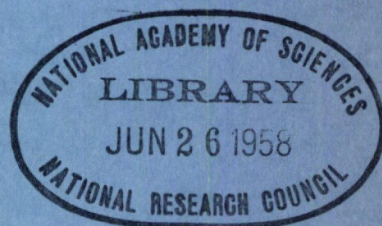


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Bulletin 183

Soil Stabilization Studies
1957



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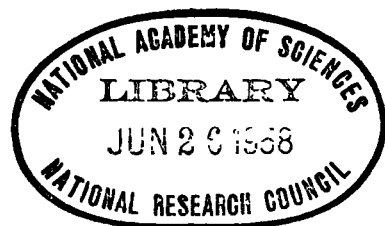
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Salt Stabilization on Ohio's Secondary System

J. W. REPPEL, Engineer of Maintenance, Ohio Department of Highways

● OHIO'S state road system consists of about 18,000 miles of various types of highways. The most expensive maintenance costs are incurred on those roads which carry the least amount of traffic, the farm-to-market portion of our highway net. The annual maintenance cost of the poorest of this mileage has averaged about \$1,000 per mile over the last ten years and some 1,500 miles are in critical need of repairs at this time.

This secondary mileage, generally carrying from 100 to 300 vehicles per day, is of trafficbound or trafficbound surface-treated type. These roads have been gradually built up over the years by the occasional addition of aggregate, application of dust control materials, and routine surface and ditch maintenance; some have been surface treated, with resultant winter and spring failures and increased maintenance costs. It was considered essential to strengthen bases and remedy deficiencies permanently by incorporating additional aggregate in varying quantities and adding a stabilizing agent, and experimental work was undertaken to develop a low cost construction method to accomplish this purpose.

Different materials have been used to stabilize surfaces and shoulders on selected sections with varying degrees of success. Some 25 miles of trafficbound surfaces were rebuilt in 1955 by state forces with rock salt as an additive, and results conclusively indicated the desirability of utilizing this type of construction to improve existing unsatisfactory surfaces.

Several factors were considered in setting up the program. Limited funds made it imperative to keep the cost down, and a brief general specification was prepared to permit maximum job control under experienced field supervision. Required operations could be done prior to the regular surface treating season, thus giving a large group of regular bituminous contractors the opportunity to perform early work at a probable cost advantage to the state. Existing roadway materials were used in the work, and aggregate was added as required. Existing intermediate satisfactory road sections were undisturbed, and contracts were set up to cover only weak and unstable portions. Detour routes were generally unavailable, and all work was accomplished under traffic.

Specifications were prepared and 28 projects varying from a single section 3 miles long to a group totaling 21 miles in length were sold in March 1956 as purchase order contracts to complete about 260 miles of contract stabilization work on secondary surfaces in five highway divisions, with headquarters at Ashland, Newark, Chillicothe, Marietta, and New Philadelphia. Completed work averaged about \$3,500 per mile, and all jobs were finished by June 30.

The purchase order contracts covered furnishing all services, labor, materials, and equipment to recondition existing base and surface in conformance with modified stabilized base course specifications at designated locations by scarifying, pulverizing, incorporating sodium chloride (furnished by the state f.o.b. state storage), additional aggregate, and water, mixing, compacting, reshaping and adjusting to uniform grade and cross-section, and applying a bituminous prime with cover chips.

Contracts listed a minimum number of individual pay items to complete the work. These items consisted of (a) cubic yards of aggregate to be furnished, hauled, and tailgate spread at a stipulated rate per mile; (b) square yards of surface reconditioned in conformance with base course specifications, including addition of sodium chloride at a prescribed rate per square yard per inch; (c) thousands of gallons of water, furnished and applied as directed; (d) gallons of bituminous material furnished and applied at a stipulated rate per square yard; (e) cubic yards of aggregate furnished, hauled and spread at a stipulated rate per square yard as cover; (f) lump sum premium on industrial insurance; and (g) lump sum for lights, signs, and barricades.

Operations started by scarifying existing surfaces to a depth sufficient to eliminate all surface irregularities and provide sufficient soil binder to consolidate finished mixture; specifications provided a minimum scarification depth of 4 in., but it was generally necessary to go deeper to obtain desired results. This material was then broken



Figure 1. Typical surface condition prior to stabilization.

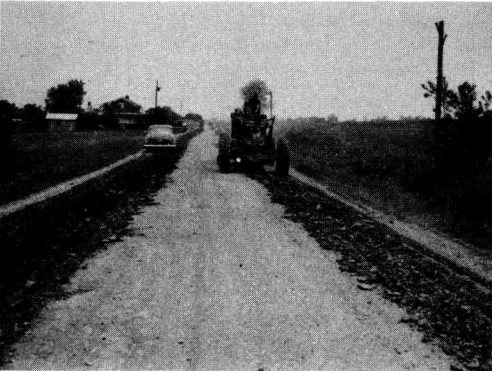


Figure 2. Scarifying existing surface.

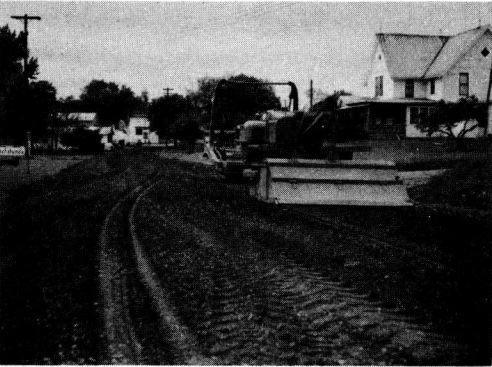


Figure 3. Pulverizing existing base material.



Figure 4. Existing base material windrowed to sides.

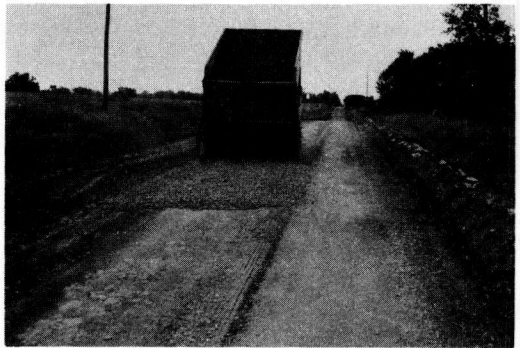


Figure 5. Spreading aggregate.



Figure 6. Mixing base materials and sodium chloride.

up and reduced to a uniform condition, with a maximum particle size of 2 in. Power driven rotary pulverizers were used for this work by most contractors with good results.

All loose material was then windrowed to the sides in preparation for spreading additional aggregate to specified widths. The base was sprinkled, if necessary, and aggregate was tail-gate spread at rates varying from 500 to 750 tons per mile; additional aggregate was usually crusher run or pit run stone, air cooled slag, or gravel, with a top size of 1 in. Some graded sizes were specified, but uniform and dense mixes were obtained with crusher or pit run. About one-half of the windrowed material was bladed on top of the new aggregate, and sodium chloride was applied at the rate of one-half pound per square inch of thickness of conditioned material. The remainder of the

windrow was placed over the sodium chloride, sprinkling was started, and materials on the base were mixed together. Power driven rotary type mixing machines, travel plants, or motor graders were permitted in the specifications; most of the mixing was done with graders, with materials being handled a minimum of three times on the mold board.

The addition of water was a critical job item, and good judgment on the part of the engineer was imperative. Specifications stipulated that the mixture should have "proper moisture content" for maximum compaction, and permitted the engineer to add water as required. Extreme caution was exercised by the contractors in controlling water content, because a "too wet" mix required

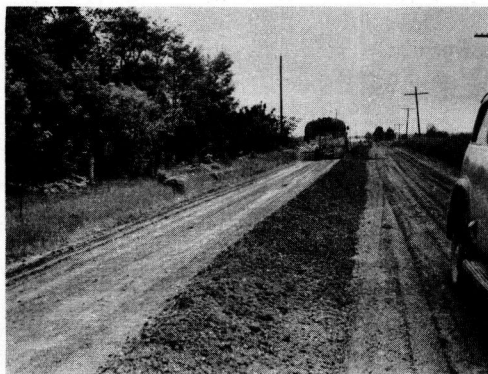


Figure 7. Applying water.

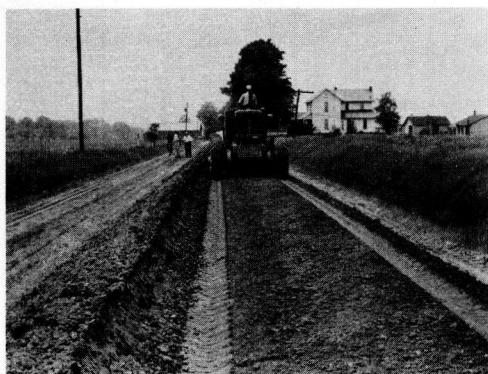


Figure 8. Mixing base material.

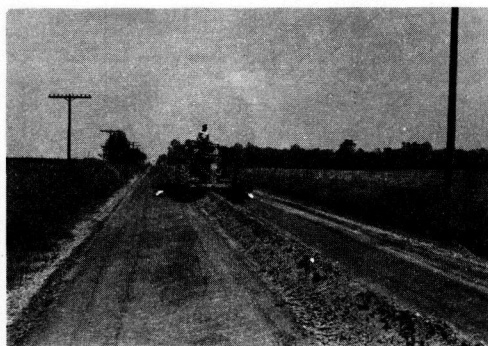


Figure 9. Spreading top course.

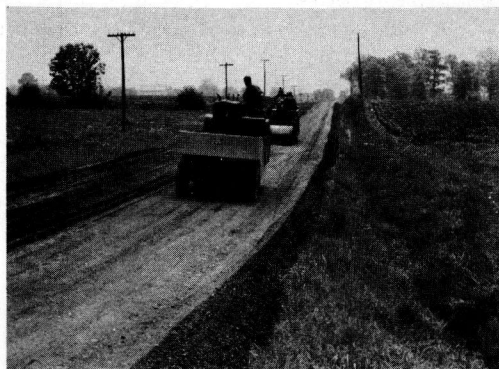


Figure 10. Rolling with tandem and pneumatic tired rollers.



Figure 11. Applying bituminous prime and cover chips.

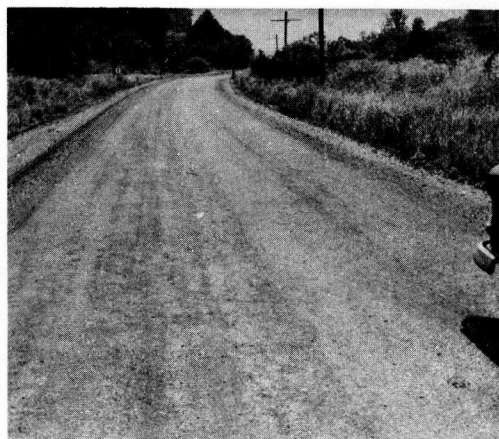


Figure 12. Finished surface.

drying out with resultant mixing delays. Material was windrowed and manipulated to permit continued traffic movement, and placed in uniform layers of about 3-in. depth, loose measurement. Each layer was compacted with tamping or pneumatic tired rollers, with final compaction obtained with a ten ton flat wheeled roller. The top layer was finished to a smooth contour and a crown of one-half inch per foot.

The road surface was lightly bladed and kept in repair until placement of the bituminous prime coat, which was specified not less than seven days after completion of reconditioning work. The prime coat consisted of an application of about one-third of a gallon per square yard of light viscous bituminous material and approximately ten pounds $\frac{1}{2}$ -in. top size chip cover. This completed the contractor's obligation and state forces assumed maintenance responsibility until bituminous seal treatments were placed.

Bituminous seals were sold as separate contracts, and applied as soon as possible after completion of the stabilization projects. Minor raveling occurred on these jobs, and some spot priming was required; traffic and weather located a few pockets of unsatisfactory material, and these were patched out prior to seal placement. Bituminous seals consisted of application of about three-tenths of a gallon per square yard of heavy viscous bituminous material, and 20 to 25 lb of cover aggregate at an average cost of \$900 per mile.

Recent inspections indicate that all projects are in good condition. Mixes are well consolidated and stable, and surfaces are tight and reasonably waterproof. The specifications covering this type of work, which permitted maximum field control, appear to be adequate, and the program, completed as it was prior to summer surface treatment operations, obtained good results at reasonable cost. Reconditioning existing surfaces by the addition of aggregate and sodium chloride, with a preservative bituminous prime and seal, has successfully stabilized 260 miles of the secondary system, and is believed to be a satisfactory low cost method of reducing maintenance costs and furnishing better traffic service to Ohio's rural highway user.

Effect of Density on Strength of Lime-Flyash Stabilized Soil

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The strength of an artificially cemented soil mass, such as soil-cement or soil-lime-flyash, is theoretically highly dependent on the intimacy of grain-to-grain contact. The controlling factor here should be degree of compaction. With this in mind, various soil-lime-flyash mixes were compacted at four different controlled densities and the specimens were moist cured at normal temperature and tested. Three soils were used: an Iowa silt (loess), a Kansas dune sand, and a Texas coastal plain clay. The lime was calcitic (high calcium) hydrated lime. Mixes were prepared with 25 percent lime-flyash and with different ratios of lime to flyash. Specimens were soaked in water and tested after 7 and 28 days.

Evaluation of the compressive strength data shows that density is indeed a highly important variable. Compaction to above standard Proctor density increased 7-day strengths on the average 100 percent and 28 day strengths, 70 percent. A higher compaction to modified Proctor density raised the average increases to 120 and 110 percent. Compaction to a super-modified Proctor increased the averages to 150 and 130 percent over strengths previously realized at standard Proctor density. It is concluded that density is not only important but that it may also be an economical consideration in design. The silt also showed influence from overcompaction, but the influence vanished on 28-day curing. The clay gave the best response to increased compaction, and strengths with modified Proctor density were approximately three times those obtained at standard Proctor. With modified density all soils showed 28-day strengths of the order of 600 to 1,000 psi with ordinary room temperature moist curing.

Attendant with this investigation was an evaluation of an optimum lime-flyash ratio. With most soils the ratio was not critical, but highest strengths were realized with a lime-flyash ratio of 1:9 or 2:8. A ratio of 1:9 was nearly a universal optimum for all three soils regardless of compactive effort.

● OBJECTIVES of this research were to study the effect of degree of compaction on the strength of lime-flyash-soil mixtures. Four compactive efforts were chosen: one to give densities equivalent to standard Proctor, one to give densities between standard and modified Proctor, one to duplicate modified Proctor, and one to give densities greater than modified Proctor (Table 1). A second objective was to determine the effect of a variable compactive effort on the selection of an optimum lime-flyash ratio.

MATERIALS

Soils

Three soils were selected for this study: a sand, a silt and a clay. The sand is from a stable dune area associated with the Arkansas River in south central Kansas. The silt is a friable, calcareous loess from the deep loess area in western Iowa. The clay is a deltaic deposit from the coastal plain region in Texas; it was sampled a few miles south of Houston. Field information on the three soil samples is in Table 2, and laboratory data are given in Table 3. ASTM procedures were followed for laboratory testing except where otherwise noted.

Lime and Flyash

The hydrated lime is a calcitic lime from the Linwood Stone Products Co., Buffalo, Iowa. A laboratory analysis furnished by the manufacturer is given in Table 4. The

flyash is a fine ash with low loss on ignition; it is from Paddy's Run Station, Louisville Gas and Electric Co., Louisville, Kentucky. Data by the Robert W. Hunt Co., Chicago, are given in Table 4.

METHODOLOGY

Correlation Study

Because of the advantage of small specimen size for rapid molding and testing, the 2-in. diameter by 2-in. high size was used in this study. The 2-in. height gives the advantage of molding in one layer, the compactive effort being applied at both

TABLE 1
DESIGNATIONS OF
COMPACTIVE EFFORT

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

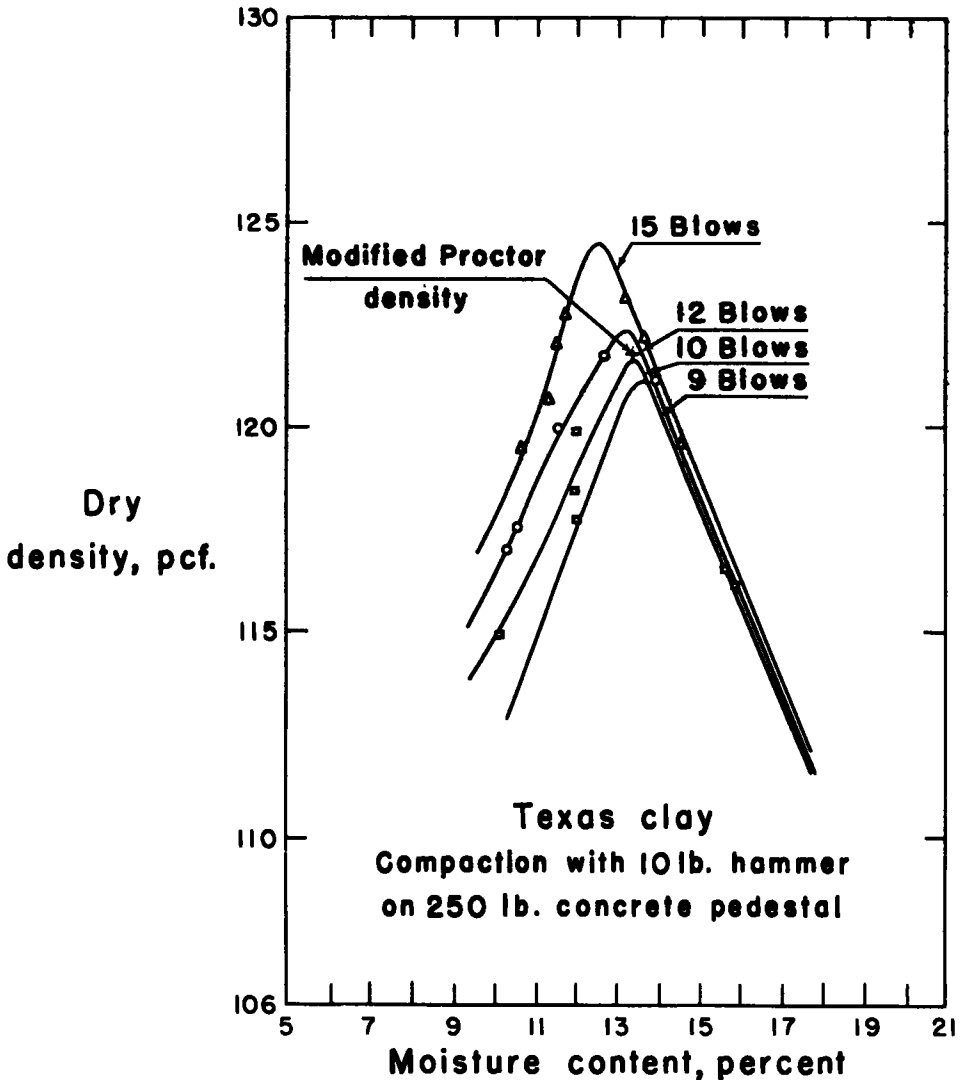


Figure 1. Typical moisture-density relationships from 2-in. x 2-in. specimens. Ten blows with a 10-lb drop hammer give a maximum density and optimum moisture content close to modified Proctor.

