FACTORS INVOLVED IN STABILIZING SOILS WITH ASPHALTIC MATERIALS

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SYNOPSIS

This paper describes various observations made on the properties of soils during investigations conducted to obtain information which would assist in the design of soil-asphalt mixtures for structural use. Rather close adherence to experimental data has been maintained in its presentation so that the opinions advanced are limited to results recorded for the selected test conditions. Extended theoretical discussions have been avoided since in the case of such poorly defined material as compacted soil, they are apt to lead to conclusions difficult to demonstrate conclusively.

In addition to the liquid and plastic limits, the exudation value or the amount of liquid retained under a selected compaction loading has been employed to classify a soil and supply information for the design of a soil mixture. A mutual relationship apparently exists between these values which can be expressed mathematically, thereby indicating that they are all based upon the same fundamental soil property

The usual soil constants attempt to portray the behavior of the unconfined soil in the presence of water and give little information on the properties of soil in the highly compacted state in which it is employed in earth constructions. A series of constants have been developed which give this information and thus aid in the further characterization of soils from the standpoint of their practical manipulation.

Except for lime which appears to be specifically adapted for improving poor soils, the use of chemicals in general for improving the properties of poor soils so that they may be satisfactory for construction purposes appears to have little practical value

By using non-aqueous liquids instead of water, compacted soils having much greater load-bearing strengths can be produced, however they generally show lower dry soil densities Light petroleum distillate oils appear to be the most effective in this respect.

In soil design, the following principles should be taken into consideration (a) the angle of failure and therefore, the angle of load distribution does not have a constant value of 45 deg but decreases with increase in soil thickness, approaching the limit of 0 deg to the vertical, namely simple shear, (b) the load-bearing capacity varies according to the 1 5 power of the thickness of the compacted soil, (c) the greater the number of layers used to build up a given thickness of compacted soil, the greater the strength and the soil density, (d) tensile strength rather than shear strength is determinative for the load-bearing capacity of compacted soil at its point of failure

For preparing soil-asphalt mixtures, satisfactory results will be obtained when the ratio of the soil fraction passing a No 10 sieve but retained on a No 200 sieve to the soil fraction passing the No 200 sieve is greater than 1 75 to 2 0 For soils showing a smaller ratio, special studies should be made.

This presentation is an abridgement of the original paper which has been prepared to conserve paper in accordance with the regulations of the War Production Board.

Interest in improving the properties of soils employed in road construction to withstand better the detrimental effects brought about by the changing ambient moisture conditions associated with their topographical and climatic locations has already yielded a considerable literature on the subject Soils causing the greatest concern from a moisture stability

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standpoint are those high in silt and clay and the various treatments which have been developed to improve their quality can be grouped as

(a) The dilution procedure in which poor soil is diluted with granular aggregate until the deleterious properties of the poor soil are reduced in amount until they cease to be significant;

(b) Treatment with chemicals so that the soil will maintain a suitable moisture content under fluctuating climatic conditions and thereby insure its stability,

(c) The addition of hydraulic setting material by which the soil is cemented into a more or less completely stabilized mass;

(d) The addition of waterproofing material such as bituminous binders by which it is attempted to render the soil more or less impervious to water and thus retain originally designed properties

The Esso Laboratories have been particularly interested in the last procedure and have already presented some of their findings and conclusions on the use of asphaltic materials in the stabilization of soils[3,4] The investigations have been continued on various factors which it was felt would contribute to the understanding of the principles underlying the asphaltic stabilization of soils and the design of stabilized soils mixtures Other workers may have already worked along similar lines and disseminated their findings which, however, have not become familiar to the writers.

CONSTANTS OF THE SOILS

One of the useful constants in the design for a stabilized soil is the exudation value, namely, that quantity of liquid which a soil can contain without being squeezed out under consolidation or without producing a plastic mixture In addition to being related to the specific properties of the soil, the exudation is further dependent upon the liquid added, such as water, asphalt or their mixtures, the consolidating load applied and whether the loading is applied continuously or intermittently. In compacting the soil to note its exudation point, it is advisable to employ the single plunger rather than the double plunger to more accurately gauge the results, especially when plastic mixtures are obtained.

The exudation values for the 50:50 water–MC-2 cutback mixtures are generally around 60–70 per cent greater than those for water alone.

Because of the existence of voids in the soil-liquid mixtures at their exudation point, it appears that asphalt-stabilized soils should tend to become richer in character under traffic consolidation due to the gradual removal of the voids, although they may appear somewhat lean at the time of construction. The reproducibility of the exudation point

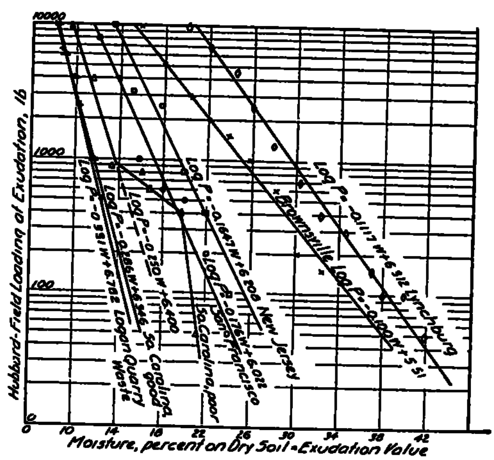

Figure 1. Exudation Values of Soil—Water Mixtures vs. Compaction Loading

appears to be within ±1 5 per cent from the average of a series of values

Figure 1 illustrates the variation between compaction loading and water exudation value for several soils, which can be summarized into the empirical expression

$$\text{Log } P = -mw + 6.385,$$

in which P is the total compaction loading applied to the briquette on molding to produce exudation, m the slope of the line on semi-logarithmic plot which varies with the soil, and w the exudation value as per cent moisture. The value 6 385 is the average of the various intercepts of the lines on the axis of compaction loading at zero moisture If

[3] J C Roediger and E W. Klinger, *Proceedings*, Highway Research Board, Vol 18, Part II, p 299, 1938

[4] J C Roediger and E. W Klinger, *Proceedings*, Association of Asphalt Paving Technologist, Vol 10, p 1, January, 1939

the exudation value at any compaction loading has been ascertained, the use of the above relation permits estimation of the exudation value at any other compaction loading similarly applied.

It would appear that some relationship should exist between the exudation value determined under various compaction loadings and the liquid limit, plastic limit, and plasticity index. As represented in Figures 2 and 3 for the liquid and plastic limits, this relationship is linear with the lines passing through the origin. It has been graphically determined that the relationship between exudation value, compaction loading, and either the liquid or plastic limit assumes the form of

$$y = \frac{A - \log P}{B} x,$$

y being the exudation value, P the compaction loading on a 2-in diameter by 1-in. high briquette, x either the liquid or plastic limit, and A and B constants. For the liquid limit, the data indicate $A = 6\,471$ and $B = 7\,1$; for the plastic limit, $A = 7\,013$ and $B = 4\,4$ The plasticity index yields a somewhat different relationship, although linear, in that intercepts varying from 6.5 to 9.6 for 10,000 to 1000 pounds compaction loading, respectively, are obtained on the exudation value axis for zero plastic index.

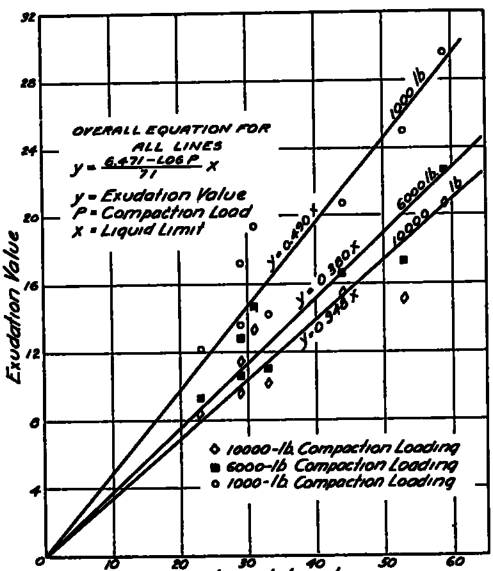

Figure 2. Exudation Values vs. Liquid Limit at Different Compaction Loadings

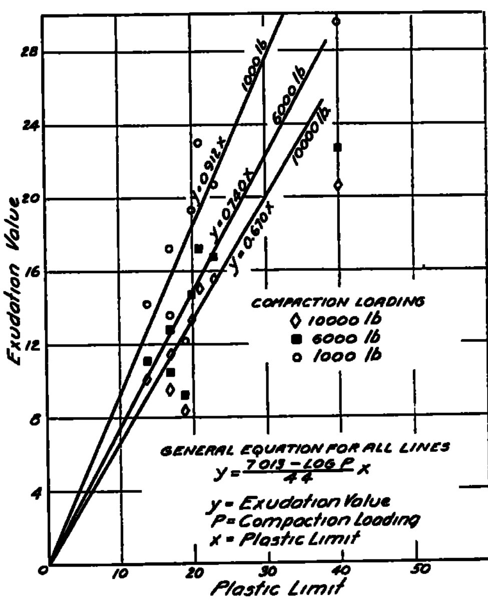

Figure 3. Exudation Value vs. Plastic Limit at Different Compaction Loadings

WETTING AGENTS AS AN AID IN SOIL COMPACTION

In the design of soil-water-asphalt mixtures, stress is laid on compacting such mixtures to their maximum densities. The maximum soil density obtained is affected by the nature and consistency of the liquid employed and the importance of soil-water mixtures in soil constructions led to ascertaining the possibility of increasing the maximum dry soil density by the addition of a wetting agent Compacted soil-water mixtures were prepared in which the water contained an effective wetting agent of the sodium tetraisobutylphenol sulfonate type, in 0 1, 0 5, and 1 0 per cent concentrations.

From the data obtained in these tests, it appears that the presence of wetting agents of the type employed is without effect for increasing the densities of compacted soil-water mixtures.

FACTORS AFFECTING THE SHEAR TEST

It is well recognized that the shear test on soils is influenced by factors introduced into the performance of the test, among which are:

(a) variation in the diameter of the plunger and of the orifice,
(b) rate of deformation during load application,
(c) height of specimen.

These several factors have been briefly investigated using 2-in diameter briquettes of a silty-clay, silty loam, sand and sand-clay soils, compacted under a total load of 6000 lb. Plunger diameters of 0 28 to 2.0 in. and orifice diameters of 0.28 to 1.75 in. were used, while briquette heights varied from 0.5 to 3 5 in. In order to ascertain the effect of

Type 2. Plunger diameter equals the orifice diameter. Punching shear results with the length of the sheared plug generally equaling the height of the briquette for the short specimens, whereas, in the case of longer specimens, a slight compression of the sheared plug may occur. Failure occurs sharply and the load bearing values are reproducible.

Type 3: Plunger exceeds the orifice diameter but is less than the diameter of specimen. Compaction followed by upward extrusion of the soil mixture past the plunger occurs until the specimen becomes thin enough to be extruded through the orifice. Sharp breaks are not obtained with this combination of plunger and orifice.

Type 4: Plunger exceeds orifice and equals the diameter of specimen and mold Simple

TABLE 1
EFFECT OF ORIFICE AND PLUNGER DIAMETER VARIATION ON LOAD AT FAILURE

Soil Mixture	New Jersey Soil + 12% Water				Beaumont Soil + 6% Water 6% RC-2 Cutback			
Briquette Height—in .	1				1			
Deformation Rate—in min	1				1			
Wet Soil Density—lb. per cu. ft	141	141	141	141	135	135 5	136	135
Dry Soil Density—lb. per cu. ft.	126	126	126	126	120 5	121	121 5	120 5
Orifice Diameter—in	9/32	9/16	1⅛	1¾	9/32	9/16	1⅛	1¾
Total Load at Failure Using Plunger Diameters of								
9/32 in	*200*	150	100	150	*70*		50	50
9/16 in	650	*400*	250	250	150	*160*	50	150
1⅛ in.	2400	2250	*800*	450	2000	1700	*300*	50
1¾ in	10000	8900	4700	*1300*			2300	*450*
2 in.		10000+	7000	2300			4000	650
Shear Strength—lb per sq. in (Orifice & Plunger having equal diameter)	230	230	230	240	80	85	85	82

briquette diameter, tests were also run on 6-in diameter briquettes compacted to the same density as the 2-in. specimens.

