FACTORS OF SAFETY TO BE APPLIED IN THE UTILIZATION OF SHEAR TEST DATA

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SYNOPSIS

The important problem is completely and accurately to determine the shearing characteristics of the soil. Careful consideration must be given to the methods of shear testing and to the analysis and interpretation of test results. Recent researches by leading authorities in the field of soil mechanics have given us certain fundamental principles. The ultimate shearing failure in soil is governed by the effective stresses carried entirely by the grain structure of the soil. A substantial part of the total applied stresses may be temporarily carried by hydrostatic stress in the pore water. which does not contribute in any way to mobilizing shearing strength. The proportion of the total applied stresses that is effective depends upon the relation between shearing stress and the volume-change that always accompany shearing deformations. The shearing characteristics are intimately related to consolidation characteristics, and are determined by the initial voids ratios and the load history by which this initial condition has been reached.

The shearing characteristics of certain cohesionless and certain cohesive plastic soils, both in the remolded and undisturbed states, are given in some detail, and are discussed in relation to shear failure phenomena. The results are analyzed as to values to be used in the analysis and solution of practical problems.

A few years ago it was thought that shearing phenomena were relatively simple and fairly well understood. But today we know that they include some of the most controversial and least understood questions of soil mechanics, and as a consequence many conflicting results have been obtained. However recent researches have established certain fundamental principles governing shearing failure.

If we are intelligently to apply a factor of safety, which is something more than a mere guess or a factor of ignorance and if we are to learn how to apply our knowledge correctly to the solution of practical problems, we must give careful consideration to:

1. The nature of shearing phenomena and conditions of failure, and the lessons learned from experience.
2. The methods used in obtaining information on the shearing characteristics of soils.
3. The analysis and interpretation of results of shearing tests.

An analysis of any given situation involves first of all attempting to understand the nature of the problem and trying to reduce the uncertainties to a minimum. Second, it involves making an estimate of the forces and physical factors governing the conditions of failure. And third, it involves attempting to define the limiting conditions in which shearing failure can possibly occur. In order to build up a body of authoritative knowledge on the subject, it is essential to make careful accurate observations in the field and systematically to collect data on the phenomena of shearing failures.

For cohesionless soils the researches of Casagrande (1) [1] (2) (3), Taylor (4), Watson (5), and for cohesive soils, the researches of Terzaghi (6) (7), Hvorslev (8), and Jurgenson (9) have given us fundamental principles governing shearing phenomena.

1. The ultimate shearing failure in soil is governed, not by the total

[1] Numbers in parentheses refer to list of references at end.

external applied stresses, but by the effective stresses carried entirely by the grain structure of the soil.

2. The relation between applied stresses and the volume-changes that accompany shearing deformations must be known in order to make an intelligent estimate of what proportion of the applied stresses may be effective for mobilizing shearing resistance.
3. The behavior characteristics of a soil in shearing depend upon the initial moisture content or voids ratio, the load history of the soil, and the consolidation characteristics of the soil.

We are concerned primarily with the safety of a mass of soil against overstress and failure by plastic flow or by rupture. This requires the determination of the shearing strength of soil under different possible conditions. The behavior characteristics of a mass of soil in shear depend upon the permeability and the compressibility of the material, the size of the mass involved, the voids ratio of the material with respect to a critical voids ratio, the volume-changes which take place during shearing, and whether the shearing failure tends to take place slowly or can take place rapidly. By varying shear test conditions indiscriminantly or by an arbitrary technique one can obtain almost any value of the angle of friction and of cohesion, between zero and its ultimate value. Such values can not be considered as soil constants and their use is dangerous. The most important tasks confronting us today are first, to develop scientifically correct methods of shear testing, which will yield significant values having a direct bearing on our problems, and second, to learn how to apply correctly the results in the solution of practical problems.

Terzaghi (6) and Casagrande (3) have stated certain conditions, which must be satisfied in order to obtain not more than one value for the shearing strength of a given material for each value of either the applied normal pressure or the minor principal stress:

1. All specimens must have the same initial moisture content or voids ratio.
2. This initial condition must be reached by the same loading procedure.
   (a) By increasing the consolidation pressure from zero to its ultimate value in a natural state of consolidation, and shearing the specimen under this pressure.
   (b) By pre-consolidating the specimen to a certain pressure and then unloading it to a certain smaller pressure under which the specimen is sheared—a state of overconsolidation.
   (c) By compacting the material to the same degree by a standard method, and then following methods (a) or (b).
3. The shear test must be run so slowly that total applied stresses are effective stresses, allowing sufficient time for the volume-change adjustments to take place and for full development of effective stresses. That is, excess hydrostatic stresses or neutral stresses in the pore water are kept to a very low value, either by slow continuous loading, or by very small increment shear loadings.

These investigators (3) (6) have emphasized the fact that the analysis of the shearing strength of either a test specimen or of a large mass of soil in nature must be based on a knowledge of effective stresses and on the possible changes of stress conditions with time, because otherwise one can not be sure that applied stresses are fully effective. A substantial part of the applied stress may be temporarily carried by the water in the voids of the soil as a neutral stress, which does not contribute in any way to mobilizing shearing strength.

