FIELD TESTS ON BEARING CAPACITY, SHEARING AND PENETRATION RESISTANCE OF SOILS

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SYNOPSIS

The first part of this report contains the results of field shear tests made on different components of flexible type road structures, during the past year The following practical factors are discussed (1) proper "seasoning," of bases following construction, (2) determination of the causes of failure, (3) surcharge effect of overlying layers as factors of safety in design, and (4) the relationship or arbitrary selected soil factors to the stability of unconfined cohesive subgrades. (For description of field shear testing apparatus and discussion of previous test data see *Proceedings*, Highway Research Board, Vol 18, Part II, pp 249 and 426 Also Vol 19, p. 484)

In the second part a method for obtaining the bearing capacity of the road structure or any of its individual components is described and the results obtained are discussed This method utilizes the anchorage obtained by placing an auger in the soil to a suitable depth The required reactive force for determining the bearing capacity of the soil is provided by operating the hydraulic jack of the shear machine against a horizontal beam attached to the upper part of the anchored auger system Relatively small circular bearing plates of different areas are recommended for use. This method obviates the use of loaded trucks or other unwieldy vehicles in obtaining bearing test data

The report also contains a description of a penetrometer and a brief discussion of the results obtained from tests on cohesive soils in the field

The development of the latter two test methods are not complete but are described at this time in the hope that others may want to use them or a modification of the principles involved

The report concludes with a suggestion that an extensive field study of cohesive soils be made in which the shearing, bearing capacity and penetration methods be employed simultaneously and the results compared Also that a correlation of these tests be madè with the laboratory stabilometer method From such a study a minimum standard for a suitable subgrade could be established and thus afford a quantitative basis from which the effect of any subgrade treatment or improvement could be measured and compared Furthermore it would establish the most practical and effective means of getting actual field data to use in the existing design formulas for flexible pavements

FIELD TESTS REVEAL THAT INADEQUATE "SEASONING" PREVENTS ATTAINMENT OF MAXIMUM STABILITY

Figure 1 shows graphically the relationship of stability to deformation for properly and improperly "seasoned" components of a soil-graded aggregate project constructed in two-courses. The bottom course was approximately six weeks old and had been covered for about five weeks with the top course at the time of test. The thickness of each course was 4 in. The figure shows that the 5-week old top course had attained a satisfactory stability of over 60 psi at a deformation of 0.10 in. The stability of the 6-week old bottom course for a similar deformation was only approximately 30 psi. The figure also shows that the bottom course attained a stability equal to that of the top course, but at a deformation of approximately 0.20 in. compared to that of 0.10 in. for the top course. This means that the top course can show signs of cracking before the full stability of the bottom course is utilized. The moisture content of the top course at time of test was 7 per cent and that of the bottom course was 10.2 per cent These high moisture contents are partially due to the high absorption of the cherty material used. From other tests made on the same project, the moisture content of this material at the time of placing was approximately 14 per cent.

The curves in Figure 1 show definitely the over-all loss of stability in a road structure constructed in two layers, when the top layer is placed before the bottom layer has properly "seasoned" (lost approximately 45 per cent of its moisture). This loss of stability in the bottom course is permanent as the 4-in. overlying course of stabilized material and the bituminous wearing course planned for this project

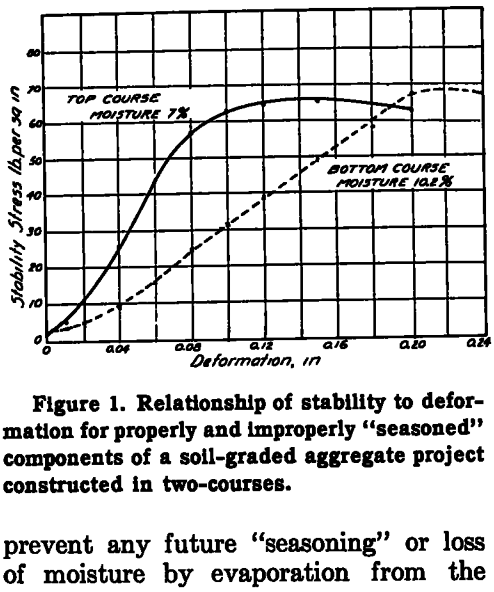

Figure 1. Relationship of stability to deformation for properly and improperly "seasoned" components of a soil-graded aggregate project constructed in two-courses.

prevent any future "seasoning" or loss of moisture by evaporation from the bottom course.

POSITIVE CAUSES OF FAILURE REVEALED BY ANALYSES OF QUANTITATIVE TEST DATA

Granular Base Failure Due to Excess of Fines (Project H).