TABLE 2
FIELD INFORMATION ON SOIL SAMPLES

Sample:	Kansas Sand	Iowa Silt	Texas Clay
Geological origin:	Recent dune sand from the Great Bend tract	Wisconsin age loess from near Missouri River	Deltaic (Beaumont fm.) clay from coastal plain
Soil Series:	Pratt	Hamburg	Lake Charles
Horizon:	C	C	C
Location:	28 mi S of Great Bend	In the town of Missouri Valley	South of Houston
Sampling depth, ft:	1½ - 3½	49-50	3¼ - 12 (Composite)

TABLE 3
PROPERTIES OF SOIL SAMPLES

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

^aDispersed by air-jet with sodium metaphosphate dispersing agent.

^bFrom differential thermal analysis of fraction passing No. 200 sieve.

^cFraction passing No. 40 sieve.

^dFrom differential thermal analysis.

ends. Specimens were molded with a drop hammer molding apparatus, and extensive correlation work was done to determine the proper hammer weights and numbers of blows for standard and modified Proctor densities. Figure 1 gives a typical set of curves for one soil and one hammer weight. In this case modified Proctor density was approximated by 10 blows on each end of the 2-in. by 2-in. specimen. The closeness

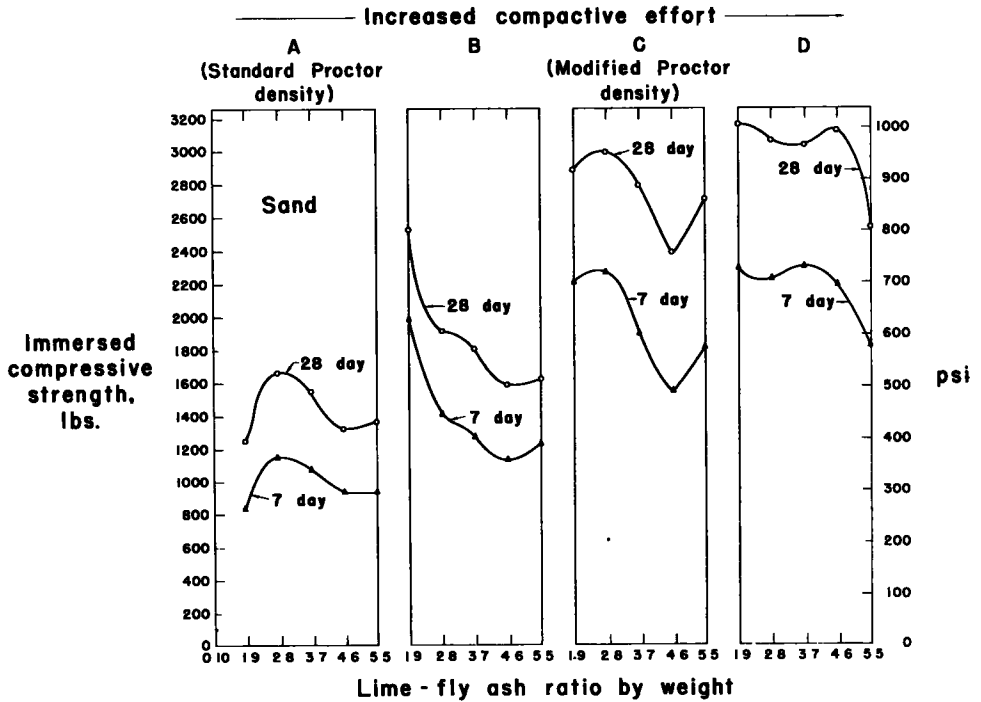


Figure 2. Effect of compactive effort on strength of Kansas sand stabilized with 25 percent lime-flyash in varying ratios.

TABLE 4
PROPERTIES OF LIME AND FLYASH

Material	Linwood Hydrated Lime	Louisville Flyash
Specific gravity	2.29	2.67
Fineness		
Passing No. 325 sieve, Percent	99.00	94.30
Specific surface, sq. cm./gm.		3,470
Chemical analysis, Percent		
Total Ca(OH) ₂	97.82	
Available Ca(OH) ₂	97.38	
MgO	0.49	0.52
CaCO ₃	0.77	8.36
Fe and Al oxides	0.82	Not determined
SiO ₂	0.80	38.90
Al ₂ O ₃		22.92
SO ₃		2.0
Free water		0.17
Loss on ignition	24.56	2.10

to which modified Proctor density can be duplicated with different soils is illustrated in Table 5. These results were obtained with 20 blows (10 on each end) with a 10-lb hammer dropping a distance of 1 ft, the molding apparatus being mounted on a concrete pedestal. Other compactive efforts used are B (Table 1) obtained by 10 blows with the same arrangement and D obtained with 30 blows. Standard Proctor density (density A) was duplicated by 10 blows from a 5-lb hammer falling 1 ft, the apparatus resting on a wooden bench.

Constants and Variables

To reduce the number of variables, a constant percentage of lime-flyash was used in all tests, the lime plus flyash making up 25 percent of the dry weight of the mixtures. Previous work has shown that 25 percent is both a satisfactory and an economical content (1).

Compaction is the major variable, as previously discussed. The moisture contents

TABLE 5
COMPARISON OF MODIFIED PROCTOR DENSITIES

Sample:	Kansas sand	Iowa silt	Texas clay
ASTM test:			
Maximum dry density, pcf	128.1	121.8	118.8
Optimum moisture content, percent	9.2	13.2	13.8
2-in. x 2-in. test:			
Maximum dry density, pcf	128.9	122.0	118.9
Optimum moisture content, percent	9.3	13.3	13.7

were adjusted to the optimums for each mixture and for each compactive effort. The optimum moisture contents of mixtures with different ratios of lime to flyash were read from a triangular chart in which optimum moisture contents of soil, of 75/25 soil-flyash, and of 75/25 soil-lime are plotted at corners of the triangle and intermediate values are found by interpolation (1, p. 81).

The second major variable is ratio of lime to flyash. Testing was continued at each of the four compactive efforts to show any change in optimum ratio. The previously found optimums with these soils has been between 1:9 and 2:8 by weight of lime to flyash. In the present study specimens were molded with ratios 0:10, 1:9, 2:8, 3:7, 4:6, and 5:5.

A third variable was age. Strengths were measured after 7 and after 28 days moist curing.

Curing and Testing

Curing was done at 70 ± 3 F and with a relative humidity near 90 percent. Specimens were not wrapped, as is sometimes done to exclude carbon dioxide from the air. After curing the specified time, specimens were immersed in distilled water at 70 F for 24 hours, then removed and tested for unconfined compressive strength. The rate of strain was 0.05 in. per min per in. of specimen height. Results are expressed in pounds; if the height-diameter ratio is neglected results can be converted to pounds per square inch by dividing by 3.14.

Other measurements include absorption and volume change during curing and immersion.

RESULTS

Results are plotted in Figures 2, 3, and 4. In most cases curves are displaced upward by increased compactive effort, and 7-day strengths were on the average about 100 percent higher with compactive effort B than at standard Proctor density A. Compaction to modified Proctor density raised this to 120 percent, and compaction to beyond

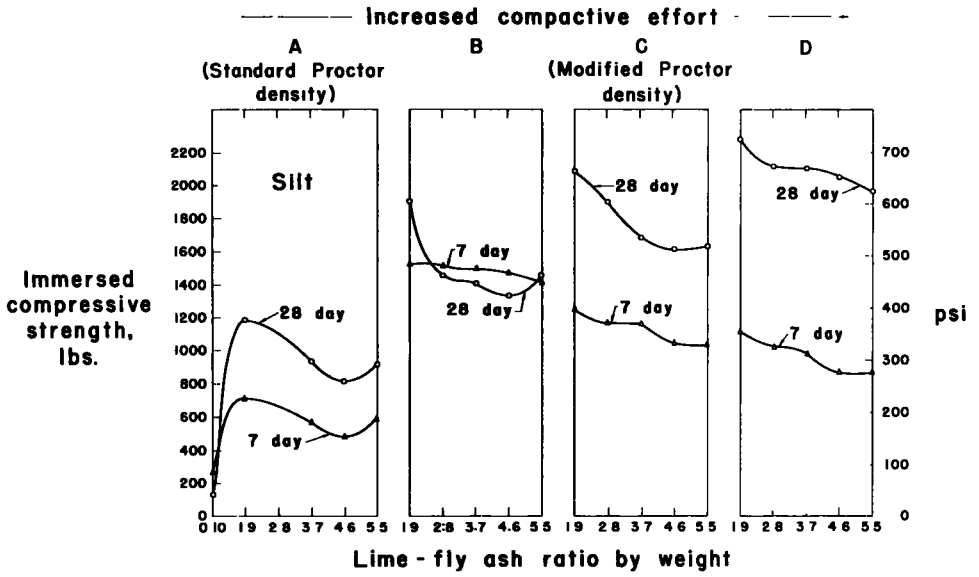


Figure 3. Effect of compactive effort on strength of Iowa silt (loess) stabilized with 25 percent lime-flyash in varying ratios.

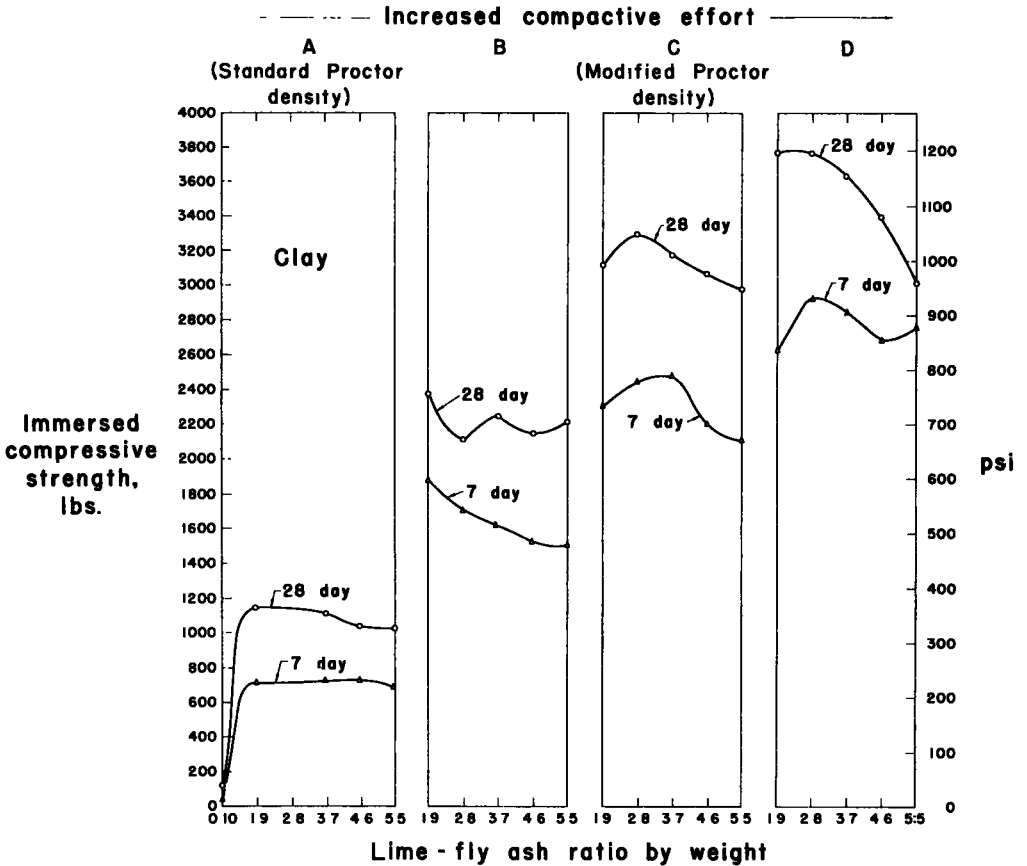


Figure 4. Effect of compactive effort on strength of Texas clay stabilized with 25 percent lime-flyash in varying ratios.

modified Proctor gave on the average a 150 percent increase in 7-day strength. Twenty-eight-day strengths reflect the same trends.

Density and Percent Solids

Density is of course dependent on compactive effort, but density also depends on lime-flyash ratio. Density is decreased by higher contents of lime because of two factors: the lime itself is less dense than soil or flyash, and lime causes aggregation of clay. The first factor is calculable and can be corrected by converting measured densities to percent solids by volume. This has been done in Figures 5, 6, and 7. In these

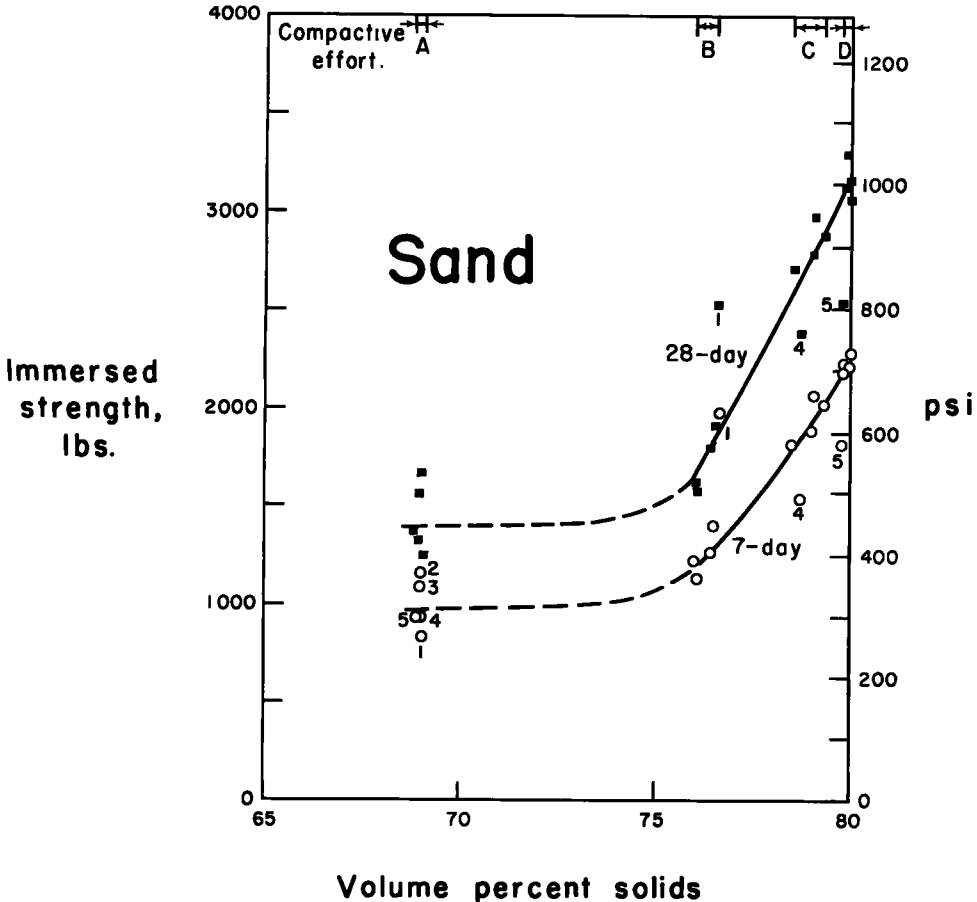


Figure 5. Relation of strength to percent solids in lime-flyash stabilized Kansas sand. figures compressive strength has been plotted against percent solids, irrespective of the lime-flyash ratio. The fact that in most cases smooth curves are obtained indicates that the lime-flyash ratio is not critical.

In the Texas clay (Fig. 7) the influence of clay aggregation on density is found to be a maximum. As an example, points labeled 1, 2, 3, 4, 5 under compactive effort A are with lime-flyash ratios of 1:10, 2:8, 3:7, 4:6 and 5:5. A higher lime content decreases the volume percent solids. With higher compactive efforts the same trend is found, but the range in percent solids shown at the top of the graph is less for B, C and D, indicating that higher effort may break down the clay aggregates and better their compaction. This tendency is particularly pronounced with the silt (Fig. 6), in which with effort A there is a wide range in percent solids depending on the lime content. The range is progressively smaller with efforts B, C and D. It is believed that the silty soil aggregates may have less strength than those formed in the clay soil and are thus easier to break down.

The sand (Fig. 5) offers a direct contrast to this. With low compactive effort addition of more lime has practically no effect on the percent solids, as shown by the narrow horizontal range in points under A. With higher compactive efforts the range is greater, as in B and C, but the range is greatly reduced, with D the highest compactive effort. Although the reason for this is not known, it is suspected that lower lime and higher flyash contents improve the gradation of the sand for compaction.