COHESIONLESS SOILS

In a laboratory test on cohesionless soil in a saturated condition a small mass of soil is involved and water can escape freely from the soil pores with volume decrease or flow into the soil with volume expansion at almost any rate of shearing. Externally applied stresses are therefore effective stresses. While the direct-shear test method is not entirely satisfactory and shows inconsistencies, the results for Ottawa sand in Figure 1 illustrate characteristic differences in the behavior of the material in the loose and dense states. In Figure 1a the ratio of the shearing stress to the normal pressure is plotted against the horizontal shearing deformation. Failure having the nature of plastic flow occurs in the loose state, where the stress gradually increases to a maximum value at a rather large horizontal deformation. Failure by rupture occurs in the dense state where there is a considerable drop in the shearing strength after the maximum has been reached, the maximum being considerably higher than that in the loose state. Taylor (4) has pointed out that while the maximum shearing strength is greatly influenced by the initial voids ratio, the more conservative ultimate is little affected, and approximately equals that of the loose condition.

The second characteristic difference is shown by the volume changes in Figure 1c, which accompany shearing deformations in the dense and loose states at the maximum stress condition. In the loose state the volume decreases and the mass of grains tends to assume a more dense state during shearing. In the dense state there is a slight initial volume decrease after which a marked expansion or loosening up of the dense structure occurs at the maximum stress.

The third characteristic difference is the effect of density on the angle of friction. Failure of masses of loose grains such as soils in shearing depends upon the internal frictional resistance to displacement of the grains that can be developed. In order to determine the angle of friction, it is necessary to establish the relation between the applied normal pressure and the maximum shearing strength of the material as shown in Figure 1b for Ottawa sand. This relation

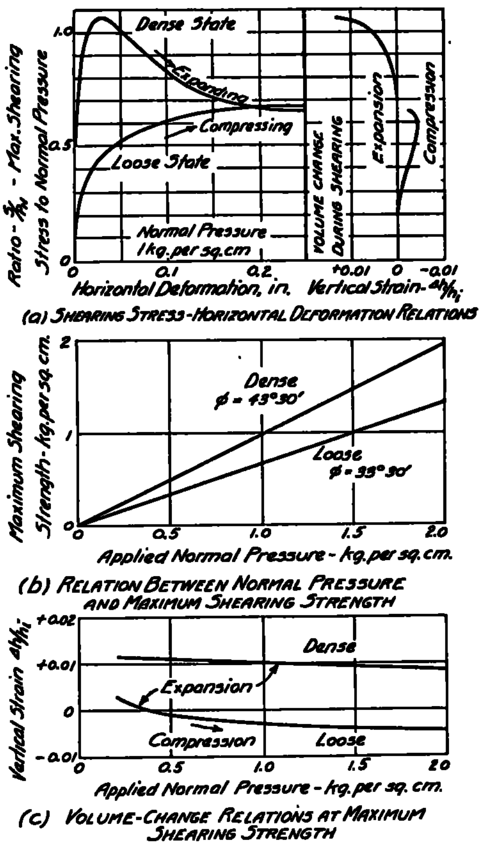


Figure 1. Shearing Characteristics of Standard Ottawa Sand, 20-30 Mesh

may be expressed by Coulomb's simple equation for cohesionless soils—

$$S = p_N \tan \phi \ldots\ldots\ldots\ldots 1$$

where

S is the maximum shearing strength,
p_N is the effective normal pressure,
ϕ is the angle of friction based on effective pressures.

The average values of the angle of friction for Ottawa sand in the loose and dense states obtained by direct-shear tests are as follows:

	Loose	Dense
Angle of friction—ϕ	33° 30′	43° 30′
Voids ratio—e	0.66	0.49

For materials which show a characteristic dropping off of shearing strength after the maximum is reached, design based on the maximum value may be decidedly on the unsafe side, while for designs based on the more conservative ultimate shearing strength, which approximately equals the loose maximum, the shearing strength can not drop below the ultimate, if progressive failure is initiated in some critical zone.

It is important in applying the results to practical problems to consider the volume-change relations at maximum stress as shown in Fig. 1c. In dealing with large masses of cohesionless soils in nature the time required for the volume-change adjustments to take place during shearing is very much longer than usual. Casagrande (1) (2) (3) has demonstrated that under such conditions a large mass of even sandy soil in a loose state may behave temporarily as relatively impermeable and hence may fail by a sudden flow failure as a result of sudden disturbance. A tendency toward volume decrease in the loose state during shearing would induce positive excess hydrostatic or neutral stresses in the pore water and only a part of the weight of the mass of soil would be effective in mobilizing shearing strength, in which case the shearing strength might be suddenly reduced to a very low value. On the other hand a tendency for volume expansion in the dense state during shearing would induce negative stresses, thereby making the effective stresses temporarily greater than the external stresses and giving the soil temporary additional strength during a disturbance.

The critical density has been defined by Casagrande (1) (2) (3) as the boundary region between the voids ratios in which volume decrease and volume expansion occur during shearing. It has been shown that the critical voids ratio is not a constant value, but decreases with increase in the applied stresses. The critical voids ratio is derived from the volume conditions at maximum stress, which, for the critical state is equal to the initial voids ratio.