Figure 2 shows graphically the relationship of stability to deformation for the different road structure components in adjacent failed and unfailed areas of the same project. The daily traffic on this road is approximately 600 vehicles of which about one-sixth are trucks. This figure reveals that the stabilized base under the unfailed bituminous mat had a good shear strength of about 50 psi at a deformation of approximately 0.10 in. (curve No. 6). The subgrade under this unfailed area had a shear strength at its "yield point" of approximately 25 psi which can be classed as good (curve No. 7).

The unfailed bituminous mat revealed both a low shear strength and a low "yield point." The shear strength was about 12 psi at a deformation of 0.04 in. which is approximately its "yield point." An appreciable quantity of free water was

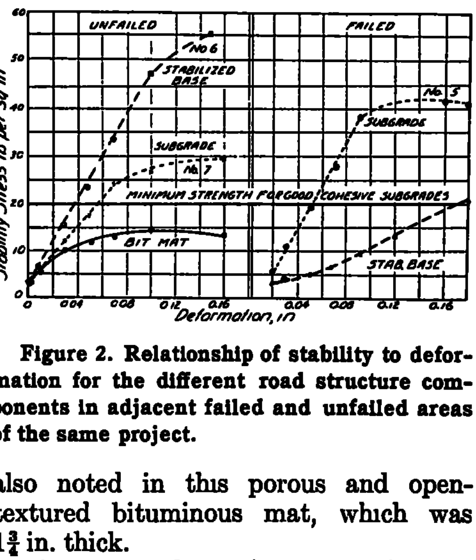

Figure 2. Relationship of stability to deformation for the different road structure components in adjacent failed and unfailed areas of the same project.

also noted in this porous and open-textured bituminous mat, which was 1¾ in. thick.

The stabilized base (curve No. 4) under the failed area had a shear strength of 21 psi but its corresponding "yield point" deformation of about 0.20 in. is abnormally high, especially when the overlying bituminous mat can only yield about one-fifth as much without cracking. Furthermore the strength of the stabilized base at a deformation of 0.04 in. is less than 10 psi, which is very low. The subgrade under the failed area had an excellent shear strength of approximately 40 psi at a deformation of 0.08 in. (curve No. 5).

Briefly reviewing Figure 2, it is noted by comparing curves 5 and 7, that the subgrade strength under the failed area is higher than that in the unfailed area, although the latter had good structural stability. Also by comparing curves 4 and 6, the stabilized base strength under the failed area is very low being only about one-fifth as great as that in the unfailed area at the lower deformations. This definitely places the cause of the failure on the stabilized base. But why did part of this base fail? Figure 3 shows graphically the granular analyses of both

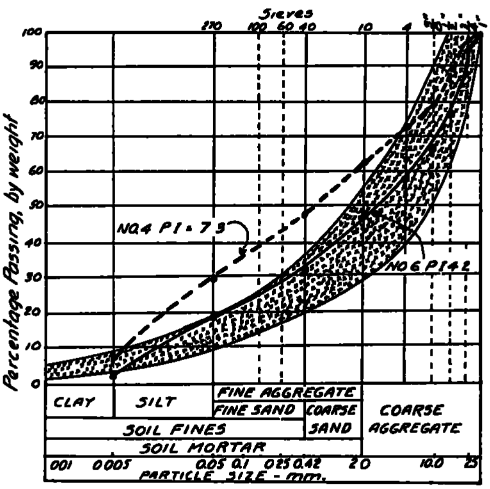

Figure 3. Stabilization chart showing the gradation curves of the failed and unfailed base materials.

the failed (No. 4) and the unfailed (No. 6) stabilized crushed stone bases and their plasticity indices. It becomes at once obvious that the gradation of the material in the base of the failed area does not meet the requirements of stabilization as there is a preponderance of detrimental fine material as shown by the 50 per cent passing the No. 40 sieve and 30 per cent passing the No. 270 sieve. The plasticity index of this material was 7.3 per cent which, although slightly high would probably not have caused trouble if the quantity of the soil fines had been less. The moisture content of this base was also quite high, being 7.2 per cent at the time of test. The source of this moisture was unquestionably from the top, as free water was observed in the overlying open type bituminous mat. It is reasonable to expect that had this bituminous mat been of the graded aggregate or denser type there would have been no failure at this area, as the subgrade had good structural quality and the low moisture content of 11.2 per cent is only 75 per cent of its lower plastic limit. It has been generally observed that an open type bituminous mix should not be placed on a stabilized base unless ample precautions are taken to provide either low shoulders

