solutions. Tečhnicki Pregled, Vol. 8, 1956, pp. 67-75, in Russian.
Mechanical and chemical properties of water-glass solutions and precipitation and control of reactions for use in soil consolidation are discussed.

75. Ingenuity Needed for Bell Caissons in Loose Sand. Western Construction, Vol. 31, No. 5, 1956, pp. 36-37.
Foundation soils vary from loose sand to clay beneath a proposed 13-story building. Soil was stabilized with sodium silicate and bicarbonate of soda injections, but the variable nature of deposits and high moisture content reduced the length of flow from the injection nozzles and prohibited effective solidification.

76. Chemical Stabilization Cuts Excavation Cost. Pacific Builder and Engineer, Vol. 62, No. 7, 1956, p. 105.
The construction of a swimming pool built in sandy and gravelly soil under an existing building, adjacent to footings and columns of the building and adjacent also to a concrete slab floor, is discussed. Commercial sodium silicate, certain metallic salts including calcium chloride, and certain dispersing gases were used to stabilize the soil. Excavation costs were reduced by 30 percent. Solidification cost was about \$25/yd^3. The capabilities of chemical soil stabilization are outlined.

77. Cambell, C. W. Chemicals Seal Foundation for New York Building. Civil Engineering, Vol. 27, No. 10, 1957, pp. 47-51.
Chemical grouting, using the Joosten 2-shot injection process with sodium silicate followed by an intrusion of calcium chloride, was used successfully as an aid in constructing foundations for the Chase Manhattan Bank building in New York City. The grouting effectively sealed off the excavation site so that the flow of water through water-bearing strata into the excavation was negligible, and the loss of ground to adjacent structures was minimized.

78. Polivka, M., Witte, L. P., and Gnaedinger, J. P. Field Experiences With Chemical Grouting. Jour. Soil Mech. and Found. Div., Proc. ASCE, Vol. 83, April 1957, pp. 1204-1—1204-31.

This paper cites many examples of the use of chemical grouts, either for improving the stability of sandy soils or for sealing porous strata to prevent water movement. The examples given include the use of sodium silicate grouts as an aid in the construction of tunnels and foundations in free-flowing sand. Laboratory control, composition of grouts, and methods of grouting are discussed. First sodium silicate, Na_2SiO_3, and then a reagent, such as calcium chloride, $CaCl_2$, are injected into a subsurface soil to form a precipitated insoluble crystal-like solid that fills the voids in the soil and cements together the loose grains. When sodium bicarbonate, $NaHCO_3$, replaces the calcium chloride, the resulting gel is weak. When sodium silicate and the reagent are injected simultaneously, the gel deteriorates in a few months. A permanent gel may be formed with dilute sodium silicate mixed with hydrochloric acid, HCl, and copper sulfate, $CuSO_4$.

79. Van Heerde, J. A., and Golder, H. Q. Solving a Problem in Soil Stabilization. Consulting Engineer, Vol. 9, May 1957, pp. 80-81.

The settlement of adjacent buildings was prevented by injecting a solution of sodium silicate-sodium carbonate followed by calcium chloride into sandy soil before excavation was begun for a new post office at The Hague, Netherlands. Samples of the injected soil showed a compressive strength of from 250 to 700 psi, depending on the origin of the sample and the pressure on the sides of the test cubes (0 to 140 psi). The results indicated a shear strength of about 70 psi. No measureable settlement of adjacent buildings was noted.

80. Chemically Made Rock Seals Foundation Pit. Construction Methods and Equipment, Vol. 39, No. 9, 1957, pp. 95-97.

Injection of sodium silicate and calcium chloride solutions under pressures of 160 to 600 psi made a subterranean cutoff wall in a wet sand-gravel stratum.

81. Pynnonen, R. O., and Look, A. D. Chemical Solidification of Soil in Tunneling at Minnesota Iron-Ore Mine. U. S. Bureau of Mines, Information Cir. 7846, 1958, 8 pp.