There are many difficulties in accurately determining the critical density by means of direct-shear tests, because of the secondary effects, and because the stress conditions in the material during shearing are not definitely known. The most satisfactory method so far devised is the tri-axial compression test for cohesionless soils developed by Casagrande (2) (3). However by analyzing the results of the direct-shear tests on Ottawa sand an approximate relation is obtained in Figure 2a, similar to that developed by Casagrande (2) (3), giving the relation between the voids ratio at the start of the test, the volume-changes during the test, and, in this case, the applied normal stress, which is constant for each test. Each point on the line of zero volume-change represents a critical voids ratio, and the corresponding effective value, which the normal stress can assume during a flow failure, if such a condition could somehow develop within a large mass of saturated cohesionless soil as a result of a sudden disturbance.

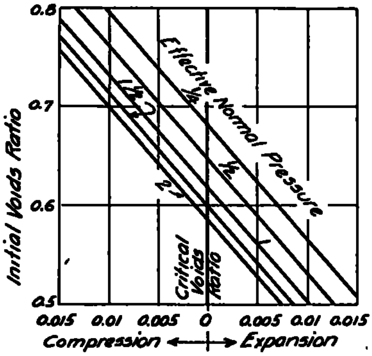
A curve representing the relation between the critical voids ratio and maximum effective shearing strength for the flow failure condition for Ottawa sand is obtained from Fig. 2a by substituting shearing strength from the relation—$S = p_N \tan \phi$ [or $S = (1 + \sin \phi) \tan \phi$, where the minor principal stress is used] as plotted in Fig. 2b. The curve is conveniently graduated for relative density in terms of the limiting loose and dense states.

$$\text{Relative density} = \frac{e_L - e}{e_L - e_D} \times 100 \quad \text{in percent} \ldots . 2.$$

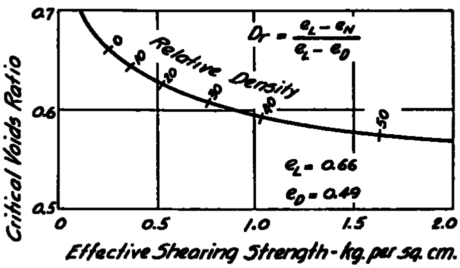
This curve represents the effective values the shearing strength can assume for each voids ratio or relative density for a critical condition during a flow failure. This curve has the appearance and many of the characteristics of the pressure-voids ratio for compression of the material, but is steeper because of the volume-change adjustments during shearing.

These relations are important because the finer-grained the material is, the greater is the probability that it has been deposited above the critical voids ratio. Casagrande (3) points out that nature deposits such materials almost without exception above the critical voids ratio. The failure of such deposits under heavy embankment loads depends on whether a flow failure condition can somehow develop, a sudden disturbance or a large foundation settlement playing the important rôle.

For embankments the Franklin Falls field compaction tests (10) (3) have demonstrated that there is no difficulty in obtaining densities greater than the critical density in cohesionless soils, even in the most heavily loaded portions of embankments of moderate height, by means of compaction equipment now used. Foundation materials of very loose character at Franklin Falls dam have recently been compacted successfully by means of explosives over a large area. The shearing strength of a well compacted mass can not be reduced by a sudden disturbance, but rather the soil mass will tend to have a greater shearing strength under such conditions, which increases the margin of safety against many of the uncertainties, particularly



(a) APPROXIMATE VOLUME-CHANGE RELATION DURING SHEARING



(b) APPROXIMATE RELATION BETWEEN CRITICAL VOIDS RATIO AND EFFECTIVE SHEARING STRENGTH

Figure 2. Approximate Critical Density Relations for Standard Ottawa Sand

as to the true stress conditions in an earth embankment about which little is actually known. On the other hand flattening the slopes, which is the usual method resorted to, does not necessarily mean increased safety for the embankment.

COHESIVE SOILS—REMOLDED

In cohesive clayey soils and in very fine, relatively impermeable cohesionless silts, the problem of determining the shearing characteristics is more complicated. The consolidation characteristics of the material must be known before one can analyze a situation and attempt an estimate of the limits within which the percentage of the total applied stresses that are effective in mobilizing shearing

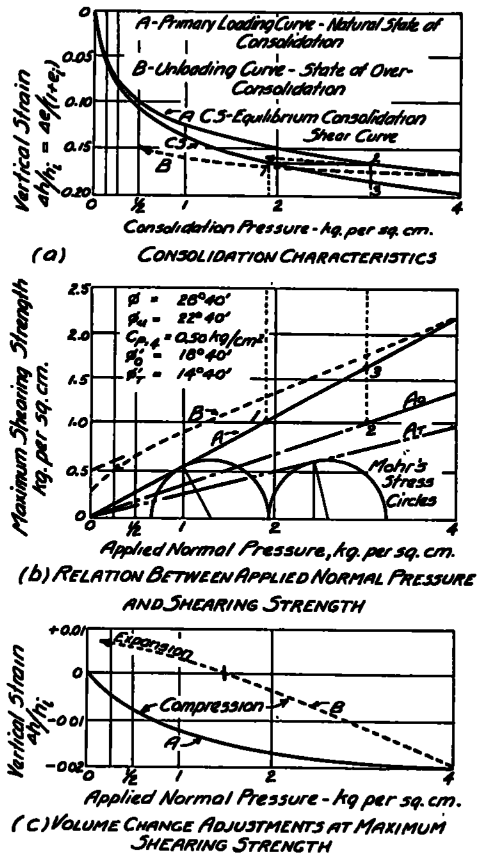


Figure 3. Shearing Characteristics of Remolded Detroit Clay

strength may lie. The permeability of these materials is so low that at ordinary rates of shearing the volume-change adjustments and full development of effective stresses can not take place. Hence a considerable proportion of the total stress may be carried by neutral stress in the pore water.

