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.

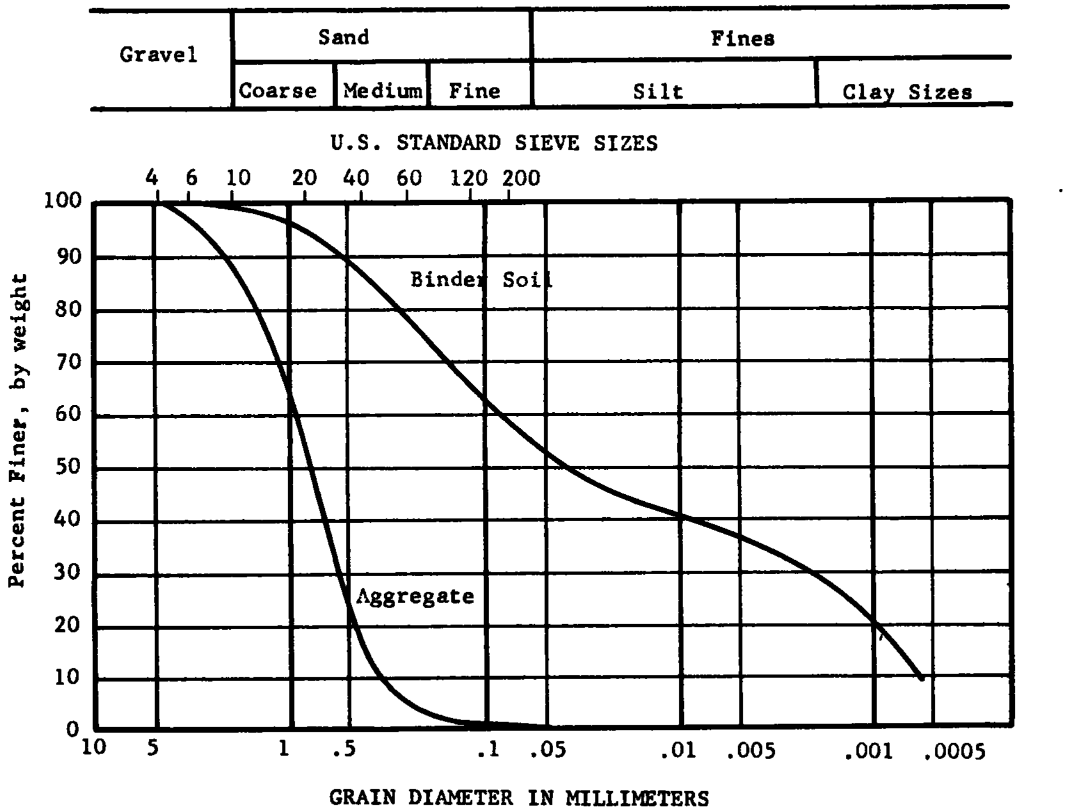
THEORIES OF PROPORTIONING SOIL MIXTURES

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 graduations (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



Soil	Liquid Limit	Plasticity Index	Specific Gravity of Solids	Remarks
Aggregate	NP	NP	2.71	Angular to subangular quartz, some mica.
Binder	31	6	2.68	Red inorganic sandy clay

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 down 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.