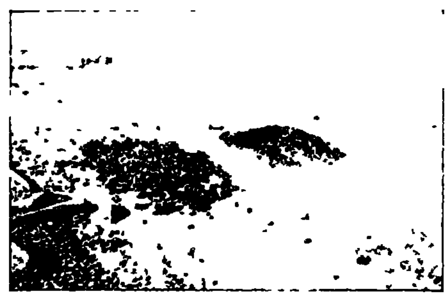

Figure 4. View showing where tests 4 and 5 (fig. 2) were made on failed area. Note pronounced cracking of the bituminous surface.

or pervious ones to facilitate complete drainage of the water from the porous bituminous structure. Figure 3 also shows that both the gradation and the plasticity index of the stabilized base (No. 6) meets the requirements of stabilization and the result is an unfailed bituminous mat. The moisture content in this stabilized base was 5.6 per cent which was not high enough to cause instability as the soil-fines and the plasticity index were low and also some of these fines were stone dust which has low moisture-volume change properties.

Figure 4 shows pictorially the condition of the bituminous mat in the failed area. The tests on the unfailed area were made about 40 ft. from this location.

Cohesive Subgrade Failure Due to Excessive Moisture (Project N).

Figure 5 shows graphically the relationship of stability to deformation for the

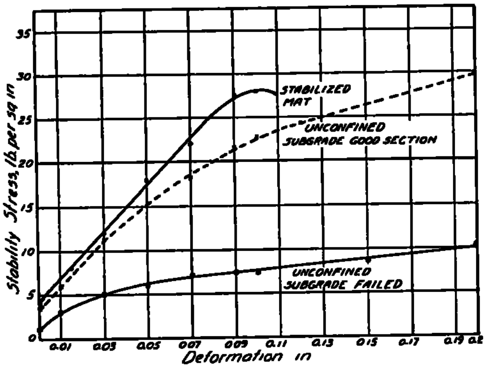

Figure 5. Relationship of stability to deformation for the different road structure components in adjacent failed and unfailed areas.

Figure 6. View showing failed and unfailed areas. The flag indicates the place where tests on unfailed area were made. (See fig. 5.)

different road structure components in adjacent failed and unfailed areas. A view showing the relative location of these areas is given in Figure 6. The bituminous wearing surface on this project was only approximately an inch thick, which is too thin for the shear machine to test.

The stability stress of approximately 28 psi for the unfailed 2.5 in. thick stabilized mat at a deformation of 0 10 in. is satisfactory The moisture content at time of test was 4.8 per cent.

The subgrade in the unfailed area had a stability stress at a deformation of 0.10 in. of 23 psi which can be classed as good. The moisture content at the time of test was 17.0 per cent which is only 75 per cent of its lower plastic limit. The dry weight of an undisturbed sample was 115 lbs. per cu ft.

The subgrade in the failed area gave a very low stability of 8 psi at a deformation of 0 10 in. and only 10 psi for a 0.20 in. deformation. This indicates a subgrade of very poor quality. The moisture content at time of test was 21.4 per cent, which is 95 per cent of its lower plastic limit. The dry weight of an undisturbed sample was only 104 lbs. per cu. ft. which is about 11 lb. less than that of the subgrade under the unfailed area.

The cause of this failure was excessive moisture in the subgrade. But why should there be more moisture in adjacent

TABLE 1

SOIL TESTS ON SAMPLES FROM PROJECT (N)

Condition of road surface	Subgrade		L L L	L P L	P I	Wt Cu Ft (Dry)	Moisture Content	M.C / L P L	Stability at 0 10 in. deformation
	General Type	Location Tested							
						lbs	%	%	p.s.i
Unfailed	Black Gumbo	0 to 3 inches under stabilized mat	55 4	22 4	33 0	115	17 0	75	23
Failed	Yellow Silty Clay	0 to 3 inches under stabilized mat	49 9	22 6	27 3	104	21 4	95	8
Failed	Black Gumbo	3 to 6 inches under yellow silty clay	63 9	26 3	37 6	93	26 7	101	13

areas on a level roadway of fairly high grade line? In the present case, the variation in the type and relative position of the subgrade soils offers an explanation. Table 1 contains the results of the physical and stability tests of the subgrades under the failed and unfailed areas.

In the unfailed area, the subgrade immediately under the stabilized soil-graded aggregate mat was black gumbo top soil, typical of this region, with a plasticity index of 33 per cent and with a moisture content low enough (17 per cent) to assure a satisfactory road foundation as revealed by the 23 psi stability. This subgrade was examined for a depth of approximately 18 in. and found to be of uniform texture and color.