The results of direct-shear tests on remolded materials, for which conditions can be fairly well controlled, give us some clues on the behavior characteristics in the undisturbed state. The consolidation characteristics of a Detroit clay are shown in Fig. 3a by a typical primary loading curve—*A*, which represents a natural state of consolidation. Specimens of remolded clay at the liquid limit were completely consolidated under a series of normal pressures, which increased to the desired ultimate value for the shearing test with complete consolidation between each load increment, as in the regular consolidation test. It was found that if any other procedure was followed in pre-consolidating the specimens, the curves differed from those in a natural state of consolidation. For the unloading branch—*B*, representing a state of over-consolidation, all specimens in this series were completely consolidated under 4 kg. per sq. cm. and then the pressure was reduced in increments with complete expansion between each to the desired normal pressure for the shearing test.

The relation between the maximum shearing strength and the applied normal pressure, which gives the shearing characteristics of the material, is shown in Fig. 3b. The primary loading curve—*A*, in which the sample is sheared under the same normal pressure to which it was pre-consolidated, is a straight line through the origin. This means that a cohesive plastic remolded soil in a natural state of consolidation starting from the liquid limit, behaves essentially the same as a cohesionless material and therefore Coulomb's simple law expressed by Eq. 1 for cohesionless materials applies. In the slow shear test of about 24 hours duration, curve—*A*, the total applied normal stresses are effective stresses in mobilizing shearing strength, giving an angle of friction of 28° 40′ for the Detroit clay.

The curve of volume-change adjustments at maximum shearing strength in Figure 3c shows that for the primary loading curve, additional consolidation takes place during shearing under all normal pressures. In the slow shear test the volume-change requirements in shear are satisfied and effective stresses are fully developed. In a rapid shear test, excess hydrostatic or neutral stresses would be induced in the pore water by the tendency for consolidation, which could not be dissipated during the shearing test by the escape of water, and therefore applied stresses would not be effective stresses.

By applying the ultimate volume-change adjustments for the slow shear test of Figure 3c to the primary loading branch of the consolidation curve—*A* of Figure 3a, an equilibrium consolidation-shear curve—*CS* is obtained for which applied total normal stresses are effective stresses. At point—*2* on the primary consolidation curve—*A* of Figure 3a the material is initially fully consolidated under the given normal pressure—p_2. When the shearing stress is increased to the maximum value in a slow shear test, further consolidation takes place to point—*3*, which lies on the equilibrium consolidation-shear curve—*CS*. A corresponding point—*3* is found on curve—*A* of Figure 3b for the slow shear test, which gives the maximum shearing strength for the given normal pressure—p_2. This point lies on the curve defining the true angle of friction.

Consider for a moment the conditions in the soil during shearing if no drainage is permitted, that is, if the specimen is sealed. There are difficulties in obtaining the curve for the angle of friction for zero volume-change by the usual quick shear test method, because, if it is run too rapidly, viscous effects make the results too high; and if run too slowly, some consolidation takes place, and the results are again too high. The direct-shear test method also gives results too high as compared with those of the tri-axial compression test, because of differences in the way the material reacts under test conditions. The material was initially consolidated at point—*2* under the normal pressure—p_2. When the shearing stress

TABLE 1

DETROIT CLAY

Sample—p-1

Remolded and tested at the liquid limit.

Liquid limit	30.0
Plasticity index	17.0
Specific gravity	2.70

CONSOLIDATION CHARACTERISTICS

Initial voids ratio at start of test, e_i	0.80
Voids ratio at pressure, p′ of one kg. per sq. cm., e_v	0.57
Compression index, (slope of semi-log curve), c_v	0.059
Coeff. of consolidation, (½ to 1 kg.), c_c	0.015 cm.²/min.
Coeff. of compressibility, (½ to 1 kg.), c_a	0.28 1/kg. per sq. cm.
Coeff. of permeability, (½ to 1 kg.), k	8.0 × 10⁻⁶ cm./min.

SHEARING CHARACTERISTICS

	Angle of friction		Shear strength kg. per sq. cm.	
Primary loading curve—*A*	ϕ	28° 40′		
Unloading curve—*B*	ϕ_u	22° 40′	$p_N = 0$	0.25
Zero drainage, curve—A_0 estimated	ϕ'_0	18° 40′	Cohesion	0.50
Zero drainage, curve—A_T Mohr's stress circles	ϕ'_T	14° 00′		