One inch high briquettes were tested for strength using orifice diameters of 9/32 to 1¾ in in combination with plunger diameters of 9/32 to 2 in with the results shown in Table 1. Figure 4 shows the four types of failure observed along with a typical plot of total load at failure versus plunger diameter which indicates the range within which these types of failure occur for briquettes laterally confined in the two inch diameter mold The types of failure may be described as follows·

Type 1 Plunger diameter is less than the orifice diameter A truncated cone type of failure combining both shear and tension results and sharp failure occurs at low load bearing values

extrusion takes place and the load bearing values are quite reproducible.

From Table 1 it can be seen that for a given orifice diameter, the strength increases in an exponential manner with increase in diameter When the plunger diameter equals the orifice diameter, the indicated shear strength in pounds per square inch of sheared surface area are substantially equal for diameters, 9/32, 9/16, 1⅛ and 1¾ in using 2-in. diameter by 1-in high briquettes.

Comparison of the regular Hubbard-Field deformation rate of 2 4 in per min. with the rate of 1 in. per min. has shown that the total load values at both rates of deformation are roughly equal.

The effect of height of specimen on the load bearing properties of soil-water mixtures was evaluated on 2-in. and 6-in. diameter briquettes ranging in height from 0.5 in. to 3.5 in. with the orifice and plunger each 1.125 in. in diameter. For the specimens of the Brownsville silty-clay and New Jersey silty-loam soils 2 in. in diameter and greater than 1.5 in. high, upward extrusion of the soil around the plunger, accompanied by considerable cracking of the top surface, as well as compression under the sheared plug took place, which effects were not evident on specimens 1.5 in. or less in height. These changes only occurred with sandy soil to a

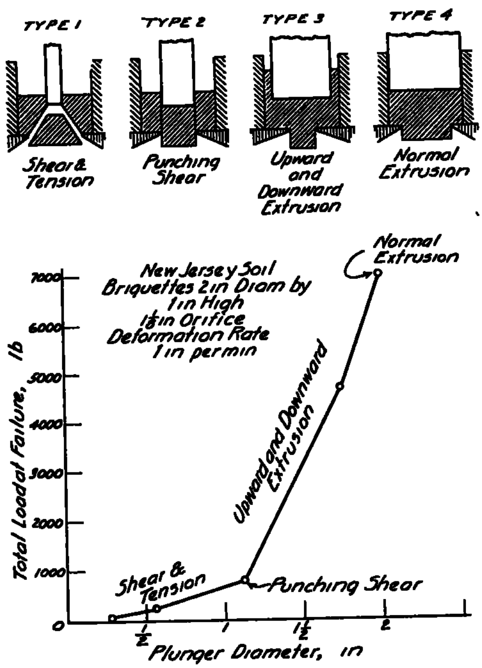

Figure 4. Diagrammatic Sectional Views of Type of Failure with Change in Diameters of Orifice and Plunger.

rather negligible extent on the thicker specimens. This suggests that 1.5 to 2.0 in. is the maximum height for proper evaluation, using the methods employed. The total punching shear strengths at failure obtained in these tests appear to vary in an exponential manner with the thickness of the specimens as indicated by the logarithmic plot (Fig. 5). The specific power of the thickness corresponding to the total punching shear load at failure is a function of the properties of the soil, the shear for the fine silty-clay soils tends to become a linear function of the specimen height, in contrast to the typically sandy soils in which the total punching shear tends to vary roughly with the cube of the specimen height. This is a striking illustration of the difference in behavior between the sandy and the fine grained soils, which is further illustrated in Figure 6 representing the relationship between the punching shear per square inch and the specimen height in inches. Punching shear strength per unit area can be calculated upon the sheared area over the

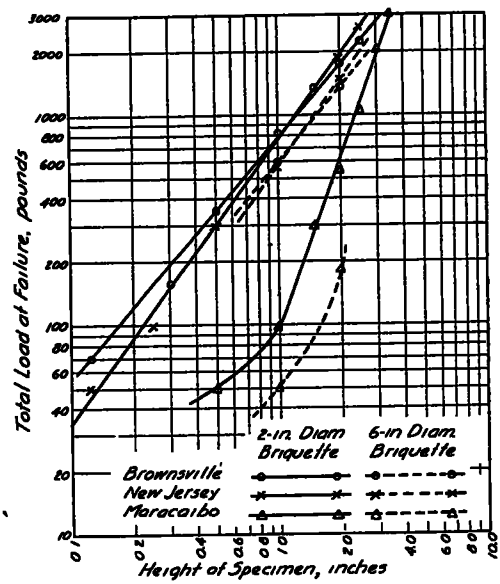

Figure 5. Effect of Briquette Height on Total Bearing Capacity of Soil-Water Mixtures Equations for 2-in. Diam. Briquettes.

Brownsville Log P = 1.157 Log H + 2.898
New Jersey Log P = 1.340 Log H + 2.878
Maracaibo Log P = 2.784 Log H + 1.978
P = Total Shear Load at Failure, lb.,
H = Height of Specimen, in.

total thickness of the specimen or that of the lateral area of the soil plug punched out. The line for the Maracaibo soil (2-in diameter specimens), Figure 6, vividly portrays the behavior of a sandy soil under the punching shear test by showing that sand exhibits little unit shear resistance in comparatively thin layers and it is for only appreciable thickness of compacted soil, that the shear resistance becomes important. At the greater thicknesses, consolidated sand may exhibit greater shear resistance than the fine grained soils. The two fine grained soils employed in these punching shear tests, decrease in the value of their indicated unit shear with decrease in specimen height due to the diminution of the effects of lateral support of the walls of the mold and that of the compacted soil. The effect of the wall support on the unit punching shear value is indicated in Figure 6 by the values for identical specimen heights for 2-in. and 6-in. diameter specimens at 1-in. and 2-in. height. The 2-in. diameter specimens give greater indicated unit shear values than those of 6-in diameter. The limiting specimen diameter beyond which uniform

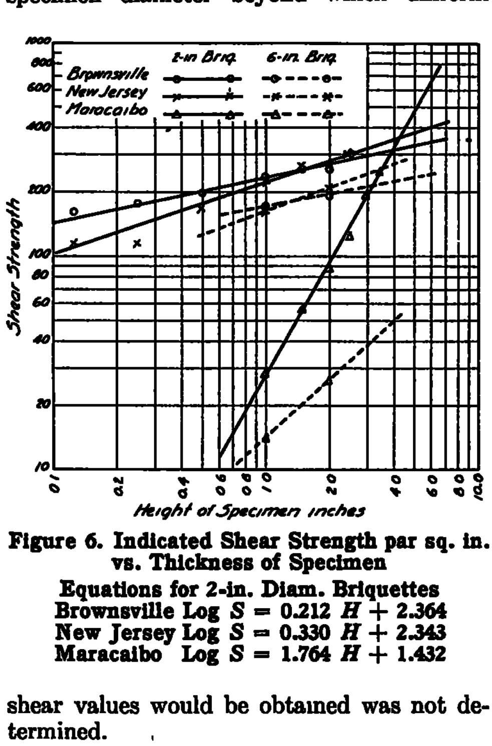

Figure 6. Indicated Shear Strength par sq. in. vs. Thickness of Specimen

Equations for 2-in. Diam. Briquettes
Brownsville Log $S = 0.212\ H + 2.364$
New Jersey Log $S = 0.330\ H + 2.343$
Maracaibo Log $S = 1.764\ H + 1.432$

shear values would be obtained was not determined.

EFFECT OF LIQUID DISTRIBUTION ON SOIL DENSITY AND STRENGTH

Investigation of a soil for its bituminous stabilization requires preparation of mixtures of soils, water and bituminous materials by a mechanical mixer. The appearance of the final mix is described in various terms such as lean, good, fat, rich, spotty, etc., but in nearly all cases, the compacted briquettes contain soil lumps which have not been disintegrated or wetted by the liquids. The effect of these hard soil lumps in the field has been a much debated question and their existence has been used to explain the differences noted between laboratory and field results. A short study was carried out to remove a part of the conjecture on this topic. Selected amounts of water were added to a soil rather difficult to stabilize with asphaltic binder, the largest amount of water employed corresponding to its previously determined water exudation point and briquettes prepared from the soil-water mixtures after undergoing various treatments to obtain distribution of the water throughout the soil. The presence and distribution of dry soil particles was determined by rubbing out the soil under a stiff knife to note streaks or lumps of dry material The data presented in Table 2 indicates the following conclusions.

(a) Improving the water distribution in a soil-water mixture has only small effect on the wet and dry soil densities of the compacted briquettes, but materially increases their strength as the distribution approaches completeness

(b) The best results were obtained by adding a large excess of water such as corresponds to the liquid limit and drying to the required moisture. The long period of drying required indicates this procedure to be generally unattractive from a practical standpoint

(c) The results next in order of effectiveness were obtained by thorough rubbing out of the soil after the soil-water mixture had been allowed to stand overnight This procedure is tedious to assure best results and makes difficult the control of the moisture from the excessive exposure of the soil-water mixture during its manipulation.

(d) Simple standing of the soil-water mixtures in closed containers gave optimum results on the overnight period, the 1 hr. period giving briquettes of lower strength The difference between the soil-water mixtures allowed to stand 1 hr and overnight decreases as the moisture content increases until it practically disappears at exudation moisture.

(e) Heating the soil and water in an autoclave failed to bring about any distribution of the moisture, the discharged material in all cases containing dry soil under a layer of mud.

These observations indicate that improving the moisture distribution in a soil-water mixture has no noteworthy effect on wet and dry soil densities, but improves the strength that can be obtained on the compacted soil, and that this distribution is best accomplished by over-wetting the soil and then drying to the desired moisture. Because of difficulties associated with the over-wetting procedure, it appears preferable to allow the well-stirred soil-water mixtures to stand overnight in tightly closed containers before compacting into briquettes.

TABLE 2

EFFECT OF UNIFORMITY OF WATER DISTRIBUTION ON PROPERTIES OF SOIL

(All Soils Compacted and Tested According to Hubbard-Field Procedure)

Preparation of Soil and Tests	Air Dry	New Jersey Red Soil — Nominal Water Content 5	8	12	14	16
1. Water added, mixture allowed to stand 1 hr. in closed container before compacting						
Appearance and Condition of Soil	Dry and dusty	Uniform		Slightly spotty		Small mud balls, uneven
Exude	No	No	No	No	Sl.	Yes
Moisture Determined—wt.%	3.06	5.0	8 02	11.98	13.71	15.72
Wet Soil Density—lb. per cu.ft	123 3	130 0	136 5	141.5	140.0	138.3
Dry Soil Density—lb per cu.ft	119 5	123 5	126 0	126 3	122.5	119 8
Hubbard-Field Strength—lb.	4700	4200	4400	2125	1220	600
2. Water added, mixture allowed to stand overnight in closed container before compacting						
Appearance and Condition of Soil		Few hard lumps		Uniform—no spots shown on rubbing out with spatula		
Exude		No	No	No	Sl	Yes
Moisture Determined—wt %		4 99	7 94	12 15	13 79	15.75
Wet Soil Density—lb per cu ft		127 5	136 0	141 0	139.5	138 3
Dry Soil Density—lb. per cu.ft		121 5	126 0	125 5	122 5	119.4
Hubbard-Field Strength—lb		6725	5075	2275	1175	600
3. Water added, mixture allowed to stand 3 days in closed container before compacting						
Appearance and Condition of Soil		Uniform—no spots or streaks on rubbing out with spatula				
Exude		No	No	No	Sl	Yes
Moisture Determined—wt.%		4.67	7 38	11.62	13 70	15 71
Wet Soil Density—lb. per cu ft		128 0	136 0	141 5	140 8	137 0
Dry Soil Density—lb per cu.ft.		122 5	126 5	127 0	124.3	118.5
Hubbard-Field Strength—lb.		5950	4775	2175	1175	550
4. Water added, mixture allowed to stand 7 days in closed container before compacting						
Appearance and Condition of Soil		Uniform throughout				
Exude		No	No	No	Sl	Yes
Moisture Determined—wt %		4 7	7 5	11 6	13 7	15 5
Wet Soil Density—lb per cu ft		128 5	136.5	141 8	140 5	140 0
Dry Soil Density—lb per cu ft		122 7	126 9	127 0	123 7	121.1
Hubbard-Field Strength—lb		6250	5125	2400	1150	650
5. Water added, allowed to stand 20 hr in closed container, then rubbed out with knife and mixed to produce uniform moisture throughout mix						
Appearance and Condition of Soil		Uniform throughout				
Exude						
Moisture Determined—wt.%		4 8	7 68	10 7	13 0	15 35
Wet Soil Density—lb per cu ft		128 7	135 8	140 3	139 8	139.0
Dry Soil Density—lb. per cu ft		122 9	126 2	126 9	123 7	120 3
Hubbard-Field Strength—lb		6375	5525	3150	1450	700
6 Water added to liquid limit, allowed to stand 20 hr , then dried to required moisture, pulverized through No 60 sieve, compacted						
Appearance and Condition of Soil		Uniform throughout				
Exude						
Moisture Determined—wt %		3 36	5 2	7 2	8 10	9 71
Wet Soil Density—lb. per cu ft		122 5	129 4	135 6	136 8	141.1
Dry Soil Density—lb per cu ft		118 3	123 0	126 3	126 3	129.0
Hubbard-Field Strength—lb.		5250	8600	7500	6600	4800