In the failed area, the top 3 in. of the subgrade was of entirely different material as it consisted of a yellow silty clay with a plasticity index of 27 per cent and a high moisture content (21.4 per cent) and low stability (8 psi). Immediately under this yellow silty clay was a layer of at least 12 in. of typical black gumbo, the top 3 in. of which had a moisture content slightly higher than its lower plastic limit. The stability of this material (13 psi) although about 60 per cent greater than the yellow silty clay, must still be classed as rather poor. The arrangement of these two subgrade soils was such that the one more susceptible to moisture changes and movements occupied the critical position immediately under the stabilized mat. Furthermore the moisture was probably supplied to this upper layer by capillarity and freezing action from the underlying layer of black gumbo, which had a higher moisture content but also a higher stability. The yellow silty clay was from a lower stratum in the area now used as a side ditch, as this was the source of the grade material.

This localized failure was due to the combination of dissimilar soil material so arranged in the subgrade profile that the one possessing the least desirable properties, for the prevailing environmental conditions, was located in the upper part of the subgrade. In adjacent areas where the subgrade consisted of only black gumbo top soil the road surface was intact.

SURCHARGE EFFECT OF OVERLYING LAYERS

Effect Greatly Diminished in Cohesive Subgrades of Low Quality

It was stated in last year's report[1] that there probably would be no beneficial effect of an overlying layer on a

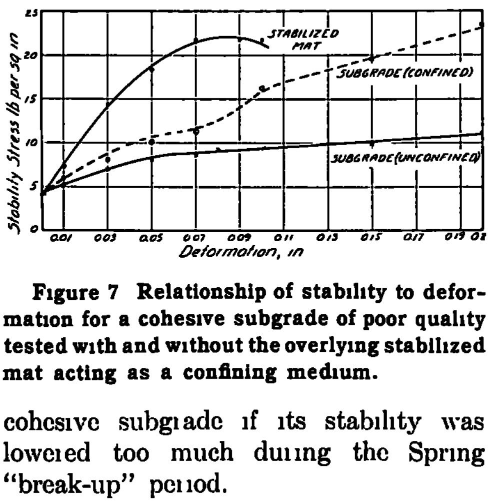

Figure 7. Relationship of stability to deformation for a cohesive subgrade of poor quality tested with and without the overlying stabilized mat acting as a confining medium.

cohesive subgrade if its stability was lowered too much during the Spring "break-up" period.

In the Spring of 1940 tests were made to determine the surcharge effect of overlying layers on the stability of cohesive subgrades of poor quality. Figure 7 shows graphically the relationship of stability to deformation for a typical cohesive subgrade of poor quality tested with and without the overlying stabilized mat acting as a confining medium.

The unconfined test was made on the subgrade after removing the stabilized

[1] Fred Burggraf, "Field Tests of Shearing Resistance," *Proceedings*, Highway Research Board, Vol. 19, p. 484.

surfacing. The confined test was made by testing the subgrade with the overlying stabilized surfacing in place and acting as a surcharge.

Figure 7 shows that the stability of the confined subgrade is influenced very little at deformations in the vicinity of and below the "yield" point of the overlying stabilized mat. The subsequent increases in the stability of the confined subgrade are of no practical importance as they occur at deformations which are beyond the "yield" point of the overlying layer. This means that cracking would take place in the stabilized surfacing before this increased stability of the subgrade could be utilized, provided the thickness of the surfacing was not great enough to distribute the load over a sufficient area of the subgrade

Effect Greatly Augmented in All Road Components of Satisfactory Quality.

Figure 8 shows graphically the relationship of stability to deformation for the various components in a road structure, tested in confined and unconfined conditions. This was a 3 component road structure. The bituminous wearing surface was 1¾ in. thick, the calcium chloride stabilized gravel base was only 2.5 in. thick, and the subgrade was of loamy texture. This project is located in one of the northern States and had passed through three Spring "break-up" periods with no signs of failure

Figure 8 shows that the densely graded bituminous mat was of excellent quality as the stability at its "yield" point was approximately 50 psi. The temperature of this mat one inch below the surface was 70° F. at the time of test (11 A.M. May 11, 1940).

