A STUDY OF THE APPLICATION OF SHEAR STRENGTH
FOR EMBANKMENT CONTROL

by 680

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ASTM	American Society for Testing and Materials
B_j	The average effect of using dry density j relative to grand average.
C	Cohesion
D	Density, Diameter
D.F.	Degrees of freedom
E_{ijk}	The random error represented in the particular shear strength from a combination of ith moisture, jth dry density and kth run
h	Height
H_o	Hypothesis to be tested
H_a	Alternate hypothesis
i	Treatment number
j	Block number
k	Run number
k.s.f.	Kips per square foot
M.S.	Mean square
NID	Normally, and independently distributed
P	Number of means to be tested
psf	Pounds per square foot
$R_p^{0.05}$	Shortest significant range at 5% level
S.	Shear strength
S.S	Sum of squares
S_x	Standard error of varietal mean
t.s.f.	Tons per square foot

T_i	The effect on average shear strength of using moisture i, as compared with grand average
$(TB)_{ij}$	The additional effect beyond and added to T_i, and B_j of combining the ith treatment and jth block
U	Grand average shear strength
U_a	Pore air pressure
U_w	Pore water pressure
V	Volume
w	Water content
W	Total weight
W_w	Weight of water
W_s	Weight of solids
X_{ijk}	The kth observation with moisture i and dry density j
ϕ	Angle of internal friction
σ	Total stress, standard deviation
$\bar{\sigma}$	Effective stress
χ	A factor depending primarily on degree of saturation, but which may also be influenced by stress history, wetting or drying sequence, and soil type.
α	Region of rejection
*	Significant at $\alpha = 0.05$
**	Significant at $\alpha = 0.01$

INTRODUCTION

Although soil has been used since the beginning of civilization as a structural material for embankments to carry traffic or to retain water, a scientific method of design was not applied until some one hundred years ago. This rationale of design was based on experience and theory and was the result of military and civilian demand for higher earthen embankments. With the development of modern earth moving equipment, the desire and need for improved standards of design, and recent developments in soil mechanics techniques, the depth of embankments have increased almost without bound. Highway fills in the past were limited to only minor cuts and fills but it is now not uncommon to construct fills as much as four hundred feet in height when circumstances require it.

This increase in the depth of earthen embankments has required ever increasing sophisticated construction methods and selection of materials to prevent failure. Most high embankments must be greatly overdesigned from two standpoints:

1. The slope is constructed at a lower angle than necessary.
2. Selected materials are hauled in at great expense to form the base of the fill that have a greater shearing resistance than that necessary for shearing resistance.

If these are not observed problems of intolerable settlements and slope failure occur. At the present time the quality of these fills is controlled by the moisture-density relationships established by Proctor (1) in 1933, by specifying some percentage of the indicated maximum dry density. It is recognized that the various changes in compaction standards to meet the demands of these deeper fills have required ever increasing compactive effort and these now have reached limits where other methods of evaluation must be used. Recent work has indicated that over compaction, especially at higher moisture contents, actually reduces the shearing resistance in materials containing clay.

One approach to overcome such like ambiguities may be the use of some type of shear test since failure due to slope instability in general is a shear failure. According to J. E. Roberts and J. M. Desouza (2) the magnitude of the normal stresses applied to soil by these high embankments is far below that required to produce crushing of grains (Customary stresses are in the range of 0 - 50,000 Psf. as compared to stresses of around 500,000 Psf. for crushing of the grains). The failure of embankments or fills is usually due to the sliding of large numbers of soil grains over one another which occur when the shear forces resulting from applied loads exceeds the shear strength of soil mass.

It is necessary that a thorough search of existing literature be made to guide a modest experimental study in the laboratory investigating the possible use of shearing resistance for embankment control in lieu of the now required maximum dry density-optimum moisture content of the compacted soil.

THE PURPOSE OF THE STUDY

The purpose of this study was to investigate:

a. The feasibility of employing shear strength characteristics of the soil for control of compaction of embankments.

b. To determine the relationship between water content and dry density which yields the maximum shear strength for a selected soil sample.

c. To develop a reliable and simple test for the determination of in situ shear strength for field control of soil compaction.

THE SCOPE OF THE STUDY

The soil used for testing was obtained from an embankment at Saylorville Dam and Reservoir near Des Moines, Iowa, as shown on the following map. The shear strength for the remolded samples of the above soil was determined in the laboratory by a direct shear device at different combinations of moisture and dry density. Five different combinations of moisture content, 90, 95, 100, 105 and 110 percent of optimum moisture content, were used for each of the three different dry density levels, 90, 95 and 100 percent, as determined by the standard Proctor test.

Three determinations for each of these fifteen different combinations were obtained.

The shear strength of the embankment as compacted was determined in place, using a Bore Hole Direct Shear Device to determine the applicability of this device for the field control of the compaction process.

Figure 1. Map of Saylorville Reservoir showing sampling location.

REVIEW OF LITERATURE

General

With the greatly increased depths of embankments in recent years shear failures have become common even though the amount of testing and inspection for control has been expanded. It is widely recognized that new tests and testing procedures must be developed. The following review of literature traces the development of control procedure, the theory of shear in soils, and compaction procedures.

According to T. E. Stanton (3) the first work along these lines was done by the California Division of Highways in 1929 when an extensive series of tests were conducted from which was developed the field equipment and methods of compacting soil samples to determine the optimum moisture content required and subsequently the relative compaction of the completed embankment.

About 1933, the engineers of the Bureau of Waterworks and Supply of the City of Los Angeles conducted a similar study. The results of this study were described in a series of articles by R. R. Proctor (1), the Field Engineer for the Bureau, published in several issues of engineering News Record, beginning August 31, 1933.

Proctor described field compaction equipment somewhat different from that developed by the Division of Highways, but using similar compaction procedures.

The Proctor Method of compaction control became widely known and led to the wide-spread adoption of control test procedures adopted today in the ASTM Standard Test method D698-64T (4). With the tremendous expansion of military construction, particularly the airfields during the war years, 1941-1945, the Corps of Engineers stepped up the compaction requirements by developing the compaction procedure known as the Modified ASTM Standard Test Method D-1557-64T (4), which sets a much higher standard for density.

The great majority of state highway departments are presently using dry density as the principal criterion for judging the quality of compacted earthwork. This criterion implies that the increased dry density produces improved engineering properties in the material. Although the use of dry density for field control can be easily accomplished, particularly with the increasing use of nuclear devices, its value as a usable criterion is only valid in so far as dry density does in fact indicate the strength properties of the material. It was reported by Hveem and Vallerga (5) that an increase in the density is not beneficial in some soils depending upon the type of soil, degree of compaction, and moisture content. The two most important and generally applicable properties are the shear strength and compressibility characteristics of the compacted material.

The first formal analysis of the mechanism of shear in soil was made by Coulomb in 1776 (6). In this classic work it was reported that the shear strength of a soil was dependent upon a component of cohesion and a component dependent upon friction. The relationship between normal force and friction was established and the general shear equation

$$S = C + N \tan \Phi \tag{1}$$

was given.