is increased gradually in a test where drainage is prevented, there is still the tendency for volume decrease, which is now restricted. Therefore excess hydrostatic or neutral stresses are induced in the pore water, which now carry part of the original pre-consolidated pressure—p_2. The normal pressure is no longer effective either as a consolidation pressure or in mobilizing shearing strength. Since the effective pressure on the grain structure of the soil is reduced, the voids ratio tends to adjust itself to the smaller pressure by expanding along the flat expansion curve dotted through point—*2*. But this expansion is also restricted, because the sample is sealed. The tendency does have the effect, however, of reducing the magnitude of the neutral stress in the pore water, and the equilibrium conditions are shifted back somewhat. Under these conditions the material has tended both to expand and consolidate along the horizontal line from point—*2* toward point—*1*, which, at the maximum shearing strength, is possibly located on the equilibrium *CS* curve. If this conclusion is substantially correct, the two points—*1* and *2* would be interrelated in such a way that they also represent the conditions, *1* when effective stresses are considered, and *2* when total stresses are considered under conditions of zero drainage. A corresponding point—*1* is obtained on curve—*A* in Figure 3b from the slow shear test in which the pressure—p_1 was the effective normal pressure throughout the test, as well as at the maximum shearing strength. A point—*2* on a curve—A_0, defined by the intersection of a vertical line through point—*2* of Figure 3a and a horizontal line through point—*1* of Figure 3b, would represent the conditions for zero drainage, in which the pressure—p_1 is the effective stress, and the pressure—p_2 is the total stress. Corresponding points for other horizontal lines in Figure 3a fall approximately on curve—A_0. The relations depend on how accurately the original consolidation curve and the equilibrium consolidation-shear curve—*CS* were determined. If these conclusions are substantially correct, the pressure—p_1, corresponding to pressure—p_2, as defined by a horizontal line cutting the two curves, may be considered to be the *equivalent effective pressure* acting at the time of maximum shearing stress during the test without drainage. The pressure—p_2 is the total pressure and the neutral stress—$u=(p_2-p_1)$. The shearing strength for the two points becomes—

$$S=p_1 \tan \phi = p_2 \tan \phi_0' = (p_2-u) \tan \phi$$

where ϕ is the value of the angle of friction—28° 40′, obtained by the slow shear test, and ϕ_0' is the apparent angle of friction for zero drainage, 18° 40′.

The value of the apparent angle of friction—ϕ'_0, obtained by direct-shear test methods is greater than the theoretical value of 14° 0′ obtained by consideration of total and effective stresses in the tri-axial compression test, which is about half of the true angle of friction. This is due to the essential difference in the way the soil reacts to the conditions which obtain in the direct-shear and the tri-axial compression tests. The Mohr stress circles for the direct-shear test intersect, while in the tri-axial compression test they are theoretically tangent to each other, but actually separated slightly, which is a phenomena resulting from the lateral bulging effects under vertical loading during the test.

It therefore appears that the direct-shear test, for the case of zero drainage, is influenced by inherent secondary effects which does not permit accurate determination of the angle of friction or of effective stresses. But for the slow shear and complete drainage and volume-change adjustments during shearing the angle of friction should be in substantial agreement with the true angle of friction obtained by the tri-axial compression test.

Theoretical considerations of the tri-axial compression test, however provide a means of estimating the approximate percentage of the total applied stresses, which are effective in mobilizing shearing strength for the extreme case of zero drainage as a limiting condition, as illustrated by the Mohr stress circles in Fig. 3b. Such a condition is almost sure to obtain in a large mass of very impermeable clay, where drainage and consolidation under a superimposed load takes place with extreme slowness.

Considering the behavior characteristics of the material in a state of overconsolidation, the unloading curve—*B* in Figure 3b for shearing strength *vs.* normal pressure is characteristic just as the unloading or expansion curve—*B* of Figure 3a is characteristic for consolidation, representing a part of the hysteresis loop of the loading cycle. Curve—*B* is considerably flatter than the primary loading branch, but is linear over much of its length, giving an angle of friction ϕ_u for the state of overconsolidation of 22° 40′. Under the lighter normal pressures it curves downward and intersects the shearing strength axis at a positive value for p_N equal to zero of 0.5. It has been the usual practice to consider the intercept for the linear portion of the curve produced to intersection with the vertical axis as the cohesion. The particular value of the cohesion obtained depends first of all upon the maximum pressure to which the material has been pre-consolidated, and second on the moisture content at the particular stage of unloading as defined by the expansion curve for the consolidation test. This so-called cohesion represents a residual shearing resistance that remains from the maximum direct loading. It is apparent that cohesion is merely a phenomena of the hysteresis effect of a loading and unloading cycle. The values of cohesion and angle of friction so obtained are not soil constants, but vary widely with the method of testing and the consolidation characteristics of the material. This is one of the most controversial and least understood matters in soil mechanics.

An examination of the volume-change relations in Figure 3c for maximum shearing strength show that in a state of overconsolidation the compression during shearing decreases with a decrease in the normal pressure until a critical condition or voids ratio is reached at which the volume-change is zero. For still lighter normal pressures a considerable volume expansion or swelling occurs, as might be expected from unloading phenomena. In a slow shear test these volume-changes have time to take place and the sample absorbs water, where expansion occurs. But in a rapid test negative hydrostatic stresses are induced, and tension in the water now contributes to the shearing strength of the soil by temporarily increasing the effective stresses above the applied stresses. Hence the value of cohesion obtained by means of a slow shear test represents the ultimate value that can be counted upon, a value which is considerably smaller than that for the rapid test.