Unconfined Compressive Strength for Evaluation of Stabilized Soils

Unconfined compressive strength is primarily influenced by cementation and does not give a true measure of the frictional strength developed in a confined state. There-

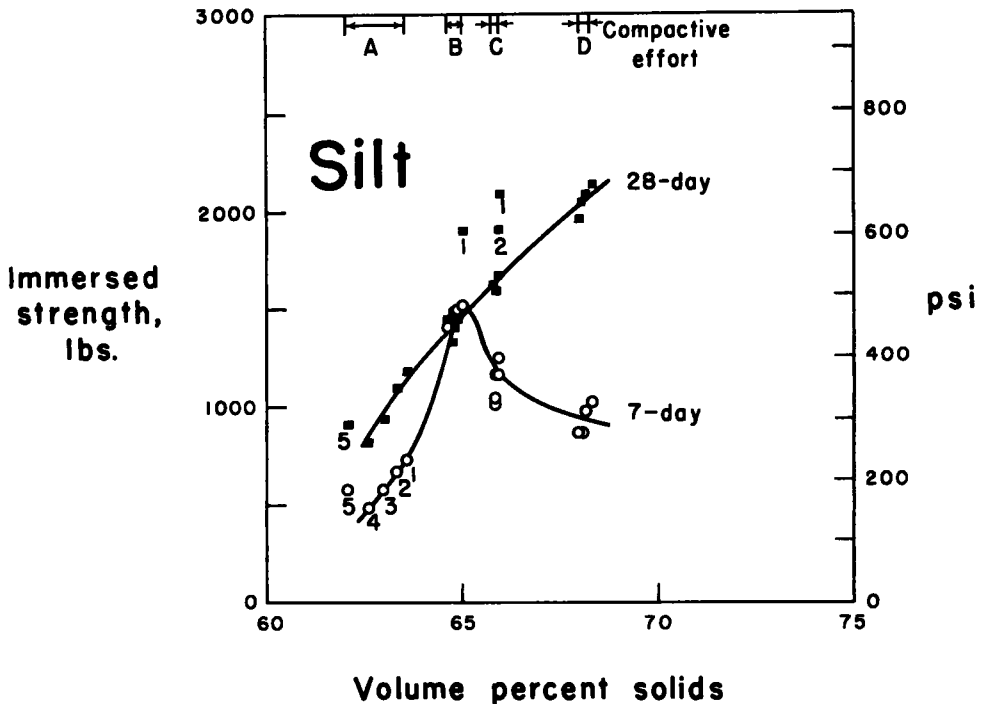


Figure 6. Relation of strength to percent solids in lime-flyash stabilized Iowa silt.

fore, a stabilized granular material with relatively low unconfined compressive strength may show satisfactory stability. It is known that for a given stabilized soil the CBR values are directly proportional to unconfined compressive strength (2), and it has been found that for example a lime-flyash stabilized sand with an unconfined compressive strength of 138 psi has a CBR of 213, while a stabilized clay must have an unconfined compressive strength of 705 psi to develop the same CBR (3).

Strength vs Percent Solids

Figures 5, 6 and 7 show the relationships between strength and percent solids. Points have been plotted without regard to lime-flyash ratio, and the striking feature is that most of the points fall on or very close to the curves. The exceptions are numbered to indicate their ratios, which are either very low (1:10 or 2:8) or very high (5:5).

Curves for the different soils show a similarity in that strength is approximately proportional to percent solids, and the proportionality factor indicated by the slope of the curve is much the same. An exception is noted in the case of sand, where the strength gain between efforts A and B is not nearly in accord with the increase in percent solids. Apparently cementation of the sand is not greatly improved until compaction reaches above a critical percent solids, in this case about 75 percent. For some

reason a critical degree of packing is necessary before grain contact and cementation are improved.

Overcompaction and Tendency to Heal

In one curve (Fig. 6), strength loss from overcompaction is evident. This is the 7-day strength curve for the silt. The same trend can be seen in Figure 3, where the 7-day curves in B, C and D are progressively lower even though density is increased and absorption and volume change are reduced by the greater compaction. It is believed, therefore, that strength loss may be due to shearing displacements in the specimen causing intrinsic planes of weakness.

Particularly significant is that at 28 days the strength curve follows a normal pattern, and the shear planes, if they existed, have apparently healed. Such a tendency for healing of overcompaction failure planes could be of considerable importance in field construction. Presumably continued intimate contact would be necessary for failure plane healing.

Optimum Lime-Flyash Ratio

Previous work has shown that for highest strength with standard Proctor compaction the optimum lime-flyash ratio (1) is usually in the range 1:9 to 2:8.¹ In the present study,

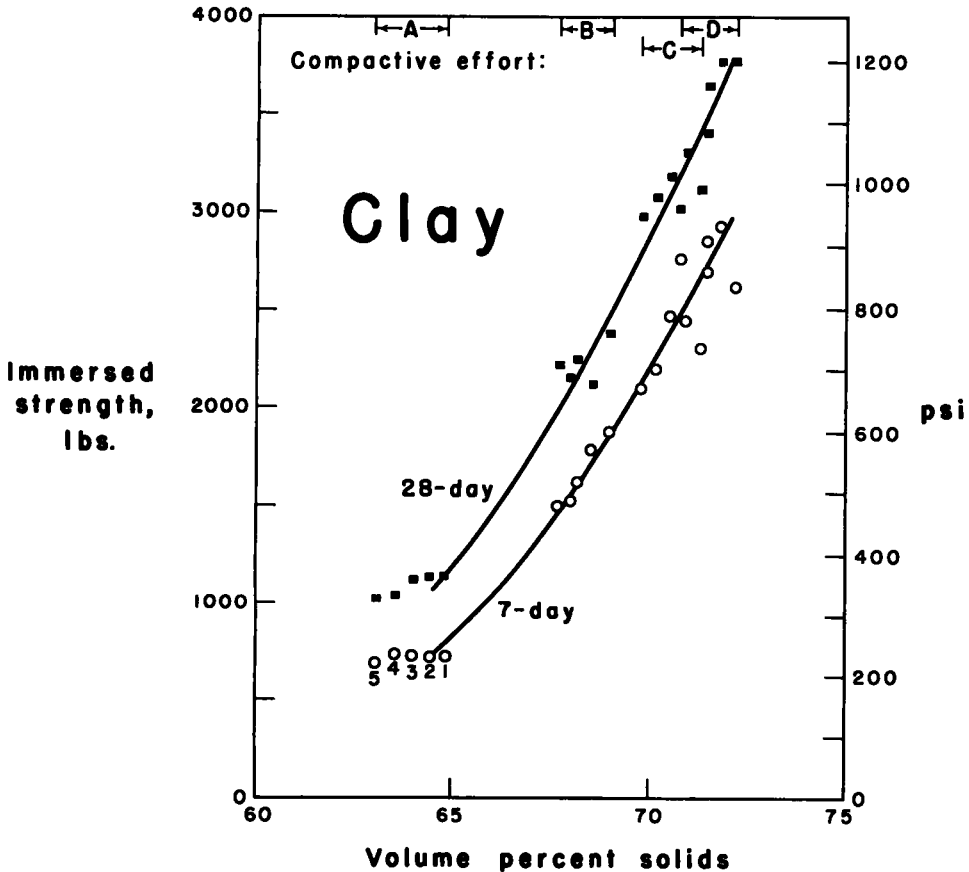


Figure 7. Relation of strength to percent solids in lime-flyash stabilized Texas clay. increased compaction does not greatly or consistently change the optimum ratio (Fig. 2, 3 and 4). From an economic standpoint a low ratio is desirable, since the cost of

¹An exception was noted for an halloysitic clay, which requires more lime.

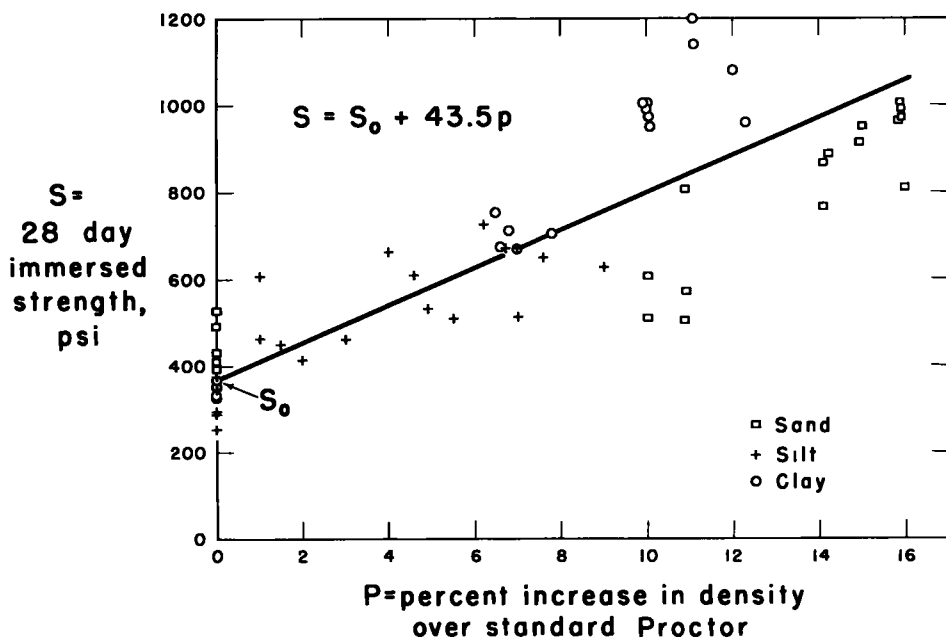


Figure 8. Average effect of increase in density on compressive strengths of a sand, a silt, and a clay.

flyash is usually a fraction of the cost of lime. A limit is imposed by difficulties in mixing and securing uniform distribution of very small percentages of lime. Experience has shown that ratios of 1:9 and 2:8 can be successfully handled in construction with a wide variety of soil textural types.

SUMMARY

A summary of the relation between compaction and compressive strength is illustrated in Figure 8. The curve represents an average for all three soils; the scatter of points is greater than in Figures 5, 6 and 7 because of disregard of other variables such as soil type and the dependence of density on percent lime. The average increase in compressive strength is 43.5 p, where p is the percent increase in density over standard Proctor. That is, $S = S_0 + 43.5 p$, where S_0 is the strength in psi at standard Proctor density. On the average a 10 percent increase in density will about double the unconfined compressive strength. This density is approximately equivalent to modified Proctor for the sand and the clay; because of poor gradation it is not readily obtainable with the silt.

CONCLUSIONS

1. Strengths of lime-flyash stabilized soil after 7 and 28 days are greatly increased by increased density and compaction, but the optimum lime-flyash ratio is little influenced. The optimum ratio for these soils remained 1:9 or 2:8.
2. Increasing the additions of lime to the clay and silt soils results in a decreasing percent solids with the same compactive effort, probably because of clay aggregation by lime. The resulting decrease in strength is approximately proportional to the decreasing percent solids. This relationship was not found in the case of the sand.
3. Lowered strengths of stabilized silt due to overcompaction were evident after 7-days curing, but at 28 days the influence had vanished. It is concluded that overcompaction shear planes in lime-flyash-soil tend to heal on long curing.

ACKNOWLEDGMENT

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The authors wish to acknowledge the help of W. L. Goecker, former Graduate Assistant, and other members of the Station's Soil Research Laboratory staff.

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The Strength Characteristics of Soil-Aggregate Mixtures

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● THE PROBLEM of improving the physical properties of soil, particularly the strength, is growing in importance because of the rapid increase in wheel loads of trucks and of aircraft. Although many methods of soil improvement or stabilization have been devised, the combining of different soils to produce a mixture which is superior to any of its components is probably the most attractive since it utilizes the cheapest ingredients—soils. Although mixing (termed mechanical stabilization) has been widely employed in highway construction, there is little known about the mechanism by which the stabilization takes place. Much of the research on mechanical stabilization has been directed toward the development of standard specifications for the mixtures. Unfortunately the performance of soil mixtures designed in accordance with the specifications is sometimes not satisfactory, indicating that further study is necessary. It was the purpose of this research to investigate the effects of varying the proportions of coarse and fine-grained soils on the strength of the resulting mix.

THEORIES OF PROPORTIONING SOIL MIXTURES

What constitutes the best mixture of soils to form a stabilized material obviously depends on what soil property is considered to be the most important. For subgrades, strength is the most important property with incompressibility a close second. For large fills incompressibility is the most important with strength a close second.

Both of these vital properties are influenced by soil density. Field experience and laboratory tests show that the strength of a soil is increased (up to a point) and the compressibility is decreased by an increase in density. As a corollary to this, it is often argued that when two different soils are compacted by the same method, the denser will be the stronger and the less compressible. On this basis, therefore, the determination of the best soil mix for mechanical stabilization resolves itself into the determinations of the mix which yields the greatest compacted density. At first glance this appears to be logical, but a careful consideration shows that the two conditions are not quite comparable. In spite of this inconsistency, most research on mechanical stabilization has been directed toward obtaining the densest possible mixture.

Two different approaches have been followed in obtaining the maximum density. One involves the ideal gradation concept. This is illustrated in the following manner: A quantity of the largest particles are arranged in their most dense state; and the next largest grains added are those that will just fit in the voids between the largest grains. Each succeeding smaller size is that which will fit into the voids between the next larger grains. If this process is continued to infinity, the results will be a solid mass. For any given shape of particle the grain size curve of the ideal gradation will follow a definite mathematical progression, extending to infinity. This approach has been used in the study of concrete aggregate gradations (1, 2, 3) but the mathematical curves developed from these studies have had serious shortcomings in soil engineering. The ideal gradation concept becomes hopelessly complex when widely different particle shapes are involved, such as bulky quartz grains and flakey mica. The grain size distribution curves which are produced by mixing different soils are usually quite irregular, regardless of the proportions. Therefore, re-sorting of the soils would be required to achieve anything approaching an ideal mix. The gradation concept does not take into account the absorbed water of the clays which renders the ideal gradation of these particles meaningless.

Another approach is the aggregate-binder concept. The soil is considered to be made up of two components: the aggregate, composed of the larger grains; and the

binder, consisting of the finer grains and the clay. The aggregate should be compacted to its densest state. The voids between the grains are filled with compacted binder, to produce the maximum density. This approach was first suggested by Macadam's method of pavement construction, but it has been extended by more recent research (4, 5). In soil work, there are serious objections in that the division of a soil into aggregate

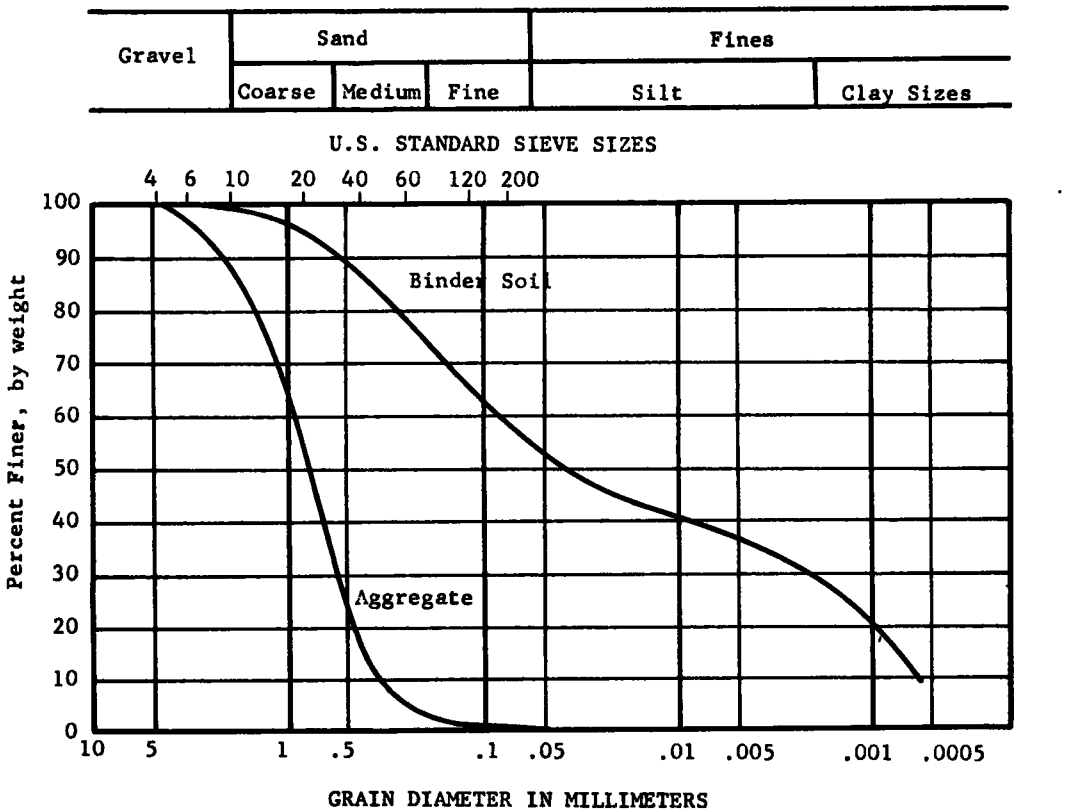


Figure 1. Grain size distribution for aggregate and binder soil.

and binder must be arbitrary and artificial, and, although it is not too difficult to compact the aggregate alone, there is no way at the present time to introduce compacted binder into the voids of the compacted aggregate.

Because of the shortcomings of both approaches, engineers have fallen back on empirical rules guided by these theoretical concepts but based on experience and tests. Examples of the rules based on the gradation concept are the standard specifications for mechanically stabilized soils adopted by the ASTM (6) and by many highway departments. Supplementary rules for proportioning mixes have been proposed (7, 8, 9); similar rules involving binder and aggregate proportions have been proposed (10). Particular attention has been paid to the quality of the binder component and various methods for control of the binder quality have been advocated (11, 12).

Some work has been done to evaluate the factors which control the penetration resistance (5) of soil aggregate mixtures. However, little research has been devoted to the components of the strength of the soil: the "cohesion" and the angle of shear resistance (or angle of internal friction). It was the purpose of this research to study the effect of soil proportioning on these components of strength. In doing so, the aggregate-binder concept was employed, since it more closely represents the way in which two component soils are handled in the field than does the ideal gradation concept.

EXPERIMENTAL PROCEDURE

Two natural soils, such as would be used in mechanical stabilization, were obtained for this study. The first was a coarse to medium grained angular to sub-angular river sand from Cartersville, Georgia. The grains were largely quartz, but because of small percentages of barite grains and mica the specific gravity of the solids was 2.71 instead of the customary 2.66. The second soil was a low plasticity inorganic sandy clay. It came from the B-horizon of a residual soil derived from the weathering of the gneiss bed rock of the Atlanta area. The grain size and plasticity properties of these soils are given in Figure 1.

Each of the soils was prepared for compaction testing according to ASTM Method D698-42T. Various mixtures of the two soils were made ranging from 100 percent sand (aggregate) to 100 percent clay (binder) with the greatest number in the range from 0 to 50 percent binder. A compaction test was run on each mixture using ASTM Method D698-42T except separate portions of soil were used for each determination of moisture and density. The maximum densities and the corresponding optimum moistures are shown in Figure 2.

Additional samples were prepared at the optimum moisture for each mixture and the samples were compacted to the maximum density. Portions of each sample were trimmed into 1.4-in. diameter cylinders which then were subjected to triaxial shear tests. Confining pressures of 5, 15 and 30 lb per sq in. were employed. The samples were loaded axially immediately after the confining pressures were applied, so that the entire testing of each portion required less than 10 min. Mohr's circles were drawn from the test data and Mohr's envelopes plotted tangent to them. The envelopes for all the tests were found to be essentially straight lines. The intercept on the vertical axis is the "cohesion" and the angle the envelope makes with the horizontal axis is the angle of shear resistance or the angle of internal friction. The values for these two parameters of the soil strength are shown in Figure 3.