The importance and use of this theory was used by a few but ignored by most until the work of Terzaghi in 1925 (7). In this important work in soil mechanics Professor Terzaghi formalized three important areas of soil behavior:

1. The consolidation of clays.
2. The principle of effective stress in the shear strength of soil.
3. The geometry of the shear plane in a soil mass in failure.

Considering only the work related to shear the theories of Terzaghi altered the Coulomb formula to:

$$S = C + \bar{\sigma} \tan \Phi \tag{2}$$

in which $\bar{\sigma}$ represents only the effective stress. The total stress on the shear plane must be reduced by an amount equal to the pore water pressure.

Since the soils encountered in engineering practice are neither purely cohesionless nor cohesive, the shear strength characteristics for both have been discussed separately.

Shear Strength of Cohesionless Material

L. J. Langfelder and V. R. Nivargikar (8) have reported that the shear strength of cohesionless material is essentially controlled by the following five factors:

1. Mineralogical composition.
2. Size and gradation of individual particles.
3. Shape of the individual particles.
4. Void ratio or dry density.
5. Confining pressure.

Assuming that the shear strength can be expressed by Coulomb failure criterion (6) for zero cohesion, the first four factors mentioned affect the angle of internal friction, whereas the fifth factor controls the normal stress. The first three factors are the properties of the material and therefore are dependent on the material encountered. The confining pressure is principally governed by the amount of overburden. Increased confining pressures for a given cohesionless material will produce larger shearing resistance and will affect the stress-strain behavior of the material. The magnitude of the confining pressure also affects the dilation characteristics and consequently affects the shearing resistance. Primarily it is only the dry density or void ratio that can be significantly changed during the process of compaction.

For a given cohesionless material it appears that the shear strength is directly related to density but is independent of the compaction process used. Data presented by Means and Parcher (9) indicates that for a particular granular material the angle of internal friction is inversely related to the void ratio. The change in the angle of internal friction with a change in void ratio appears to differ somewhat depending upon the soils being tested - varying from two degrees for silty sands to about six degrees for uniform gravels for each 0.1 change in void ratio.

Based on the effective stress theory it is seen that the effective stress may either increase or decrease with increasing water content along a compaction curve on the dry side of optimum, but the effective stress will always decrease with increasing water content along the compaction curve on the wet side of optimum. The shear strength, being directly proportional to the effective stress, will increase or decrease accordingly.

As reported by L. J. Langfelder and V. R. Nivargikar (8) the most important factors that will produce increasing shear resistance are:

a. increasing angularity of particles,

b. increasing surface roughness,

c. improved gradation.

Improved gradation and possibly increasing amount of larger grained material mainly increase the amount of dilation during shear, which leads to increasing shear resistance.

Shear Strength of Cohesive Material

According to L. J. Langfelder and V. R. Nivargikar (8) the shear strength of a purely cohesive soil is primarily affected by:

1. Normal effective stress acting on failure plane.
2. The water content.
3. Gradation.
4. Dry density.
5. Soil structure.
6. Thixotropy.

The effective stress that acts on an element of soil is produced by external pressure, such as overburden, and internal pressure exerted by the apparent negative pore water pressure. The overburden pressure on subgrades are quite small, therefore the major contribution to the effective stress would be the internal pressure. The water content that influences the shear strength is not only controlled by the molding water content, but includes any changes in moisture conditions that occur after placement. The dry density is controlled by the amount of compactive effort expended during compaction, the water content at which compaction takes place, the method used to compact the soil and any density changes that occur after initial compaction. The soil structure is controlled by the method of compaction used and water content relative to the optimum water content. The thixotropic effects for a given soil depend upon the time allowed for strength changes to occur and the strain level at which strength is defined.

Influence of Effective Stress

Bishop (10) proposed the following expression for defining the effective stress in an unsaturated soil:

$$\bar{\sigma} = \sigma - \chi U_W - Ua\,(1-\chi) \qquad (3)$$

$\bar{\sigma}$ = effective stress

σ = total stress

Ua = pore-air pressure

U_W = pore water pressure

χ = A factor depending primarily on the degree of saturation, but which may also be influenced by stress history, wetting or drying sequence, and soil type.

The solution to this expression requires a knowledge of χ, Ua and U_W. The pore air and pore water pressure can be measured by using modification of the pressure plate procedure as described by Richards (11). The determination of the χ-factor requires the testing of the duplicate samples of saturated and unsaturated specimens, and the assumption that the angle of internal friction remains constant upon saturation.

On the dry side of the optimum water content, the air permeability is high and therefore the pore air pressures produced by compaction should be rapidly dissipated. At optimum and slightly wet of optimum, although the air permeability is quite small, the χ factor is large but the term Ua $(1-\chi)$ should be small compared to U_W in eq. 3. Assuming the Ua $(1-\chi)$ term can be neglected, eq. 3 degenerates to $\bar{\sigma} = \sigma - \chi U_W$. For a constant value of total stress the effective stress becomes a function of χ factor and U_W. Assuming that the χ factor is only related to degree of saturation, and pore water pressure is related to the water content similar to the data presented by Lambe (12), Bishop and Blight (13) and Olson and Langfelder (14), Fig. 2, 3 and 4 schematically represents the relationship of dry density, χ factor and pore water pressure to the molding water content.

It is clear from Fig. 2, 3 and 4, that, on the dry side of optimum, U_W becomes less negative as molding water content and dry density increase, and the χ factor continuously increases, therefore, the effective stress may either increase or decrease depending on the interaction of the two factors. This implies that increased dry density does not necessarily result in increased effective stress.

On the wet side of the optimum the degree of saturation is essentially constant beyond optimum water content and thus χ is essentially constant. However, U_W continues to be increasingly less negative as the molding water content increases. This implies that the effective stresses must decrease on the wet side of optimum.