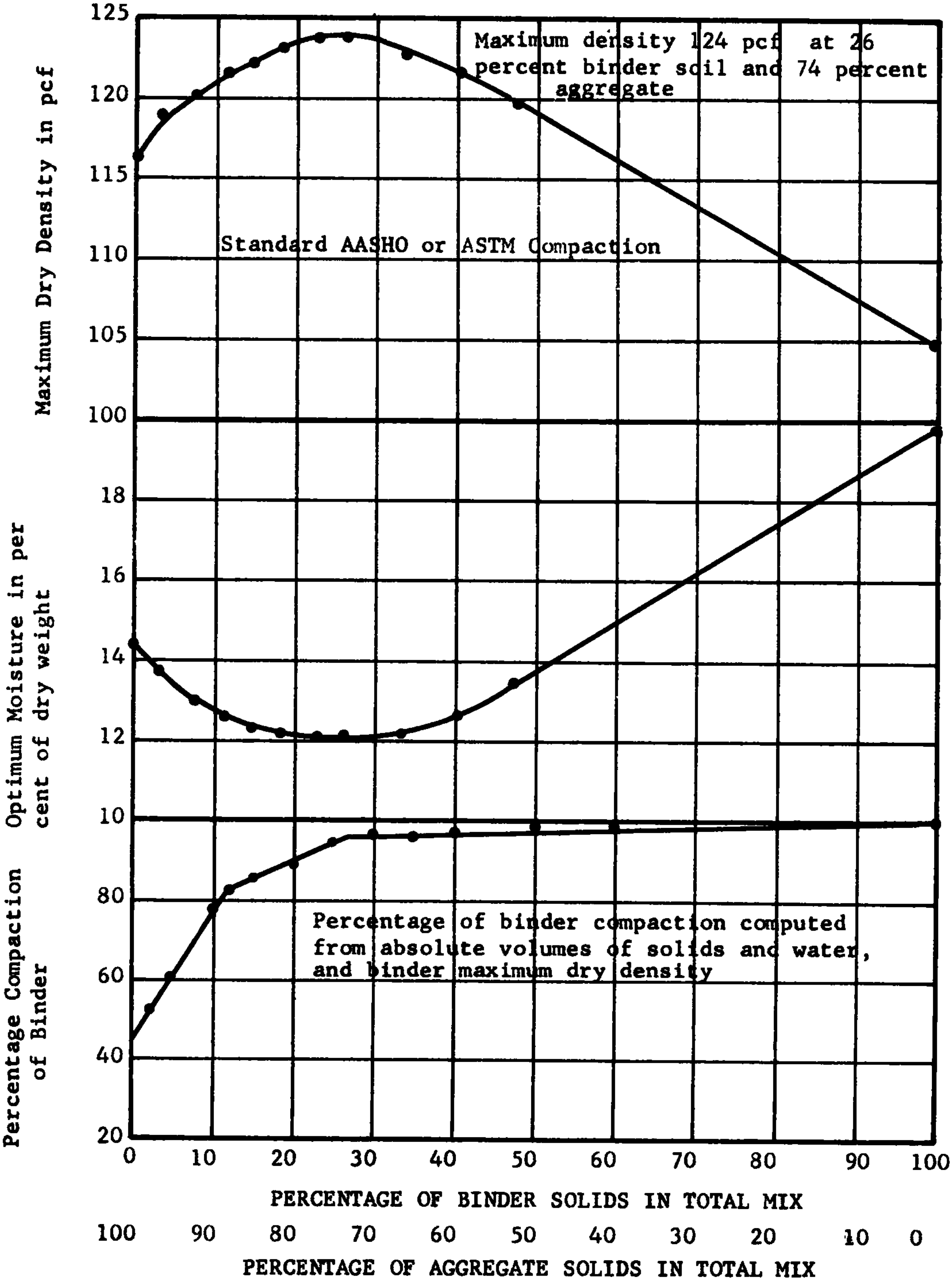


Figure 2. Maximum density, optimum moisture, and percentage of compaction of binder for various proportions of binder soil and aggregate.

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 ¾ 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):

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

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.