RESULTS

The relationship between maximum density and the proportions of aggregate and binder (Fig. 2) indicates that the highest density is produced by 26 percent binder and 74 percent aggregate by weight. The maximum density is greater than that of either the aggregate or the binder, and it is about 12 lb per cu ft heavier than their average. If the volume of the voids in the compacted aggregate is assumed to be just filled with compacted binder the resulting proportion would be 22 percent binder and 78 percent aggregate. The maximum density of this theoretical mix would be 148 lb per cu ft which is far greater than the greatest observed maximum density of the mixtures. These facts indicate that the aggregate-binder concept is not strictly correct. Further light is shed on the mechanism by the relationship of binder compaction to the proportions of aggregate and binder (Fig. 2). The first part of the curve, from 0 to 12 percent binder, shows a rapid linear increase in binder compaction from 50 percent to 82 percent. Since 82 percent compaction is approximately the density which is obtained by merely dumping the moist binder into a container, it appears logical that this part of the curve (from 0 to 12 percent) represents the filling of the voids with the loose binder.

From 12 percent to 26 percent binder, the curve of binder compaction is flatter but also linear. This appears to represent actual compaction of the binder rather than the mere filling of open voids in the aggregate. At 26 percent binder, the binder soil is almost 100 percent compacted. Probably the aggregate particles, by creating hard spots that bridge over the looser matrix of binder, prevent 100 percent compaction.

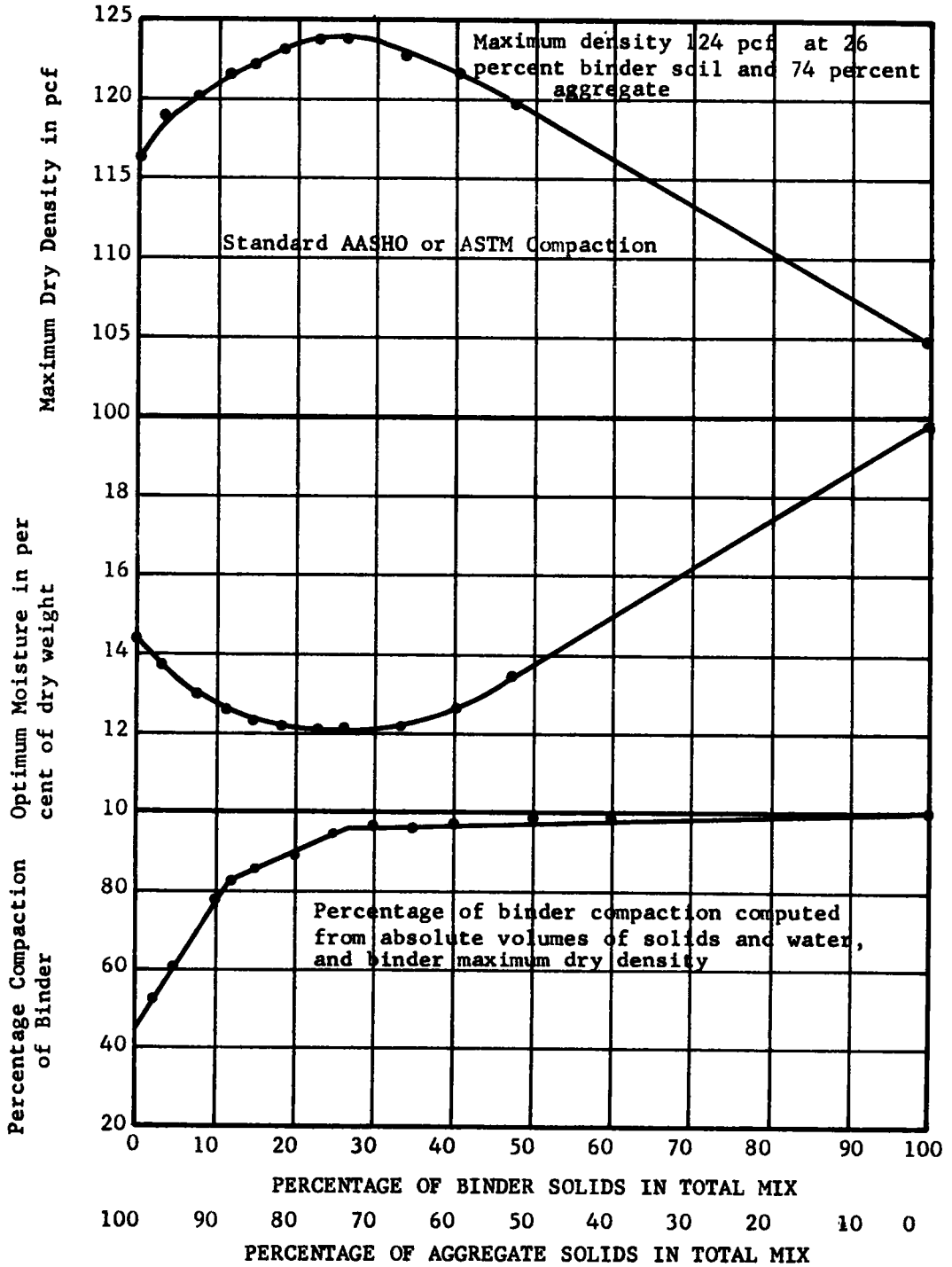


Figure 2. Maximum density, optimum moisture, and percentage of compaction of binder for various proportions of binder soil and aggregate.

Beyond this point the binder compaction increased slowly, as expected.

The curves of cohesion and internal friction shed further light on the binder-aggregate behavior. The aggregate alone has a high angle of internal friction but no cohesion. The addition of a small amount of binder produces a sharp drop in the angle of friction and a rapid increase in cohesion. This indicates that some of the binder is trapped between some of the aggregate particles, preventing aggregate to aggregate contact. As the amount of binder increases from 10 percent to 26 percent the cohesion increased but at a decreasing rate. This reflects increasing binder compaction and a greater degree of void filling by the binder. The changes in curvature occur at about the same point as the changes in the percent binder compaction curve, as would be expected. The internal friction in the range from 10 to 26 percent binder decreases slightly with increasing binder, showing that there is little additional soil trapped between the aggregate particles.

Both the cohesion and the internal friction change rapidly in going from 26 percent to 33 percent binder. The internal friction drops to that of the binder alone, while the cohesion increases to nearly $\frac{3}{4}$ of that of the binder alone. The change begins at the same point the binder compaction curve reaches 97 percent compaction and breaks sharply. At this point the aggregate particles begin to be surrounded by compacted binder; beyond 33 percent binder the aggregate particles float in the compacted binder. There is no change in the internal friction beyond this point. The cohesion increases gradually, with increasing binder, since the total cohesive force across any plane of shear increases when the volume of aggregate decreases.

Figure 4 shows the hypothetical grain structure for different amounts of binder and aggregate, based on the observed binder compaction, cohesion, and internal friction curves of the soil mixes tested. Figure 4a represents compacted aggregate alone. Figure 4b shows the structure with up to 10 or 12 percent binder. Part of the binder is highly compacted between the contact points of the aggregate while the loose remainder partially fills the aggregate voids. Some of the aggregate particles still make direct contact, maintaining their friction. In Figure 4c (12 to 26 percent binder) there is highly compacted binder between the contact points of the aggregate and partially compacted binder filling the voids. Figure 4d illustrates the aggregate floating in a matrix of compacted binder, when the binder exceeds about 33 percent. Between 26 and 33 percent there is a transition with the amount of highly compacted binder between the contact points decreasing and the degree of compaction in the voids increasing.

Because the cohesion increases and the internal friction decreases with increasing binder content, it is not immediately apparent what proportions produced the greatest soil strength. Since the strength of such soils depends on the degree of confinement, it was assumed that the soil was employed as a subgrade beneath a 10-in. thick pavement. The bearing capacity of the subgrade was computed on the basis of a uniformly loaded circular area having a 10-in. diameter (similar to a rubber tire) on the pavement surface. The tire load was assumed to spread out through the pavement as if the pavement formed a truncated cone with its sides sloping 2 (vertical) to 1 (horizontal). The bearing in kips per square foot was computed by the formula (13):

$$qc = \frac{\gamma d}{2} \tan^5 \left(\frac{45 + \phi}{2} \right) + q' \tan^4 \left(\frac{45 + \phi}{2} \right) + 2c \left[\tan \left(\frac{45 + \phi}{2} \right) + \tan^3 \left(\frac{45 + \phi}{2} \right) \right]$$

in which

- d = diameter of the area;
- γ = weight per cubic foot of soil;
- q' = pavement weight;
- c = cohesion; and
- ϕ = angle of internal friction.

The results of the computations are shown in Figure 3c.

The bearing increases with increasing binder up to 26 percent. The increase is rapid at first but the curve levels off above a binder percentage of 10 or 12 percent. This is the same point at which the percentage of compaction curve changes slope.

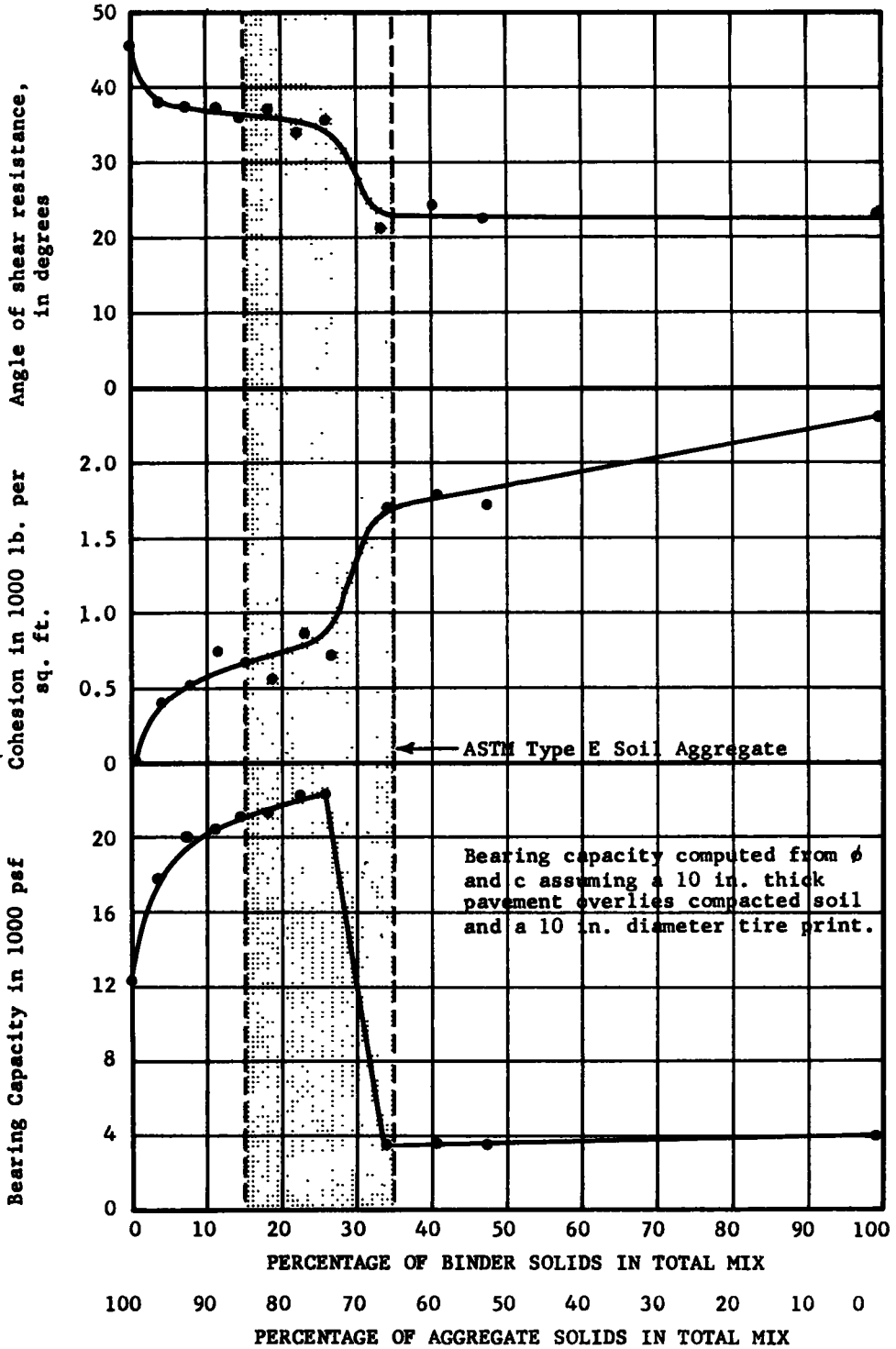
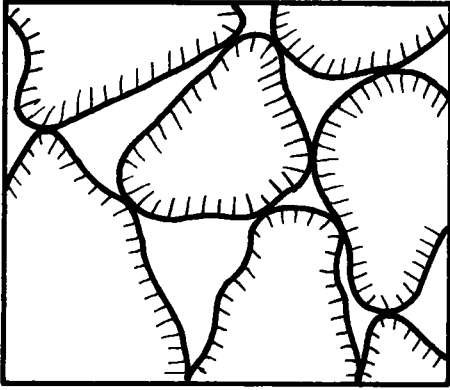
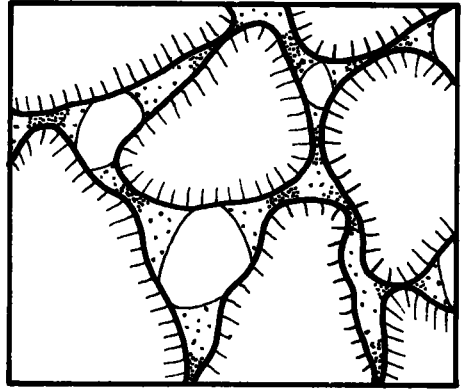


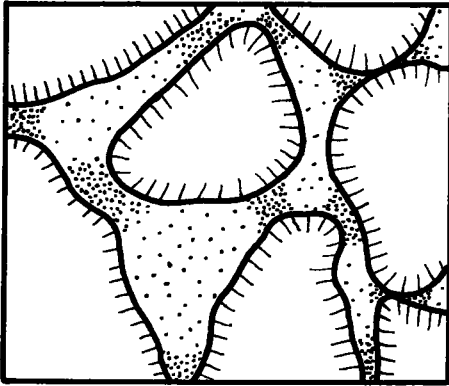
Figure 3. Strength and computed bearing capacity of soil-aggregate mixtures for various proportions of the binder soil and aggregate.



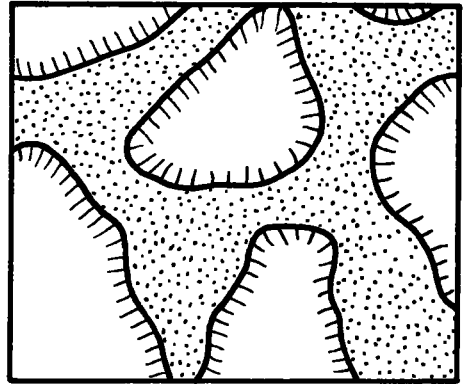
a. Compacted aggregate alone with grain to grain contact and high internal friction.



b. Aggregate with small amount of binder. Binder highly compacted between contact points of aggregate, and loose in voids. Some open voids and grain to grain contact persists.



c. Aggregate with sufficient binder to fill voids loosely. Binder highly compacted between contact points of aggregate, loose in between.



d. Aggregate floating in a matrix of uniformly well compacted binder.



Loose Binder
(Less than 80%
Compaction)



Well compacted
binder (95-100%
Compaction)



Highly compacted
binder (over 100%
Compaction)

Figure 4. Grain structure of soil aggregate mixtures.

When the amount of binder exceeds 26 percent, the bearing falls off very sharply to a minimum at 33 percent binder. With more binder the strength increases slightly but it never approaches the peak value. The results of these computations show that the customary assumption that maximum density produces greatest strength is justified in this case. A slight increase in binder beyond that which produces maximum density does not produce a significant decrease in density but it does produce a very marked drop in strength. A decrease in binder below that required for the maximum results in only a small decrease in strength, however. Mix proportions which would satisfy the ASTM requirements for granular stabilization were found from grain size analyses

to range from 15 to 35 percent binder. These limits are shown in Figure 3. The proportions for maximum strength and density lie within this specified range. However, the sharp drop in bearing also occurs within the range, which indicates that the specifications could result in both a very strong soil and a weak one.

RECOMMENDATIONS

On the basis of these tests a simple method of determining the best mix for bearing capacity is proposed. The aggregate alone and the binder alone are compacted. The weight of compacted binder required to fill the aggregate voids is computed. The actual amount of binder specified could vary from 50 percent of the computed value to 100 percent of the value without a significant change in soil bearing.

The conclusions reached in this investigation were based on only two soils. Although these were representative materials, additional research is certainly necessary to establish the applicability of this research to other materials.

ACKNOWLEDGMENT

This investigation was conducted by E.A. Miller in the Soil Mechanics Laboratory, School of Civil Engineering, Georgia Institute of Technology, under the direction of Professor Sowers. It is part of the continuing program of basic research in soil compaction of the laboratory.

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Discussion

EUGENE Y. HUANG, Research Assistant Professor of Civil Engineering, University of Illinois—The effect of soil proportioning on the strength components of the resulting mixture has been realized for a good many years. However, it has rarely been corroborated by quantitative data such as those presented in this paper. The paper is an interesting and valuable addition to the literature of mechanical soil stabilization.

Although the data in Figure 3 indicate that the strength components of the soil-aggregate mixtures are definitely affected by the proportions of binder soil and aggregate, material proportioning must never be regarded as the only factor that affects the strength characteristics of a soil-aggregate mixture. Internal friction is the resistance of soil grains to sliding on any plane through the material by the interlocking or mutual support of adjacent particles; cohesion is the resistance of soil grains to displacement by the bond developed at the surfaces of contact of very fine-grained soils as a result of electro-chemical forces of attraction. It may be said that internal friction is primarily contributed by aggregate and cohesion is primarily contributed by clay, thus an increase in the amount of each constituent generally results in an increase in its corresponding strength component; it must be realized, however, that the resulting effect is attributable to a number of factors.

Internal friction largely depends upon angularity, shape, surface texture, and size of particles; gradation and density of the mixture; and the amount of pressure exerted on the sliding plane. Cohesion is largely dependent upon the kind and relative abundance of clay minerals and the moisture content. Because some of these factors are more or less correlated with each other, the pure influence of these variables on the strength characteristics of soil-aggregate mixtures can only be determined by a system of polyfactor analysis based on the data of a variety of mixtures.