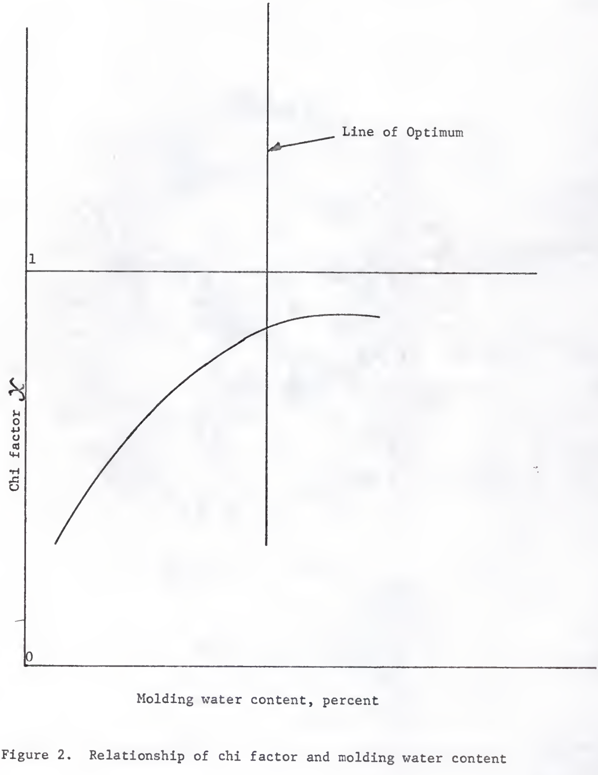

Figure 2. Relationship of chi factor and molding water content

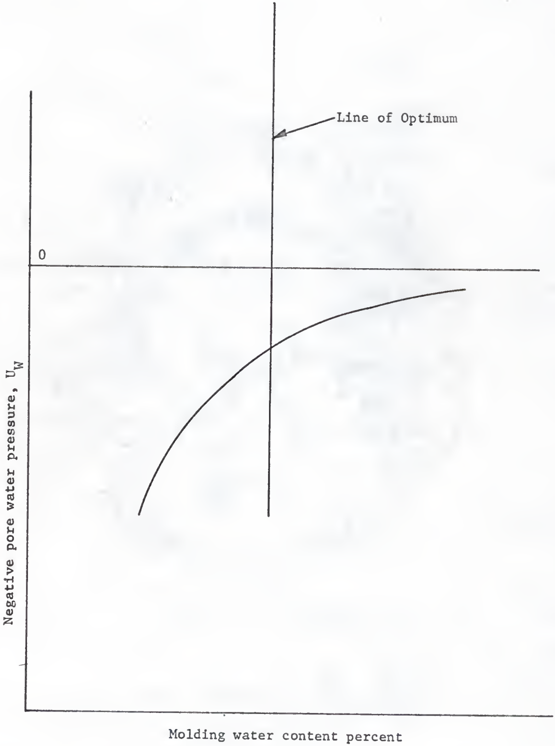

Figure 3. Relationship of pore water pressure and molding water content.

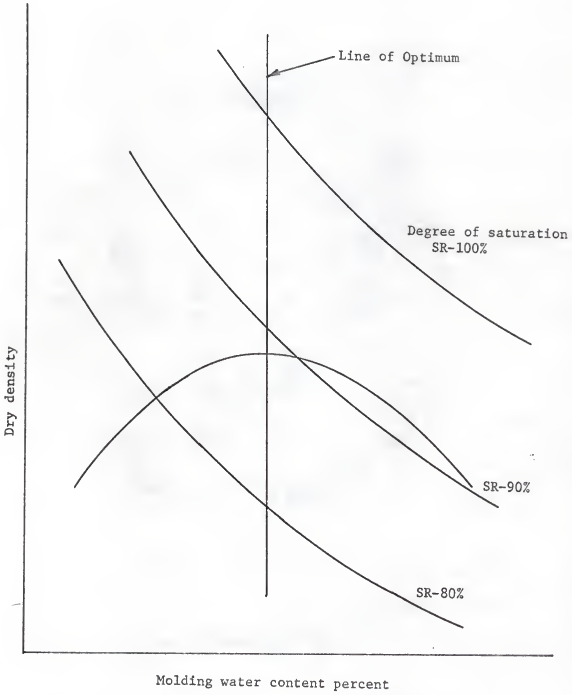

Figure 4. Relationship of dry density and molding water content.

The change in shear strength is essentially a function of effective normal stress on the failure plane and change in frictional resistance. This change in frictional resistance will vary along the compaction curve and therefore it is not possible to establish the change in shearing resistance along the compaction curve from consideration of effective stress alone.

Effect of Molding Water Content and Soil Structure

As reported by Langfelder and Nivargikar (8) varying the molding water content of a compacted cohesive soil will have an effect on

1. The initial soil structure.
2. The magnitude of initial pore water pressure.
3. The dry density of the material.
4. The swelling characteristics.
5. Pore water pressures developed during shear.

Each of these factors in turn, influence the shear strength of material.

According to Seed, Mitchel and Chan (15) the soil structure at low water content is flocculated because of insufficiency of the water available for formation of the double layer, the absence of interference of the adsorbed water films, and the attraction of the negatively charged surface of the clay for the positively charged clay edges and any other cohesion present. As the water content increases there is a tendency for greater interference of the water film and if an opportunity for particle rearrangement exists, the soil will tend towards a more dispersed structure. Therefore as the molding water content is increased it should be expected that the shear strength should decrease based upon this change in structure.

Various compaction theories (Proctor (16); Hogentogler (17); Lambe (18); and Olson (19) have attempted to define the mechanism by which molding water tends to affect the dry density and in turn the shear strength, that can be obtained by a specific compaction technique. It is generally agreed that the addition of water to a dry cohesive soil first allows the particles to be more easily packed (up to optimum). After optimum water content is reached, the addition of more water acts to dispel soil particles. Considering this as compacted state of the cohesive soils, all the available data indicates that for any constant value for dry density the shear strength will decrease with an increase in molding water content. In fact CBR data from a series of Waterways Experiment Stations Publication (20, 21, 22) indicate that for water contents up to approximately 10 percent dry of the optimum the strength in almost all cases decreases or remains essentially constant with increasing molding water content, even though the density increases with increasing water content on the dry side of optimum. These data imply that if increased strength is the primary engineering property sought it would be advantageous to compact the soil well dry of the field optimum water content. This would be particularly the case where the natural water content is less than the optimum water content and water must be added.

Effect of Method of Compaction

A comprehensive study of the effect of method of compaction and water content on soil structure and soil strength was done by Seed and Chan (23) in 1959. It was shown that for a soil susceptible to dispersion by shear strain the greater the shear strain, during compaction the greater is the degree of dispersion of compacted soil.

For samples of the same soil having the same density and water content, it has been found that the greater the degree of dispersion of clay particles the lower is the strength of soil at low strains and the greater is the shrinkage of the sample. For the value of stress required to cause 5 percent strain for samples prepared to the same dry densities and water content by static, vibrating and kneading compaction, it was found that on the dry side of optimum water content, the method of compaction has no effect on soil strength, indicating that the various methods of compaction produce similar structures. This is to be expected since none of the methods of compaction induces shear strains at water content below optimum. During compaction on the wet side of optimum moisture, however, the different methods of compaction induce increasing shear strains and thus greater dispersion in the order of static (being least), vibratory, and kneading. It is seen that the strengths of the resulting samples decrease in the same order.

It should be noted however, that the effect of method of compaction on soil structure can vary markedly in different soils. In a comparison of the effect of kneading and static compaction on strength measured at low strains in three different soils, the relative susceptibility of the soils to structural alteration by shear strains varied considerably. This is probably due primarily to the different status of the inter-particle forces in the clay fraction of the soils. If these interparticle forces (which in the clay fraction of the soils are very strongly attractive) are strong the clay particles will tend to assume an aligned arrangement, regardless of any attempts to disperse them by shear strains. If the inter-particle forces are strongly repulsive, the particles will assume a dispersed arrangement whether the method of compaction induces shear strains or not. It is in those soils where the balance of inter-particle forces is not strongly attractive or repulsive that shear strains and method of compaction can have the greatest effects.

Effect of Dry Density

The changes in shear strength that are produced as a function of changes in dry density alone can be determined by using several different compaction energies and comparing the strength at constant value of molding water content. This procedure assumes that there is no effect of the possible change of the soil structure as optimum water content decreases with increasing compaction energy. This assumption is only valid if the strength is measured at large strains, however, if the strength is measured at low strains, the influence of change in void structure should not be neglected. According to Langfelder and Nivargikar (8), it appears that the method of compaction influences the response of the shear strength to change in dry density at constant molding water content.