The effect of better liquid distribution was further investigated on a soil-water-cutback asphalt mixture along the lines given under

Method 5, with the results presented in Table 3. Although compacted briquettes having much greater strength are obtained, which is especially pronounced with the mixtures containing the greater amounts of liquids, the water resistant properties of the briquettes are not improved

STRENGTHS OF SOIL-WATER MIXTURES AT PROCTOR DENSITY AND AT DENSITIES OBTAINED UNDER 10,000 AND 6,000-LB. COMPACTION LOADINGS

The Proctor procedure for obtaining maximum soil density is based upon the idea that a soil-water mixture compacted to maximum dry soil density would be the most stable toward subsequent variations in moisture content of the soil and future compaction with time, the load bearing capacity of the Proctor compacted soil being generally obtained by the application of the Proctor needle Later experience has shown that the Proctor density as usually determined is not the maximum dry soil density that can be attained in the field so that various modifications of the original procedure have been introduced which seek to represent more closely the densities obtained in practice, such as increasing the weight of the plunger, the height of its drop and the number of blows. All such modifications must necessarily be more or less arbitrary. An examination has therefore been made of other methods for compacting and indicating the load bearing capacities of soil mixtures, which involve the compaction of the soils to much greater densities than obtained by the Proctor procedure. For this investigation soil containing a range of moisture contents passing through the Proctor optimum moistures was tested in the form of 2-in. diameter by 1-in. high briquettes:

(a) According to the Hubbard-Field procedure by compacting under 10,000 lb. total load in the Hubbard-Field machine, determining the Hubbard-Field strength and the dry and wet soil densities.

(b) By compacting under 6000 lb. total load in the Hubbard-Field machine and determining the punching shear strength using 1.125-in. plunger and orifice. The dry and wet soil densities were also obtained.

(c) By compacting under a loading in the Hubbard-Field machine to produce a briquette density equal to that produced by the Proctor compacting procedure and determin-

TABLE 3
EFFECT OF GOOD DISTRIBUTION OF LIQUIDS ON SOIL-CUTBACK ASPHALT-WATER MIXTURES
NEW JERSEY SOIL

1. Mixes prepared in usual manner				
Water—wt %	3.67	3.75	3.9	3.4
MC-2 Cutback—wt %	4	6	8	10
Mix Evaluation	*V.L.*	*V.L*	*L*	*F*
Wet Soil Density—lb. per cu ft.	130.0	132.3	134.5	134.5
Dry Soil Density—lb. per cu. ft	120.7	120.6	120.3	118.8
Hubbard-Field Strength—lb.				
Before 7-day Water Absorp.[b]	4075	3375	2175	1625
After 7-day Water Absorp	Disint.	Disint.	150[a]	300
Water Absorp.—g. per 100 sq cm.			27.3	14.1
Volumetric Swell—%			14.6	9.3
2 Same mixtures as above but pulverized and manipulated with spatula to produce a uniform texture				
Water—wt %	3.0	3.0	2.54	2.7
MC-2 Cutback—wt %	4	6	8	10
Mix Evaluation	*L*	*L*	*L*	*F*
Wet Soil Density—lb per cu. ft.	128.0	127.5	131.5	134.5
Dry Soil Density—lb. per cu ft	119.5	117.0	119.0	119.4
Hubbard-Field Strength—lb.				
Before 7-day Water Absorp	4900	4900	4600	3900
After 7-day Water Absorp.	Disint	Disint	Disint.	200
Water Absorp —g per 100 sq. cm				17.9
Volumetric Swell—%				10.3

[a] Cracked on edges
[b] Water absorption on 2-in. diam by 1 in high briquettes immersed to a depth of ½ in.
Mix Evaluation· *V L* = very lean; *L* = lean, *F* = fat.

ing the Hubbard-Field strength and 1.125-in. plunger-orifice shear strength on the compacted briquettes. The dry and wet soil densities were also obtained

Table 4 gives the properties of the soils at their maximum dry soil densities under the several compacting loadings From the large amount of data procured from these tests, the following items appear worthy of mention

1. In agreement with observations by other workers in this field, the maximum dry soil density increases with increases in compaction pressure and is accompanied by progressively lower moisture contents; simple linear relationship exists between the maximum dry soil density and the corresponding moisture content One point on this line may be that representing the Proctor maximum dry soil density and its corresponding Proctor optimum water and the other, the density of the solid aggregate in pounds per cubic feet having zero moisture (=62 4 × specific gravity of dry soil) The slopes of the lines or the change in maximum dry soil density with corresponding changes in moisture content as shown in Table 4 indicate that good soils exhibit a more rapid increase in dry soil density than the poorer soils From this it may be concluded that the sandy or good soils consolidate more rapidly than the poorer clayey or silty soils

2 The static compaction loading required to form 2-in by 1-in briquettes having the same density as that obtained on the soil by the application of the regular Proctor procedure is not constant but decreases with increasing moisture content of the soil-water mixture. The required compaction loading is largely dependent on the nature of the soil, the present data indicating 4000–5000 lb. for good sandy soil as compared to 400–700 lb. for the clayey-silty soils when they contain less than their critical moisture. The relative loadings required to obtain Proctor densities apparently have possibilities for classifying soils and for correlation with their load bearing capacities and suitability for stabilization

3. The maximum dry soil density obtained under the Proctor method of compaction occurs at higher moisture contents than for mixtures having maximum Hubbard-Field strength. The moisture at maximum Hubbard-Field strength corresponds closely to that indicated for maximum dry soil density when the soil is compacted under 10,000-lb. loading. Sandy soils exhibit no such points of coincidence since no maximum strength is indicated in these tests for such soils compacted under Hubbard-Field loading to Proctor density The existence of maxima in two properties of soil-water mixtures compacted to Proctor densities, namely, dry soil density and maximum Hubbard-Field strength, except in the case of sandy soils, raises the question as to which should be employed in soil designs Soil designs in general tend to employ too large amounts of moisture so that mixtures containing less water are to be preferred, such, for instance, as that indicated by the maximum dry soil density, obtained under the 10,000-lb Hubbard-Field loading

TABLE 4
PROPERTIES OF SOILS AT THEIR MAXIMUM DRY SOIL DENSITIES

Soil	South Carolina Good			South Carolina Poor			New Jersey Red			San Francisco		
Specific Gravity		2 647			2 662			2 749			2 736	
Compaction Loading on 2 in by 1 in Briquette	300	6000	10000	525	6000	10000	500	6000	10000	425	6000	10000
Moisture at Max Dry Soil Density—%	10 1	8 7	8 5	12 5	9 9	8 9	17 4	11 0	9 8	15 3	11 4	10 8
Maximum Dry Soil Density	125 2	130 5	131 0	119 0	126 0	129 0	112 0	124 9	127 0	113 5	126 3	129 2
Coefficient Density vs. % Moisture		−3 91			−3 54			−2 07			−3 30	
Coefficient—Log Compaction Loading vs % Moisture		−0 936			−0 377			−0 183			−0 302	
Coefficient—Log Compaction Loading vs Maximum Dry Soil Density		+0 246			+0 132			+0 087			+0 089	
Calculated Compaction Loading to Produce Maximum Dry Soil Density at 0% Moisture		$3\ 28 \times 10^{12}$			$1\ 15 \times 10^{7}$			$5\ 0 \times 10^{6}$			$2\ 53 \times 10^{8}$	

4. The change in compaction loading with variation in moisture content at maximum dry soil density could possibly serve as a soil characteristic and may be expressed as the corresponding change in the logarithm of the compaction loading, thus

$$\log P = -mx + b,$$

P being the compaction loading on a 2-in by 1-in briquette, x the moisture content and m and b constants The value of m from the data presented is approximately -0.9 for sandy soils and around -0.3 for the poorer soils In other words, it is easier to dehydrate a poor soil by increasing the load upon it than is the case for a good sandy soil.

5 The rate of increase in maximum dry soil density with increase in compaction loading is dependent upon the specific soil, and may be expressed as the change in the logarithm of the compaction pressure with the change in maximum dry soil density, thus

$$\log P = nD + c,$$

P being the compaction loading on a 2-in by 1-in. briquette, D the maximum dry soil density, n and c constants The value for n from the data presented amounts to $+0.246$ for the good sandy soil and around 0 1 for the poor soils As a matter of interest, the calculated loading required to produce maximum density, namely 62.4 times specific gravity of the dry soil indicate 3.28×10^{12} lb. for the sandy soil and 1.15×10^{7} and 2.53×10^{8} for the poor soils, thus showing the greater resistance to ultimate consolidation for the sandy soils Such values might also serve as qualifying soil indices

Summarizing the information on the behavior of the soil-water mixtures of the soils employed in these tests, the preferred procedure for soil design appears to be to compact the soil into briquettes of 2-in. diameter by 1-in high under 10,000-lb loading and evaluate their strength by the Hubbard-Field test The optimum moisture will be that indicated for the mixture showing maximum dry soil density under this procedure

CHEMICAL TREATMENT OF SOILS AS A PRELIMINARY STEP IN THEIR STABILIZATION

Soil properties are determined by the relative proportions of its constituents which may be broadly grouped into the granular material such as gravel, sand and silt characterized by lack of bonding action and the clays having great bonding action It is the clay fraction which primarily contributes to the undesirable changes in a soil under the influence of moisture when present in excessive amounts, although this is shared to some extent by the silts Attempts have been made by various workers to modify the properties of these fine soil fractions so as to improve the soil constants Results obtained by the use of chemicals to render soils suitable for bituminous stabilization are presented.

For this investigation a soil refractory to stabilization treatment was employed. This was an active, silt-clay, falling within the A5–6 classification of the Public Roads Administration, and very sensitive to the amounts of water which could be admixed before becoming plastic and difficult to work. To 50 g of the soil, selected quantities of chemicals in aqueous solution were added to give a total mixture of 250 cc, the suspension allowed to settle, the supernatant liquid removed by decantation and the settled treated soil evaluated for its soil constants. The chemical treatments were grouped as follows:

(1) To note the relative effects of cations, a series of chlorides was employed with the results given in Table 5

(2) The relative effect of the anion was noted by employing a series of sodium salts varying in their anion, with the results given in Table 6.

(3) The effect of treating the soil with acid (hydrochloric acid) has been summarized in Table 7

(4) The results of the use of selected acids, bases and salts have been summarized in Table 8.

(5) The behavior of the soil treated with a selected series of chemicals, when subsequently treated with MC-2 cutback asphalt was examined in the form of compacted soil briquettes, the positive results have been recorded in Table 9

Effect of Cations

As summarized in Table 5, it appears that the cations affect but slightly the liquid limit of the soil with the possible exception of potassium which slightly increased the liquid limit Where effective, it appears that the cation tends primarily to increase the plastic limit, which is to say that the soil can retain more moisture before becoming unduly plastic. As a corollary to this, the plasticity index is decreased. As a rule, it is considered that a soil improves in quality for construction purposes with decrease in the plasticity index, so that the changes wrought by the effective cations may be considered beneficial. Increased settling volumes were observed for the aluminum, chromium and ferric chlorides but these are fictitious since the increased settling volumes were largely due to the hydroxides of these metals which formed in contact with the soil.

The most effective salts in this series for improving, that is, for reducing the plasticity index of the soil, appear to be, in order of decreasing merit, aluminum chloride, chromium chloride, ferric chloride and sodium chloride.

Effect of Anions

From the data in Table 6, it appears that the anions are much more active in influencing the properties of soils than the cations. In a few instances the cations affect the plastic limit. The anions, where effective, tend primarily to increase the liquid limit, the most effective anions being the silicate and oxalate followed by the hydroxide and carbonate. The result of this effect is to increase the moisture holding power of the soil before it will tend to flow Although the anions in general exert little influence on the plastic limit, the hydroxyl is outstanding in this action. In addition, the hydroxide ion and the chloride ion tend to decrease the plasticity index as shown by sodium hydroxide and sodium chloride It may be concluded that the effect of the active anions is to increase the plasticity index, but instances occur in which the plasticity index is decreased because of the increase in the plastic limit as for example, with sodium hydroxide or sodium chloride, or as a result of a decrease in the liquid limit as occurs with sodium nitrate, sodium dichromate and sodium acetate.