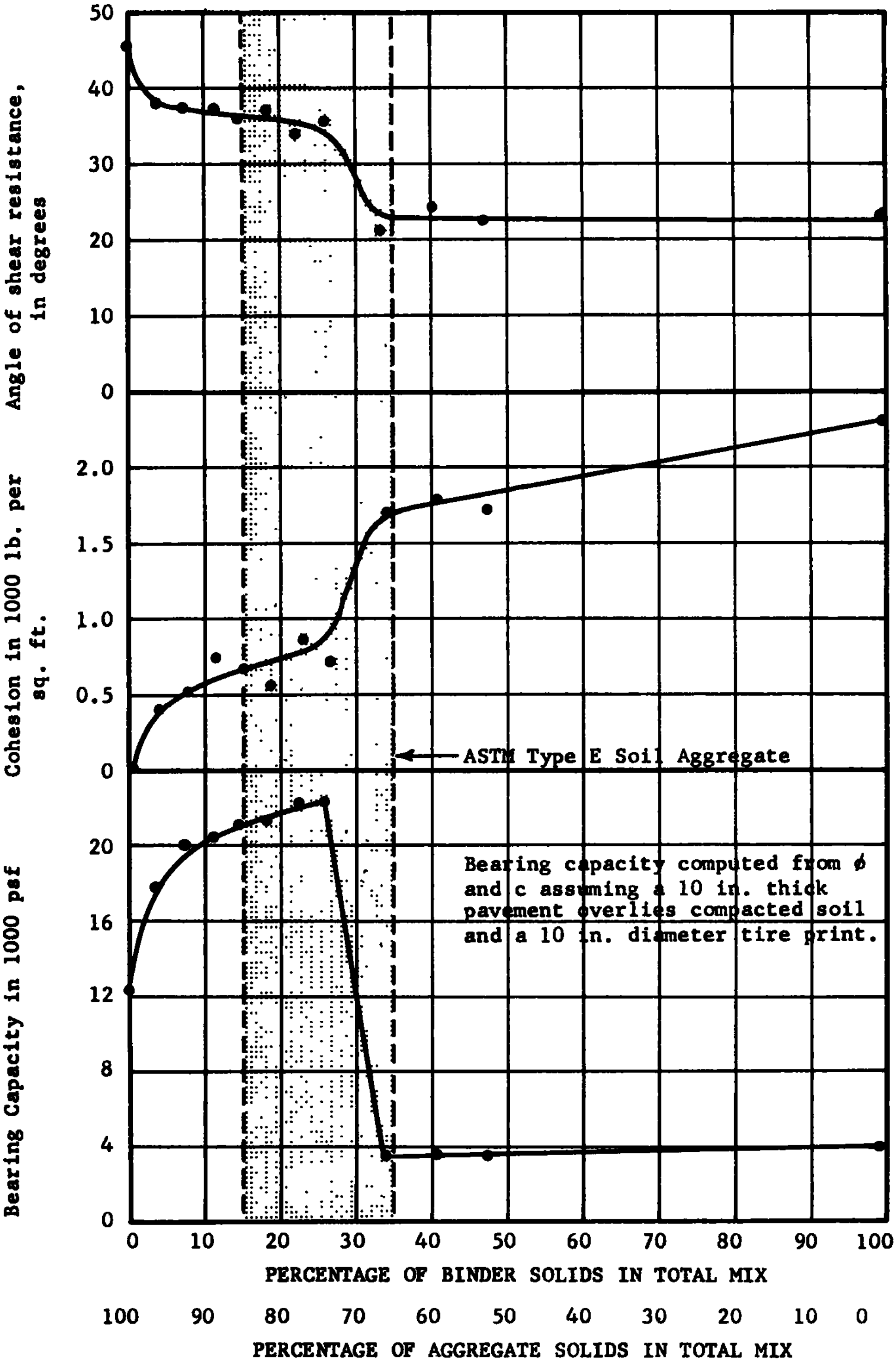
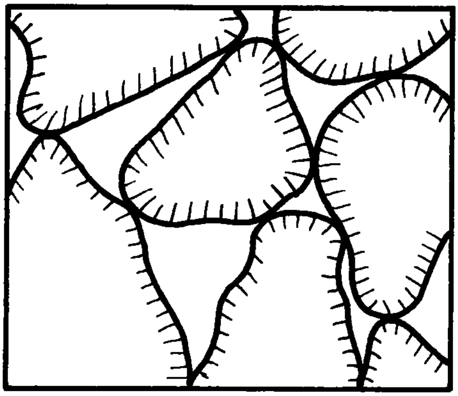
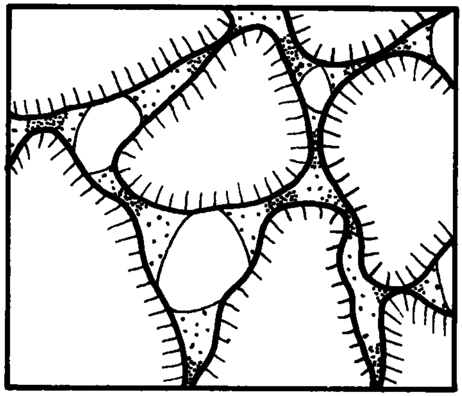