Seed and Moni-Smith (24), Seed, Mitchell and Chan (15), and Casagrande and Hirschfeld (25) have presented data on the relationship between dry density and shear strength at different molding water contents. All these data indicate that an increase in dry density will cause an increase in shear strength for a given water content, provided the shear strength is defined at both large strains and moderate confinement pressure. In general the rate of increase in shear strength with an increase in dry density is largest for the lowest value of water content. As the molding water content increases the increase in shear strength decreases depending upon the soil being investigated. If the stress mobilized at low strains is plotted against dry density for constant values of water content in soils compacted by different methods of compaction, it can be shown that the relationship between stress and dry density depends on the water content and method of compaction. For a moderate confining pressure ($1 Kg/cm^2$) statically compacted samples exhibit an increase in shearing resistance with increasing density. However for kneading compacted samples there is a marked decrease as water content increases. It is seen that the decrease in stress for the higher densities with an increase in dry density is most pronounced for 1 percent strain data and, except for the greatest water content, non-existent for 20 percent strain data.

It is interesting to note that both field and laboratory compacted CBR data exhibit relationships between strength and dry density similar to the relationship found at low to medium strain level for kneading compacted specimen tested in the Triaxial apparatus.

The relationship between the shear strength after soaking and the initial dry density depends on the amount of swelling that takes place during soaking, the compaction method used and soil type.

Seed and Chan (26) have shown, however, that the soaked strength of a compacted cohesive soil will increase with an increase in initial dry density regardless of the compaction method, soil type (although the soils investigated were limited), amount of swelling during soaking, and strain level. An exception to this conclusion occurs where strength is defined at low strain and soil is compacted by a method that produces large shearing strains.

Thixotropic Considerations

The process of strength changes with time at a constant water content is generally referred to as thixotropy in soil mechanics literature. This property is important when attempting to predict field strengths at some time after compaction from laboratory tests that are generally performed soon after compaction or when soaking has been completed.

Mitchell (27) has hypothesized that the cause of thixotropy as being the creation of a new equilibrium condition resulting from the cessation of external compaction forces. In order to obtain increase in shear strength with time it is necessary that the final equilibrium condition be conducive to a flocculent structure and structure immediately after compaction be a relatively dispersed structure. This condition can be produced in certain soils by using kneading compaction methods even up to water contents slightly wet of optimum. In conjunction with this change in soil structure it was found that the pore water pressure decreases during aging and also the pore water pressures developed during shearing are smaller for aged samples. It is quite likely, therefore, that there is an increase in strength in terms of total stress but strength remains constant in terms of effective stress.

In addition to the influence of molding water content on the amount of strength gain, the strain at which failure is defined also determines the measured amount of strength increase.

DESIGN OF EXPERIMENT

For the purpose of determining whether shear strength could be used for embankment control a series of samples at selected combinations of moistures and densities as shown in Table I, were prepared and tested for shear strength.

The combinations of moisture and densities were selected in such a way that the range of proctor densities in current use would be covered. Five different combinations of moisture content 90, 95, 100, 105 and 110 percent of optimum moisture content, were used for each of the three different dry density levels 90, 95 and 100 percent.

Three samples for each level of moisture and density or 45 samples in all were prepared and tested to reduce experimental error in so far as possible.

To evaluate the significance of the findings from the experiments the data was analyzed by the analysis of variance (28), and by the Duncan New Multiple range test (29).

EXPERIMENTAL PROCEDURE

The experimental procedure involved the preparation of three specimens for each of the fifteen different combinations of moisture and dry density. Each sample within a set had to be prepared at a relatively exact moisture content and to a prescribed density. In order to accomplish this, samples of a known volume and weight of water and soil solids were prepared.

The samples used were 2.5" in diameter and 1" high. The volume of the fabricated samples is shown in the following computations

Table I. Combinations of moistures and densities to be used in experimental procedures.

		% Optimum Moisture				
% Dry Density		90	95	100	105	110
	1	X	X	X	X	X
90%	2	X	X	X	X	X
	3	X	X	X	X	X
	1	X	X	X	X	X
95%	2	X	X	X	X	X
	3	X	X	X	X	X
	1	X	X	X	X	X
100%	2	X	X	X	X	X
	3	X	X	X	X	X

1. To compute the volume of the samples

$$V = \frac{\pi D^2}{4} h = \frac{\pi (2.5)^2}{4} \times 1 = 0.00284 \text{ cu. ft.}$$

2. The weight of oven-dry soil used for each set of samples was computed by the formula:

$$D = \frac{W}{V(1+w/100)}$$

and these weights are tabulated in Table II.

The water content required was computed by the formula

$$w = \frac{W_w}{W_s}$$

$$W_w = wW_s$$

and is shown in Table II for each of the various sets.

PREPARATION OF SPECIMEN

For a particular combination the required weight of oven dry soil passing U. S. Standard sieve #10 was taken and thoroughly mixed with the amount of distilled water as given in Table II. This sample was kept in a moist cabinet for 24 hours to insure uniform content throughout the sample.

Table II. Weights of oven-dry soil and water required for the preparation of specimens of different combinations.

Dry Density, 90%

Moisture %	Total weight, W, gms.	Weight of solids, W_s (gms)	Weight of water W_w (gms)
90	151.0	135.5	15.5
95	152.00	135.5	16.5
100	152.90	135.5	17.4
105	153.80	135.5	18.3
110	154.70	135.5	19.2

Dry Density 95%

Moisture %	Total weight, W, gms.	Weight of solids, W_s (gms)	Weight of Water W_w (gms)
90	160.32	143.64	16.68
95	161.24	143.64	17.60
100	162.20	143.64	18.56
105	163.05	143.64	19.41
110	164.02	143.64	20.38

Dry Density 100%

Moisture %	Total weight, W, gms.	Weight of solids, W_s (gms)	Weight of Water W_w (gms)
90	168.22	150.72	17.50
95	169.20	150.72	18.48
100	170.17	150.72	19.45
105	171.08	150.72	20.36
110	172.11	150.72	21.39

The total weight was statically compacted to a height of 1" in a floating piston apparatus shown in Fig. 5, compressed by a Soil Test Versa-Tester (see Fig. 6).

It was observed in the trial runs before starting the actual experiment that the loss of water content in molding was about 1 percent. In molding the specimens 1 percent additional water was added.

A determined effort was made to control the moisture within limits of $\pm$ 0.5 percent and the density within $\pm$ 0.5 lbs per cu. ft. This requirement was generally met, however in no case was the discrepancy of more than $\pm$ 1 lb per cubic ft. in density permitted.

TESTING OF SPECIMEN

The specimens after having been removed from the mold were immediately set in a strain-controlled direct shear machine for shear strength determination.