In addition to increasing the liquid limit of the soil, and altering the plasticity index the anions also frequently act to reduce the shrinkage limit. This is tantamount to saying that the range in moisture content over which shrinkage occurs on drying has increased and thus the soil has been rendered less desirable for construction purposes Sodium hydroxide appears to be an exception to this behavior in that it increases the shrinkage limit.

The shrinkage ratio is, in general, little affected by the anion employed except that it appears to be decreased by sodium hydroxide and sodium oxalate, thus to increase the volume of the dried soil, and increased by sodium carbonate, thus to decrease the volume of the dried soil.

Notable increases in the settling volumes of the soil were noted with the use of sodium oxalate, sodium carbonate and sodium silicate, the settling volume decreasing in the order given Sodium silicate is often used to assist in the dispersion of a soil for sedimentation

TABLE 5
EFFECT OF CATIONS UPON SOIL CONSTANTS

Kind of Salt	Wt % on Soil	pH	Settling[a] Volume-cc	Liquid Limit	Plastic Limit	Plastic Index
Barium Chloride	5	7 1	88	40 9	22.5	18 4
Magnesium Chloride	5	7 0	91	40 8	21.4	19 4
Calcium Chloride	5	7 0	91	41 1	21 8	19 3
Potassium Chloride	5	7 1	92	46 0	24 9	21 1
Sodium Chloride	5	7 5	94	41 0	24 6	16 4
Copper Chloride	5	6 5	94	42 1	20 4	21 7
Manganese Chloride	5	6 5	98	43 7	22 8	20 9
Ferric Chloride	5	6.6	112	40 8	25 3	15 5
Aluminum Chloride	5	6 4	121	39 5	27 2	12 3
Chromium Chloride	5	6 6	141	42 2	28 3	13 9
Water		7 4	94	42.6	21 5	21 1

[a] Apparent volume of soil after settling for 2 hr. in 250 cc. graduate.

analysis, but these data suggest that both sodium oxalate and sodium carbonate would be better suited for this purpose

Summarizing, of the various salts listed in these tests, the sodium chloride and sodium hydroxide appear to have beneficial effects for reducing the plasticity index, whereas the sodium silicate, sodium oxalate and sodium carbonate act to increase it.

TABLE 6
EFFECT OF ANIONS UPON SOIL CONSTANTS

Kind of Salt	Mols per 50 g Soil	Settling Volume-cc	Liquid Limit	Plastic Limit	Plastic Index	Shrinkage Limit	Shrinkage Ratio
Sodium Chloride	0 015	67	41 0	24 6	16 4	15 4	1 79
Sodium Sulfate	0 015	74	39 8	'21 0	18 8	17 1	1 73
Sodium Nitrate	0 015	68	38 4	21 6	16 8	16 1	1 73
Sodium Hydroxide	0 015	78	47 5	36 5	11 0	19 1	1 66
Sodium Carbonate	0 015	130	46 3	20 8	25 5	12 3	1 87
Sodium Dichromate	0 015	69	39 0	21 0	18 0	15 5	1 71
Sodium Acetate	0 015	66	39.0	21 4	17 6	15 3	1 72
Sodium Oxalate	0 015	186	52.0	22 2	29 8	10.9	1 66
Sodium Silicate	0 015	90	59.0	22 3	36 7	7 1	1.73
Water		72	42 6	21 5	21 1	- 15 9	1 76

Effect of Acid

The effect of acids upon the soil constants is indicated in Table 7, from which it is seen that the main result of such treatment is to reduce the liquid with practically no effect on the plastic limit with a resulting decrease in the plasticity index. These effects are, however, only attained for acids having a pH of approximately 6.1 or less. To produce a plasticity index of approximately 3 for the soil in question requires an acid (hydrochloric acid) having a pH of 1.9.

TABLE 7
EFFECT OF ACID (HYDROCHLORIC ACID) UPON SOIL CONSTANTS

Hydrochloric Acid		Liquid limit	Plastic limit	Plastic index
pH	Mols per 50 g soil			
7 1	0 015	41 0	21 0	20.0
6 2	0.020	41 6	21.4	20 2
6 2	0 0215	42 0	20 6	21 4
6 0	0 03	40 0	20 2	19 8
6 1	0 05	39 6	22 6	17 0
5 8	0 10	37 0	21 7	15 3
4 7	0 20	32 6	20 5	12 1
2 5	0 30	31 2	20 6	10.6
2 2	0 40	31 9	21 3	10 6
1 9	0 50	25 0	21 8	3 2
Water		42 6	21 5	21.1

Effects of Various Acids, Salts and Bases

An additional series of acids, bases and salts were investigated as given in Table 8. In this series no pronounced effect was produced upon the liquid limit, whereas, except for phosphoric acid and to a lesser extent nitric and acetic acids (which decreased the plastic limit), the remaining materials, where effective and, notably lime, tended to increase the plastic limit, or as a corollary to decrease the plasticity index. The effective materials also tended to increase the shrinkage limit so that their net results would appear beneficial. These changes are accompanied by a decrease in the shrinkage ratio.

TABLE 8
EFFECT OF SELECTED ACIDS, SALTS, AND BASES UPON SOIL CONSTANTS

Reagent	Mols per 50 g soil	Liquid limit	Plastic limit	Plastic index	Shrinkage limit	Shrinkage ratio
Sulfuric Acid	0 015	42 0	21 7	20 3	16 2	1 78
Nitric Acid	0 015	42 9	18 9	24 0	15 8	1 77
Hydrochloric Acid	0 015	41 0	21 0	20 0	16 5	1 75
Phosphoric Acid	0 015	38 1	15 5	22 6	14 9	1 79
Acetic Acid	0 015	40 2	18 9	21 3	15 0	1 78
Boric Acid	0 015	37.4	27 8	9 6	18 6	1 70
Oxalic Acid	0 015	41 4	22 3	19 1	15 8	1 76
Di-Sodium Phosphate	0 015	43 0	21 7	21 3	15 1	1 78
Calcium Acid Phosphate	0 015	41 7	19 7	22 0	15 4	1 72
Borax	0 015	37 9	29 6	8 3	19 2	1 68
Potassium Permanganate	0 015	42 0	28 9	13 1	19 9	1 65
Lime	0 015	39 7	30 8	8 9	24 2	1 58
Lime Solution—Sat'd		42 2	21 2	21 0	16 1	1 72
Lime	10%	38 2	36 3	1 9	28 6	1 50
Water		42 6	21 5	21 1	15 9	1 76

Stabilization of the Chemically Treated Soils

The chemically treated soils were further investigated by the regular procedure of adding cutback asphalt, preparing molded specimens and submitting them to the usual water absorption test The successful treatments are presented in Table 9.

The complete data indicate that lime pretreatment of the soil gives by far the best results; potassium permanaganate has moderate beneficial action; phosphoric acid and boric acid have some beneficial effect; the other chemicals, although varying in the results produced, appear to be without practical interest

TABLE 9

EFFECT OF SELECTED CHEMICAL AGENTS UPON THE STABILIZATION OF SOIL BROWNSVILLE SOIL

Treatment					
Water, wt %.	10 0				
Chemical Agent	Acid		Base		Salt
Kind	H_3PO_4	H_3BO_3	Hydrated Lime		$KMnO_4$
Amount, mols per 50 g. soil	0 015	0 015	0 015	0 068	0 015
wt %	2 94	1 86	2 22	10 0	4 72
MC-2 Cutback, wt %	10 0	10 0	10 0	10 0	10 0
Soil Constants					
Liquid Limit	38 1	37 4	39 7	38 2	42 0
Plastic Limit	15 5	27 8	30 8	36 3	28 9
Plastic Index	22 6	9 6	8 9	1 9	13 1
Shrinkage Limit	14 9	18 6	24 2	28 6	19 9
Shrinkage Ratio	1 79	1 70	1 58	1 50	1 65
Mix Evaluation	G	G	G	G	G
Compaction Loading, lb.	10,000	10,000	10,000	10,000	
Exudation	—	—	—	—	—
Briquettes					
Wet Soil Density, lb per cu. ft	128 5	127 5	128 0	127 5	130 0
Dry Soil Density, lb. per cu ft.	107 0	106 2	106 6	106 2	108 3
Hubbard-Field Strength, lb					
Before 7-day water absorption	1400	1400	1850	3100	1350
After 7-day water absorption	100	100	900	900	300
Water Absorption, g per 100 sq. cm.	57.6	63 7	14 0	6 7	36 8
Volumetric Swell, %	12 8	19 0	7 6	2 7	11 2

HEAT TREATING SOILS PRELIMINARY TO BITUMINOUS STABILIZATION

Refractory soils do not respond readily to simple treatment of water and asphalt Such soils can often be handled by the addition of chemical agents, of which lime has been the most successful, or partial curing of the soil mixture before compacting. Another possibility of modifying soil properties preparatory to stabilization with bituminous materials is by heat treating the soil at a moderate temperature. In order to investigate this feature, two refractory soils were heat treated at 140 F , 250 F. and 390 F. for two days before the application of water and cutback asphalt. One of the soils was investigated both with and without the addition of lime. These tests are presented in Table 10, which also includes data on the air-dried soil for comparison

The information obtained on the effects of heat treatment before stabilization appears to be as follows·

(1) No change is produced in either the dry or wet soil densities

(2) For the Hubbard-Field strength test on the freshly compacted briquettes, heat treating at 140 F produces somewhat lower strength than was obtained for the air dried soil, but at increased temperatures, these strengths increase until at 390 F they somewhat exceed those obtained with the air-dried soil

(3) After 7 days water absorption the Hubbard-Field strengths increase with increase in temperature of heat treatment. Also, after absorption, for the soil containing lime, strengths ranging from 600 to 1400 lb. were

obtained from heat treatment as compared to 300 lb for the air-dried lime-treated soil.

materials. The relation between time of heating and temperature was not investigated.

TABLE 10
EFFECT OF HEAT TREATING SOIL ON ITS STABILIZATION

Soil	A		B			
Water Added—wt %	8	8	10	10	10	10
MC-2 " — "	4	8	5	10	5	10
Lime " — "	0	0	0	0	5	5
Air dry—not heat treated						
Mix Evaluation	*L*, Spotty	*F*, Sl., Spotty	*L*, Spotty	*F-G*, Sl., Spotty		*G*
Wet Soil Density, lb per cu ft.	137.9	134.4	133.5	127.5		122.5
Dry " " "	123.0	115.9	136.0	106.2		98.0
Hubbard-Field Strength, lb.						
Before Water Absorption	2475	900	2825	900		1250
After " " (7 days)	125	175	Disintegrates			300
Water Absorption, g per 100 sq cm	21.5	9.6		a		7.8
Swell, vol %	9.2	8.1				5.9
Heat treated 2 days at 140°F.						
Mix Evaluation	*L*	*F-G*, Sl., Spotty	*L*, Spotty	*F-G*, Sl., Spotty	*L*	*F-G*, Sl., Spotty
Wet Soil Density, lb per cu. ft.	137.1	138.4	131.4	125.8	130.6	125.6
Dry " " "	122.5	119.3	114.3	104.8	108.8	100.3
Dry " " " (Soil + Lime)					114.2	105.3
Hubbard-Field Strength, lb						
Before Water Absorption	1825	775	1500	650	2975	1050
After " " (7 days)	150	150	Disintegrates		600	325
Water Absorption, g per 100 sq. cm.	18.7	9.1		a	12.3	11.0
Swell, vol %	10.2	10.0			7.9	7.4
Heat treated 2 days at 250°F						
Mix Evaluation	*L*	*F*, Spotty	*L*, Spotty	*F*, Spotty	*L*	*F*, Spotty
Wet Soil Density, lb per cu ft	136.9	133.5	132.1	127.8	131.3	127.9
Dry " " "	122.0	115.0	115.0	106.4	109.6	102.2
Dry " " " (Soil + Lime)					115.0	107.3
Hubbard-Field Strength, lb						
Before Water Absorption	1875	825	2225	975	3900	1550
After " " (7 days)	175	250	Disintegrates		1075	700
Water Absorption, g per 100 sq cm	19.3	9.0			11.4	6.8
Swell, vol. %	9.5	5.5			5.0	5.0
Heat treated 2 days at 390°F						
Mix Evaluation	*L*	*L-G*, Sl., Spotty	*L*	*G*	*L*	*F-G*, Sl., Spotty
Wet Soil Density, lb per cu. ft	138.0	134.8	134.5	128.5	131.0	128.0
Dry " " "	123.2	116.2	117.0	107.0	109.2	102.3
Dry " " " (Soil + Lime)					114.7	107.5
Hubbard-Field Strength, lb						
Before Water Absorption	2800	1050	3225	1000	5150	1750
After " " (7 days)	300	500	Disint [b]	100[c]	1400	750
Water Absorption, g per 100 sq cm	18.3	18.7(?)	"	17.4	9.8	5.8
Swell, vol %	7.5	4.2	"	12.6	5.2	5.0

L = Lean, *F* = Fair, *G* = Good
[a] Binder stripping from soil
[b] Cracked on the top of the briquette
[c] Cracked on sides of briquette near bottom

(4) Water absorption tends to decrease slightly with increase in temperature of heat treating.