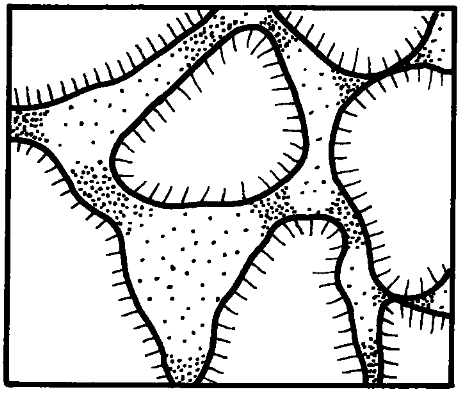
Figure 3. Strength and computed bearing capacity of soil-aggregate mixtures for various proportions of the binder soil and aggregate.



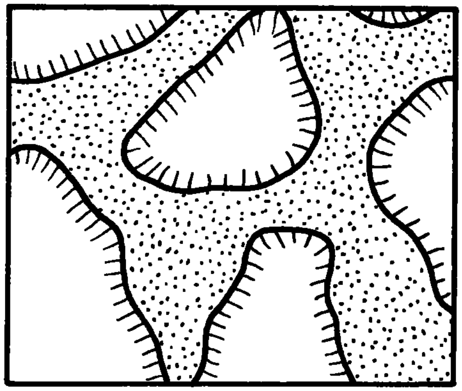
a. Compacted aggregate alone with grain to grain contact and high internal friction.



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-aggreate 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:

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

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.