Construction of a conveyor tunnel at an iron ore mine was stopped when a 22-ft bed of saturated fine gray sand was encountered. Solutions of sodium silicate and calcium chloride were forced into the water-bearing sand, consolidating it and removing the water problem. Excavation was done with pneumatic spades.

82. Zhmako, N. M., and Ioseleva, M. A. Effect of Various Sulfates on the Strength of Fixation of Sandy Soil During Silicatization. Belorus. Politekh. Inst. Im. I. V. Stalina. Sbornik Nauch. Rabot., No. 59, 1958, pp. 149-153, in Russian.

Test cubes of sand were treated with 50 cc of 2 N water glass solution and from 6- to 12-g portions of hydrated sulfates of Be, Mg, Zn, Cd, Mn, Fe, Co, Ni, Al, Cr, Cu, and a Ca-Mg mixture. The strength of the cube (maximum 10 kg/cm^2) is greater the lower the atomic weight of the metal is.

83. Chemical Modification of Soil-Cement. Soil Stabilization Res. Rept., Phase 10, M. I. T., Cambridge, 1958, pp. 22-42.

Sodium metasilicate is one of a group of sodium compounds found to be effective additives for improving the properties of soil-cement. Use of 7 percent cement and 1 percent sodium metasilicate in a clean sand resulted in higher early strength and comparable 28-day strength when compared with 10 percent cement alone. With a silt of low plasticity (PI = 8), 5 percent cement and 1 percent sodium metasilicate gave strengths comparable to those with 10 percent cement. With 2 heavy textured soils (PI = 30), 1 percent sodium metasilicate improved the 7-day strength of 10 percent cement by 50 percent. Sodium metasilicate was most effective in sandy soils that are deficient in reactive silica. The effectiveness of sodium compounds decreases with increasing plasticity or organic matter content of the soil or both.

84. Handy, R. L., Jordan, J. L., Manfre, L. E., and Davidson, D. T. Chemical Treatments for Surface Hardening of Soil-Cement and Soil-Lime-Fly Ash. HRB Bull. 241, 1960, pp. 49-66.

Laboratory experiments were conducted to determine the effectiveness of surface applications of chemical solutions on specimens of lime-fly ash and cement-stabilized friable loess [PI = 6 percent, L_w = 34 percent, A-4(8)] and a 75:25 sand: loess mixture [PI = 3 percent, L_w = 19 percent, A-4(1)]. Soil-lime-fly ash was benefited most by application of sodium silicate solutions. The sodium silicate penetrates into the soil and reacts with calcium and magnesium ions from the lime to produce insoluble calcium and magnesium silicates. Ratios of Na_2O:SiO_2 and lime to fly ash were not critical, and the treatment was only moderately sensitive to kind of lime. A single application of sodium silicate followed by continual moist curing approximately doubled the bearing strength, and sodium silicate treatment followed by daily applications of water increased bearing capacity in the surface layer about 4 times. The silicate crust also forms an effective seal against entry of surface water and loss of soil moisture due to drying. Laboratory data indicate that a road surface treatment of 1 gal/sq yd of 20 percent solution gives a threefold to eightfold increase in bearing strength. One gal/sq yd of 20 percent sodium silicate raised the bearing value of silt-cement about 35 to 50 percent. However, sodium silicate was most beneficial to sand-cement and, when used in a 5 percent solution in the amount of $\frac{1}{2}$ gal/sq yd, doubled the bearing value.

85. Herzog, A. The Use of Additives in the Stabilization of Poorly Reacting Soils With Cement. Constructional Review, Vol. 33, No. 9, 1960, pp. 35-36.

The compressive strength of soil-cement mixtures made from poorly reacting soils was greatly increased by chemical additives. Calcium chloride was most effective on sand and soils of low plasticity; sodium metasilicate and, to a lesser extent, ferric chloride were effective on plastic soils that responded least to calcium chloride. Additive concentrations used were 0.5 and 1.0 percent based on dry weight of treated soil.

86. Vaisfel'd, G. B. A New Formula for a Single-Solution Method for the Silicate Stabilization of Loose Ground Masses. Sb. Nauchn.-Issleda. Institute Osnovanli i Podzem, Sooruzhenii, Akad. Stroit. i Arkitekt. SSSR, No. 39, 1960, pp. 14-27, in Russian.