(5) Volumetric swell tends to decrease with increase of temperature of heat treating.

It appears that heat treating refractory soils can improve their properties from the standpoint of stabilization with bituminous materials. The relation between time of heating and temperature was not investigated.

EFFECTS OF VARIOUS LIQUIDS ON PROPERTIES OF COMPACTED SOILS

Soil stabilization with asphalt employs almost exclusively cutbacks which are mixed with the soil previously moistened with water to assist in the distribution of the cutback and to improve the water proofness of the mixture. The dry soil densities of the compacted soil-water-asphalt mixtures, however, are as a rule, not as great as can be obtained with water alone and the mixtures exhibit lower strengths. High dry soil density is one of the objects sought in compacted soils in road and embankment construction. It was thought that this would be facilitated by the presence of certain wetting agents which had the property of lowering the surface tension

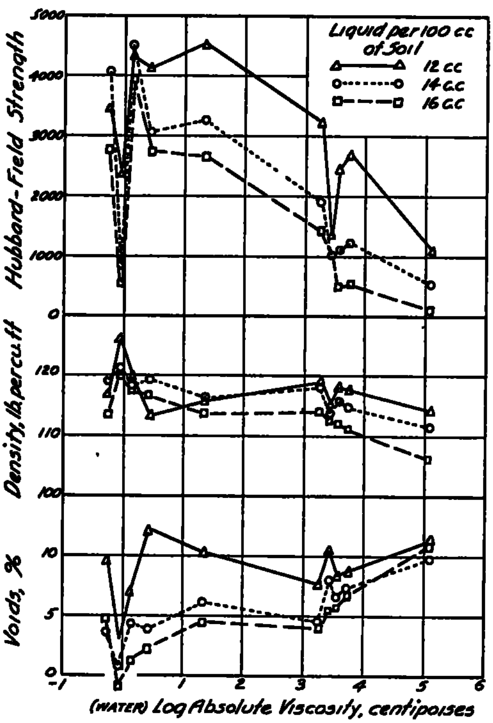

Figure 7. Relative Effects of Water and Oils and Compacted Soils. New Jersey Soil. Briquettes compacted under 10000—lb. total load.

of the water, but such agents appear to be without effect.

The greater strengths and dry soil densities usually observed with the straight soil-water briquettes as compared to those containing bituminous materials has been ascribed to the lubricating action of the water. This feature has been given some study by employing liquids and oils covering a wide range of viscosities. The high strengths obtained although combined with comparatively low dry soil densities suggest that dry soil density may lack some of the importance ascribed to it when non-aqueous liquids are employed for preparing soil mixtures. Some of the liquids investigated are obviously impractical for field use but the high strengths obtained, although associated with relatively low dry soil densities, warrant further study of the use of the non-asphaltic petroleum oils with soils. The information here presented must be considered only as indicating the results which can be obtained by incorporating relatively low viscosity liquids in soil mixtures.

A series of tests was performed with the New Jersey Red Soil using petroleum oils of several types and covering a wide range of consistency. The oils used were:

Naphtha No 1	Cylesso 150 (Cylinder oil)
Refined Oil No. 1	RC-2 Cutback Asphalt
Gas Oil	MC-2 Cutback Asphalt
Marcol	SC-2 Cutback Asphalt

Similar results were obtained in any one series of mixtures using the same volume of liquid. Table 11 illustrates the effects obtained with 12 cc of the liquids per 100 gms. of soil Figure 7 shows graphically the complete data obtained in these tests.

From the results obtained with the New Jersey soil it may be said that greater strengths accompanied by greater voids contents and lower dry soil densities are obtained with the soil-oil mixtures than for the soil-water mixtures and that the best results are obtained with petroleum oils having not more than approximately 22 centipoises absolute viscosity at 77 F (85 SUV/100 F). Higher compaction loadings are required to produce the same dry soil densities for the soil-oil mixtures as compared to the soil-water mixtures due to the poorer lubricating properties of the oil even though having the same viscosity This may be partially explained by the poorer wetting properties of oils as compared to water, for hydrophilic mineral matter.

During the course of these tests the inferior lubricating properties of the oils for compacting soils was clearly shown by the relative compaction loadings required to produce definite dry soil densities as compared to those for the soil-water mixtures, namely, 1500–10,000 lb. versus 550–600 lb. for the soil-water mixtures. It was indicated that within limits, the load bearing properties of a compacted soil apparently show little relation to the compaction pressures employed.

Dry soil density by itself appears to have little relation to the strength of compacted soil For any one liquid there exists a maximum dry soil density as the quantity of added liquid is varied and the soil-liquid mixtures are compacted under the same loading.

TABLE 11
COMPARISON OF EFFECT OF WATER AND OILS ON PROPERTIES OF COMPACTED SOILS
NEW JERSEY RED SOIL (SPEC. GR = 2 749)
Liquid Used in All Mixes—12 cc. per 100 g of soil

Liquid	Properties of Liquid			Exude	Soil Density lb per cu ft		Voids	Hubbard-Field Strength
	Sp Gr 60°F.	Viscosity at 77°F						
		Centistokes	Centipoises		Wet	Dry		
							%	lb
Water—wt %	1 000	0 890	0 890	No	141 0	12 0 126 5	3.0	2350
Naphtha No. 1—wt. %	0 755	0 730	0 550	No	127 3	9 1 116 8	9.5	3475
Refined Oil No 1—wt. %	0 807	1.71	1 37	No	131 3	9.7 119 9	7.1	4350
Gas Oil—wt. %	0 859	3 12	2 66	No	125 0	10 2 113.3	12 3	4125
Marcol—wt. %	0 844	26 2	21.9	No	127.5	10 1 115 8	10 3	4550
Cylesso—wt %	0 905	2010	1809	No	132 0	10 8 119 0	7 7	3250
RC-2—wt %	0 943	2964	2777	No	128 3	11 3 115 2	10 6	1350
MC-2—wt %	0 952	5832	5523	No	131 0	11 4 117.6	8 8	2700
SC-2—wt. %	0 959	3790	3616	No	131 8	11 5 118 1	8 3	2475
No 11 Flux—wt %	0 977	129600	125840	No	127 5	11 7 114 2	11 3	1100
Water(a)—wt. %	1 000	0 89	0 89		130 6	12 0 1250(b) 116 4	9 6	1125

(a)—Compacted to average dry soil density of soil-oil briquettes.
(b)—Compaction loading required to produce dry soil density of briquettes made with corresponding volume of oil.

Effects of Liquids Having the Same Viscosity as Water

It has been shown that soil mixtures having greatly increased strengths are obtained by using petroleum oils and other non-aqueous liquids in place of water in the preparation of the compacted soils. The oils used in these tests covered a wide range of viscosity, from somewhat below that of water to that many times its value, with the result that indications were obtained to the effect that the strength of the compacted soil is not closely related to the viscosity of the liquid, at least when liquids of different natures are employed. In order to have more direct information on the effect of the nature of the liquid on the properties of compacted soils, briquettes were prepared with the following liquids having similar absolute viscosities:

Water	Isobutylene trimer
Amylacetate	Varsol—narrow cut
Carbon Tetrachloride	Absolute Ethyl Alcohol
Xylene	

As shown in Table 12, the descending order of briquette strength for the series of liquids is:

1. Varsol—narrow cut, *Best*
2. Carbon Tetrachloride
3. Isobutylene trimer
4 Xylene

5 Absolute ethyl alcohol or amylacetate
6 Water, *Worst*

The aromatic liquids (xylene) appear to be inferior for producing strength in soil mixes to the paraffinic solvents, varsol and isobutylene trimer, and are again surpassed by chlorinated liquids, carbon tetrachloride. One of the interesting and unexpected results revealed in these data is that some liquids, for example absolute ethyl alcohol and amylacetate, are capable of yielding briquettes having greater dry soil density than can be obtained with water This may have practical implications and merits further interest since one of the things currently desired in earth construction is the compaction of soils to their maximum dry soil density It must be emphasized that none of the non-aqueous liquids in the present tests were selected because of their practical value, but the results indicate possibilities which might be obtained with further investigation along this line From these data it appears that the results to be obtained in the production of compacted soil mixtures are practically independent of the viscosity of the liquid provided this is not so great as to interfere with the mixing and compaction operations, but are largely conditioned by the nature of the liquid employed. Water, in general, produces compacted soil mixtures much lower in strength than can be obtained with non-aqueous liquids, having approximately the same viscosity.

In soil constructions, great stress is placed on dry soil density, but the important thing would appear to be the production of structures having maximum strength with secondary consideration being given to dry soil density. The present work suggests that this can be promoted by the use of non-aqueous liquids For soil stabilization, the results point to a preference for light petroleum oils. It is possible that other liquids than light petroleum oils can be similarly employed in soil stabilization

PROPERTIES OF SOIL-WATER-ASPHALT MIXTURES

In a survey on the properties of soil-water-asphalt mixtures it appears that maximum dry soil densities, maximum strengths after water absorption, minimum water absorption and volumetric swell do not occur at identical compositions of the briquettes, but for any one series of briquettes made with constant amounts of water and increasing amount of asphaltic binder, these maxima occur at increasing amounts of cutback in the order of the property given The possibility of obtaining maximum wet or dry soil densities varies with the particular soil in question and it frequently happens that points of minimum water absorption and volumetric swell cannot be established over the practical range of briquette composition Improved results may be obtained on the uncured briquettes by aging This may have practical significance as indicating that this improvement acts as a safety factor for the accepted design.

It has been observed that for a given percentage of total liquids, that is cutback asphalt plus water, the dry soil density increases and the voids decrease in the bri-

TABLE 12

COMPARISON OF EFFECTS OF LIQUIDS HAVING THE SAME ABSOLUTE VISCOSITY ON THE PROPERTIES OF COMPACTED SOILS

New Jersey Red Soil (Sp Gr.—2 75)

Liquid					Compaction Loading	Exude	Soil Density lb per cu ft		Voids	Hubbard-Field Strength
Kind	Sp Gr. 77°F.	Abs Visc 77°F — Centipoises	cc per 100 g Soil	Wt			Wet	Dry		
				%					Vol, %	lb
Water	1 000	0 89	14	14 0	10000	Slight	138 9	121 2	0 8	850
Amylacetate	0 866	0 92	14	12 1	10000	Slight	134 0	119 2	3 9	1825
Carbon Tetrachloride.	1 583	0 88	14	22 2	10000	No	137 0	112 0	5 6	3950
Xylene	0 855	0 59	14	12 1	10000	No	131 5	117 3	5 8	3550
Isobutylene-trimer	0 738	0 89	14	10 5	10000	No	129 3	117 1	5 6	3800
Varsol Cut	0 766	0 89	14	10 8	10000	No	128 0	115 8	7 5	4325
Abs. Ethyl Alcohol	0 785	1 30	14	11 1	10000	No	135 2	121 5	2 0	1675

quettes compacted under a given compaction loading (10,000 lb. in these tests) with increase in the percentage of water in the total liquids This observation suggests that maximum dry soil density is only obtained by using water alone in comparison to MC-2 cutback as the lubricating liquid as pointed out in a previous paragraph and this is confirmed to a considerable degree by results on soil-water mixtures.

Obviously the voids content and total liquids present in a soil mixture are closely correlated in the manner that the greater the total liquid content, the less the voids in the compacted soil-liquids mixture. Indications have been obtained that the amount of total liquids required to produce a briquette (under 10,000-lb. compaction) with zero voids is a characteristic of the soil, as the following approximate figures illustrate:

Volume Percent of Total Liquids Required to Obtain Zero Voids

South Carolina GOOD Soil, 21 5
South Carolina POOR Soil, 30 0
New Jersey Red Soil, 34.0
San Francisco Soil, 33 5

From these figures, it may be stated good soils attain zero voids with much less liquids than the poor soils, which may be an underlying cause in their better behavior under service This property might also serve as a useful soil constant.