Of particular importance to the strength of a soil-aggregate mixture, from the practical standpoint, is the field moisture condition of the mixture. Both cohesion and internal friction are affected by the moisture content and are well preserved only when a mixture is relatively dry. As the moisture content increases, the adsorbed moisture films between fine soil grains will be thickened and cohesion will decrease. The increase in moisture content will also cause volume change in the binder soil which will expand the solid framework and decrease the mechanical contact between soil particles. Thus the interlocking of granular particles and the mutual support so important to the internal friction is readily eliminated. With these soil properties in mind, the proportioning of materials must aim at combining the high internal friction of the aggregate with the beneficial cohesion of the binder soil in such proportions as to avoid detrimental characteristics of absorbed moisture. When a binder soil is incorporated in an aggregate for the purpose of supplying cohesion to stabilized the otherwise loose mass, the amount of the binder soil must be determined not only by the cohesive property but also by the swelling property of the soil, taking into consideration local climatic and drainage conditions. A mixture should never be considered satisfactory without due regard to the field moisture conditions.

To determine what proportions of aggregate and binder soil would produce the greatest soil strength the authors have computed the bearing capacities of various mixtures, assuming they are employed as subgrade materials beneath a 10-in. pavement (Fig. 3). Based on values of cohesion and angle of internal friction, the writer has estimated the bearing capacities of seven soil-aggregate mixtures using the following formula:

$$q_d = \frac{1}{2} k d \gamma (N_\phi^2 - 1) + q' N_\phi^2 + 2c \sqrt{N_\phi} (N_\phi + 1)$$

In this formula, q_d is the ultimate bearing capacity, or the maximum pressure that can be sustained by a soil-aggregate subgrade; q' is the surcharge pressure due to the weight of a 10-in. thick pavement and is assumed to be 115 lb per sq ft; γ is the unit weight of the soil-aggregate mixture, c is the cohesion shearing resistance of the mixture; and N_ϕ , commonly known as flow value, is dependent upon the angle of internal friction ϕ of the mixture, and is equal to $\tan^2 (45^\circ + \phi/2)$. The factor k is a rough measure of the depth of the material involved and is assumed to be $\sqrt{N_\phi}$ in all cases.

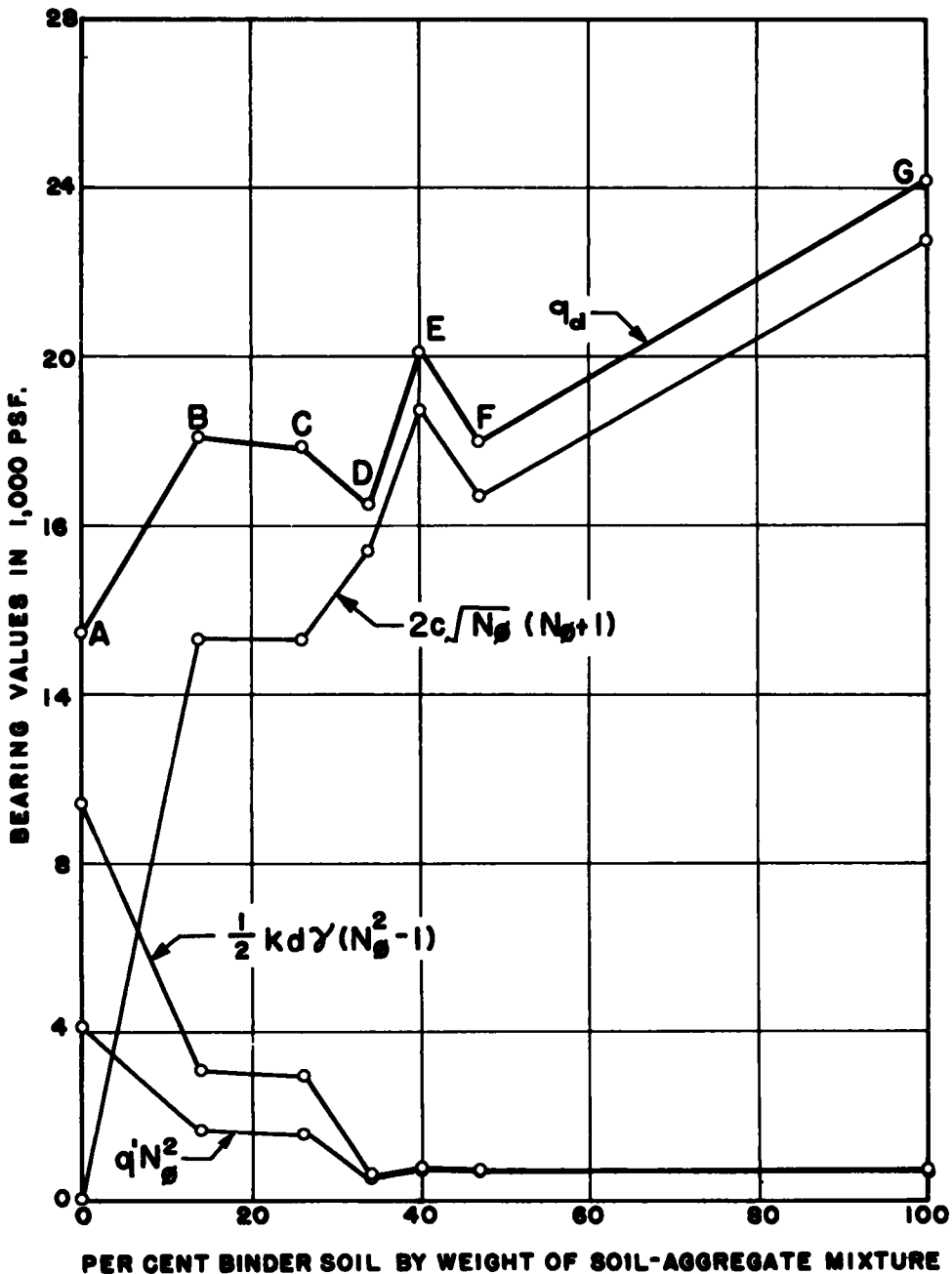


Figure 5. Computed bearing values of soil-aggregate mixtures.

Following loading conditions assumed by the authors, the diameter of the circular bearing area, d , is 1.67 ft.

The formula affords nothing except an approximate method for evaluating the influence on bearing capacity of internal friction and cohesion. It is derived, according to the principles of statics, originally for a rough computation of the bearing capacity of continuous footings, assuming that the subgrade material under load acts in compression similar to a specimen in a triaxial shear test. It may be noted that the formula is almost exactly like the one used by the authors.

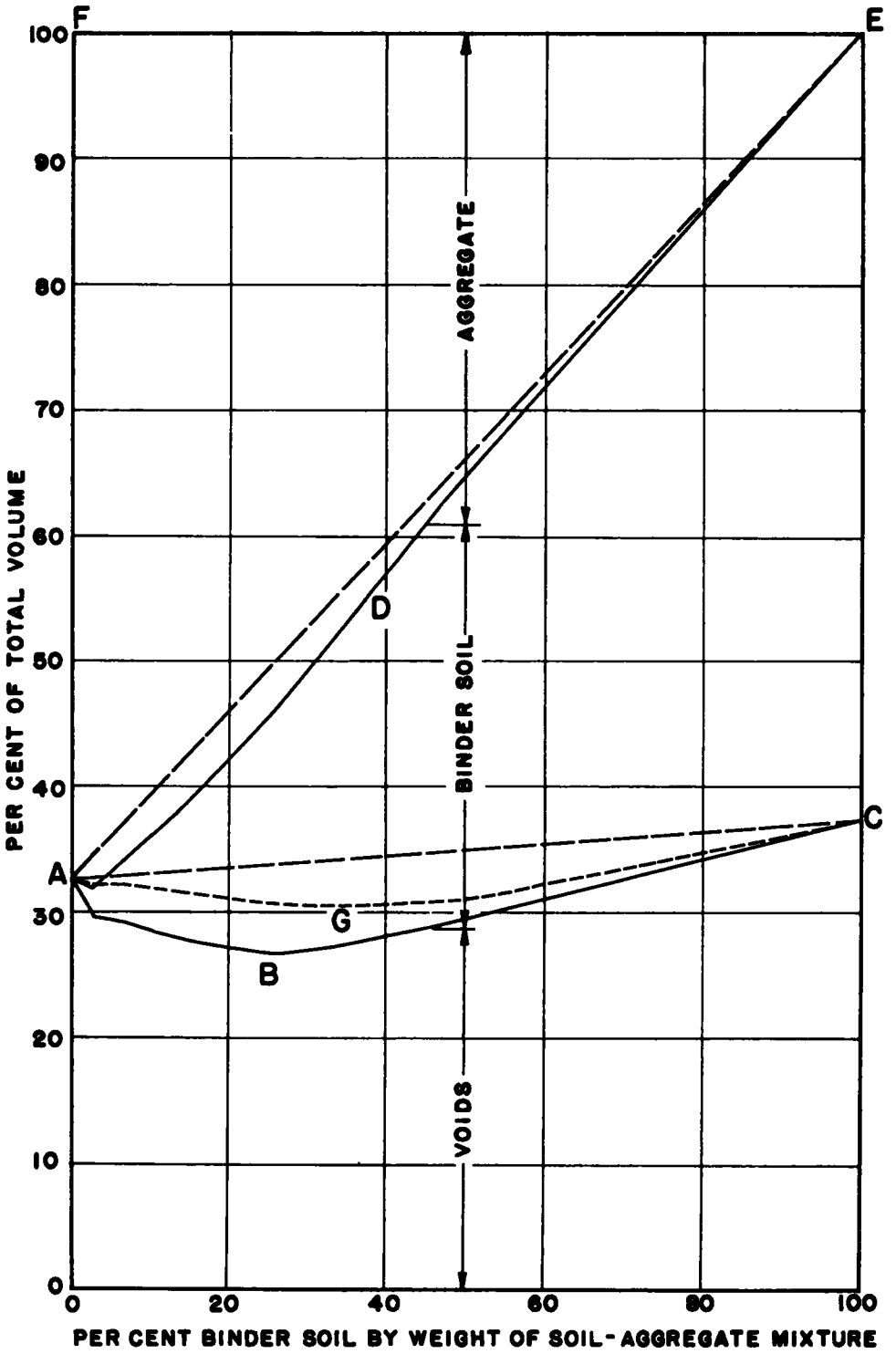


Figure 6. Absolute volumes of solids and voids in soil-aggregate mixtures.

The bearing capacity of a soil-aggregate mixture, as indicated by the formula, is derived from three sources: (a) The friction due to the weight of the soil-aggregate mixture, (b) the friction due to the surcharge or the weight of the pavement, and (c) the cohesion shearing resistance of the soil-aggregate mixture.

The results of the computations are tabulated in Table 1 and also shown in Figure 5. The data indicate that both the friction due to the weight of the mixture and that due to the surcharge decrease gradually with increasing binder soil. The cohesion shearing resistance, however, increases concurrently at a much higher rate. As a result of the

TABLE 1
COMPUTATION OF BEARING VALUES

Mixture Designation	Aggregate in Total Mixture (% Wt.)	Binder Soil in Total Mixture (% Wt.)	Unit Wt., γ (pcf)	Cohesion Value, c (psf)	Angle of Internal Friction, ϕ (deg)	Flow Value, $N\phi$	Diameter of Loading Area, d (ft)	Pavement Weight per Unit Area, q' (psf)	$\frac{q' \sqrt{N\phi} (N\phi - 1)}{c}$ (psf)	$q' N\phi^2$ (psf)	$2c \sqrt{N\phi} (N\phi + 1)$ (psf)	R_b (psf)
A	100	0	133	0	45.5	5.97	1.87	115	9,410	4,100	0	13,500
B	86	14	137	700	36.0	3.87	1.87	115	3,110	1,710	13,300	18,100
C	74	26	139	720	35.5	3.77	1.87	115	2,980	1,640	13,300	17,900
D	66	34	138	1,700	21.0	2.12	1.87	115	580	520	15,400	16,500
E	60	40	137	1,800	24.0	2.37	1.87	115	820	650	18,700	20,200
F	53	47	136	1,720	22.5	2.24	1.87	115	680	580	16,700	18,000
G	0	100	126	2,300	23.0	2.28	1.87	115	670	600	22,800	24,000

combined effect, the bearing capacity of the soil-aggregate mixture shows an increase by addition of binder soil and reaches its peak value at 100 percent of the soil. It must be noted, however, that the soil-aggregate mixtures were tested for strength at optimum moisture content. As previously discussed, the increase in bearing capacity at this particular moisture content as a result of an increase in cohesion does not necessarily indicate that the mixture is becoming more desirable, since this strength component is not reliable under all conditions. It may also be noted that the maximum density, which occurs at 26 percent of binder soil, did not produce the greatest bearing strength.

As a basis for examining the behavior of the binder soil and the aggregate at optimum moisture content in various mixtures, the writer has used the data of compaction tests from Figure 2 and computed the absolute volumes of aggregate, binder soil, water, and air in these mixtures. The results are presented in Figure 6. The absolute volume of voids (water plus air) in the soil-aggregate mixtures is indicated by curve ABC. The ordinates between curves ADE and ABC represent the absolute volume of the binder soil; those between lines FE and ADE represent the absolute volume of the aggregate.

Assuming that both the binder soil and the aggregate had no change in their individual void characteristics when they were combined, the absolute volumes of these two constituents in various mixtures would be indicated by lines AE and AC (Fig. 6) and the total amount of voids in these mixtures (water plus air) would be represented by the ordinates of line AC. However, both the actual absolute volume of the aggregate and that of the binder soil, under standard compaction conditions, are larger than those assumed. The correct volumes are indicated by the lines ADE and ABC. The amount of the undervaluation in the absolute volume of aggregate is represented by the vertical ordinate between the lines AE and ADE. The undervaluation in the binder soil volume is represented by the vertical ordinate between AC and ABC, minus the ordinate between AE and ADE. This is represented as the ordinate between AC and AGC (Fig. 6).

Figure 6 indicates that, when the binder soil was incorporated in the aggregate, it apparently reduced the particle interference and facilitated the rearrangement of aggregates being compacted into closer association. The absolute volume of the binder soil was concurrently increased attributable to its filling into the voids of the aggregate

and/or an increase in its degree of densification. It is difficult to discriminate the influence of these two factors. It appears, however, that the void-filling effect was probably predominating when the amount of binder soil was relatively low. For a mixture with a large amount of binder soil, the aggregate particles were floating in the soil mass. The increase in the absolute volume of the binder soil would be primarily attributed to the increase of its degree of compaction. At all events, the data in Figure 6 suffice to show that the binder soil did not, in the main, function as a filler of voids within the structure formed by the aggregate.

The writer expresses his appreciation to Richard A. Davino, Research Assistant in Civil Engineering at the University of Illinois for reading the first draft of this paper and checking the computations in Table 1.

W. H. CAMPEN, Manager, Omaha Testing Laboratories, Omaha, Nebraska—The most important point brought out by this paper is that in order to obtain maximum strength the mixture must be such as to produce maximum density. In other words, the mixture must contain all the aggregate it can possibly carry. In spite of the fact that the authors seem to be of a different opinion, the writer is sure that most of those engaged in this field are aware of the principle involved and have been following it.

The method suggested for determining the relative amounts of aggregate and binder has merit. However, it might be difficult to determine the weight per cubic foot of some coarse aggregates, and for this reason it might be necessary to make moisture-density tests with mixtures containing the calculated amount of coarse aggregate as well as a few percent more or less.

The strength of soil-aggregate mixtures is susceptible to moisture content. Therefore, the use of as little as 50 percent of the binder necessary for maximum density might be undesirable for the reason that the mixture would be able to take in more water than indicated by the optimum moisture.

W. A. GOODWIN, Research Engineer, Tennessee Highway Research Program, University of Tennessee, Knoxville—This subject is one of considerable interest. It is encouraging to know that research work of this nature is being carried on and is finding its way into the literature. Information of this type is of immeasurable benefit to those who use soils as an engineering material.

Data presented in Figure 3 is most significant. It can be seen that there is an optimum percent binder for maximum strength as indicated by the curve for bearing capacity versus percentage of binder solids in total mix. This optimum occurs at about 26 percent. As stated in the paper, present ASTM specifications permit a wide range in percent binder which could result in both strong and weak mixes. In view of these data, the recommendations in the paper for determining the best mix seem to be justified.

In support of the data presented the writer wishes to submit the following summary of a paper which was presented at the annual meeting of ASCE in New York in November 1951. The paper was entitled "Clay Mineralogy and Soil Stabilization," by James H. Havens and W. A. Goodwin, Highway Materials Research Laboratory, Lexington, Kentucky.

The strength characteristics of soils, as presented in this discussion are based largely upon relationships developed experimentally in connection with a long-range program of research on clays and their influences on the fundamental properties of soils. The objective of the over-all program is to develop data on the occurrence and distribution of clays in Kentucky and eventually to correlate these findings with soil formations as well as with the engineering properties of the soils.

For this phase of the project, about 40 lb of a mixed illite-kaolinite clay, smaller than 1 micron (μ), was separated from one soil and another 40 lb of a mixed montmorillonite-kaolinite clay, also smaller than 1 μ , was recovered from another soil. The separations were accomplished by sedimentation procedures. About 50 lb of silt was separated from a third soil, also by sedimentation. The recovered clays, with both Na⁺ and Ca⁺⁺ modifications, were combined in definite proportions with the silt to form synthetic soils of known composition.