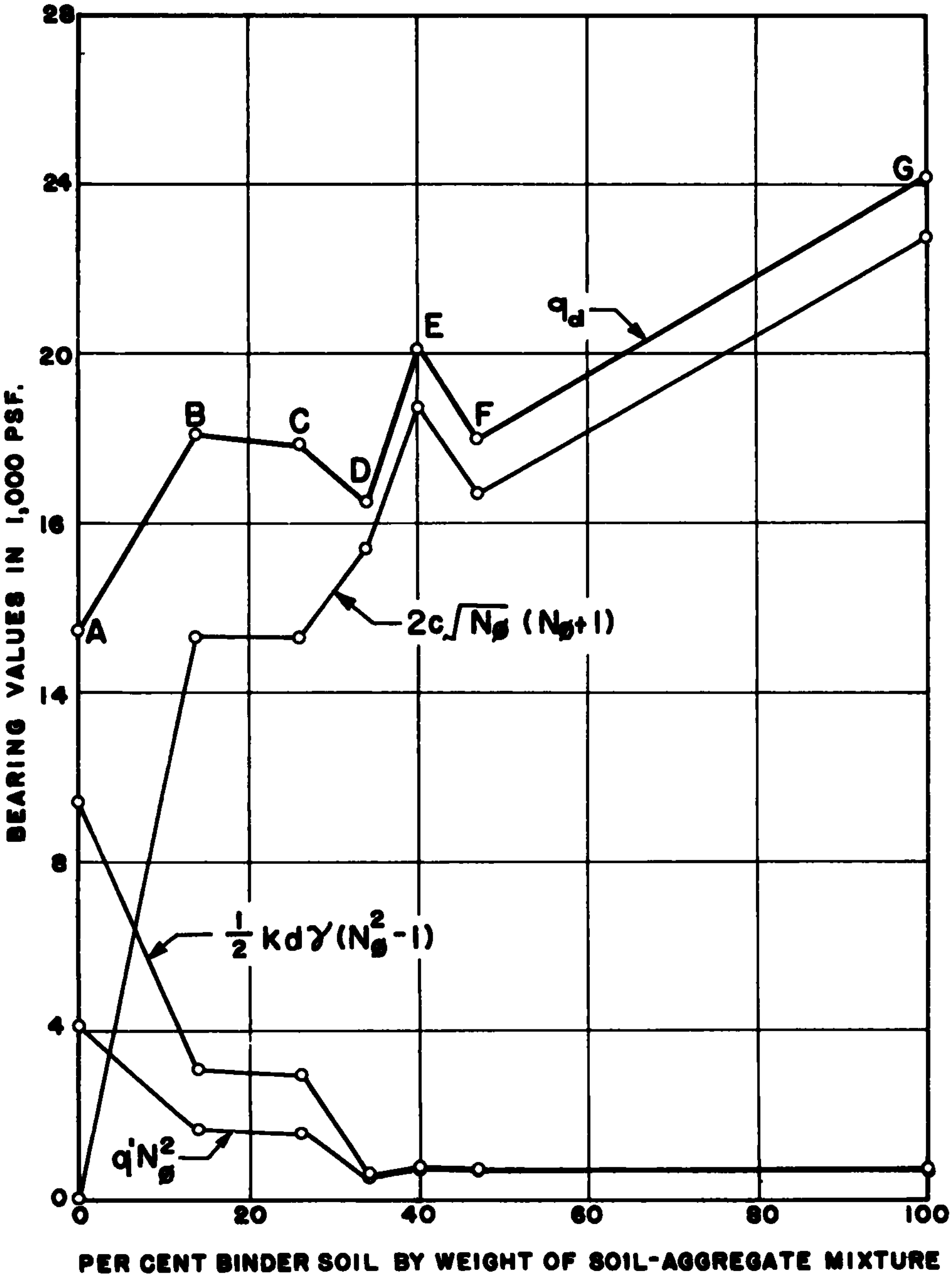

Figure 5. Computed bearing values of soil-aggregate mixtures.

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 text. It may be noted that the formula is almost exactly like the one used by the authors.