The unconfined stabilized base possessed a satisfactory stability of 33 psi at a deformation of 0 10 in. The gradation of this material was as follows: 100 per cent passing ⅝-in., 85 per cent passing ⅜-in., 58 per cent passing No. 10 sieve, 29 per cent passing No. 40 sieve and 12 per cent passing No. 270 sieve. The liquid limit was 19 3 per cent and the plasticity index was 6.3 per cent. The field moisture content was 2 8 per cent and the dry weight per cu. ft. was 141 lbs

The 27 psi stability of the loamy unconfined subgrade indicates very good quality, and undoubtedly accounts for the good service rendered by this road structure. This stability indicates the minimum attained by this subgrade as it was taken immediately after the frost

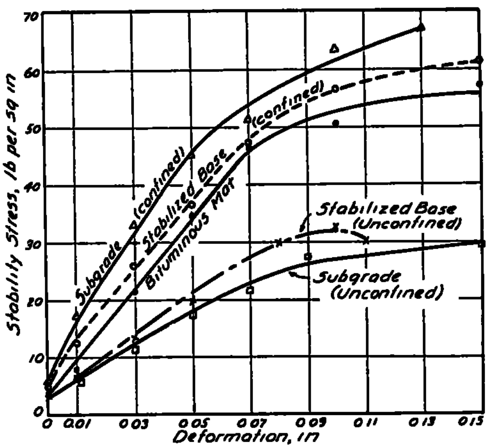

Figure 8. Relationship of stability to deformation for the various components in a road structure, tested in confined and unconfined conditions.

left the ground in the Spring. The moisture content at time of test was 19.7 per cent which is only 72 per cent of its lower plastic limit of 27.3 per cent. The plasticity index was 15.7 per cent and the dry weight per cu. ft. was 103 lbs.

Referring again to Figure 8 we note that the stability of the stabilized base increases from 33 to 57 psi when the 1¾-in. bituminous wearing surface acts as a surcharge or confining medium. This may partially account for the general observation that stabilized bases covered with heavier bituminous mats give much better service than similar bases covered with light bituminous treatments. Also the stability of the subgrade increases from 27 to 60 psi when tested with the stabilized base and bituminous wearing surfaces as overlying layers. A comparison of the stress-strain curve for this confined subgrade with that shown for the confined subgrade of poor quality in Figure 7 is rather interesting. The one in Figure 8 shows from the very beginning the beneficial influence of the overlying layers, whereas that in Figure 7 does not reveal this influence until considerable deformation has taken place in the poor quality subgrade due to inherent compaction.

These results indicate that the beneficial effects of overlying layers are of varying magnitude, depending mainly on the strength and thickness of the overlying layers and the quality of the confined cohesive medium. Also that the beneficial effect is approximately the same for a high quality cohesive material (27 psi cohesive subgrade P.I. = 15.7) as for a more frictional material (33 psi stabilized base P.I. = 6.3).

This beneficial action of overlying layers apparently needs to be given some consideration before "Factors of Safety" are established for this type of road structure.

A DIRT ROAD OF SATISFACTORY STABILITY

While making some tests on a stabilized project, several gravel trucks were observed traversing a cross road of ordinary dirt. An inspection of the surface of the dirt road showed it was carrying this type of traffic satisfactorily and naturally there was a desire to determine quantitatively its stability. Figure 9 shows graphically the relationship of stability to deformation for this dirt road. This figure shows that the stability of the top 3 in. of the road surface was, at its "yield" point, approximately 20 psi which has tentatively been designated as the lower limit of good quality for an unconfined cohesive subgrade. This figure merely confirms quantitatively the tentative limit for good subgrades and explains why the truck traffic was not rutting-up the road surface. The favorable weather conditions prevailing were also a contributing factor for this temporary good service behavior.

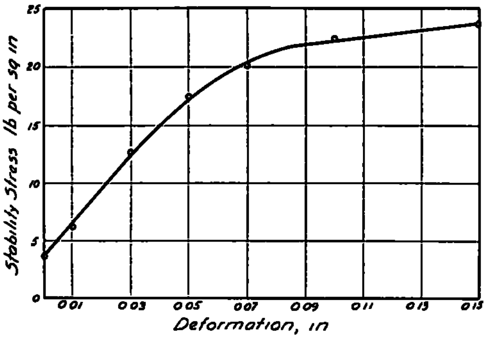

Figure 9. Relationship of stability to deformation of a dirt road which was carrying truck loads satisfactorily.

STABILITY VERSUS AN ARBITRARY SOIL-FACTOR

Figure 10 shows graphically the relationship of stability to an arbitrary soil-factor—moisture content divided by lower plastic limit—for finely divided cohesive subgrades with plasticity indices between 15 and 40. This figure shows that over most of the range there is a general tendency for the stability to decrease as the field moisture content approaches or exceeds the lower plastic limit value of the respective soils. The rather wide range in the stabilities at moisture contents equal to or greater than the lower plastic limit is probably due to the difference in the organic contents of the soils. The general observation has been that the heavy black gumbo soils possess more stability than the lighter colored clays or silty clays, when this soil factor ratio is constant but high.