Maximum densities and optimum moisture contents, under a given method of static

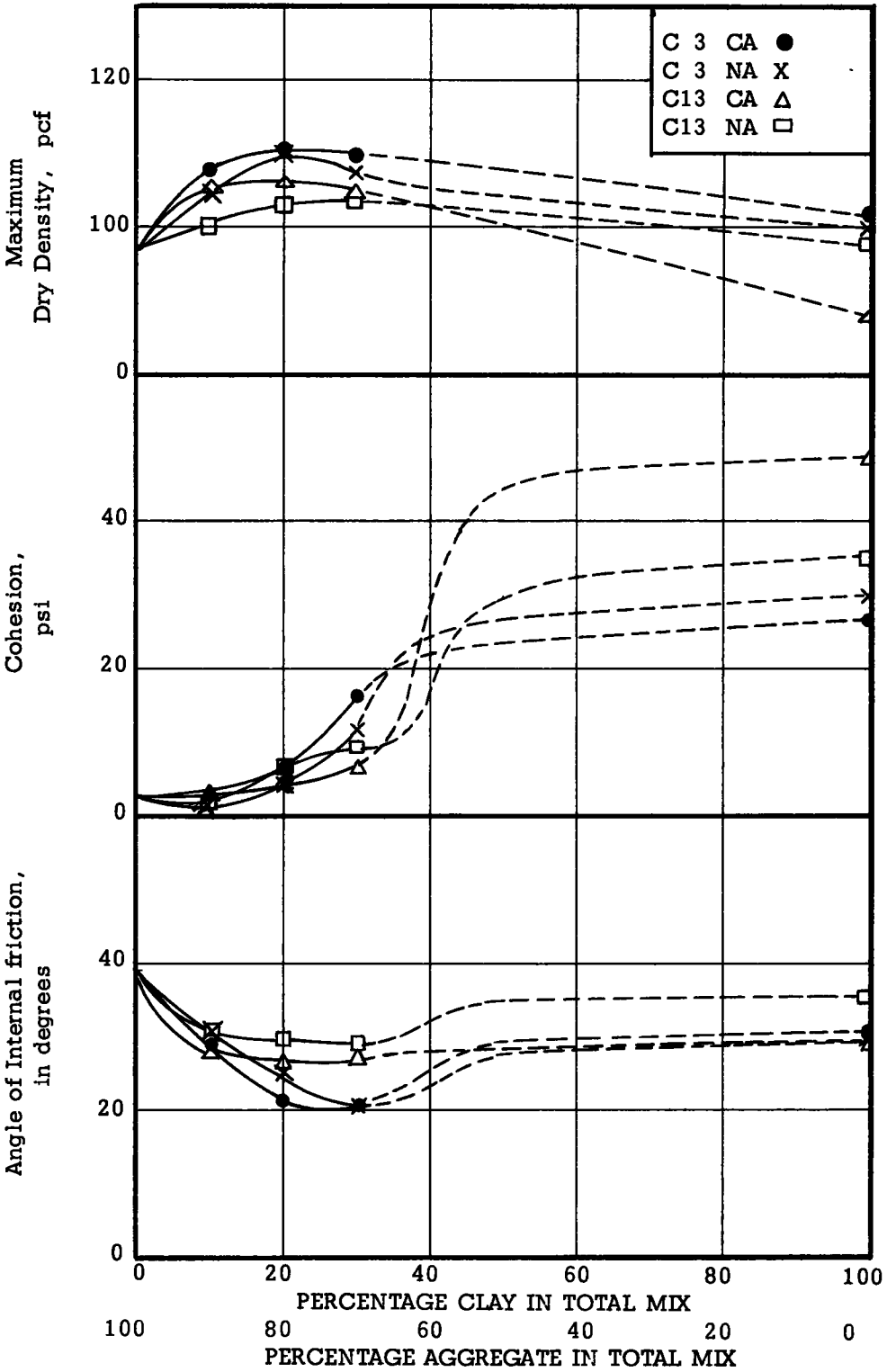


Figure 7.

compaction, were determined and these formed the basis for preparation of triaxial-test samples for combined stress analysis. Further comparisons or evaluations were made through the Atterburg limits tests.

X-ray diffraction patterns for both clays indicated that Samples C-3 is a yellow clay composed of illite and kaolinite. Sample C-13 is a red clay composed of montmorillonite and kaolinite. The silt is composed of angular quartz grains ranging in size from 5 to 75 μ , the largest portion occurring at about 25 μ .

Cationic modifications were introduced during the process of recovery by adding to the suspension of clay, chloride salts of sodium and calcium in excess of the amount necessary to produce flocculation. The flocculated clays were separated from the remaining water by vacuum filtration. The recovered clays were then air-dried, pulverized to pass the No. 200 sieve, and combined with the silt.

Triaxial specimens were prepared by adding sufficient water to the mixtures to bring them up to optimum moisture contents. The specimens were formed in a 2-in. diameter split-mold under a compressive load of 1,500 lb. They were sealed and allowed to age a minimum of 21 days prior to testing. In terms of triaxial nomenclature, the loading conditions approximated the so-called consolidated quick-type of test. Confining pressures of 0, 5, 10, and 15 lb per sq in. were used throughout this study.

Since the synthetic soils used in this study were formed by combining two clays with a silt, it might be well to consider them separately and then collectively. The clays were a flat, flaky, fine-grained material which could hold considerably more water than the silt before their strengths were materially reduced. The silt was composed of relatively clean, angular to spherical-shaped particles. Its strength was sensitive to small changes in moisture.

The combined influence of clays and granular (silt) materials in a mixture are best understood by their influence on the total voids of the mixture. It is the clay-water system within the voids which reflects the strength of the mix.

An idealized concept of the system can be shown by considering three methods of packing uniform spheres. Let A represent the loosest arrangement, B the densest, and C an intermediate stage. All three conditions are independent of size and quantity of spheres so long as they are uniform.

A has a calculated porosity of 47.6 percent; B, 26 percent; and C, 39.7 percent. A can accommodate, in its interstices, an equivalent number of spheres whose size is 0.732 times the diameter of the large spheres. These smaller spheres reduce the porosity of A to 27 percent. C will accommodate an equivalent number of small spheres 0.528 d in diameter, but they only reduce its porosity to 31.8 percent. B, which is the densest arrangement to begin with, will accommodate two sizes of small spheres, 0.414 d and 0.225 d. Together, they reduce the porosity of B to only 19.86 percent. The average for all conditions in which the small spheres are included is 26 percent voids, whereas without the small spheres the average for all three conditions is 37.8 percent voids.

Taking the average of A, B, and C with their corresponding small spheres as representative of a well-graded granular material, 26 percent voids could well be considered typical of naturally-occurring soils. Since the range is only about 20 percent to 32 percent, the assumption of 26 percent is within the predictable limits of 6 percent.

The introduction of sizes larger than the interstices for any condition would expand the structure and increase the porosity. Furthermore, a volume of a clay-water mixture in excess of the porosity of any granular structure would tend to float the strengthening structural members, and the strength of the mass thereafter would be determined by the strength of the clay-water system alone.

Although these concepts are elementary in nature, they have a bearing on the interpretation of results and should be kept in mind. The extrapolated or dashed portion of the curves between 30 and 100 percent clay (Fig. 7) represent the writer's conceptions of the relationships, and they lack verification by experimental data.

Maximum Dry Density vs Percent Clay

As the percent clay is increased the maximum density of the mass increases to the place where the clay begins to over fill the voids. When this occurs, it is apparent

that the clay has begun to expand the granular structure and, consequently, further consolidation of the mass is dependent upon the susceptibility of the clay-water system to consolidate. This relationship is shown in Figure 7.

Cohesion vs Percent Clay

In considering the cohesion of the mixtures, reference should be made to the "minimum porosity of the granular structure." With respect to the silt used in these experiments, the silt alone at maximum density has a calculated porosity of 43 percent which means that 43 percent of its bulk volume may be occupied by a clay-water system without disrupting its structure.

The silt alone is cohesive to the extent of the meniscus tension of the water within its voids. As clay is added in small increments, the water content required by optimum conditions is still very large in terms of clay content. In fact, it is so large that the clay-water system exists there in a state of fluidity incapable of contributing further cohesion to the silt. As more clay is introduced, the moisture content, with respect to the clay only, decreases. Accordingly, the moisture content of the clay is higher than its liquid limit until about 22 percent clay has been added. Above 22 percent, the clay very rapidly approaches its plastic limit, which serves to explain the sharp inflections in the cohesion curves.

At clay contents above the critical volume, determined by the minimum porosity of the silt, cohesion is no longer dependent upon binding the silt grains together but is dependent upon the cohesion of the clay-water system itself. Throughout this higher range of clay content, cohesion is constant provided the moisture content of the clay is constant, which it is assumed to be.

Angle of Internal Friction vs Percent Clay

Below the critical minimum porosity of the silt in Figure 7, the angle of internal friction for the mass is determined by the degree of frictional contact existing between the silt grains. As the clay becomes more concentrated with respect to the water in the voids, it departs more and more from a state of fluidity and merges into a state of low plasticity. In doing so, it becomes increasingly capable of lubricating the silt grains which explains the proportional decrease in the angle to about 30 percent clay content.

As the clay content approaches the minimum porosity of the silt, the angle rapidly becomes less dependent upon the inter-granular friction of the silt and more dependent on the degree of friction inherent in the clay-water system.

The following summary is quoted from the original paper:

"Throughout the discussion of this group of curves, the most outstanding feature common to all has been the volume relationships existing between the clay-water systems and the void volume of the granular structures. These have shown up repeatedly as critical points of inflection in the curves.

"As an approach to soil stabilization, these relationships emphasize the necessity of defining a soil in terms of its clay-water system and granular structure. As an approach to the evaluation of clay-water systems, they confirm the necessity of testing them independently of a granular structure. When tested independently, they demonstrate the profound characteristics inherent in the mineralogical identity of the clay and the extent to which cationic modifications influence the physical properties of clay-water systems."

GEORGE F. SOWERS, Closure—The authors wish to thank Mr. Huang for his interesting analysis of their data. They agree that absorbed moisture should also be considered, particularly when the soils are exposed to extreme weathering. Huang's analysis of bearing capacity tends to underrate the contribution of internal friction to bearing capacity, compared to the author's analysis. Furthermore, the author's analysis is low compared to the widely-used method of Terzaghi. Therefore, Figure 5 may give the false impression that bearing increases with increased binder amounts. It has been the author's experience that there is an optimum binder amount that is in the neighborhood of that shown in the paper.

Mr. Campen's comment regarding the importance of maximum density is welcomed. However, exception is taken to the statement that most engineers engaged in the field are aware of it. Too many highway engineers place blind faith in rigid specifications for soil aggregate mixtures which may not yield the maximum density. The most important fact brought out by the paper is the sharp drop in strength produced by amounts of binder only slightly in excess of that required for maximum density.

It was not intended to imply that the mix proportions are the only factor affecting soil strength. Certainly, if the soil is subjected to extreme moisture conditions a shortage of binder might be detrimental, unless the binder has swelling properties. In the latter case a shortage of binder would be helpful.

It is gratifying that the work of Havens and Goodwin confirms some of the findings presented in this paper. It is particularly encouraging to see the emphasis placed on the volume relationships of the soil and aggregate. The volume approach is a much sounder basis for stabilization than one based purely on gradation. Of course, the mineralogy of the binder is extremely important, as Mr. Goodwin demonstrates, and any consideration of volume should include the volume changes of the clay components.

Expanded Shale as an Admixture In Lime Stabilization

RAYMOND F. DAWSON, Professor of Civil Engineering, Associate Director of the Bureau of Engineering Research, The University of Texas, Austin; and
CHESTER MC DOWELL, Senior Soils Engineer, Texas Highway Department, Austin

The pozzuolanic action between lime and volcanic ash or flyash has long been known. It is also known that other materials when combined with lime will produce a similar action, but the investigation of these other combinations has been limited. This paper gives the results of the use of dust from expanded shale aggregate as an admixture in lime stabilization. The effect of expanded shale and the unburned raw material on the compressive strength of the mixtures is given as well as a limited amount of data on the variation of the quantities of the expanded shale dust. In general, the results show considerable increase in the strength of the lime stabilized mixtures for two different aggregates, one aggregate having a high clay content and the other being a sandy gravel.

● THE POZZUOLANIC ACTION between lime, volcanic ash, flyash and some other materials has long been known. The use of artificial materials in combination with lime is also quite old, although not nearly as old as the use of the natural volcanic materials. India has long used burned clay as a pozzuolanic material. Reports of early use of burned clay in Egypt have also been found, although complete references are limited. Mortar composed of lime and burned clay was used extensively in the construction of the Asyut Barrage completed in 1902 across the Nile River as reported by George Henry Stephens (1). He describes in detail the test of lime and clay mortars in which was found considerable increase in strength with the addition of burned clay to the lime mixtures. The soft-burned clays gave greater strengths than the hard- and very hard-burned clays. Although no reason was given for the greater strength obtained with the soft-burned clays, the writer believes that it might be possible that the softer clays were actually ground much finer than the harder materials thus producing much greater action. "The burnt clay after grinding was passed through a sieve having 400 meshes per square inch." This, of course, would be a No. 20 sieve and therefore a rather coarse grind.

McDowell found that the addition of small quantities of dust from expanded shale aggregate also had beneficial effects on lime stabilized soil mixtures. This paper is a report of the investigation of the use of expanded shale dust as an admixture in lime soil stabilization.

MATERIALS

Lime was freshly burned, hydrated lime obtained from the Austin White Lime Company and was sealed in tight containers prior to use.

Expanded shale was dust passed through a 200-mesh sieve, obtained from pulverizing light weight concrete aggregates that had been made from burning shale materials.

Two different aggregates were used in these investigations. One was the WASHO gravel which had a liquid limit of 24, a plasticity index of 4, a shrinkage ratio of 1.71 and a grading as shown in Figure 1. This material had only a small percentage of the clay sizes. The second aggregate was Post Oak gravel from near Austin, Texas which had a liquid limit of 53 and a plasticity index of 33. It can be seen (Fig. 1) that the Post Oak gravel all passed the $\frac{1}{4}$ -in. sieve, and although the percentage of clay was not high, most of the clay fraction was smaller than 0.001 mm in diameter.

Figure 2 shows the moisture density relationship of the Post Oak gravel as obtained naturally and the effect of the addition of lime and shale to the mixtures. These density tests were made with the Harvard miniature compaction apparatus.

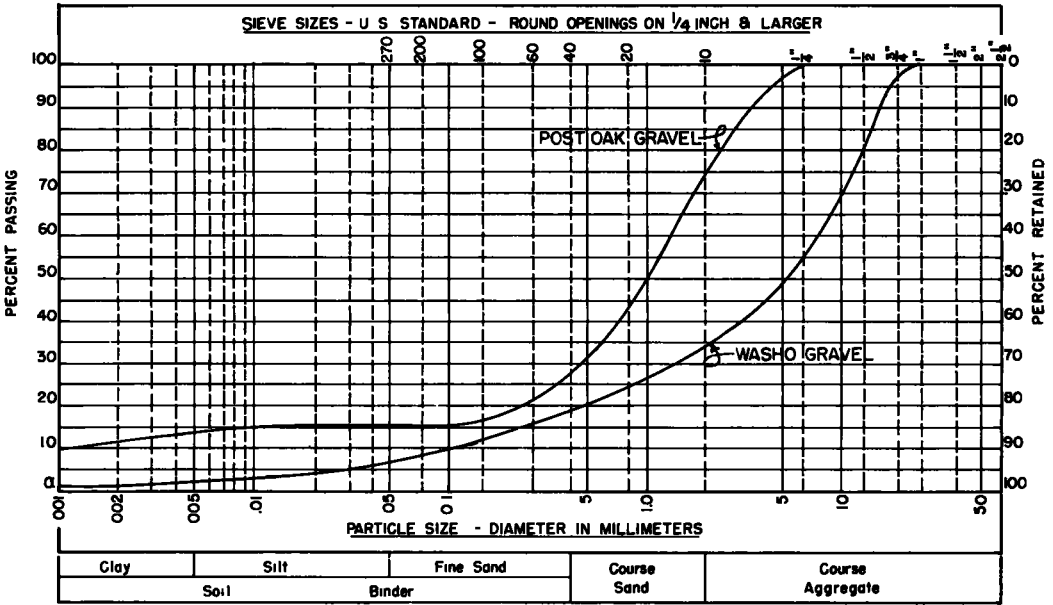


Figure 1. Grain size accumulation curve, Post Oak and Washo gravels.

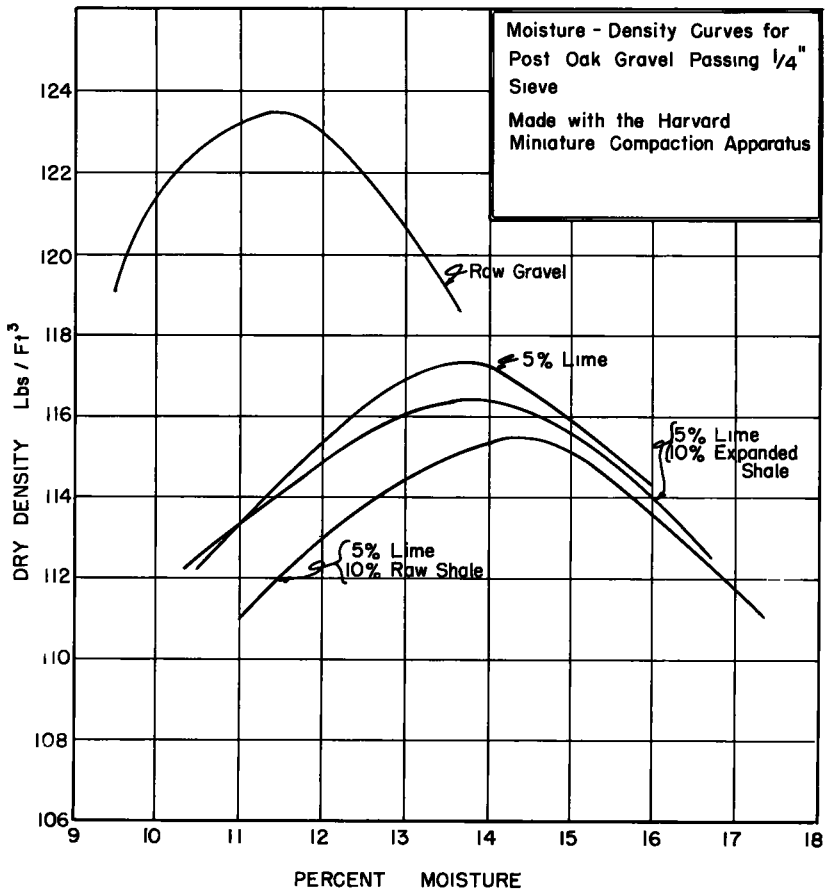


Figure 2.