Sandy and loose ground soils, unsafe for the support of buildings, are stabilized by injecting into them solutions that interact to form cementing materials of sufficient strength to consolidate the granular mass into a monolithic structure. Solutions possessing the necessary qualities, such as low viscosities, easily regulated setting times, good cementing properties, and adequate final strength, and usually made up by mixing dilute acids or acid salts with dilute water glass set into solid gels at pH 3.7 to 7.0. A new formula in which phosphoric acid, H_3PO_4, is replaced with sulfuric acid, H_2SO_4, with considerable reduction in cost calls for a solution 1.32 to 1.40 N with respect to sulfuric acid and 0.36 to 0.38 N with respect to aluminum sulfate, $Al_2(SO_4)_3$, to which is added a solution of water glass of density 1.19 to 1.20 in the ratio of acid solution:water glass of 1.3 to 1.4. Its viscosity is 1.7 cp; its setting time is 10 to 16 hours. Structures so hardened have a crushing strength of about 45 psi.

87. Volkova, V. L. A Single-Solution Method of Claying. Uch. Zap. Vses. Zaochn. Institute Inzh. Zhelezno-Dorozhn. Transp., No. 6, 1960, pp. 190-195, in Russian.

Stabilizers and coagulators for clay solutions used for stabilizing weak soils and rocks were selected. When water glass was introduced as a stabilizer and, at the same time ferric chloride, $FeCl_3$, was introduced as a coagulator into the clay suspensions, it was possible to get clay mixtures fulfilling all requirements for performing the claying with 1 solution. The described claying method in which ferric chloride, $FeCl_3$, was used as a coagulator can be recommended for anchoring acid soils.

88. Cunningham, L. J. Use of Chemicals in Mine Grouting. Canadian Mining and Metallurgical Bulletin, Vol. 55, No. 603, 1962, pp. 480-483.

Use of sodium silicate, chrome lignin, and AM-9 chemical grouts is discussed. The procedure for surveying and determining the mine water problem and the methods of sodium silicate application (used as a primer or lubricant), properties, and equipment requirements are presented. A comparison of chemical grouts with cement is made.

89. Higuchi, Y. Studies on Admixtures of Cement Grout for Filling Fine Voids. Trans., Japan Society of Civil Engineers, No. 81, 1962, pp. 27-50, in Japanese with English abstract.

A survey of the history of cement and chemical grouting is presented. The successful treatment of cracks larger than 0.01 mm or voids among soil particles larger than 0.1 mm by combining cement and sodium silicate is reported, and recommendations for using quick-setting cement and expansive grout are provided.

90. Katti, R. K., and Brave, A. G. Effect of Inorganic Chemicals on the Consistency Properties of an Expansive Soil Sample, HRB Bull. 349, 1962, pp. 1-8.

This paper reports the results of the effect of 20 inorganic chemicals on the consistency properties of an expansive soil. The soil was treated with amounts of the inorganic chemicals ranging from 0.1 to 10 percent to take into consideration both practical and academic interests. The data indicate that sodium silicate brings about dispersion of the soil mass. For chemical concentrations below 3 percent, the liquid limit and plasticity index of the soil increase greatly with increasing concentration, whereas the plastic limit and the shrinkage limit of the soil are only slightly affected. Beyond 3 percent, because of the high dispersive action of the chemical, it was not possible to run the liquid limit and plastic limit tests properly; but it was determined that the shrinkage limit was increased threefold over that of the control soil when the chemical concentration was 10 percent.

91. Manvelyan, M. G., Babayan, G. G., Galstyan, V. P., Gevorkyan, S. V., and Aslanyan, D. G. Reaction of Aqueous Solutions of Lithium and Potassium Carbonate With Calcium Metasilicate. Izvestiya Akademii Nauk Armyanskoi S. S. R. Khimicheskie Nauki, Vol. 16, 1963, pp. 437-441, in Russian.

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