(See Table 1.) The explanation for this is probably the difference in the textural structure of the finely divided particles and colloids. Due to their sponge-like structure, the organic colloids have a much greater surface area than that possessed by the denser inorganic colloids of the lighter colored soils. This means that for the same relatively high moisture content an inorganic clay or silty clay soil will show less stability than an organic type soil due mainly to the fact that, in the first case, practically all of the water accumulates on the surface of the particles and acts as a detrimental lubricating medium, whereas in

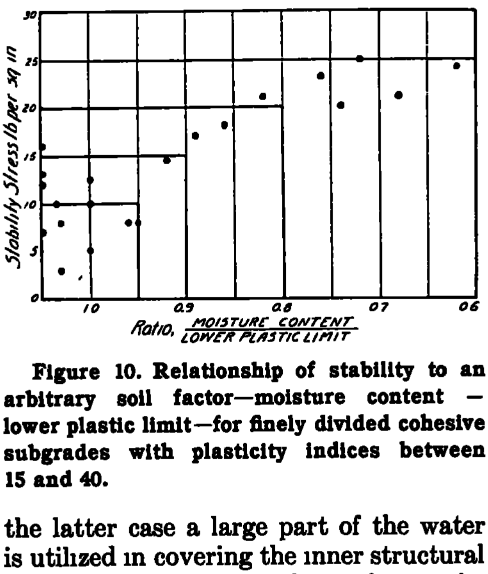

Figure 10. Relationship of stability to an arbitrary soil factor—moisture content – lower plastic limit—for finely divided cohesive subgrades with plasticity indices between 15 and 40.

the latter case a large part of the water is utilized in covering the inner structural area and consequently the surface moisture film thickness is less and the stability of the soil mass is greater.

In general Figure 10 shows for soils falling in this category, that the moisture content should not be more than 80 per cent of the lower plastic limit to assure a 20 psi stability or one classed as good. If the moisture content is between 80 and 90 per cent of the lower plastic limit the soil is of fair quality and if over 95 per cent it is definitely a poor subgrade material. These limits are only true for cohesive subgrades with plasticity indices between 15 and 40 which do not contain any frictional material. The indications are that for cohesive subgrades with a plasticity index less than 15 the 20 psi standard of good subgrade quality is attained at a lower ratio of soil moisture to lower plastic limit. This is probably due to the lower surface area which permits more of the water to act as a lubricant between the soil particles.

Although Figure 10 shows only a general trend for subgrades of a limited classification, it is believed that further studies may establish better relationships between stability and some arbitrary soil factor One that would include density and plasticity index in addition to the values given would be of interest The great need of establishing these relationships is to increase the practical value of the laboratory determined soil-constants.

TEST METHODS

The description of the two test methods that follow and the discussion of the data obtained from pilot determinations are given at this time for general information only. Neither the apparatus nor the testing technique are fully developed but they are being briefly described at this time in hope that other investigators may want to use them or a modification of the principles involved.

Apparatus for Determining Bearing Capacity.

This method utilizes the anchorage obtained by placing an ordinary soil auger in the soil to a suitable depth and operating a hydraulic jack against a horizontal beam attached to the upper part of the anchored auger system. Figure 11 shows a close-up view of the attachment on the upper part of the anchored auger system for holding the horizontal beam. Figure 12 shows a general view of a field set-up for making bearing tests. After the auger has been screwed into the soil for a distance of at least 3 ft. or more if necessary, it is anchored securely by slipping a lever through the yoke and pulling upward. The horizontal beam, the hydraulic jack and dials are then placed. The jack on the right is used only to keep the system approximately horizontal, which is indicated by a spirit level placed on top of the horizontal beam. This assures more consistent dial readings. The load is applied by the hand wheel and recorded on a pressure dial gauge located in the box at the right.

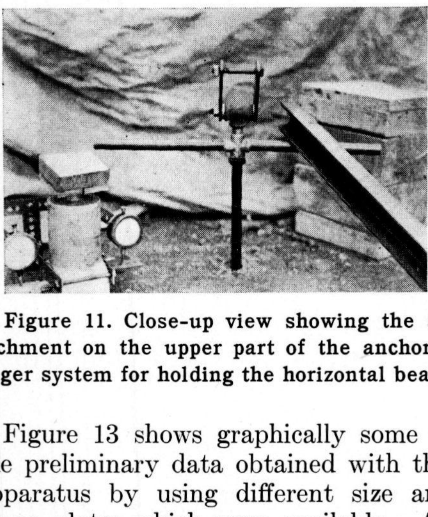

Figure 11. Close-up view showing the attachment on the upper part of the anchored auger system for holding the horizontal beam.