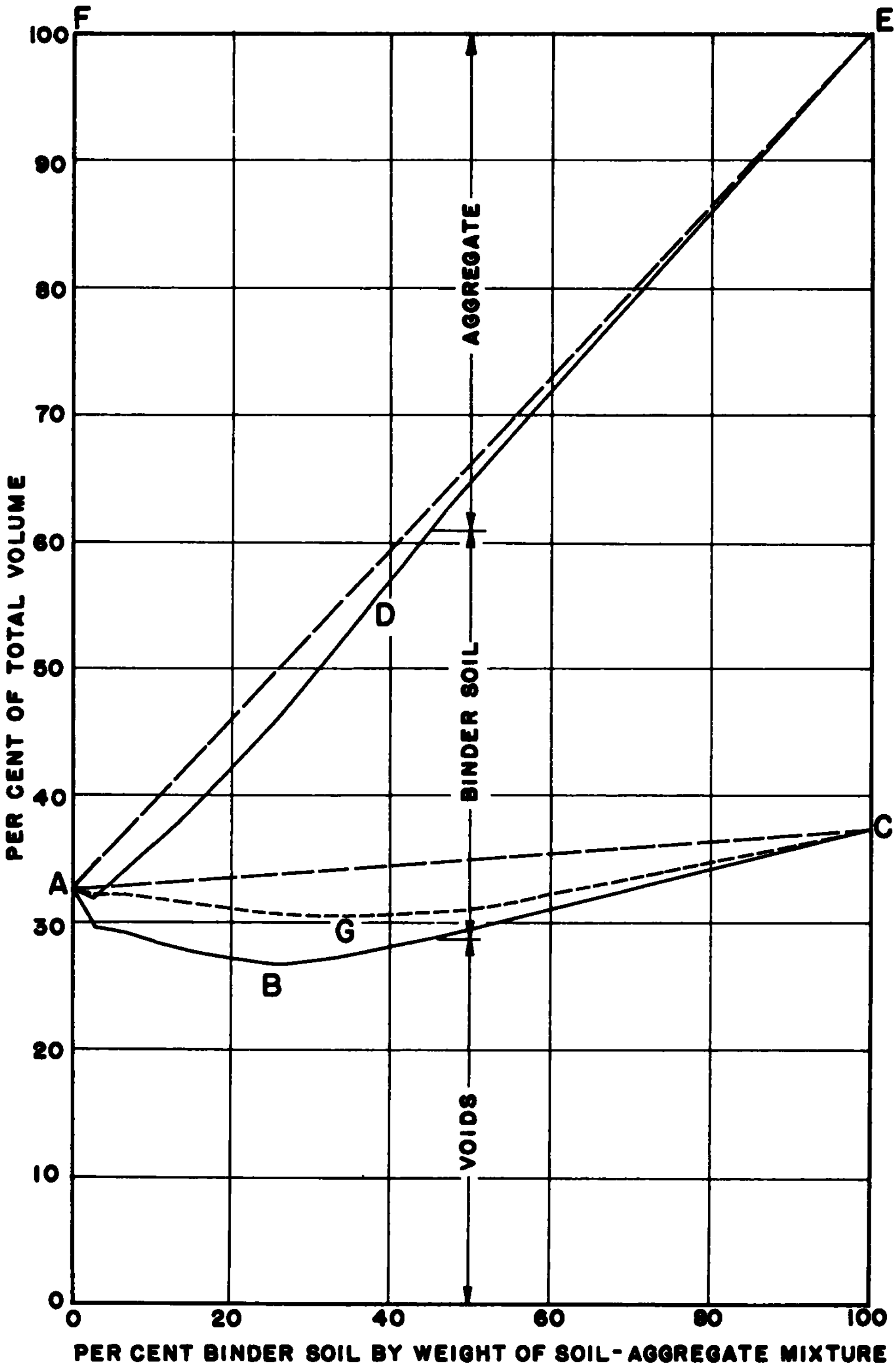

Figure 6. Absolute volumes of solids and voids in soil-aggregate mixtures.

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 Desig-nation	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_ϕ	Diameter of Loading Area, d (ft)	Pavement Weight per Unit Area, q' (psf)	$\frac{\gamma d}{2}\sqrt{N_\phi}(N_\phi^2-1)$ (psf)	$q'N_\phi^2$ (psf)	$2c\sqrt{N_\phi}(N_\phi+1)$ (psf)	q_d (psf)
A	100	0	133	0	45.5	5.97	1.67	115	9,410	4,100	0	13,500
B	86	14	137	700	36.0	3.87	1.67	115	3,110	1,710	13,300	18,100
C	74	26	139	720	35.5	3.77	1 67	115	2,980	1,640	13,300	17,900
D	66	34	138	1,700	21.0	2.12	1.67	115	580	520	15,400	16,500
E	60	40	137	1,800	24.0	2.37	1.67	115	820	650	18,700	20,200
F	53	47	136	1,720	22.5	2.24	1.67	115	680	580	16,700	18,000
G	0	100	126	2,300	23.0	2.28	1.67	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