STRENGTH OF STABILIZED SOILS

After a soil has been investigated and a decision made on the treatment which should be applied for its stabilization, there remains the question as to how the soil mixtures should be handled in the field to give satisfactory performance. This generally requires decision on the thickness to be used, about which there has been considerable discussion, particularly on the simple soil-water mixtures. An endeavor has been made to obtain some specific information which would be applicable to stabilized soil mixtures It appears that the soil-water and the soil-asphalt-water mixtures exhibit the same mechanism of failure so that the same treatment for strength design can be applied to each

The procedure followed in these tests consisted of preparing the usual Hubbard-Field briquettes from the soil mixture and testing them for Hubbard-Field strength, shear strength and compressive strength, and then comparing these results with the load bearing capacity and behavior under load of 6-in. diam. briquettes made to various thickness and tested with a concentrated load applied at the center of the briquette through a 1-in. plunger. The stresses involved in the failure of the 6-in briquettes under this method of testing are both tensile and shear, but the attempt to determine which one predominates has not heretofore been too successful Sufficient evidence has been obtained, however, to indicate that compressive strength is not closely correlated with the strength properties of a soil and that failure of the 6-in unsupported compacted soil specimens is primarily through lack of tensile strength

Two soils were employed in these tests, namely, New Jersey Red Soil and Brownsville Soil See Table 13 for properties.

TABLE 13

Soils	New Jersey Red Soil a silty-loam	Brownsville a silty-clay
Specific Gravity	2 749	2.654
Liquid Limit	31 1	52 5
Plastic Limit	20 2	21 4
Plasticity Index	10 9	31 1
Field Moisture Equivalent	24 9	25 0
Shrinkage Limit	15 3	14 3
Shrinkage Ratio	1 81	1 92
Exudation Point		
Water only—10,000 lb compaction	14	18
50 50 Water—MC-2 10,000 lb compaction	19	21
Wet Screen Analysis		
Through No 4 on No 10	0 6	0 03
" " 10 " " 20	1.9	0 15
" " 20 " " 40	3 8	0 04
" " 40 " " 60	7 5	0.11
" " 60 " " 100	9.5	0 12
" " 100 " " 200	11 5	0 13
" " 200 " " 325	5 3	Thru No 200 99 42
" " 325	59 9	

TABLE 14

Gravity deg API	16 0 = (0 9593 sp gr at 60 F)
Flash (Tag O C) deg F	150 +
Furol Viscosity at 140 deg. F	180
ASTM Distillation	
IBP—deg F	391
374	0
437	18 1% of O H
500	56 5% of O H
600	85 9% of O H
680	100% of O H
Residue—Vol % by diff	75 2

For the soil-water-cutback mixes, the regular grade of MC-2 cutback, Venezuelan Binder "C" base having the typical inspections in Table 14 was used. Distilled water was employed in making all mixtures

Experimental Procedure

To the air dry soil whose moisture content had been previously determined, water was added to give the desired total moisture content and the soil-water mixture allowed to stand overnight. For the soil-water only mixtures, briquettes were prepared the next morning, in case of the soil-water-cutback-asphalt mixtures, the required amount of cutback asphalt was incorporated the next morning into the dampened soil and the total mixtures allowed to stand 4 hr. before preparation of the briquettes The mix evaluation, or the ease of incorporating the cutback and the uniformity of the distribution and completeness of coating the soil particles, were noted during these mixing operations. Three series of briquettes were prepared

(1) 2-in diam by 1-in high, made according to the regular Hubbard-Field procedure which includes preliminary tamping with 25 blows of the No. 1 tamper and final compaction under 10,000 lb Hubbard-Field loading. In case exudation occurred, the final compaction was made on a fresh charge under a compaction loading just short of the exudation pressure.

(2a) —Two sets for Hubard-Field and Shear strengths, 2-in diam. by 1-in high were made with preliminary tamping with 25 blows of the No 1 tamper and then compacted under a loading to produce the dry soil density obtained for the 6-in briquettes prepared as described in (3).

b—One set for compressive strength, 2-in diam. by 4-in. high, was made similarly and compacted to the dry soil density obtained for the 6-in briquettes prepared as described in (3)

(3) 6-in diam. briquettes covering a range in height from 1 to 6 were made according to the regular Hubbard-Field procedure with preliminary tamping with the No 1 tamper and final compaction of 10,000 lb total Hubbard-Field loading. In case exudation occurred, a fresh mixture was compacted under a loading just short of the exudation pressure.

The 2-in. diam by 1-in high briquettes were evaluated for their several strengths on the Hubbard-Field machine according to the standard procedure using the 2-in diam by 1-in high briquette confined in a 2-in. diam. mold placed over a 1.75-in diam. orifice with the load applied on a 2-in diam plunger. The specimens fail by a combination of shear and extrusion

The Shear tests were conducted in a manner similar to that of the Hubbard-Field test using 2-in diam. by 1-in. high briquettes confined in the 2-in mold, except that the plunger was 1 75-in in diam used in combination with the 1 75-in. orifice. The specimens fail in this case primarily through shear which results in the punching out of a plug of the soil mixture This method has been designated as the Hubbard-Field or "modified" shear test. Although the effects of side wall support enter to an appreciable extent into the final strength figure as pointed out in a previous section, no attempt was made to correct for this in this series of tests

The compression tests were performed on 2-in. diam by 4-in high unconfined briquettes with the load applied through a 2-in. plunger on the top surface

The 6-in diam. briquettes were tested by confining them in a 6-in. mold with the specimen unsupported on the bottom except around the circumference over a 5.75-in. orifice, thus giving a bearing surface 0 125-in. wide around the outer edge. The load was applied to a centrally located 1-in. diam. plunger The specimens failed by a combination of shear and a breaking away of a conical plug having curved lateral surfaces whose outline depended upon the thickness of the specimen.[5] The size and shape of the cones was more conveniently measured on the opening formed in the residual shell remaining in the mold rather than on the cone itself. The extensive data are not being presented

[5] It is realized that in addition to specimen thickness, side wall support also exerts its effect on determining the contour of the cones, particularly in those cases where the edge of rupture is at the supporting ring. This has been omitted from consideration in the present discussion, being reserved as a subject for future study

but a typical set of cone contours is illustrated in Figure 8.

There appears to be an approximate relation between the Hubbard-Field strength and Hubbard-Field or "modified" shear strength, which present data indicate to be.

$$\text{Log } S = 0.867 \, H.F. - 0.528$$

S being the Shear strength and $H.F.$ the Hubbard-Field strength. The value of the slope of the line appears to be fairly definitely established but not that of the given constant.

As observed in this investigation, no apparent relation exists between the compression and shear strengths from which it might be inferred that compressive strength is of little use in the design of soil mixtures.

Angle of Break of Soils at Failure

On increasing the load to failure on the 1-in. plunger at the center of the 6-in briquettes supported only on a 0.125-in. edge around the circumference at the bottom, the compacted soil breaks out in the form of a frustum of a circular cone with the lateral surface concave, as illustrated in Figure 8, the upper base having a diameter equal to that of the plunger and the bottom base varying in diameter up to a maximum equal to the diameter of the supporting edge of the orifice (5.75-in. for the present 6-in. specimens). The contours of the sides of the cones vary in curvature depending upon the thickness of the specimen under test and the physical properties of the soil mixture In addition, it appears that the lateral surface of the cone immediately adjacent to the plunger breaks out along a straight line in cross-section Inspection of a large number of graphs of the cone contours reveals that the angle to the vertical varies with the *height of the cone* broken out, *not with the original thickness of the specimen* The tangent of the angle to the vertical of the lateral surface of the cone adjacent to its upper base is a linear function of the thickness of the cone. The exact relation appears dependent upon the properties of the specific soil mixture, which prevents the formulation of any general relationship between the tangent of the angle to the vertical of the lateral cone surface at the upper base and the cone depth, beyond the knowledge of the existence of its linearity.

In designing for thickness of stabilized soils, it has been customary to assume that the soil fails along a surface making an angle of 45 deg. with the vertical The present data suggest that this may not be entirely true, but that the angle is related to the thickness of the compacted soil undergoing failure and that as the thickness increases, the soil will fail in a manner approaching simple shear This conclusion refers to results obtained by the use of 6-in specimens

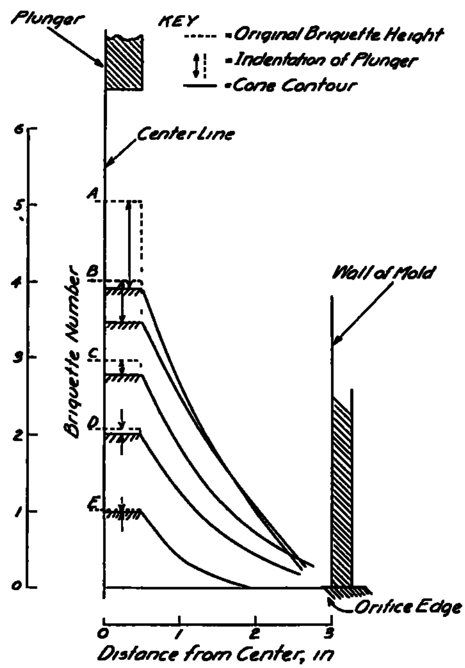

Figure 8. Typical Cone Contours

laterally confined by the walls of the mold and unsupported at the bottom except by a narrow edge around the circumference. Larger specimens supported uniformly over the bottom surface and free from rigid lateral restraint might produce different indications than those described.

Penetration of Plunger at Failure vs Thickness of Briquette

On applying the 1-in. plunger to the compacted soil specimen confined in the 6-in. mold and supported only at the periphery by the 5 75-in orifice, the plunger tends to penetrate the specimen a certain distance before failure occurs, the extent of the penetration depending upon:

1. the strength and plastic properties of the soil-liquid mixture, and
2 the thickness of the specimen

For any one series of the 6-in. briquettes prepared from a given soil-liquid mixture, the method of the preparation of the briquettes, the compaction loading employed and the wet and dry soil densities were practically identical so that the only variable entering into their formation would be that of thickness These tests indicate that the plunger penetrates the compacted soil when its thickness is greater than approximately two in , irrespective of whether the soil is friable, well compacted or in a rather plastic condition. Upon plotting the experimental results, it appears that the relationship between the penetration of the plunger at failure and the thickness of the briquette may be approximately expressed as

$$P^{\frac{1}{2}} = 0\ 33\ (T - 2)$$

P being the penetration of the plunger and T the original thickness of the briquette unsupported at the bottom except for a width of 0 125-in around the peripheral edge

This observation suggests that soils should not be compacted to greater thickness than 2 in in layers thicker than 2 in The penetration of the plunger into the briquette before failure may be related to the shear properties of the compacted soils in such a manner that when the 1-in plunger penetrates the soil specimen, a shearing stress is set up which is equivalent to the total load divided by the circumference of the plunger. The ratio of this unit peripheral stress for the 6-in briquettes to the unit Hubbard-Field shear obtained on the 2-in diam by 1-in high soil specimens should show some definite relation to the penetration of the plunger at the time of failure It has been found that penetration of the soil occurred when this ratio exceeded unity and this occurred when the 6-in diam briquettes had greater thickness than 2 in Beyond this point there appears to be no definite relation between these two quantities indicating that when failure occurs in the soil, the behavior of the soil from then on is unpredictable

The indicated critical ratio of unity suggests that until further information becomes available, the loading on a soil should be so distributed that its unit peripheral shear does not exceed that of the Hubbard-Field or "modified" shear It would seem, however, that only a moderate increase in penetration should be permitted with increase in specimen thickness of compacted soils beyond the limiting thickness of 2 in , thus indicating intrinsic stability of the soil mixture

To estimate the limiting loading at which only negligible penetration of the compacted soil should be produced, the Hubbard-Field strength and unit Hubbard-Field shear strength have been plotted against penetration on semi-log arithmic paper by which a linear relationship is apparently obtained. The curves indicate that for a compacted soil to exhibit no penetration under loading as applied through the 1-in plunger on the 6-in. briquettes, the tests on the 2-in diam by 1-in specimens prepared and tested according to the procedure outlined in this report should show around 1,900 lb Hubbard-Field strength or around 130 lb unit Hubbard-Field shear.