The machine used (Fig. 7) was modified from a stress-controlled direct shear device, in the workshop at KSU. The strain was applied by an electric motor with variable speed reduction which fed a finely threaded rod into a threaded tube which in turn through a proving ring pushed the upper portion of the sample box applying a shear force to the sample. A strain-dial attached to the top shear box gave the strain on the sample. The proving ring was calibrated to give shear force in pounds. The normal load was applied to the specimen through weight applied on a hanger system. A normal load of 0.925 tons per sq. ft. was applied. No drainage was permitted by using non-porous stones at top and bottom

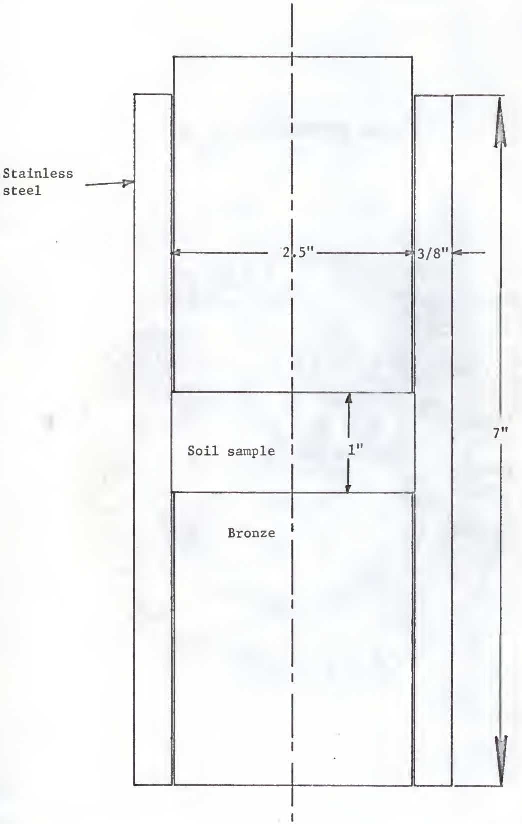

Figure 5. Floating piston apparatus for molding specimens.

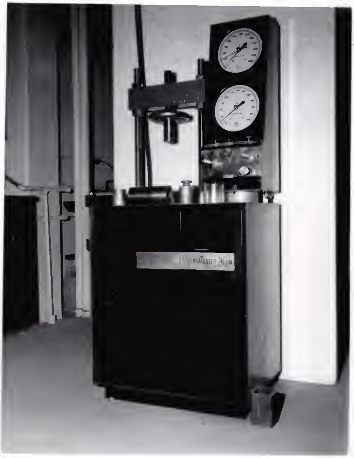

Figure 6. A Soil Test Versatester for molding samples.

Figure 7. Direct shear test machine.

of the specimen.

The test was conducted at a rate of 0.02 inches per minute until the peak strength was obtained.

The test results were graphed with the abscissa representing the strain and the ordinate the corresponding shear stress, as usual. Three different observations were graphed on the same paper for a normal load of 0.925 t.s.f. The peak shear strengths obtained from these graphs are tabulated in Table IV.

PRESENTATION OF DATA

For classification of the soil and to determine its other characteristics, grain size analysis and Atterberg limits were carried out in accordance with ASTM standards (5) D-423-63 and D-423-61T and D-424-59 (1965), respectively. The general characteristics of the soil are shown in Table III. The plots of grain-size distribution and Standard Proctor Density are given in Appendix A.

Table III. Soil characteristics of the raw soil.

The physical properties of the soil were as follows:

1. Geological classification................ Wisconsin Glacial Till
2. Specific Gravity of Solids...................................2.723
3. Atterberg limits..........
   a. Liquid limit..25.3
   b. Plastic limit...13.8
   c. Plasticity Index......................................11.5
4. Group Index..5.60
5. Standard Proctor Compaction..............
   a. Optimum moisture content percent......................12.9
   b. Maximum dry density lbs per cu. ft.....................117
6. Grain-size analysis percent..............
   a. Passing # 200 sieve....................................60
   b. Clay (smaller than 0.002 mm.)..........................12
   c. Silt (0.002 to 0.05 mm.)...............................40
   d. Sand (greater than 0.05 mm.)..........................43.2
7. Uniformity Index...43.2
8. Granulometry............................very non-uniform soil
9. HRB Classification..............................A_4 Silty soil

The peak shear values for the forty five samples tested are shown in Table IV. These peak values were obtained from the stress-strain curves graphed for each of the direct shear tests.

Table IV. Peak shear strength in k.s.f. for selected combinations of moisture and dry density.

Percent Max. Dry Density	Percent Optimum Moisture Content				
	90	95	100	105	110
90	2.06	2.20	2.22	1.91	1.78
	2.07	2.32	2.26	1.90	1.70
	2.37	2.00	2.10	1.92	1.70
95	2.24	2.30	2.70	2.12	2.13
	2.22	2.30	2.52	2.18	2.00
	2.22	2.16	2.60	2.20	2.04
100	2.36	2.39	2.68	2.27	2.12
	2.40	2.50	2.48	2.20	2.10
	2.26	2.54	2.72	2.20	2.18

ANALYSIS OF DATA

After the data was collected, it was analyzed to determine the effects of moisture, dry density, and combinations of both on the shear strength of the soil. The significances of the experiments were interpreted by analysis of variance described in Appendix C.

Computations from Table V:

Densities, sum of squares:

$$\frac{(30.5)^2 + (33.93)^2 + (35.40)^2}{15} - \frac{(99.84)^2}{45}$$

$= 222.3086 - 221.51$

$= 0.7986$

Moistures, sum of squares:

$$\frac{(20.20)^2 + (20.71)^2 + (22.28)^2 + (18.90)^2 + (17.75)^2}{9} - \frac{(99.89)^2}{45}$$

$= 222.85 - 221.51$

$= 1.34$

Subclass, sum of squares:

$$\frac{(6.50)^2 + (6.52)^2 \text{ .... } (6.67)^2 + (6.40)^2}{3}$$

$= 223.85 - 221.51$

$= 2.34$

M. X D. Sum of squares:

= 2.34 - (1.34 + 0.7986)

= 0.2014

Total, sum of squares:

$$(2.06)^2 + (2.07)^2 + (2.37)^2 \ldots (2.12)^2 + (2.10)^2 + (2.18)^2 - \frac{(99.84)^2}{45}$$

= 224.0825 - 221.51

= 2.5725

Analysis of Variance

Source	D.F.	S.S.	M.S.	F.	$F_{0.05}$
Moisture, m	4	1.34	0.335	$\frac{0.335}{0.00775}$ 43.3**	3.84
Density, D	2	0.798	0.40	$\frac{0.40}{0.00775}$ 51.5**	4.46
Moisture X Density M. X D.	8	0.2014	0.0252	$\frac{0.0252}{0.00775}$ 3.25*	2.27
Samples same moisture and density or Remainder	30	0.2325	0.00775		
Total	44	2.5725			

* Significant at $\alpha = 0.05$

** Significant at $\alpha = 0.01$

Table V. Shear strength of soil samples with provision for study of the interaction between moisture and dry density.

$$\text{Model} - X_{ijk} = u + T_i + B_j + (TB)_{ij} + E_{ijk}$$

Percent Max. Dry Density	Percent Optimum Moisture Content					Block Total
	90	95	100	105	110	
90	2.06	2.20	2.22	1.91	1.78	
	2.07	2.32	2.26	1.90	1.70	
	2.37	2.00	2.10	1.92	1.70	
	6.50	6.52	6.58	5.73	5.18	30.51
95	2.24	2.30	2.70	2.12	2.13	
	2.22	2.30	2.52	2.18	2.00	
	2.22	2.16	2.60	2.20	2.04	
	6.68	6.76	7.82	6.50	6.17	33.93
100	2.36	2.39	2.68	2.27	2.12	
	2.40	2.50	2.48	2.20	2.10	
	2.26	2.54	2.72	2.20	2.18	
	7.02	7.43	7.88	6.67	6.40	35.40
Column Totals	20.20	20.71	22.28	18.90	17.75	