In rapid shearing the unloading curve —*B* for the slow shear test is in effect rotated about the point of zero volume change so that for pressures less than the critical value the rapid shear test gives values which are too high, and for greater pressures gives values which are too low. Thus the cohesion is higher and the angle of friction is lower than the values for the slow shear test. Only for the unloading curve obtained by a slow shear test are the conditions fully known so that Coulomb's more general law can be applied—

$$S = C_p + p_N \tan \phi_u \text{........} 3.$$

where C_p is the cohesion obtained for a maximum overconsolidation pressure—*p*, and is defined by the linear portion of the unloading curve produced to intersection with the shearing stress axis; and ϕ_u is the angle of friction obtained for the unloading cycle. The value of the cohesion depends upon the maximum pressure to which the sample has been consolidated, the overconsolidation pressure and the moisture content, as deter-

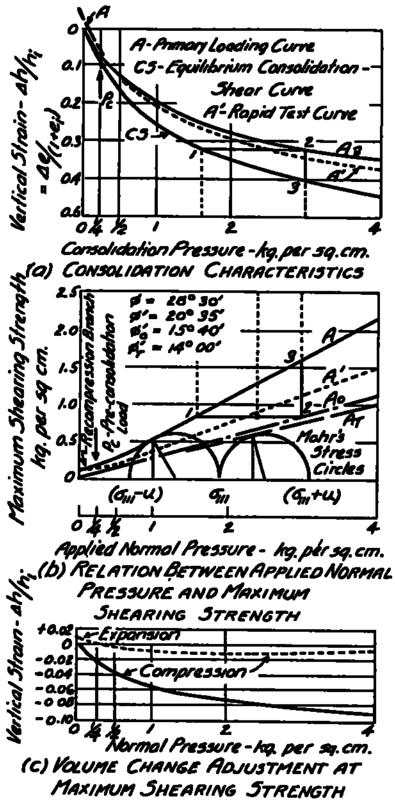


Figure 4. Shearing Characteristics of Flushing Meadow Silt-Clay

mined by the expansion characteristics of the material. The angle of friction for the unloading cycle appears to be independent of the overconsolidation pressure, but does depend upon the consolidation and expansion characteristics of the material.

UNDISTURBED COHESIVE SOILS—NATURAL STATE OF CONSOLIDATION

The consolidation and shearing characteristics of a typical sample of the Flushing Meadows, Long Island, soils are given in Figure 4. The material in the ground was in a natural state of consolidation under the weight of the overburden. In order to utilize the results of the tests we must separate the curves of Figure 4 into two parts. From the loading cycle relations for remolded materials in Figure 3 we recognize the first portion of the curves of consolidation and shear to be the hysteresis loops of the reloading cycle. This is a consequence of the removal of the sample from the ground and the reduction of the natural stresses, the pre-consolidation load as defined by Casagrande (11), to zero, with the accompanying swelling of the material. This reloading cycle does not represent actual condition in the ground, but merely the test phenomena. For the second part of the curves we observe that after the pre-consolidation load has been reached the curve becomes tangent to and practically coincides with the primary loading branch of the soil. The curve above the pre-consolidation load now represents approximately the conditions which actually obtain in the ground. It is characteristic for the primary loading branch—A of the curve in Figure 4b for a natural state of consolidation to be a straight line passing through the origin. Again Coulomb's simple law for cohesionless materials as expressed by Equation 1 applies. It is evident that only for pressures equal to or greater than the pre-consolidation load are test conditions representative of natural conditions or of subsequent conditions as the material is subjected to foundation or embankment loads.

The volume-change adjustments during shearing are given in Figure 4c. Applying these to the primary consolidation curve—A of Figure 4a, the equilibrium consolidation-shear curve—CS is obtained for the slow shear test. Curve—A' represents the conditions for the usual rapid shear test in which applied stresses are not effective stresses.

Applying the notions of an equivalent effective pressure, the horizontal lines in Figure 4a and 4b define corresponding points—*1* and *2*, which indicate that for the direct-shear test the applied stresses were only $p_1/p_2 \times 100$ or 53 per cent effective in mobilizing shearing strength in consolidation load. The results are therefore on the safe side and the quick shear test or unconfined compression test could be used as rapid routine tests to obtain information on the shearing strength of the material for filling and grading operations in the development of the site

TABLE 2

FLUSHING MEADOWS SILT-CLAY

Samples—9-4 and 9-5, Bridge C, Horace Harding Boulevard. Flushing Meadows, L. I.

Liquid limit	100 and 93
Plasticity index	44 and 42
Specific gravity	2.73

CONSOLIDATION CHARACTERISTICS

Depth of samples	42 ft. and 52 ft.
Overburden pressure	0.55 and 0.60 kg. per sq. cm.
Pre-consolidation load, p_c	0.25 kg. per sq. cm.
Voids ratio at pre-consolidation load, e_c	2.96
Initial voids ratio at start of test, e_i	3.00
Voids ratio at pressure, p' of one kg. per sq. cm., e_v	2.17
Compression index (slope of semi-log curve), c_v	0.36
Coeff. of consolidation, ($\frac{1}{4}$ to $\frac{1}{2}$ kg.), c_c	0.0018 cm.^{2} per min.
Coeff. of compressibility, ($\frac{1}{4}$ to $\frac{1}{2}$ kg.), c_a	1.44 1/kg. per cm.^{2}
Coeff. of permeability, ($\frac{1}{4}$ to $\frac{1}{2}$ kg.), k	8.8×10^{-6} cm. per min.