It would appear that only a moderate increase in penetration of the plunger should occur with increase in specimen thickness of the compacted soils by which the intrinsic stability of the soil mixture is indicated. Some insight into the relative stabilities of soils can perhaps be gained by considering the fundamental nature of the quasi-extrusion Hubbard-Field strength test and the simple Hubbard-Field shear test, both of which are performed on the same size specimen and in which the soil is forced through the same size orifice The excess force required by the Hubbard-Field test over that for the shear test must indicate the effort to crowd the soil outside of the dimension of the potential shear plug into its lateral boundaries This process requires the forcing of the soil particles over each other; in other words, the overcoming of the internal friction of the soil specimen, so that the ratio of the values obtained for the Hubbard-Field strength and shear tests may be presumed to give some indication or measure of the stability of the compacted soil. From a plot of the ratio on log-log paper against the penetration of the 1-in plunger into 6-in diam by 4-in high soil specimens supported ⅛ in around their bottom, linear relations are obtained. The slopes of the lines for the New Jersey soil-water and soil-water-cutback mixtures lie close to each other, indicating the same intrinsic stability of the soil, the difference in values for the two series of mixtures originating probably from the effect of the liquid binder employed in the two cases The slope for the Brownsville soil-water mixtures is much greater, suggesting that this soil is intrinsically more stable than the New Jersey soil This feature demands further investigation before drawing definite conclusions.

Load Bearing Strength versus Thickness of Compacted Soil

Considerable discussion has already been devoted by various investigators to the subject of the required thickness to support anticipated loads but practically no information is available for the soil-water-bitumen mixtures To supply this information was one object of this investigation The failure of compacted soils by the application of concentrated loads through a small plunger at the center of a briquette supported only around its lower edge appears analogous to that of circular metal plates submitted to the same treatment Formulae have been developed for the latter which indicate that the strengths of circular plates varies as the square of their thickness Compacted soil-liquid mixtures will not necessarily show the same strength relationships as metal plates.

From a series of tests on compacted soils, the concentrated loads obtained at failure with a 1-in plunger applied at the centers of 6-in briquettes ranging in thickness from 1 to 6, confined within the 6-in mold and supported around the circumference over a 5 75-in orifice have indicated the powers of the thicknesses as related to the loads supported at the point of failure given in Table 15

The strength of a soil appears to be more closely related to the thickness to which it is compressed at the moment of failure than to initial thickness, thus indicating that design based on original thickness is apt to be misleading unless some relation between it and final cone height at failure can be established. The exact exponent correlating thickness to load bearing capacity varies according to the properties of the soil mixture which in turn is dependent upon both the fluid employed and the soil entering into the mixture The combined effect of these several factors introduces uncertainty into the thickness design of soil mixtures which necessitates factors of safety of considerable magnitude However, for design purposes, the data indicate that, until additional information becomes available, the average the load bearing capacity of a compacted soil construction may be considered to vary as the 1 5 power of the thickness

Forces Acting at the Failure of Unsupported Soil Briquettes

There has been considerable discussion as to whether road surfaces fail through the action of shearing or tensile forces It is obvious that under traffic the road surface is subjected to two main forces, the horizontal shoving caused by the advance of the vehicle and the vertical pressure due to its weight.

TABLE 15
EXPONENT OF THICKNESS VERSUS LOAD AT FAILURE FOR COMPACTED SOILS

Based On	Original Thickness of Briquette	Height of Cone Punched Out
New Jersey Soil Soil-Water Mixtures	1 330	1 448
New Jersey Soil Soil-Water-Asphalt Mixtures	1 431	1 662
Brownsville Soil Soil-Water Mixtures	0 934–1 409	1 616–2 244

Because of greater simplicity for experimental investigation and mathematical analysis, this study has been limited to the effects of the application to compacted soils of vertical loads analogous to those produced by vehicles at rest If failure under these conditions consists in the cutting through of the surface accompanied by further consolidation of the material directly under the load (the sides of the depression assuming an angle of approximately 90 deg. with the surface and without appreciable deflection of the surface around the area of failure) the failure may be assumed to have resulted primarily from simple shear effects Thus any of the simple shear tests such as the Hubbard-Field shear employed in this investigation, would serve as a measure of the bearing capacity of the surface construction This type of failure rarely, if ever, occurs Such a failure would mean no distribution of load beyond the area of its application and there is ample experience to indicate the contrary. Load distribution on earth has been customarily considered to take place through a cone whose sides make an angle of 45 deg. with the vertical; but as shown in a previous section, this angle for compacted soils laterally confined but unsupported on the bottom varies with the thickness of the construction. This discussion attempts to analyze the various forces involved in the failure of 6-in diam. soil specimens of various thicknesses, confined on the sides in a 6-in. mold but unsupported at the bottom except around the edge resting on a 5 75-in. orifice and tested by applying a load to failure through a 1-in. diam. plunger at the center of the upper surface.

The contour of the cone indicates that the forces are combinations of shear and tension varying from primarily shear within the immediate region of the plunger to primarily tension at the outer edge of the specimen; particularly the latter when the break occurs away from the edge of the supporting orifice. Since the walls of the cone form an inclined surface, the treatment of the forces should be analogous to that of a plane of failure under tension for ordinary construction materials. Although the contours show that the surfaces of the cones are decidedly curved, it has been assumed for purposes of approximate analysis that the surface of the cone is produced by revolving about the axis straight line joining the end of a top diameter to the end of the corresponding base diameter.

The following is the treatment of the data employed in this discussion with the results presented in Table 16

Let L = Total load at failure in pounds
h = Height of cone in inches
θ = Angle to vertical of side of cone in degrees
a_1 = Lateral area of cone
S_t = Tension per unit lateral area of cone perpendicular to shear plane
S_s = Shear per unit lateral area of cone parallel to shear plane
L_t = Total tension in effect at failure—pounds
L_s = Total shear in effect at failure—pounds

The angle θ was calculated from its tangent as estimated from the graphs of cone contours; the lateral area a_1 of the cone was calculated from the dimensions of the cone. Using these symbols the following relations can be formulated:

$$\frac{L}{a_1} = \text{Load per unit lateral area of cone}$$

$$S_t = \frac{L}{a_1} \sin \theta$$

$$S_s = \frac{L}{a_1} \cos \theta$$

The component of tension parallel to the direction of the applied force then becomes

$$S_t \sin \theta = \frac{L}{a_1} \sin^2 \theta$$

from which the total component of the tension effective in promoting rupture is seen to be

$$L_t = a_1 S_t \sin \theta = L \sin^2 \theta.$$

The component of shear acting in line with the direction of the applied force then becomes

$$S_s \cos \theta = \frac{L}{a_1} \cos^2 \theta$$

from which the total component of shear acting in line with the direction of the applied force becomes

$$L_s = a_1 S_s \cos \theta = L \cos^2 \theta,$$

L_s is more conveniently calculated as the difference between the total applied load L and L_t the effective vertical tension.

The relative magnitudes of the unit shear and unit tension acting at the surface of the assumed cone vary with the shape or height of the cone. It may be assumed that whatever force primarily determines the strength of the soil mixture should reveal a practically constant unit value per unit area, and from the values in Table 16 it appears that this is the vertical component of the tension A further indication that shear plays a relatively minor role in the failure of the briquettes in tests in this investigation is the relatively small calculated unit shear values as compared to the Hubbard-Field unit shear values in the table; these approach each other when the soil mixtures become decidedly plastic.

Further evidence that compacted soils fail primarily through tension has been obtained from 6-in. soil briquettes having alternate layers of the New Jersey Red soil and thin layers of gray Brownsville soil. These specimens were loaded to failure by the 1-in. plunger, the cone and remaining shell cut mixtures when failure occurs, and that this involves a high percentage of shear resistance, although at the lower portions of the cone there was no evidence of such flow This indicates failure in this portion primarily through tension.

In connection with the relative effects of tension and shear in the failure of the unsupported 6-in. briquettes, it may be of interest

TABLE 16

ANALYSIS OF FORCES AT FAILURE OF 6-IN. BRIQUETTES—NEW JERSEY SOIL

6-in Briquettes With Load Applied to 1-in Diameter Plungers, Assumed Cone of Straight Sides

Water on Dry Soil	H F Shear on 2-in by 1-in Briq at Soil Density of 6-in Briq	Load at Failure L	Height h	Angle of Side to Vertical θ	Lateral Area a_1	Load per sq in. of Lateral Area $\frac{L}{a_1}$	Calcd Tension per sq in Lateral Area $S_t = \frac{L}{a_1} \sin\theta$	Calcd Shear per sq in of Lateral Area $S_s = \frac{L}{a_1} \cos\theta$	Vertical Component of Tension $\frac{L}{a_1} \sin^2\theta$	Total Vertical Effect of Tension $L_t = L \sin^2\theta$	Vertical Component of Shear $\frac{L}{a_1} \cos^2\theta$	Total Vertical Effect of Shear $L_s = L \cos^2\theta$
wt %	*lb per sq in*	*lb*	*in.*	*deg.*	*sq in.*	*lb*						
4	72 5	40–	1	62 8	20 3	1 97	1 75	0 90	1 56	32	0 45	8
		134	2	49 8	32 8	4 09	3 12	2 64	2 38	78	1 70	56
		271	3	38 4	40 5	6 70	4 16	5 25	2 58	105	4 12	167
		579	4	34 4	49 3	11 73	6 62	9 69	3 74	184	8 00	394
		943	4 875	26 0	57 4	16 42	7 20	14 78	3 16	181	13 27	762
8	103	119	1 11	60 3	20 8	5 72	4 98	2 84	4 33	90	1 41	29
		359	2 10	52 6	33 5	10 7	8 5	6 50	6 85	230	3 94	129
		543	2 94	39 0	40 1	13 55	8 5	10 53	5 35	215	8 19	328
		915	3 72	32 5	46 7	19 6	10 5	16 52	5 65	264	13 93	651
		1227	4 39	28 5	52 8	23 2	11 1	20 40	5 3	280	17 91	947
10	98	163	1 12	60 1	20 8	7 84	6 8	3 91	5 90	123	1 95	40
		351	2 04	49 4	33 2	10 57	8 01	6 88	6 08	202	4 48	149
		667	2 89	39 4	39 7	16 80	10 68	12 98	6 78	269	10 01	398
		847	3 92	31 2	48 7	17.40	9 01	14 88	4 67	228	12 72	619
		1421	4 27	29 1	51 7	27 50	13 38	24 00	6 50	336	20 97	1085
12	92 6	119	0 99	55 7	13 5	8 82	7 29	4 98	6 02	81	2 80	38
		371	2 06	45 6	28 6	12 92	9 24	9 04	6 60	189	6 32	182
		559	2 86	39 7	39 4	14 18	9 05	10 90	5 78	228	8 30	331
		865	3 74	32 4	47 0	18 41	9 86	15 57	5 29	249	13 15	616
		1219	4 81	26 2	56 9	21 45	9 46	19 27	4 18	238	17 23	981
14 5	85	119	0 99	49 3	10 3	11 60	8 80	7 56	6 66	68	4 93	51
		363	1 98	47 1	28 6	12.70	9 31	8 65	6 83	195	5 89	168
		583	2 88	39 6	39 6	14.72	9 39	11 33	5 99	237	8 74	346
		727	3.32	35 6	43 2	16.85	9 81	13 70	5 71	247	11 58	480
		1055	4 38	28 5	52 8	20 00	9 55	17 58	4 55	240	15 45	815
20	17 2	40–	0 99	56 6	14 1	2.84	2 38	1 56	1 98	28	0 86	12
		136	2 09	48 1	33 5	4 06	3 05	2 68	2 29	77	1 77	59
		200	2 18	47 5	34 2	5 85	4 32	3 96	3 18	109	2 68	91
		230	2 38	45 0	35 6	6 45	4 56	4 56	3 22	115	3 23	115
		240	2 12	48 2	33 7	7 12	5 30	5 25	3 88	131	3 50	109

across their diameters and the cross-sections photographed. It appears that the failure has been primarily through tension since distortion along the surface of rupture does not show Compression of a top portion of the cone has taken place but evidently this has not affected the adjacent soil from which the cone was punched, except for plastic mixtures. The pictures of the plastic mixture were interesting in showing the flow of such soil

to compare the calculated average tension values for the New Jersey soil-water mixtures with some preliminary tensile strength results obtained by experiment using the soils compacted to the same wet soil densities in a 2-in. diam mold as found for the 6-in. briquettes (see Table 17).

It will be noted that the determined values are considerably less than those calculated and that moreover the determined values indicate a very sharp maximum for the mixture containing 14 5 per cent of water This indicates the marked increase in the coherence of the soil mixture of this composition as compared to the others submitted to the tensile strength test. Much higher tensile strengths can be obtained by compacting the soil mixture to higher soil densities, but this feature was not investigated further. It would appear, however, from these few data that the tensile strength of compacted soil mixtures might serve as a better criterion in their design than the tests in vogue at the present time.