Grand Total.... 99.84

From the analysis of variance table the following inferences were drawn:

1. The hypothesis that the true average shear strength with these different moistures (90-110 percent) averaged over these three dry densities (90, 95, and 100 percent) equally are equal should be rejected with 99 percent confidence.
2. Similarly the hypothesis that the true average shear strength with these three different dry densities i.e. (90-95 and 100 percent) averaged over all the five moisture content i.e. (90, 95, 100, 105, and 110 percent) equally are equal should be rejected with 99 percent confidence.
3. Since the mean sum of squares moisture X density interaction is larger than that for samples within the same moisture, density combination, we can say with 95 percent confidence that these are two different sources of variation, and there is truly an interaction. So the decision as to what moisture content may be used to give a particular shear strength cannot be specified without specifying as to which density is to be used and vice-versa.

TEST OF SIGNIFICANCE FOR MEAN STRENGTH OBTAINED WITH DIFFERENT COMBINATIONS OF MOISTURES AND DRY DENSITIES

For test of significance Duncan's New Multiple Range Test has been used (29).

Mean square for error = 0.00775

$$S_x = \sqrt{\frac{0.00775}{3}}$$

$$= 0.0508$$

Region of rejection $\alpha = 0.05$

$p = 15$

$5\%q = 3.44$

$R_p^{0.05} = 0.17476$

Table VI. Mean shear strength in k.s.f. for selected combinations of soil sample.

Dry Density percent of max.	Moisture content percent of optimum				
	90	95	100	105	110
90	2.167	2.173	2.193	1.91	1.73
95	2.23	2.253	2.61	2.17	2.06
100	2.34	2.48	2.63	2.223	2.133

Any means underscored or bracketed by the same line are not significantly different.

FIELD EXPERIMENTS

The shear strength of the compacted material at Saylorville Dam was determined with the help of, "Soil Bore Hole Direct Shear Test Device" patented by the "Soil Technical Associates" of Des Moines, Iowa. (See Appendix C).

The moisture content as determined in the field was 13.4 percent and dry density 114.5 lbs/cu. ft., which corresponds to 104 percent of optimum moisture content and about 98 percent of the maximum dry density. The value for the shear strength corresponding to a normal load of 0.925 Ton/sq. ft. as applied in the laboratory experiment was found to be about 2770 lbs per sq. ft. which is about 20 percent higher than the values obtained in the laboratory.

DISCUSSION OF RESULTS

The mean shear strength for selected combinations of the soil samples are summarized in table VI. The means which are not significantly different, as shown by statistical analyses ($\alpha = .05$), are connected by underlining or by brackets.

The results indicate that with a definite increase in the moisture content for a fixed dry density the shear strength increases up to the optimum moisture content, and decreases beyond the optimum. For a fixed moisture content the shear strength increases with increase in the dry density.

Statistical analyses of the data indicates that both moisture content and dry density affect the shear strength of the soil, so both moisture content and dry density should be specified to insure given strength.

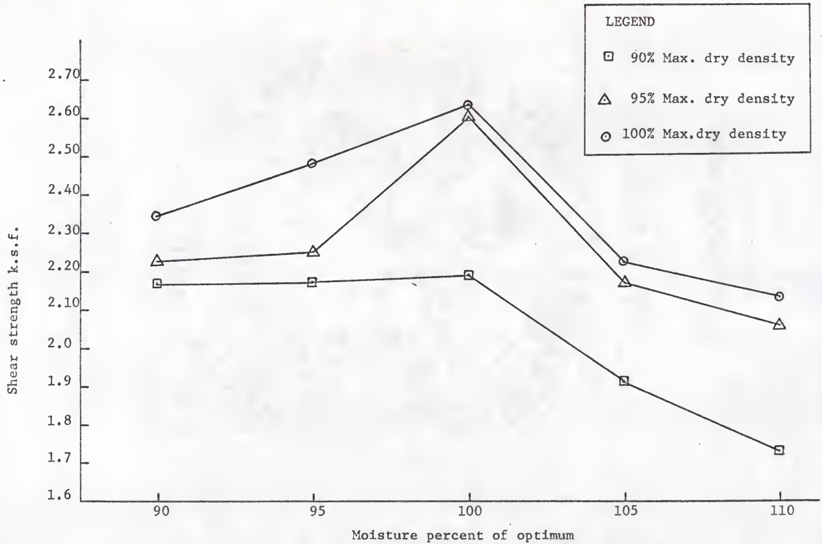

Figure 8. Average shear strength vs. moisture content at different densities.

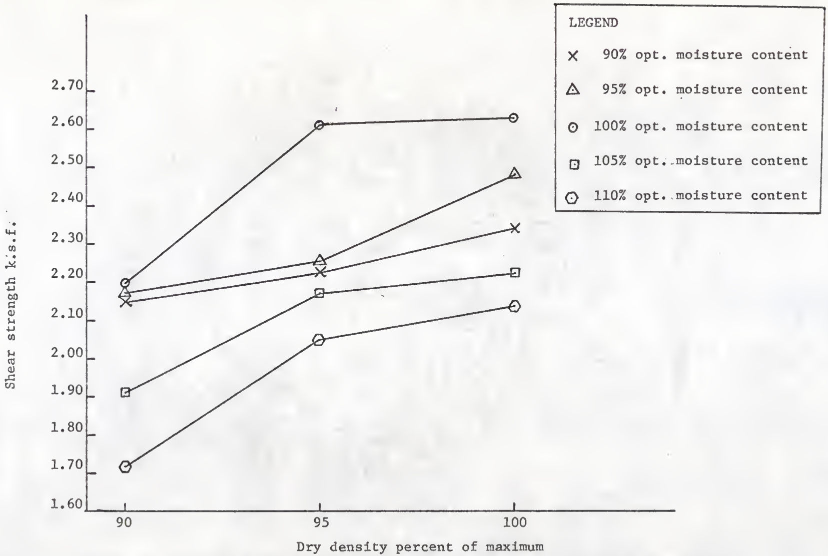

Figure 9. Average shear strength vs. dry density at different moisture contents.

CONCLUSION

(a) Results from the present study reveal that it is not feasible to employ shear strength measurements for compaction control of embankments.

(b) The test results indicate that with definite changes in moisture content and dry density it is not possible to predict the change that will occur in shear strength.

(c) The contrast of mean shear strengths for the interaction term, moisture content (M) x dry density (D), shows that, for fixed moisture content, the shear strength does not increase in the same manner when compared for five different levels of moisture content. Also, for fixed dry density, the shear strength does not increase in the same manner when compared for three different levels of dry density. As a result, it would not be possible to take a value obtained in the field and determine whether moisture content, dry density, or both, should be altered in order to bring field value into compliance with some specified value.

(d) A Soil Bore Hole Direct Shear Test Device may be useful to determine the in situ shear strength in the field, but it requires further improvements and more work to establish this.