Figure 13 shows graphically some of the preliminary data obtained with this apparatus by using different size and shape plates which were available. Although the bearing plates used had different shapes the figure shows the typical trend of increased deformation for a unit bearing stress, as the size of the plates increase. Also the decrease in unit bearing stress for a constant deformation, as the size of the bearing plates increase.

The two lower curves also show approximately a 1 to 3 relationship between shearing and bearing stresses when the shape and size of the plates are constant.

The main value of this testing method is that the individual units of the apparatus are easily handled, transported and when assembled offer very little inconvenience to traffic or other operations. The method also obviates the use of loaded trucks or other unwieldly vehicles in obtaining bearing values of

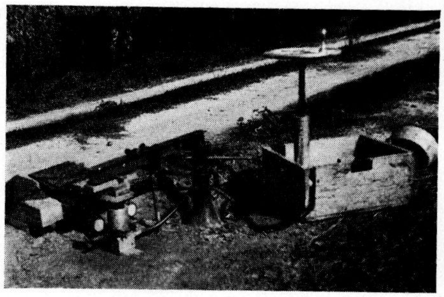

Figure 12. General View Showing Field Set-up for Making Bearing Tests

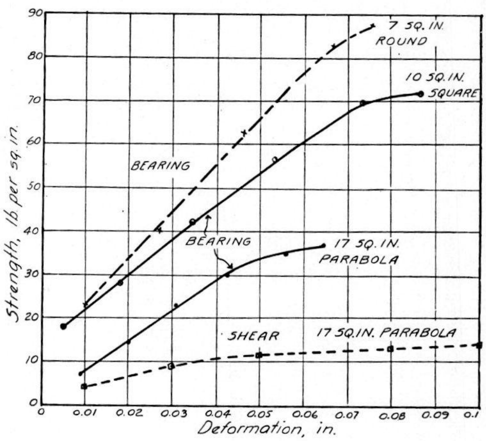

Figure 13. Relationship of, (1) bearing strength to deformation for different size and shape plates and (2) shearing to bearing strength for same size and shape plate.

subgrades or other road components. Its utilization also includes the measurement of the "softening action," sometimes observed in the upper part of stabilized bases under thin bituminous mats, during Spring "break-up" periods.

The following factors were scheduled for investigation when the work was discontinued: (a) The use of two augers which would permit a simple beam arrangement instead of a cantilever type as used. This would require greater dislodgment pressures and eliminate the use of the levelling jack. It required about 1200 lb. to dislodge the 8-in. long and $1\frac{1}{2}$-in. diameter screw auger when anchored at a depth of approximately 40 in. in an ordinary cohesive soil. This would vary with the type and nature of the material in the lower strata.

(b) The use of larger diameter and longer screw augers and other anchoring devices such as those used to anchor guy wires. This would permit the use of larger area bearing plates.

(c) The use of circular plates of different areas to obtain data on the perimeter-area relationship.

(d) General improvement in testing technique.

Penetrometer Apparatus.

The use of a penetration method to measure the resistance of soils is not new. The two most comprehensive reports on this subject are: (1) "Determination of Consistency of Soils by Means of Penetration Tests", by Charles Terzaghi, *Public Roads,* February 1927, and (2) "A Penetration Method of Measuring Soil Resistance", by W. S. Housel, *Proceedings,* American Society for Testing Materials, Vol. 35, Part II, p. 472 (1935).

In the present case, the main objective for using a penetrometer was to obtain quickly additional structural data on undisturbed cohesive soils as they were being tested by the shear machine and to study the interrelationship of these values.

Figure 14 gives a general view of the penetrometer apparatus. It consists mainly of a 2-in. diameter pipe 30 in. long with a blunt pointed $\frac{5}{8}$-in. diameter rod 30 in. long firmly attached to the lower end. This rod passes through metal bearings held in a wooden frame, which rests on the road surface, so that the rod and pipe system is held in a vertical or upright position at all times. A 10-lb. weight, $1\frac{3}{4}$ in. in diameter and 15 in. long is located inside the 2-in. pipe and this is raised to the maximum height by a small rope passing through the upper cap of the pipe and allowed to