TABLE 17
TENSILE STRENGTHS ON NEW JERSEY SOIL

Water	Lateral Area of Cone	Vertical Component of Tension, calculated from cones	Determined Tensile Strength
% by wt	*sq. in*	*lb per sq in*	*lb per sq in*
4	4.57	2 68	1 15
8	8.72	5 49	1 78
10	9.58	5 99	1 78
12	8 98	5 57	2 55
14 5	9 37	5 95	9 55
20	3 92	2 91	2 29

Effect of Layer Compaction on the Load Bearing Strength of a Soil

It was suggested previously that in compacting soils to greater thicknesses than 2 in layers prefereably not greater than approximately 2 in. should be used. This indication has been further investigated by compacting soil in a 6-in. mold in layers under 10,000-lb. loading on each layer, and then without removing the specimen from the mold, evaluating its load bearing capacity by applying the load to the 1-in. diam plunger centrally located on the upper surface of the briquette under two conditions, namely.

a Supporting the 6-in briquette on a 5 75-in. diam orifice so that it was resting only on a ring 0.125 wide around the circumference with no support underlying the remaining bottom surface

b Supporting the 6-in briquette over the entire bottom surface by resting the specimen on a solid support The results from these two test conditions as applied to soil-water mixture using New Jersey Red soil indicated that

1. Compaction in layers produces greater soil density which increases progressively with decrease in the thickness of the layer compacted, in other words, the greater the number of layers by which the soil is compacted to a given thickness, the greater the soil density The limiting thinness, however, was not estimated

2 Compaction in layers produces a progressive increase in the load bearing capacity of a compacted soil for a given thickness of construction.

It was also considered of interest to note if any relationship existed between the load bearing capacities for the soil tested under the two conditions of support, and for this purpose the ratios of the two values obtained in this series of tests were plotted It appears that the same load bearing strength will be indicated for either case when the thickness of the compacted soil is 7–8 in. or greater. This indication might be employed for establishing the minimum thickness for compacted soil construction. It also suggests that any discussion of the relative merits of supported and unsupported 6-in. briquettes will be limited to soil thickness below the limit at which the same failure loading is obtained for either of the two types of briquette support.

Water Absorption of 6-in. Diameter Briquettes

The effects of water upon stabilized soils is customarily evaluated by exposure of small compacted specimens to various moisture conditions which in our case consists of immersion of 2-in diam by 1-in. high briquettes to a depth of ⅛ in in distilled water. The behavior of the 6-in briquettes from 1 to 5 in. thick on exposure to water has been noted by immersing them to one-half their height in water for 85 days with periodic inspections of their water absorption and volumetric swell, with the following results.

1. Volumetric swell remains relatively low in value and generally reaches its maximum value in from 14 to 36 days. For the soil mixtures employed, the 1-in. high briquettes exhibit practically the same volumetric swell whereas the briquettes of other heights vary in this property, which tends to decrease with increasing amount of asphalt in the mixture.

2. Water absorption is relatively high and requires approximately 50 days to reach its maximum value. Water absorption apparently decreases with increase in the amount of asphalt in the mixture and increases with the increase in thickness of the briquette.

The erratic behavior of the softened water soaked soils under the loading as carried out in these tests was such that the design of stabilized soils, based upon properties determined on the compacted soils before being submitted to the action of moisture, does not appear to apply too closely to the construction after it has been exposed to excessive moisture. In other words, for design purposes, the properties of the compacted soils after submission to water absorption are controlling.

Conclusions

From the discussion of strength of stabilized soil, one can say that:

(1) There appears to exist a relationship between the Hubbard-Field and shear strengths which present data indicate can be expressed approximately as

$$\text{Log } S = 0\,867 \text{ Log } (H\,F.) - 0\,528,$$

S being the shear strength and $H\,F.$ the Hubbard-Field strength.

(2) Compressive strength is of questionable use in the design of soil mixtures because of its lack of correlation with other strengths which have indicated correlation through experience with the service behaviors of soils.

(3) In the rupture of compacted soil by the application of a concentrated load at the center of a 6-in. briquette, it appears that the angle to the vertical of the cone broken out varies with the height of the segregated cone and not with the original thickness of the specimen and moreover that the tangent of the angle of the lateral surface of the cone is a linear function of the height of the cone. These correlations are limited to laterally confined 6-in. diam soil specimens and thus include the effects of sidewall support.

(4) On testing 6-in. diam. compacted soil specimens by loading a 1-in. diameter plunger, penetration occurs on specimens having a thickness greater than 2 in. The relation existing between the penetration of the plunger and thickness of specimens appears to be

$$P^{t} = 0\,33\,(T - 2),$$

P being the penetration and T the thickness, both in inches.

(5) Until further information becomes available, it appears that the load on a soil should be so distributed that the peripheral shear around the bearing area would not exceed the Hubbard-Field unit shear without the production of measurable penetration. For soil to be satisfactory from this standpoint it appears that in the compacted condition it should show around 1900 lb. Hubbard-Field strength or around 130 lb unit Hubbard-Field shear.

(6) The intrinsic stability of a soil can apparently be indicated as the ratio of the Hubbard-Field strength to the Hubbard-Field shear.

(7) The load bearing capacity of a compacted soil appears to vary according to the height of the segregated cone raised to the 1 5 power

(8) Under concentrated load, compacted soils appear to fail when the tension component of the applied load exceeds the tensile strength of the soil. This has been indicated by the calculation of the applied loads, the photographing of sections of soil specimens after failure and the determination of the approximate shear strengths of soils.

(9) Compacting a soil in layers to a given thickness, as compared to compaction in a single layer, produces greater soil density and increases the load bearing capacity

(10) Indications have been obtained that beyond a certain thickness (7–8 in. for New Jersey soil), the same load bearing capacity will be indicated on testing the laterally confined 6-in soil specimens in a supported or unsupported condition.

(11) Water absorption, expressed as grams per 100 sq cm of exposed surface, appears to increase in amount with increase in the thickness due possibly to a more open structure of the soil with increase in its thickness.

(12) The design of soil mixtures should be based upon the properties of such mixtures after they have been submitted to the action of water since water-soaked compacted soil mixtures, at least if not completely stabilized towards the action of water, do not behave the same as the compacted soils previous to exposure to water.

THE EFFECT OF CUTBACK CONSISTENCY AND THE TEMPERATURE OF MIXING

In stabilizing soils with cutback asphalts, the binder should obviously be incorporated into the soil mixture as uniformly as possible. The ease and the thoroughness of the distribution of the liquid asphalt among the soil particles is dependent, among several factors, upon its consistency, which in practice is selected according to the gradation of the soil or aggregate being treated It can be inferred from this that other factors remaining the same, the same results ought to be obtained with products having approximately identical consistency at the time of incorporation and mixing The consistency of the cutback can be controlled within certain limits by variation in its temperature since these products have a rather high temperature coefficient of viscosity, but in controlling the viscosity of the cutback during the preparation of the mixture, the soil or aggregate should preferably be at the same temperature In order to observe the effect of consistency of the cutback upon the properties of stabilized soil, a number of tests have been made with the following grades of cutback asphalts mixed with a soil fairly difficult to stabilize with asphaltic binders. RC-2, MC-2, SC-2, RC-4, MC-4, SC-4

From the results of these tests it appears that increasing the viscosity of the cutbacks from that of the No. 2 Grades (100–200 Furol viscosity at 140 F) to that of the No. 4 grades (125–250 Furol viscosity at 180 F.) decreases the wet and dry soil densities of the compacted soil, decreases the obtainable Hubbard-Field strength for the leaner mixtures but increases that for the richer mixtures before these are exposed to the water absorption test; decreases the resistance of the compacted soil to the effects of water as indicated by low Hubbard-Field strength after the water absorption test, increased tendency for the compacted briquettes to disintegrate in water and greater water absorption and volumetric swell.

Lowering the viscosity of the cutback by increasing the mixing temperature, permits production of increased soil densities, increases Hubbard-Field strength both before and after water absorption, but decreases resistance to the effects of water as indicated by increased water absorption and generally increased volumetric swell.

The net effect of increasing the mixing temperature appears therefore to be negative in character.

AIR CURING SOIL-ASPHALT MIXTURE BEFORE COMPACTING

To evaluate the effect of curing the soil-asphalt mixture, overnight before compacting, soil-water-asphalt briquettes were prepared from mixtures of New Jersey soil molded immediately after mixing and after allowing the mixture to air-cure in a loose condition before molding and testing. It appears that compared with the uncured mixtures, curing the soil mixture before compacting (a) yields briquettes having much higher Hubbard-Field strengths before the water absorption test; (b) decreases the water-resistant powers of the compacted soil in that larger quantities of asphaltic material are required to prevent disintegration of the briquettes, (c) causes the briquettes to have higher water absorption and greater volumetric swell for mixtures of the same original composition. For briquettes satisfactorily water resistant, the briquettes exhibit appreciably higher strengths after the water absorption test

These observations strongly suggest that better results might be obtained in the field by following a similar procedure, namely, aerate the treated soil to remove the major portion of the moisture before compacting.

ROLE OF 200-SIEVE FRACTION IN ASPHALTIC SOIL STABILIZATION

No easily discernable relation exists between either the water exudation value of a soil or that of a 50 50 water cutback mixture and the magnitude of the soil fraction passing the No 200 sieves The role of the soil fraction passing the No 200 sieve has been further investigated in reference to the other soil fractions as defined by the quantity of material retained on No 10 sieve, passing No. 10 and retained on No 200 sieve and passing No. 200 sieve and other soil properties such as the liquid limit, plastic limit, plasticity index, P.R A classification, exudation values under 10,000-lb. compaction loading, the stabilization treatment recommended and characteristics of the treated soil. A general trend exists for increasing exudation values up to around 30 to 35 per cent with increase in the amount of minus 200 soil present but there is no definite correlation. However, on comparing the ratio of the amount of the 10/200 sieve material to the fraction passing the No. 200 sieve with the Hubbard-Field strength after the water absorption test, the water absorption and volumetric swell, it appears satisfactory stabilization is practically assured when this ratio is 1 7 to 2.0. Although satisfactory results can be obtained when this ratio has much lower values, such results can only be ascertained by special investigation. This soil fraction ratio appears to be particularly applicable to the cohesive and plastic soils since the non-cohesive soils can show a high soil fraction ratio and still have a high-water absorption

The soil-fractions ratio may be interpreted as a special application of the improvement of a poor soil brought about by the addition of granular material in which the ratio indicates the proportion of sands that must be added to the fine soil to obtain satisfactory results by bituminous treatment, with the added implication that the fraction passing the No. 200 sieve is difficult to stabilize by simple cutback asphalt treatment. The apparently satisfactory results which can be obtained at times with soils showing a lower soil-fractions ratio than 1.7 to 2.0 is undoubtedly connected with the specific properties of the fine soil fraction from a stabilization standpoint. This ratio can serve for classifying soils and for evaluating the suitableness of a soil for bituminous stabilization.

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DISCUSSION ON STABILIZING SOILS WITH ASPHALTIC MATERIALS

MR. A T. GOLDBECK, *National Crushed Stone Association:* I rather question the idea that the tensile resistance of the soil is the critical value. It may be in the described test, but I wonder if the tensile stress in the stabilized layer is the critical stress That depends upon whether the tension created in the roadbed exceeds the tensile resistance

MR. C. N. CONNER, *Public Roads Administration:* I am interested in the character of the stress. It appears to me to be similar to shear, involving diagonal tension such as that found in breaking a concrete beam. Further, I would like to know the unit stress that might have occurred along the surface of the cone.

MR. HOLMES. Answering the last question first, the tensile strength which we have found for soils varied from about 2 to 7 lb per sq. in. on the soil mixtures from which this series of cones were made, the tensile strength ran around 5 lb. per sq. in. In regard to the question of whether or not the tensile strength is the critical one, the low values of 5 as compared to around 92 lb per sq in. of shear would indicate that the tendency to fail by tension is certainly the greatest.

MR. GOLDBECK I still do not think that answers my particular question I should like to know what tensile stresses are actually created in the subgrade or in these layers of stabilized soils. Until we know whether the tensile stress created is greater than this tensile resistance that is spoken of, I still do not think we know the answer We all know that very high stabilities can be created in layers of materials that have no tensile resistance at all I have in mind, of course, a layer of crushed stone. We know very well that there may be no tensile resistance yet high stabilities are secured.

MR. HOLMES· The information we have is limited to the calculated values on the cones in these tests We have had no experience with crushed stone structures and so cannot answer the question from that standpoint.