SHEARING CHARACTERISTICS

	Slow shear test		Rapid shear test	
	Angle of friction ϕ	Shear strength kg. per cm.^{2}	Angle of friction ϕ'	Shear strength kg. per cm.^{2}
Primary loading curve—A	28° 30′	....	A' 20° 35′	....
Estimated percentage effectiveness of p_N	100%	....	67%	....
Shear strength, $p_N = 0$	......	0.098	....	0.092
Shear strength at pre-consolidation load	......	0.15	....	0.12
Zero drainage, curve—A_0 estimated	15° 40′		percentage effectiveness of p_N	53
Zero drainage, curve—A_T Mohr's stress circles	14° 00′			48.5

the rapid shear test. The theoretical percentage of effectiveness for zero drainage, obtained from Mohr's stress circles is 48.

The consolidation and shearing characteristics of samples 9-4 and 9-5 are given in Table 2.

For these soils the "quick" shear test under zero normal pressure or a simple unconfined compression test will provide an estimate of the shearing strength of the material which is somewhat lower than the shearing strength at the pre- for the World's Fair. However for stable slopes and control of deeper subsurface lateral movements in the deposit, and for estimating lateral pressures, etc., the shearing strength depends upon the effective stresses acting in the soil. Because of the very slow rate of consolidation of the material and of the thickness of the deposit, very little increase in shearing strength can be counted upon at deeper levels due to the filling and grading of the Meadows. Therefore the increased pressures are not effective in mobilizing shearing strength. The conditions are approximately given by the Mohr stress circles in Figure 4b, and the original overburden pressure is only approximately 48 percent effective.

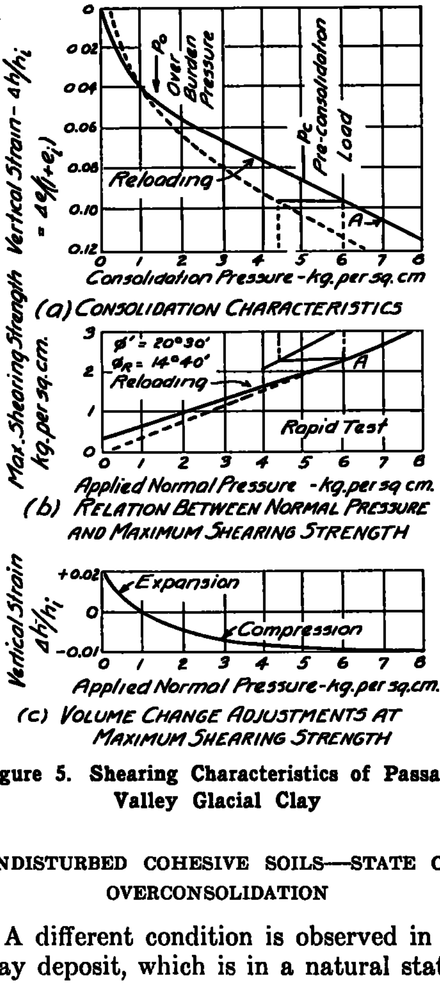


Figure 5. Shearing Characteristics of Passaic Valley Glacial Clay

UNDISTURBED COHESIVE SOILS—STATE OF OVERCONSOLIDATION

A different condition is observed in a clay deposit, which is in a natural state of overconsolidaiton. Quite frequently a clay deposit has been subjected to a greater overburden pressure sometime in its geologic history than now exists, for example when a part of the overburden has been removed by erosion. Such a condition is illustrated in Figure 5 for a glacial clay from the Passaic River Valley, N. J. The consolidation and shearing characteristics are given in Table 3.

In this case we are operating on a reloading branch of the curve—R, and conditions approximate those in the natural state for pressures equal to the overburden pressure and for pressures greater than the overburden pressure due to subsequent loadings from proposed structures. In this case the shearing strength can be expressed by Equation 2—$S=0.28+p_N \tan 14° 40'$, where p_N represents the total normal stress in this case for a fairly rapid shearing test. This relation is only applicable, however, for pressures greater than the overburden pressure and less than the pre-consolidation load, and further provided that the percentage effectiveness of the applied stresses are the same; which means that very little is known about the shearing strength in this case, because only a rapid shearing test was made.

ARTIFICIALLY COMPACTED COHESIVE SOILS

The consolidation and shearing characteristics of a cohesive soil, compacted by the standard Proctor method of compaction are given in Table 4 and shown in Figure 6. For pressures less than one kilogram per sq. cm. the material tends to expand during shearing and hence in a rapid test the effective pressures and the shearing strength are temporarily increased. For pressures greater than this the material tends to compress during shearing and part of the applied normal stresses are not effective but carried by neutral stress in the pore water. An artificially compacted soil is subjected to internal capillary pressures. These internal capillary pressures and the original equivalent compacting pressure produce a state similar to that of overconsolidation under an equivalent compacting maximum pressure. In an earth dam the portion of the initial shearing strength contributed by the internal capillary pressures will ultimately be reduced to zero by complete saturation when the entrapped air passes into solution. There-

TABLE 3

Glacial Clay, Passaic Valley, N. J.