RECOMMENDATIONS FOR FURTHER STUDY

In the present series of tests for testing the shear strength at different combinations of moisture and dry density, the moisture and the dry density were fixed and the compaction effort was kept variable to obtain the desired combinations. Moreover, in this study the soil was compacted by static compaction. It would be desirable to recheck these results using samples prepared by kneading compaction. This can be achieved in the laboratory by compacting samples at known water content and predetermined compaction effort for varying dry unit weights as suggested by Casagrande and Hirschfeld (30).

It is felt that more work is required to establish the possible use of the soil Bore Hole Direct Shear Device before it can be recommended for the control of field compaction.

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APPENDIX A

Plots of Grain-Size and Moisture - Dry Density Curve

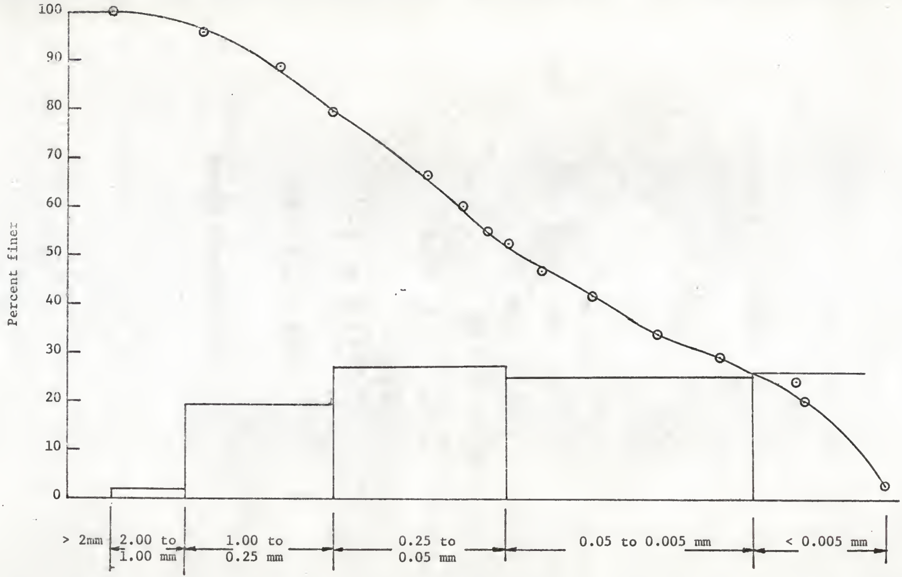


Figure 10. Grain size distribution.

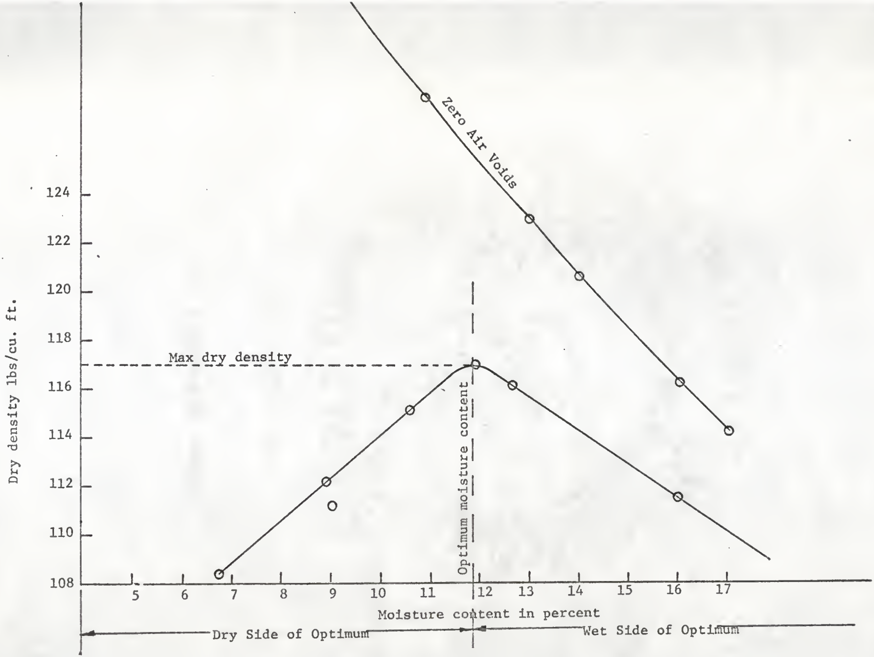


Figure 11. Moisture content - dry density curve.

APPENDIX B

Discussion of Method of Analysis of Test Data

After the experiments have been run it is necessary to determine whether the effect of moisture and dry densities on the shear strength of soil are significant.

The arithmetical discussion by which the experiment is to be interpreted is best accomplished by the "Analysis of Variance", as devised by R. A. Fisher (28). This is a simple arithmetical procedure, by means of which the results may be arranged and presented in a single compact table, which shows both the structure of the experiment and relevant results, in such a way as to facilitate the necessary test of their significance.

As pointed out by Fisher (31) the structure of the experiment is determined during planing and before the results are obtained which consist of actual shear strength of the specimens from different combinations of dry density and moisture content. The structure depends on:

i. the number of comparisons to be made.

ii. the number of replications of each obtainable

iii. the system by which these are arranged.

In this situation, randomized, complete block has been utilized.

The complete analysis of variance actually performs a dual role. In the first place we must sort out and estimate the variance components, and secondly test for significance.

In its arithmetical aspect this structure is specified by the numbers of degree of freedom or independent comparisons, which can be made between the shear strengths. Between 45 shear strengths 44 indep endent comparisons can be made so the total number of degrees of freedom will be 44. This number will be divided into 4 parts representing the number of independent comparisons:

a. Between the moistures.

b. Between the dry densities.

c. Between the interaction between moisture and density.

d. Representing the discrepancies between the relative performance of different moistures in different blocks, which discrepancies provide a basis for estimation of error.

We may specify the structure of our typical experiment by a partition of the total of 44 degrees of freedom into four parts as under.

Source of variation	Degrees of freedom
moistures	4
dry densities	2
interaction between moisture and dry density	8
Error	30
Total	44

The completion of the analysis of variance, when the yields are known, is strictly in accordance with the structure imposed by the experiment. It consists in the partition of a quantity known as the sum of squares (i.e. of deviations from mean) in to the same four parts as these into which the degrees of freedom have been divided.

After the structure has been set up the model and the hypothesis to be proved are set up, Fryer (32).

Model:

$$X_{ijk} = u + T_i + B_j + (TB)_{ij} + E_{ijk}$$

Xijk = The kth observation of shear strength of the soil with moisture i and dry density j.

u = the grand average shear strength of soil for all conceivable such combinations made with any of these specific moisture content combined with any of these specific dry densities.

T_i = the effect on average shear strength of using moisture i, as compared with the grand average u, $T_i = u_{i00} - u$ and T_i are NID $(0,\sigma_T^2)$ Variates

B_j = the average effect of using dry density j relative to the grand average.

$B_j = u_{0j0} - u$ and B_j are NID $(0,\sigma_B^2)$ Variates.

$(TB)_{ij}$ = The additional effect beyond and added to T_i and B_j of combining the ith treatment and jth block.