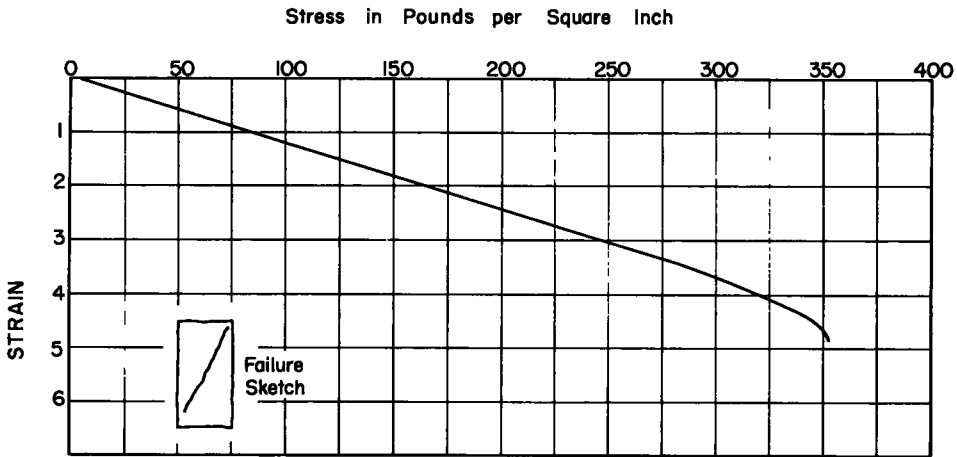


Figure 3. Typical stress strain curve, Post Oak gravel, 5 percent lime and 10 percent expanded shale. Age 28 days.

In determining the percentage of lime and shale, the total aggregate was taken as 100 percent, and the percentages of this figure were added to the aggregate in making the final mixture.

TESTING METHODS

In order to have a comparatively simple test that could be quickly made and yet give a satisfactory indication of the action of the expanded shale, it was decided to use the unconfined compression test. A few flexure tests were also made on small beams.

The unconfined specimens varied considerably in size, many of them being 2-in. by

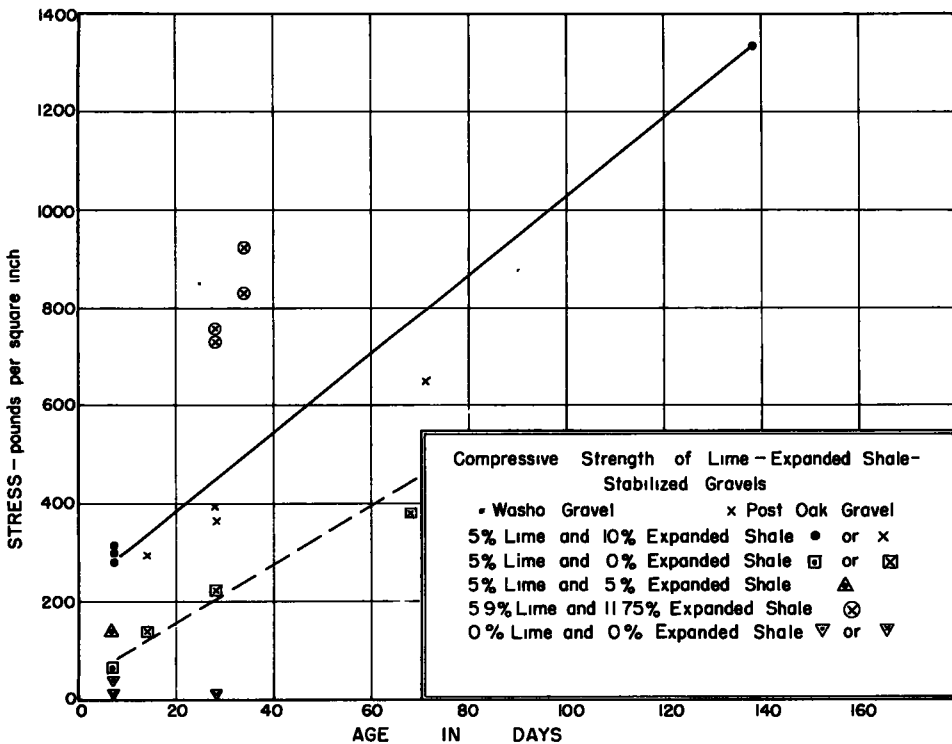


Figure 4.

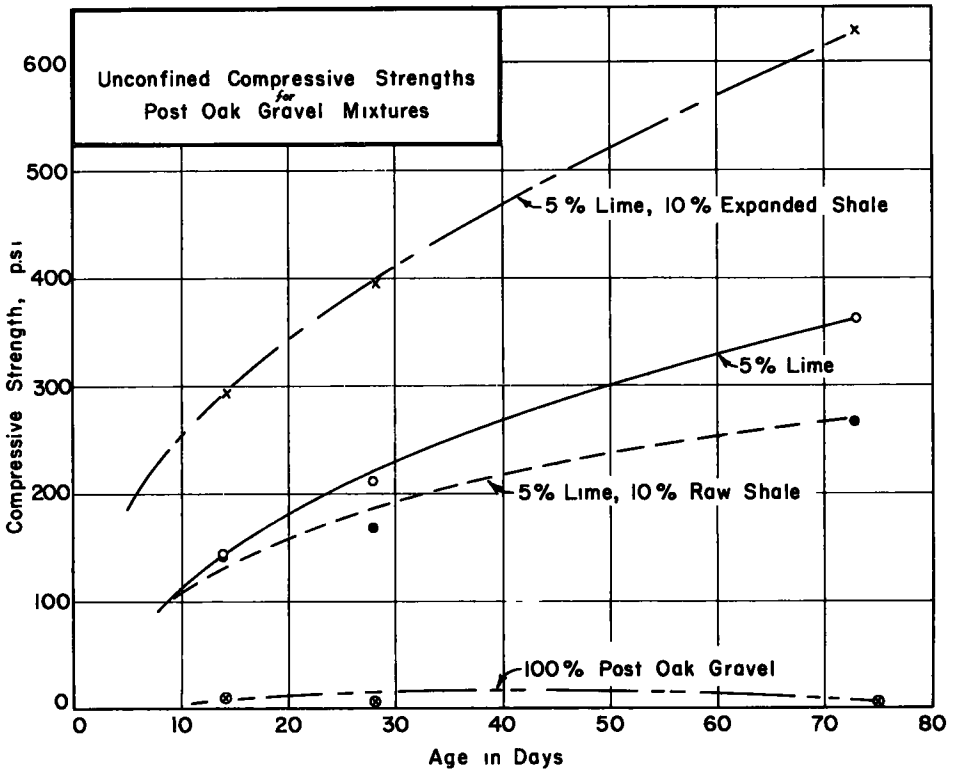


Figure 5.

4-in. cylinders made with material passing in a $\frac{1}{4}$ -in. sieve. Others were 4-in. by 8-in. cylinders, and 6-in. by 8-in. cylinders were used with the coarse WASHO gravel. The specimens were compacted to the standard Proctor density and cured under moist conditions prior to testing. Testing was done in a constant-strain testing device at a rate that required approximately 5 minutes to cause failure. Specimens were capped with plaster of Paris prior to testing and protected with Saran-wrap to prevent the loss of moisture.

Figure 3 is a typical stress-strain curve for the 2-in. by 4-in. specimens. Most of the breaks occurred rather suddenly, and the specimen shattered with little or no bulging, thus simulating a brittle material.

Flexural strength tests were made on small beams cut from cured cylinders. These beams were approximately 1.33 in. wide, 1.1 in. high and 2.8 in. long and were tested with mid-point loadings.

RESULTS OF TESTS

The flexural strength tests were limited; all were made after a considerable curing period. All of these specimens were made with WASHO gravel and contained 8 percent lime and 10 percent expanded shale. The following results were obtained:

Curing Time in Days	Flexural Strength psi
229	533.9
487	550.9
487	343.2

Although there is some variation in the strengths obtained, the tests do indicate that the material has considerable flexural strength.

Figure 4 shows the results of a number of compressive strength tests. The solid

dots are results obtained with the WASHO gravel and the x's result from specimens made with the Post Oak gravel. For combinations of 5 percent lime and 10 percent expanded shale, it can be seen that the Post Oak gravel gave values only slightly lower than the WASHO gravel, and there is considerable increase in strength with age and also with the addition of expanded shale. In one set of specimens made with Post Oak gravel, the percentages of materials were determined as a percent of the total mixture and not as a percentage of the aggregate. When these values were reduced to percentages of the aggregate, it was found that actually 5.9 percent lime and 11.75 percent expanded shale were used, thus giving considerably higher strengths. These points are indicated with x's inside the circles. One specimen was made with 5 percent lime and 5 percent expanded shale which gave lower values than the 10 percent expanded shale mixtures. Also the specimens with 5 percent lime and 0 percent expanded shale were lower than when the expanded shale was used. These values are given with dots and x's inside the squares. When no lime or lime and shale was used, very low values were obtained, as indicated by the inverted triangles.

In order to check the effect of burning on the shale, one series of tests was made in which a raw shale was used. This material is one that will expand on burning, but in these tests was merely pulverized by passing through a 200-mesh sieve. Figure 5 gives the results of these tests showing that the raw shale actually reduced the strength below that of the specimens with lime but no shale admixture. The expanded shale dust causes a considerable increase in strength over the other mixtures while the raw gravel specimens gave very low strengths.

CONCLUSIONS

Although the addition of lime to these gravels was quite beneficial as compared to the raw soils, the addition of 10 percent expanded shale dust to the lime mixtures gave added strength to the specimens. In some cases, the increase in strength was as much as 100 percent above the straight lime stabilized soil. The addition of more than 10 percent of the expanded shale indicates somewhat higher strengths than the 10 percent expanded shale mixtures. Other tests now under way indicate that the optimum amount of expanded shale ranges from about 7.5 percent to 12.5 percent.

The addition of raw shale reduces the strength of the lime-gravel mixtures as this material is merely adding more clay to the gravel.

Flexural strength tests indicate that considerable bending strength may be obtained from mixtures stabilized with lime-expanded shale admixtures.

ACKNOWLEDGMENTS

The writers wish to acknowledge the assistance and cooperation of the National Lime Association who have sponsored this project. They also wish to acknowledge the laboratory work of W. E. Strohm, Jr. and J. T. L. McNew, Jr., research assistants on the project.

REFERENCE

1. Stephens, George Henry, "Barrage Across the Nile at Asyut," Proceedings of the Institution of the Civil Engineers, Vol. CLVIII of minutes, Paper No. 3462, 1904.

Improvement of Strength of Soil-Cement With Additives

T. WILLIAM LAMBE, Director, and ZA-CHIEH MOH, Research Assistant;
Soil Stabilization Laboratory, Massachusetts Institute of Technology, Cambridge, Mass

The paper describes a search at the M. I. T. Soil Stabilization Laboratory for additives to improve the strength of soil treated with portland cement. Investigated were 29 additives, including dispersants, synthetic resins, waterproofing agents, and several salts and alkalis. The three soils studied were: a silt from New Hampshire, a clayey silt from Massachusetts, and a loess from Vicksburg, Miss. The soil samples were treated with 5 percent Type I normal portland cement plus 0.5 or 1.0 percent of the additives being studied; compacted; cured for 7 or 28 days; totally immersed in water for one day; and then tested for compressive strength.

The most effective additives were sodium carbonate, sodium hydroxide, sodium sulfate, and potassium permanganate. The trace quantities of these additives improved the strength of the soil-cement in excess of 150 percent for the two silts. Many of the other additives improved the strength of the silt significantly. The additives had a modest or no effect on the strength of the loess-cement.

Since a number of the additives that were effective are relatively cheap chemicals, they can permit economical improvement of soil-cement. The effectiveness may be limited, however, to certain soils.

● **PORTLAND CEMENT** is one of the most common and successful stabilizers for soil. Excellent results with soil-cement have been obtained in many parts of the world for a number of uses, especially for stabilizing pavement bases and subgrades. Nearly all soils which can be mixed with cement will respond to treatment. The difficulties of incorporating cement into plastic soils and the high cement requirements have, however, greatly limited its use with these soils. Acceptable clay-cement has been obtained with plant mixing and high cement concentration (> 15 percent).

Because of the present importance and considerable potential of soil-cement, the M. I. T. Soil Stabilization Laboratory has studied it for some years. Tests (Baker 1954) clearly showed a direct relation between the degree of mixing of soil and any stabilizer and the strength of the resulting product. Several studies at M. I. T. (Lambe 1954) showed that the mixing and compaction characteristics of soils could be markedly changed by the addition of trace chemicals, especially dispersants.

Based on these mixing and trace chemical studies, it was reasoned that the effectiveness of portland cement as a soil stabilizer could be enhanced with chemical additives. Denz and Steinborn (1953) investigated the effect of aggregants and dispersants on a sandy clay with 10 percent cement; they found that the chemicals could increase both the compacted density and strength a modest amount. For lower cement levels (1 percent) a higher percentage of strength increase (50 percent) was obtained. Continued study by Le Tellier and Wagner (1955) showed that considerable increase of strength could be effected on silt plus portland cement.

In 1955 a thorough review and investigation of the improvement of soil-cement with additives was undertaken. This paper, describing the most recent studies, indicates the very large improvements that are obtainable. A detailed explanation of the mechanisms employed by the various additives is not given, mainly because they are not yet well understood. Certainly, the dispersion mechanism which initiated this study is not the most important one. Some chemical reaction after compaction appears to occur in most of the systems. The authors are now studying the chemistry of the best systems, hoping to delineate the various reactions. Such an understanding should help in finding the most effective additives and in predicting the degree of response of different soils.

TABLE 1
 PROPERTIES OF SOILS EMPLOYED IN STUDY

Soil	Massachusetts			
	Clayey Silt	Viscksburg Loess	New Hampshire Silt	
Textural composition ^a percent by weight	Gravel	0	0	0
	Sand	47	10	3
	Silt	42	86	90
	Clay	11	4	7
Engineering classification ^b		A-4(4)	A-7-6(10)	A-4(8)
Physical properties	L. L.	20	41	28
	P. L.	14	26	20
	P. I.	6	15	8
	Sp. Gr. ^c	2.77	2.80	2.72
	Max. dry density ^d in lb/ft ³	122.3	104.5	99.5
	Optimum moisture ^d in percent	13.3	18.5	19.9
Chemical properties	Cat. Ex. Cap., m. e./100 gm.	10	16	3
	pH	-	4.6	5.4
	Soluble salts, m. e. NaCl/100 gm.	-	0.2	-
	Organic matter, percent	-	1.8 ± 0.1	0.4 ± 0.1
Mineral composition ^e , percent by weight	Quartz	35	30	40
	Feldspar	20	30	40
	Mica	-	-	10
	Illite	30	15	10
	Montmorillonoid	-	20	-
	Fe ₂ O ₃	2.9	1.6	1.0

Based on M. I. T. classifications: Gravel-above 2.0 mm, sand-0.06 to 2 mm, silt-0.002 to 0.06 mm, clay-below 0.002 mm.

Based on Highway Research Board system.

Determined on the fraction passing No. 10 sieve.

Determined by Harvard Miniature compaction apparatus, compacted in three layers with a 40-lb tamper, 25 blows per layer.

Determined on the fraction smaller than 0.074 mm.

The testing program described in the following pages was a screening one. For these screening tests only compressive strength has been used for evaluation; no freeze-thaw, wet-dry or abrasion tests have been run.

MATERIALS AND TESTING PROCEDURE

Materials

The research considered the effects of 29 chemicals on the strength of 3 soils of different composition stabilized with 5 percent portland cement. The 3 soils tested were a clayey silt from Massachusetts, a uniform silt from Manchester, N.H., and a uniform loess from Vicksburg, Miss. Properties of these soils are shown in Table 1.

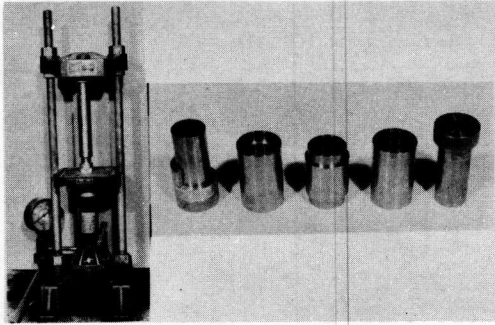


Figure 1. Assembly of static compaction apparatus.

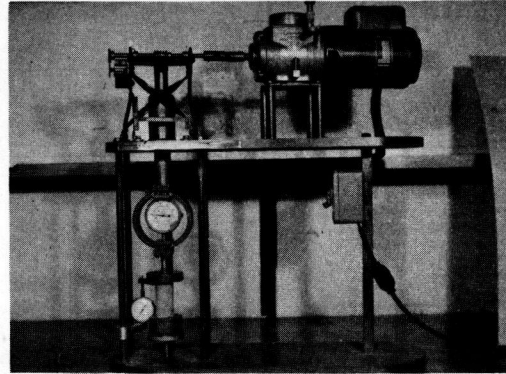


Figure 2. Testing machine for unconfined compression.

Table 2 lists the 29 chemicals — dispersants, synthetic resins, bonding and water-proofing agents, alkalies, and salts.

Testing Procedure

Preparation of Soil, Cement, and Additive Mixtures. Five percent Type I normal portland cement was used in all cases with additives at concentrations of either 1/2 or 1 percent, both cement and additive concentrations based on the dry weight of soil.

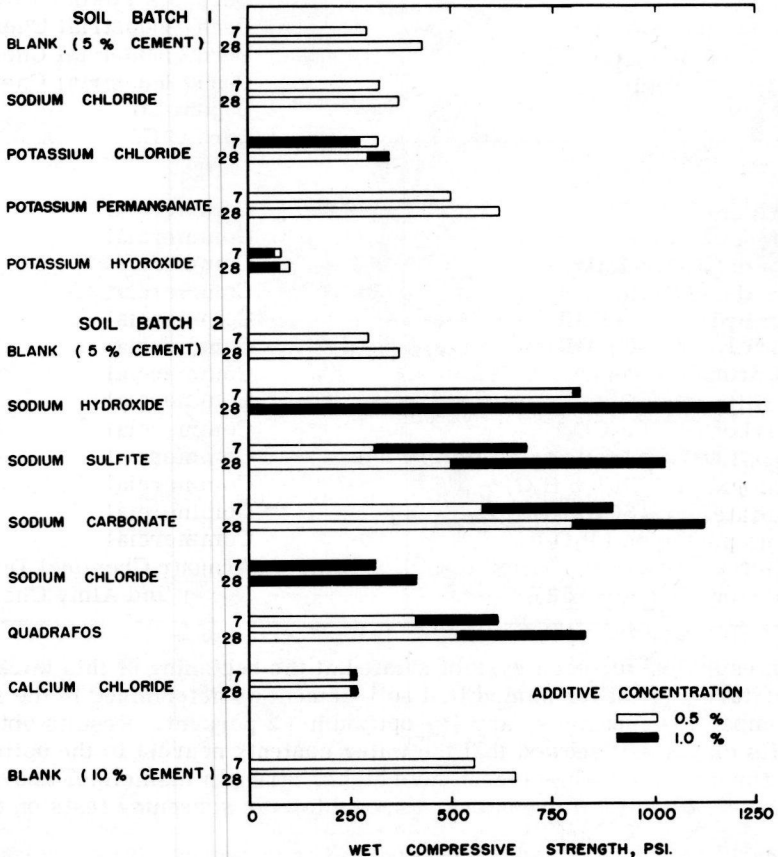


Figure 3. Effect of additives on 7 and 28 day wet compressive strength of cement stabilized Massachusetts clay silt.