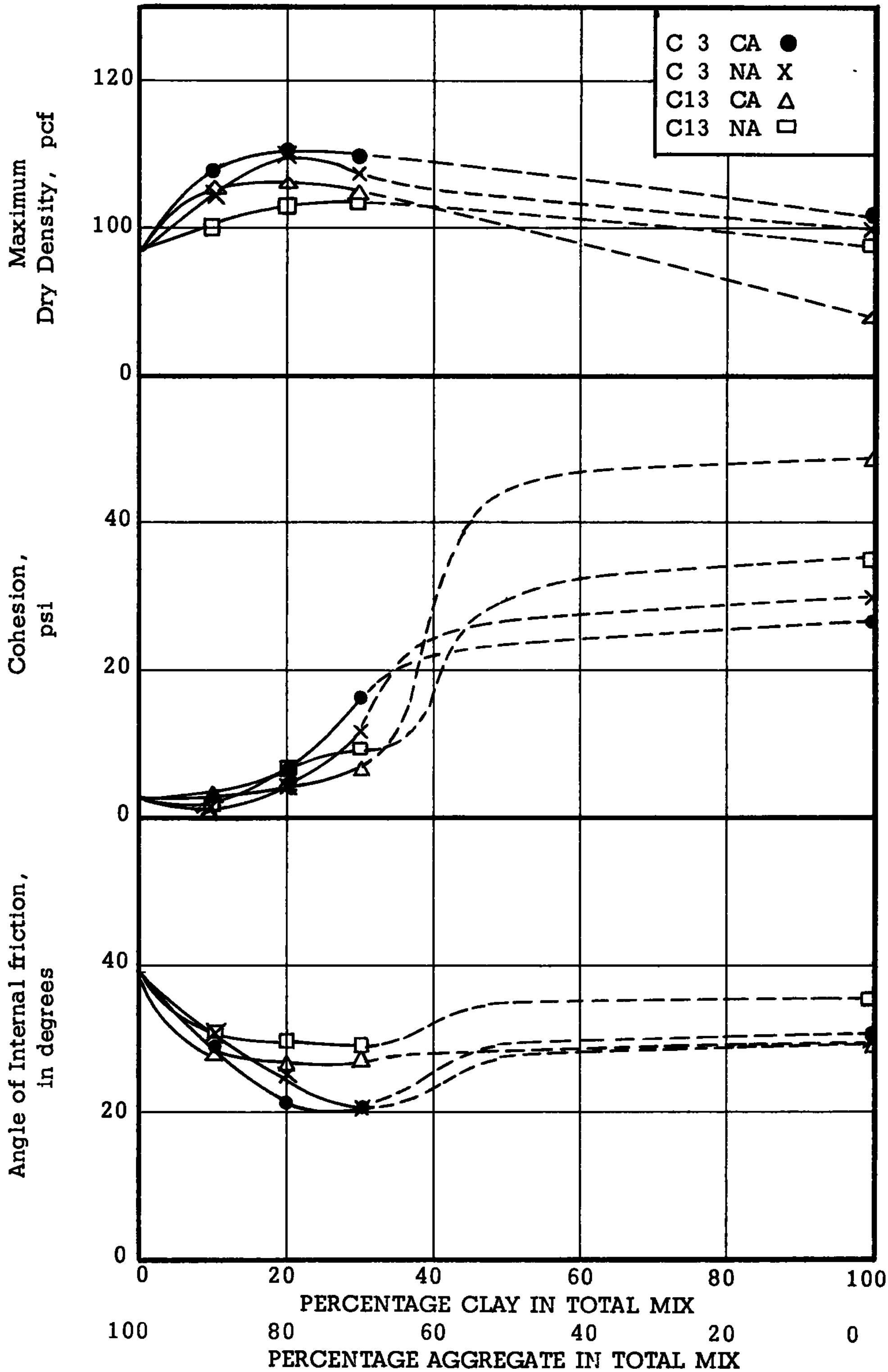
C 3 CA ●
C 3 NA X
C13 CA Δ
C13 NA □
Maximum Dry Density, pcf
120
100
0
Cohesion, psi
40
20
0
Angle of Internal friction, in degrees
40
20
0
0
20
40
60
80
100
PERCENTAGE CLAY IN TOTAL MIX
100
80
60
40
20
0
PERCENTAGE AGGREGATE IN TOTAL MIX

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 apheres 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.