$(TB)_{ij} = u_{ij0} - u_{i00} - u_{0j0} + u$, $(TB)_{ij}$ are assumed to be NID $(0,\sigma_{TB}^2)$, Variates

Eijk = the random error represented in the particular shear strength from a combination of ith moisture, jth dry density and kth run. E_{ijk} are assumed to be NID $(0,\sigma^2)$, Variates.

The following identity is used to determine the sum of squares:

$$\sum_{i=1}^{5}\sum_{j=1}^{3}\sum_{k=1}^{3}(x_{ijk} - x_{ooo})^2 = \sum_{i=1}^{5}\sum_{j=1}^{3}\sum_{k=1}^{3}$$

$$X(x_{ijk} - x_{ijo})^2 + rq \sum_{j=1}^{5} (x_{ioo} - x_{ooo})^2 + tq \sum_{j=1}^{3}$$

$$X(x_{ojo} - x_{ooo})^2 + q \sum_{i}^{5} \sum_{j}^{3} (x_{ijo} - x_{i00} - x_{ojo} + x_{ooo})^2$$

where t stands for the number of treatments, r stands for the number of replications and q stands for the number of observations, per cell.

Once the sum of squares have been computed the hypotheses to be tested are set up. To test the hypotheses analysis of variances is used as devised by R. A. Fisher (28).

It is assumed in this test that the population variances are equal, before testing for equality of means by analysis of variance.

There are three hypotheses to be tested by means of the analysis of variance,

a) H_{01} $(u_{100} = u_{200} = u_{300} \cdots\cdots = u_{500})/$

all σ_{ij}^2 are equal $\quad i = 1, 2, 3, 4, 5$

$j = 1, 2, 3$

Versus H_a (some u_{i00} are unequal) or referring to the model which is

$x_{ijk} = u + T_i + B_j + (TB)_{ij} + E_{ijk}$

H_o (all $T_i = 0$/all σ_{ij}^2 equal) Versus

H_a (some $T_i \neq 0$)

b) H_{02} (all u_{ojo} are equal)/(all σ^2_{ij} are equal)

Versus

H_a (some u_{ojo} are unequal)

or

H_o (all $B_j = 0/\sigma^2_{ij}$ are equal)

Versus

H_a (some $B_j \neq 0$)

c) H_{03} $(u_{ijo} = u_{ioo} + u_{ojo} - u)/(\sigma^2_{ij}$ are equal)

Versus

H_a $(u_{ijo} \neq u_{i00} + u_{ojo} - u)$

H_o (all $(TB)_{ij} = 0$ / all σ^2_{ij} are equal)

Versus

H_a (some $(TB)_{ij} \neq 0$)

APPENDIX C

A Soil Bore Hole Direct Shear Test Device

As reported by R. L. Handy and Nathaniel S. Fox (33) the test is essentially a direct shear test run on soil at the sides of a bored hole. The hole is bored first, the soils sampled and identified; and critical strata and depth are selected for tests. The method for shearing involves application of a measured pressure normal to an incipient shear plane, and gradual application of a measurable shearing stress to cause failure.

A normal pressure can readily be applied inside a hole by means of an expansion device. The apparatus utilized two diametrically opposed expansion plates grooved to engage soil at the sides of a bored hole, expanded hydraulically by means of two automotive brake cylinders. After expansion for a measured normal stress, the device is pushed or pulled axially by attachment to a wrist pin.

Operation

The test device is lowered on a cable to the shallowest desired testing depth into the hole. A predetermined expansion pressure is applied and maintained constant until the desired amount of consolidation has occurred. The pulling force is applied with a steady rate of strain. Stress vs. displacement may be recorded and corrected for stretching of the cable, but usually only the maximum pulling stress is noted and recorded.

After a maximum shearing stress has been reached the pulling stress is reduced to zero, and the normal stress is increased to a predetermined amount. The soil is again allowed to consolidate, and pulling is repeated to give a new maximum shearing stress corresponding to the second normal stress.

Shearing stress vs. normal stress can be plotted. Although two points theoretically determine C and ϕ of the failure envelope, three or more are preferred.

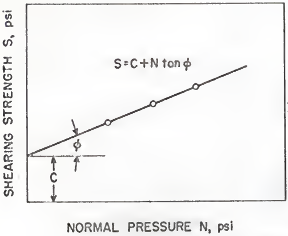


Figure 12. Soil shear strength relationship of Coulomb; φ is the soil angle of internal friction and c is the cohesion.

Figure 13. The bore-hole shear device ready for test.

Figure 14. Conducting a shear test. Expansion pressure is maintained by the pump at the left while a steadily increasing pulling force is applied, done here hydraulically.

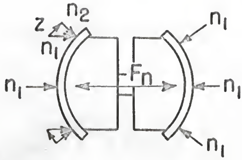
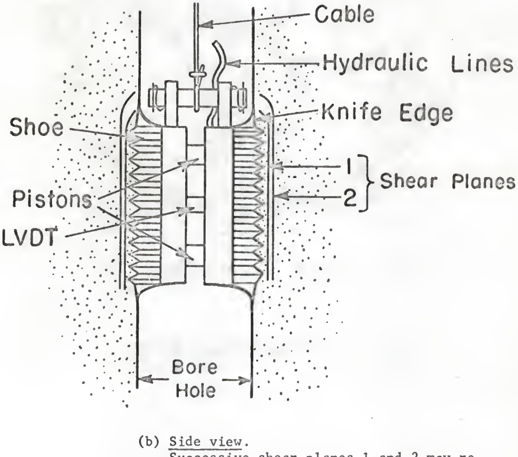


Figure 15. Schematic of soil bore-hole shear device.

(a) Top view.
Reduction of circumferential shearing resistance Z should give more even distribution of normal pressures n_1.



(b) Side view.
Successive shear planes 1 and 2 may relocate outward as soil compacts close to the apparatus.

A STUDY OF THE APPLICATION OF SHEAR STRENGTH
FOR EMBANKMENT CONTROL

by

JAGAN NALT GOSAIN

B. Sc. Engineering (Civil), Panjab University, 1961

AN ABSTRACT OF A MASTER'S THESIS

submitted in partial fulfillment of the

requirements for the degree

MASTER OF SCIENCE

Department of Civil Engineering

KANSAS STATE UNIVERSITY
Manhattan, Kansas

1969

ABSTRACT

For the control of field compaction of earthwork, it is customary to specify it by percent of the maximum dry density and optimum moisture content as obtained by Standard Proctor Soil Compaction test.

The purpose of this study was to investigate the feasibility of using shear strength characteristics of soils for control and to recommend a reliable field test to determine the in situ strength.

The shear strength of the soil was tested for fifteen selected combinations of moisture and dry density. In all, 45 samples were tested.

The results of the study revealed that it was not feasible to employ the shear strength of the soil as the controlling factor in field compaction. Different combinations of moistures and dry densities yield the shear strength which are not significantly different from each other. This indicates that this test is not sensitive enough for field control.

It was further concluded that though the bore hole direct shear device, patented by Soil Technical Associates of Des Moines, Iowa, may be useful to determine the in situ strength of the compacted fill, it will require further work and improvement to prove its value.