Sample—5-b. Varved clay with silt partings.

Liquid limit	53.8
Plasticity index	24.8
Specific gravity	2.70

CONSOLIDATION CHARACTERISTICS

Depth of sample	40 ft.
Overburden pressure	1.40 kg. per sq. cm.
Voids ratio at overburden pressure	1.125
Pre-consolidation load, p_c	5.0 kg. per sq. cm.
Voids ratio at pre-consolidation load, e_o	1.05
Initial voids ratio at start of test, e_i	1.20
Voids ratio at pressure, p' of one kg. per sq. cm., e_v	1.32
Compression index (slope of semi-log curve), c_v	0.17
Coeff. of consolidation, (1 to 2 kg.), c_c	0.045 cm.^{2} per min.
Coeff. of compressibility, (1 to 2 kg.), c_a	0.036 1/kg. per cm.^{2}
Coeff. of permeability, (1 to 2 kg.), k	3.6×10^{-6} cm. per min.

SHEARING CHARACTERISTICS

Rapid test.

Primary loading curve, produced—A	ϕ' 20° 30′
Reloading curve, R	ϕ_R' 14° 40′
Shear strength, $p_N = 0$	0.28 kg. per cm.^{2}
Shear strength at overburden	0.75 kg. per cm.^{2}

TABLE 4

Cohesive Embankment Material

Sample—Test pit sample, No. 8. Passaic River Valley, N. J.

Liquid limit	20.8
Plasticity index	17.8
Specific gravity	2.70

Maximum density, Proctor compaction test	125.6 lb. per cu. ft.
Optimum moisture content, w_o	11.6 percent.

Samples tested at 110 percent of optimum and compacted to standard degree.

CONSOLIDATION CHARACTERISTICS

Initial voids ratio, at start of test, e_i	0.343
Voids ratio at pressure, p' of one kg. per sq. cm., e_v	0.308
Compression index, (slope of semi-log curve), c_v	0.014
Coeff. of consolidation, (½ to 1 kg.), c_c	0.25 cm.^{2} per min.
Coeff. of compressibility, (½ to 1 kg.), c_a	0.022 1/kg. per cm.^{2}
Coeff. of permeability, (½ to 1 kg.), k	7.0×10^{-6} cm. per min.

SHEARING CHARACTERISTICS

Slow shear test.

Angle of friction	39° 30′
Cohesion	0.55 kg. per sq. cm.
Shear strength at p_N	0.40 kg. per sq. cm.

fore it is on the unsafe side to count upon the cohesion supplied by internal capillary pressures as permanently contributing to the stability of the structure. Casagrande (3) has emphasized that in stability analyses of earth dams only the

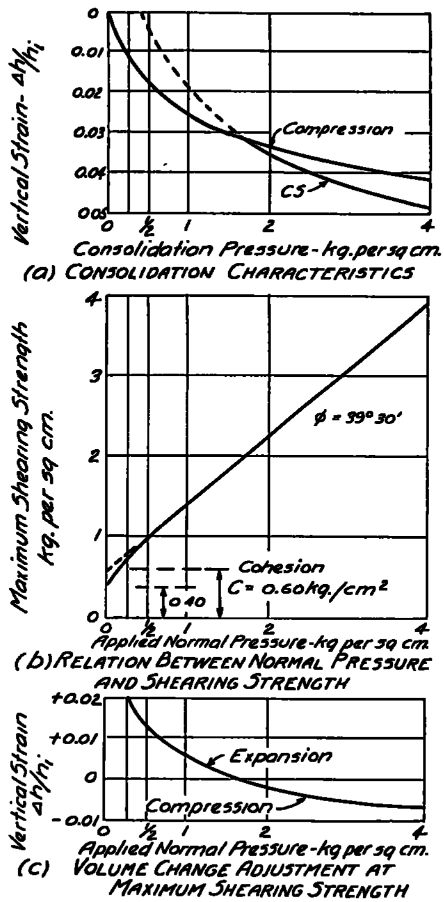


Figure 6. Shearing Characteristics of Compacted Cohesive Soil

shearing strength which can be counted upon for certainty after a number of years should be used, namely the shearing strength mobilized by permanently effective stresses due to the weight of the earth in the structure, which will allow the true cohesion existing after a number of years to provide an additional factor of safety against some of the uncertainties inherent in design. This means an attempt must be made to estimate the percentages of the total pressures due to the weight of the material itself, that are effective in mobilizing shearing resistance for certain most dangerous conditions.

1. Upstream and downstream slopes. During and just subsequent to completion of the structure before the interior portions have reached complete consolidation and equilibrium under the weight of the overlying earth.
2. The upstream slope during a rapid drawdown.
3. The downstream slope under seepage forces.
4. The foundations, when the most dangerous sliding curve passes into a weaker underlying layer of material, and this material has not completely consolidated and come to equilibrium under the weight of the structure.

The greatest uncertainties at the present time in the design of earth structures are due to the lack of accurate knowledge, on the true stress distribution in the embankment itself and how these stresses change with time; on the stresses induced in the foundations, and how they are affected by the subsurface conditions; and how the embankment and the foundations adjust themselves to these stresses by settlements and lateral movements.

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