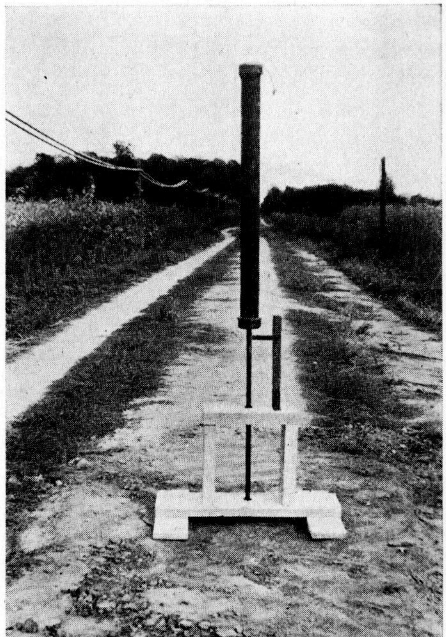

Figure 14. General View of Penetrometer Apparatus

drop by suddenly releasing the grip on the rope. By noting the relative positions of a horizontal pointer attached to the upper part of the $\frac{5}{8}$-in. rod, on a graduated scale fastened in a vertical position on the wooden frame, the degree of penetration per blow can be determined. The base of the wooden frame is so designed that there is no confinement of the soil adjacent to the area being penetrated To make a test, the operator stands on the base of the apparatus and lowers the needle system to a position where the point is in contact with the soil The pointer position on

on many more soils of varying cohesive qualities and moisture contents are necessary before conclusive deductions can be made Also the design of the penetrometer used can be greatly improved upon In general these pre-

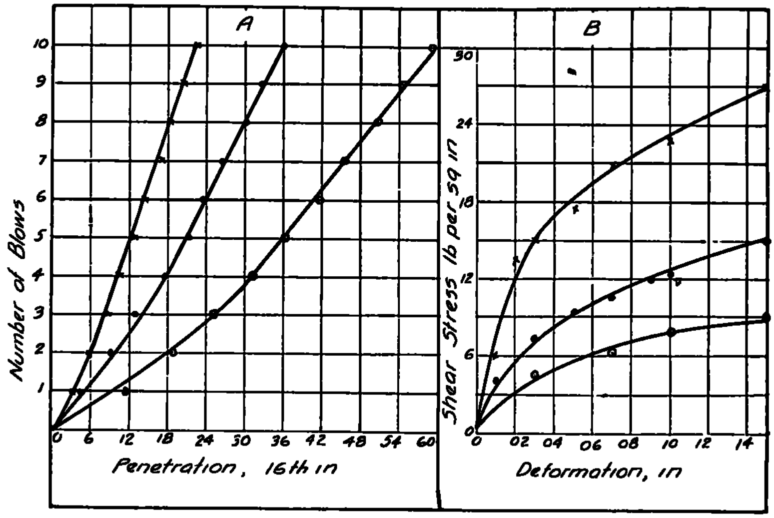

Figure 15. (A) field penetrometer testsshowing the relationship of the number of blows to the degree of penetration for soils of varying quality. (B) field shear tests showing the relationship of stability to deformation for the same soils.

the vertical scale is then recorded and the weight raised to its maximum height and allowed to drop freely. The new position of the pointer on the scale is then read and the operation repeated until any degree of penetration is attained

The curves to the left in Figure 15 show some typical results obtained by the penetrometer on unconfined cohesive soils of varying quality The curves at the right show the shear stress-strain results for the same soils. Figure 16 shows the relationship of the soil stability in shear to the number of blows required for the penetrometer rod to pass into the soil to a depth of 1 in. The quality of soil included in Figure 16 range from good to very poor from a subgrade standpoint. It is again emphasized that these are only preliminary results and that tests

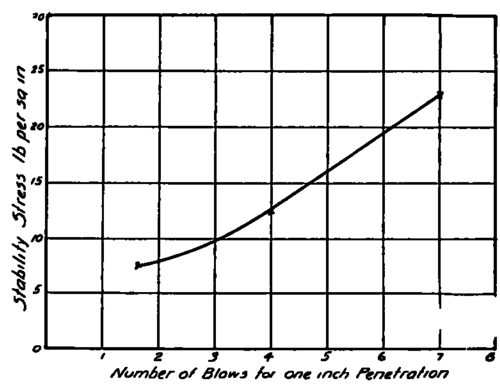

Figure 16. Relationship of soil stability to the number of blows required for the penetrometer rod to pass into the soil to a depth of one inch.

liminary results indicate that this type of test, if it can be properly correlated with some standard test method, will be a means of obtaining a large volume of valuable quantitative data on cohesive soils with a minimum of effort and time.

CONCLUSION

Based mainly on the favorable relationships obtained from the preliminary test methods described herein and on the realization that there is great variation in the effort required and the type of equipment needed to make these different tests, there appears to be a great need for an extensive field study of cohesive soils in which the shearing, bearing capacity and penetration methods are employed simultaneously and the results compared; also for a correlation of these tests with the laboratory stabilometer method. From such a study a minimum standard for a suitable subgrade could be established which would afford a quantitative basis from which the effects of any subgrade treatments or improvements could be measured and compared. Furthermore it would establish the most practical and effective test method for getting actual field data to use in the existing design formulas for flexible pavements.

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