Air-dried soil passing No. 10 sieve was handmixed with a predetermined amount of cement. The chemical was dissolved in the mixing water in proper proportion and the solution added to the dry soil-cement; this procedure was followed with all chemicals with the exception of the following four synthetic resinous agents: Aroclor 4465, Vinsol, Piccolyte S-125, and Picco XX-100B. Because these resins were not water-soluble under ordinary conditions, they were pulverized and mixed dry with the soil-cement before the addition of water. In all cases mixing was done in a sigma-blade mechanical mixer for 5 min.

TABLE 2
CHEMICAL ADDITIVES EMPLOYED IN STUDY

Material	Source
Dispersants:	
Sodium tetraphosphate (quadrafos)	Rumford Chemical Co.
Pozzolith 2AA	Master Builders Co.
Daxad 21	Dewey and Almy Chemical Co.
Lignosol X2D	Lignosol Chemical, Ltd.
Synthetic resin bonding and waterproofing agents:	
Aroclor No. 4465	Monsanto Chemical Co.
Piccolyte S-125	Penn. Industrial Chemical Corp.
Picco SS-100B	Penn. Industrial Chemical Corp.
Vinsol (powdered resin)	Hercules Powder Co.
Piccopale emulsion A-1	Penn. Industrial Chemical Corp.
Piccopale emulsion A-35	Penn. Industrial Chemical Corp.
Losorb	Penn. Industrial Chemical Corp.
Polyvinyl alcohol (Grade 50-42)	Dupont Co.
Polyvinyl alcohol (Grade 5-88, 65-98)	Borden Co.
Others:	
Calcium chloride (CaCl_2)	Commercial
Sodium chloride (NaCl)	Commercial
Potassium chloride (KCl)	Commercial
Potassium dichromate ($\text{K}_2\text{Cr}_2\text{O}_7$)	Commercial
Potassium hydroxide (KOH)	Commercial
Calcium hydroxide ($\text{Ca}(\text{OH})_2$)	Commercial
Sodium hydroxide (NaOH)	Commercial
Sodium sulfite (Na_2SO_3)	Commercial
Sodium carbonate (Na_2CO_3)	Commercial
Borax ($\text{Na}_2\text{B}_4\text{O}_7 \cdot 10 \text{H}_2\text{O}$)	Commercial
Ferric chloride ($\text{FeCl}_3 \cdot 6 \text{H}_2\text{O}$)	Commercial
Ferric sulfate ($\text{Fe}_2(\text{SO}_4)_3 \cdot 6 \text{H}_2\text{O}$)	Commercial
Phosphorus pentoxide (P_2O_5)	Commercial
Arquad 2HT	Armour Chemical Division
Darax polyvinyl acetate X52L	Dewey and Almy Chemical Co.

Water contents used for each system studied at the beginning of this work were (a) optimum moisture content for unmodified soil-cement as determined in the Harvard miniature compaction apparatus, and (b) optimum ± 2 percent. Results obtained with Massachusetts clayey silt showed that the water contents nearest to the optimum moisture content for untreated soil-cement gave higher strength than either above or below optimum. Only optimum water content was used for the screening tests on the other two soils.

Molding of Specimens. Needed for the screening work was a simple method of preparing soil samples to determine the relative effect of different additives on the stability

TABLE 3
SUMMARY OF RESULTS OF BENEFICIAL ADDITIVES ON THREE SOIL-CEMENT SYSTEMS
BASED ON 7-DAY CURING STRENGTH^a

Improvement (percent)		New Hampshire Silt (percent)		Massachusetts Clayey Silt (percent)		Vicksburg Loess (percent)
20 to 50	1.0	Aroclor 4465	1.0	Vinsol	0.5	Quadrofos
	1.0	Vinsol	1.0	Aroclor 4465	0.5	Lignosol X 2D
	0.5	Quadrofos	0.5	Quadrofos	1.0	PVA (50-42)
	1.0	Piccopale emulsion A-1			0.5	Picco XX-100B
	1.0	Quadrofos			1.0	Picco XX-100B
	1.0	Calcium chloride			0.5	Sodium carbonate
					1.0	Lignosol X 2D
50 to 100	0.5	Piccopale emulsion A-35	1.0	Potassium permanganate	1.0	Sodium sulfite
	1.0	Potassium chloride	0.5	Sodium carbonate	1.0	Sodium carbonate
	0.5	Calcium chloride			0.5	Sodium sulfite
	0.5	Potassium permanganate			1.0	Sodium hydroxide
	0.5	Sodium chloride			0.5	Sodium hydroxide
	0.5	Sodium hydroxide				
	0.5	Potassium dichromate				
	1.0	Sodium chloride				
1.0	Potassium dichromate					
100 to 150	1.0	Potassium permanganate	1.0	Quadrofos		
	1.0	Sodium sulfite	1.0	Sodium sulfite		
150 to 200	1.0	Potassium hydroxide	0.5	Sodium hydroxide		
	1.0	Sodium hydroxide	1.0	Sodium hydroxide		
	0.5	Sodium sulfite	1.0	Sodium carbonate		
Over 200	0.5	Sodium carbonate				
	1.0	Sodium carbonate				

^aBlank is soil plus 5 percent cement; all other systems are soil plus 5 percent cement plus additive.

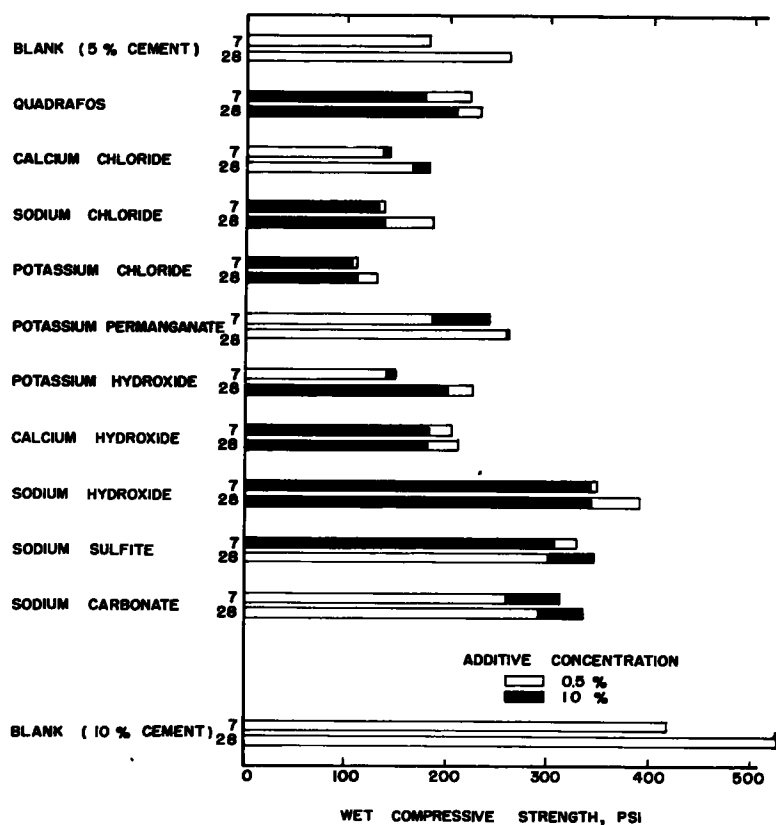


Figure 4. Effect of additives on 7 and 28 day wet compressive strength of cement stabilized Vicksburg loess.

of soil-cement. This method should be fast and simple, and yet display the full benefits of the additive.

Dynamic compaction methods were discarded since they entailed too much molding effort; static compaction from one end was also found to be unsatisfactory since it yielded a specimen denser at one end than the other. The method adopted was static compaction from both ends; a Harvard miniature-size mold (1.313 in. in diameter, and 2.816 in. high) was fitted with an extension collar and piston at each end. Compaction was then carried out simultaneously at both ends under a specified pressure by means of a hydraulic jack. This specified pressure was that required to mold soil-cement to the maximum dry density as determined by dynamic procedure, that is, compacted in three layers by a 40-lb spring tamper with 25 blows per layer. Figure 1 shows the assembly of the molding apparatus.

Static compaction admittedly is generally less representative of field compaction; however, it appeared to give satisfactory and reproducible results for this screening test program.

Curing of Specimens. Molded specimens were cured for periods of both 7 and 28 days in a desiccator maintained at 100 percent relative humidity and room temperature (about 20 to 25 C).

Testing of Specimens. All specimens were subjected to a 24-hr complete immersion in distilled water at room temperature prior to testing in unconfined compression. Complete water immersion is a most severe treatment for stabilized soils.

The apparatus used for determining compressive strength was a proving-ring type machine (Fig. 2). The maximum pressure causing failure of the specimen was taken as the compressive strength.

TABLE 4

**SUMMARY OF RESULTS OF BENEFICIAL ADDITIVES ON THREE SOIL-CEMENT SYSTEMS
BASED ON 28-DAY CURING STRENGTH^a**

Improvement (percent)		New Hampshire Silt (percent)		Massachusetts Clayey Silt (percent)		Vicksburg Loess (percent)
20 to 50	0.5	Pozzolity 2AA	0.5	Aroclor 4465	1.0	Sodium carbonate
	0.5	Lignosol X 2D	1.0	Aroclor 4465	1.0	Sodium sulfite
	1.0	Calcium hydroxide	0.5	Sodium sulfite	0.5	Sodium hydroxide
	1.0	Aroclor 4465	0.5	Quadrofos		
	0.5	Potassium chloride	1.0	Potassium permanganate		
	0.5	Aroclor 4465				
	0.5	Vinsol				
	1.0	Vinsol				
	1.0	Piccopale emulsion A-35				
50 to 100	1.0	Calcium chloride			1.0	Sodium hydroxide
	1.0	Borax				
	0.5	Calcium chloride				
	0.5	Piccopale emulsion A-35				
	0.5	Sodium chloride				
	0.5	Quadrofos				
	1.0	Potassium hydroxide				
	0.5	Potassium permanganate				
0.5	Sodium hydroxide					
100 to 150	1.0	Sodium carbonate	0.5	Sodium carbonate		
	0.5	Potassium dichromate	1.0	Quadrofos		
	1.0	Potassium chloride				
	0.5	Sodium sulfite				
	1.0	Quadrofos				
	1.0	Sodium chloride				
	1.0	Potassium dichromate				
	1.0	Sodium sulfite				
150 to 200	0.5	Sodium carbonate	1.0	Sodium sulfite		
	1.0	Sodium hydroxide	1.0	Sodium carbonate		
Over 200	1.0	Potassium permanganate	1.0	Sodium hydroxide		
			0.5	Sodium hydroxide		

^aBlank is soil plus 5 percent cement; all other systems are soil plus 5 percent cement plus additive.

RESULTS

Effect of Additives on Compacted Density

None of the trace additives studied had any large effect on the compaction characteristics of the soil and cement mixtures. Most of the dispersants, alkali reagents with sodium (ions and salts with sodium-ions, caused a modest increase in maximum compacted density (1 to 7 lb per cu ft) and a slight decrease in optimum moisture. Several additives increased strength but had no influence on the compaction characteristics. There was not any observable relation between strength increase and density increase. These facts indicate that the strength improvements are not primarily due to any effects on compacted density.

Although there are no numerical data on the degree of mixing, visual observations suggest that the use of trace additives is not a significant aid to mixing.

These and other considerations strongly point to some chemical reaction, or reactions, between various components in the system with a resulting increase of cementing action. Now in progress are a number of investigations aimed at ascertaining what reactions occur and what the effects are of the various reactions.

Effectiveness of Additives

Tables 3 and 4 show that over one-half of the chemicals tried increased the strength of the soil-cement. There were seven chemicals which, when used at concentrations of

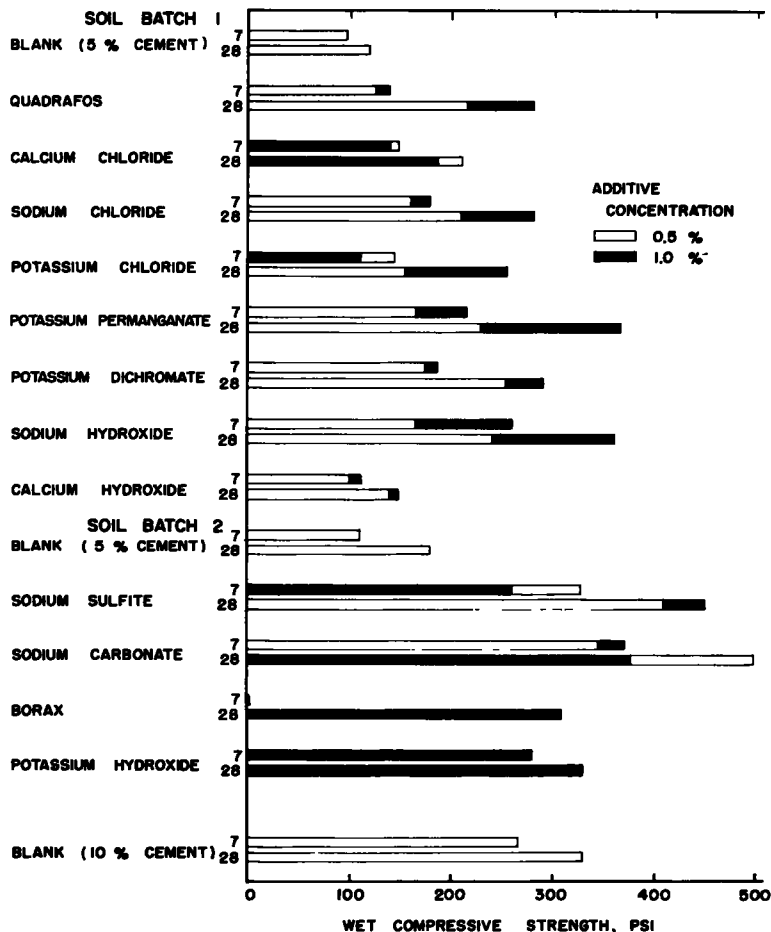


Figure 5. Effect of additives on 7 and 28 day wet compressive strength of cement stabilized New Hampshire silt.

TABLE 5

COST COMPARISON OF STABILIZED NEW HAMPSHIRE SILT WITH CEMENT ALONE AND WITH CEMENT PLUS ADDITIVE FOR A REQUIRED STRENGTH^a

	Soil-Cement		Soil-Cement Plus Additive	
Compressive strength required, psi	300	300	300	300
Concentration of admixtures required, percent	12 cement	5 cement plus 0.5 sodium carbonate	5 cement plus 1.0 sodium hydroxide	5 cement plus 1.0 sodium sulfite
Cost ^a per mile for stabilizer and additives, dollars	5,340	3,170	3,540	3,850

^aThe cost estimates are for a mile of base course 8-in. thick and 12 ft. wide, with a compacted dry density of 100 lb per cu ft. The prices of chemicals are obtained from Oil, Paint and Drug Reporter Weekly, December 10, 1956.

1 percent or less, more than doubled the strength of the silts treated with cement.

Figures 3, 4, and 5 present the strength data as bar graphs. The white bars show the strengths of soil-cement with 0.5 percent additives and black bars show the strength with 1 percent additives. For example, Figure 5 shows that 0.5 percent sodium carbonate increased the 7-day strength of New Hampshire silt (with 5 percent cement) from 100 psi to 345 psi and the 28-day strength from 180 psi to 500 psi. The 1 percent sodium carbonate treatment gave a 7-day strength of 370 psi and a 28-day strength of 375 psi. Figure 5 also shows that the silt and 10 percent cement with no additive had a 7-day strength of 265 psi and a 28-day strength of 330 psi.

The preceding example illustrates several important general points, namely:

1. The beneficial effects were not merely due to acceleration of cement hydration since these effects are apparent at 28 days. Other tests show that the benefits fully persist after four months of curing, the maximum period so far investigated.

2. The influence of chemical concentration is not predictable. An increase from 0.5 to 1 percent sodium carbonate caused a beneficial effect on the 7-day strength but a detrimental effect on the 28-day strength increase. Until the nature of the chemical reaction can be better understood, optimum treatment levels for any system will have to be determined by trial.

3. The chemical additives can be more effective than a sharp increase in the amount of cement added. For example, the addition of 1 lb of sodium carbonate gave more strength increase than 10 lb of cement.

Response of Soils to Treatment

The test data show that both cement-silts responded to chemical treatment better than did the loess from Vicksburg. This fact can be seen in Tables 3 and 4 which indicate no additive as much as doubled the loess-cement strength. During the last decade, stabilization research at M.I.T. on many soils and many stabilizers has always shown that different soils respond to stabilization in greatly different extents.

Certain general correlations between soil composition and soil responses to various types of stabilization have been noted. There are too few data and too little understanding of the principles of chemical reactions in soil-cement to draw such correlations at this time. It is apparent, however, that different soils will respond differently to cement plus chemical treatment.

Economics of Chemical Treatment

Although the test program has not advanced far enough to warrant a detailed economic study on the use of additives to soil-cement, it does indicate that significant financial savings are possible. Table 5 presents a cost comparison of New Hampshire silt-cement with and without additives. The figures show that the cost of total admixture per pile, to obtain a strength of 300 psi, is considerably less for the cement plus additives than for the cement alone (\$3,200 vs \$5,300). The use of strength alone as a performance criterion may not be justified. There would also be a saving in material handling, and possibly a saving in soil processing, with the cement-chemical combination.

Certainly, at this stage of the research, the economics of chemical treatment of soil-cement looks very promising.

SUMMARY AND CONCLUSIONS

This paper describes an experimental investigation aimed at improving the effectiveness of portland cement as a soil stabilizer. The sole criterion of improvement is based on the compressive strengths of moist-cured and water-immersed specimens.

The results show that, on two of the three soils tested, low level chemical treatment gave strengths more than double those of soil-cement specimens with no chemical additive. More modest improvements were obtained on the third soil. Since a number of the beneficial additives are relatively cheap and since the effective treatment levels are percent or less, based on the dry soil weight, the use of additives to soil-cement is economically interesting.

Not enough is yet known to explain in detail the mechanism the additives employ to increase the strength. Chemical reactions which result in further cementation are obviously occurring.

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