AN ABSTRACT OF THE THESIS OF

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Title: THE EVALUATION OF THE FIELD VANE SHEAR STRENGTH ANALYSIS OF THE GLEN AIKEN CREEK EMBANKMENT FAILURE

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A post-failure investigation of a highway embankment constructed over organic and sensitive subsoils was made with a series of field vane tests. The results indicated that the embankment was quite stable, and, therefore, the reliabilities of both the $\phi = 0$ slip-circle analysis and the field vane results seemed to be highly questionable. Undisturbed samples taken from an area unaffected by the embankment failure were used to determine the undrained shear strength of these subsoils. The results of both the laboratory vane and unconfined compression tests were appreciably below the results of the field vane tests, but they were not low enough to completely explain the instability of the embankment. It was concluded that a progressive failure of the highly sensitive organic silty clay was partly responsible for the inaccuracy of the $\phi = 0$ slip-circle stability analysis. However, the major error resulted from the inaccuracies of the field vane results. The effect of the rate of

rotation of the laboratory vane upon the undrained shear strength was found to be insignificant. Since the vane basically determines the shear strength developed on the vertical plane, the difference between the shear strength developed on the horizontal plane and vertical plane was studied. Finally, the method of evaluating rod-friction was investigated. It was found that the use of the "dummy" rod for the evaluation of the frictional resistance developed on the field vane's torque rod was inaccurate. This fact was demonstrated by the comparison of original field vane test results with results obtained from a modified slip-joint field vane, capable of directly measuring the rod-friction. It was concluded that a modified slip-joint field vane accurately measured the _in situ_ undrained shear strength.

The fact that silty clays found at the highway embankment site were sensitive was not indicated by either the original field vane or the modified slip-joint field vane. The actual sensitivity of these subsoils was determined by both the laboratory vane and the liquidity index of the soils. It was concluded that the _in situ_ sensitivity could not be obtained by the modified slip-joint field vane.

The one point that the entire investigation indicated was that before the field vane test results can be intelligently used, additional information about the subsoils is required.

The Evaluation of the Field Vane Shear Strength Analysis of the Glen Aiken Creek Embankment Failure

by

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Term	Description	Dimensions
A_f	Bishop's (1954) pore pressure coefficient at failure.	dimension-less
c	Apparent undrained cohesion.	lb/ft^2
C_V	Coefficient of consolidation	$in.^2/min$
D	Diameter of the failure cylinder that is formed by the vane.	in.
e	Void ratio	dimension-less
F. S.	Factor of safety	dimension-less
H	Height of the failure cylinder that is formed by the vane	in.
p	Overburden pressure	lb/ft^2
S	Shear strength	lb/ft^2
S_{ave}	Average shear strength	lb/ft^2
S_H	Shear strength developed on the horizontal plane.	lb/ft^2
S_V	Shear strength developed on the vertical plane.	lb/ft^2
T	Torque	ft-lb
ϕ	Apparent angle of internal friction (undrained).	degrees
σ'_1	Major principal stress	lb/ft^2
σ'_3	Minor principal stress	lb/ft^2
ϵ	Strain	%
γ	Unit weight	lb/ft^3
γ'	Effective unit weight	lb/ft^3

THE EVALUATION OF THE FIELD VANE SHEAR STRENGTH ANALYSIS OF THE GLEN AIKEN CREEK EMBANKMENT FAILURE

INTRODUCTION

In June of 1966 near Glen Aiken Creek, Oregon, a 350 foot section of a 22 foot high embankment on the Coos Bay-Roseburg highway slipped on soft organic subsoils. Although this failure did not constitute a major disaster, the Glen Aiken Creek embankment failure is interesting because of the results of the post-failure investigation. The first portion of this investigation was done with a field vane. The field vane determined the distribution of the undrained shear strength with depth at four points adjacent to the failure site. These test results indicated that the 22 foot high embankment had a factor of safety of 2.6. The investigation was complicated by the fact that construction of the embankment had been halted by poor weather and working conditions when the fill height was only ten feet. Further construction was not possible for eight months. Therefore, the subsoils were given an opportunity to consolidate under approximately half the final fill height for a period of eight months before failure occurred under the 22 foot embankment. Since the field vane tests were performed in an area unaffected by any consolidation due to the embankment load, the factor of safety of the material beneath the embankment could have been even higher than 2.6. The reliability of the undrained shear strength results of the field vane test seemed to be highly questionable.

Obviously, the field vane was inaccurately representing the shear strength of the subsoils, and, therefore, further investigation of this embankment failure was required. Undisturbed samples were taken from an area adjacent to the embankment failure site for this purpose. These samples were used to investigate the affect of the rate of rotation on the results of the vane test, the shear strength representation of the vane test, the actual undrained shear strength of the samples, and the consolidated undrained shear strength parameters of the subsoils. The embankment failure was, also, analyzed from the records of a settlement station located near the failure site.

LITERATURE REVIEW

The Royal Swedish Geotechnical Institute (Carlson, 1948) is credited with the development of the first field vane apparatus. The first practical application of the field vane was to determine the ability with which different subsoils could withstand tank loadings for the British Army. The first use of the field vane test took advantage of two of its major virtues, namely, the short time required to obtain results and the low cost of obtaining those results. The third virtue of this test is that the sample is brought to failure in its _in situ_ state with very little disturbance.

The fact that the field vane causes only slight disturbance to the element of soil which is to be failed in its _in situ_ conditions was first appreciated by both Carlson (1948) and Skempton (1948). They point out that the removal of the sample from its in situ condition results in some disturbance of the sample, and they both referred to Odenstand's (1948) paper. Odenstand attributed the reduction of the indicated undrained shear strength determined in the laboratory from that which actually exists in the field to the reduction of the "average grain pressure" during the sampling process. This same effect has subsequently been explained by failure of the saturated sample to develop a negative pore pressure equal to the _in situ_ confining stress (Ladd and Lambe, 1963). The low negative pore pressure is due to the change in the directions of the major principal stresses, the disturbances of sampling and trimming, and the time and manner of storage. Undrained shear strength differences between samples obtained by carefully trimming three inch Shelby tube samples and samples obtained by carefully trimming block samples have been found to be as high as 50 to 200 percent of the values determined from the three inch Shelby tube samples (Coates and McRostie, 1963). The negligible amount of disturbance which the insertion of the field vane causes is particularly advantageous for the determination of the undrained shear strength of sensitive soils (Carlson, 1948; Skempton, 1948; Osterberg, 1956; Eden and Hamilton, 1956).

There are numerous variations in the field vane designs, but basically they consist of a four winged vane attached to a torque rod (Figure 1). The torque rod is coupled to some device by which the torque can be applied. Originally, this device was either a spring balance (Carlson, 1948) or a torque wrench, but these methods have been abandoned. The present method (Figure 1) involves the connection of the torque rod to a torque ring. The torque ring is calibrated in terms of the deflection of the connecting steel blade, which is measured with a strain gauge. The torque rod is also connected to a large circular housing with a second steel blade. This circular houeing is divided into 5° increments, which are used to control the rate of rotation. The circular housing is then connected to a small

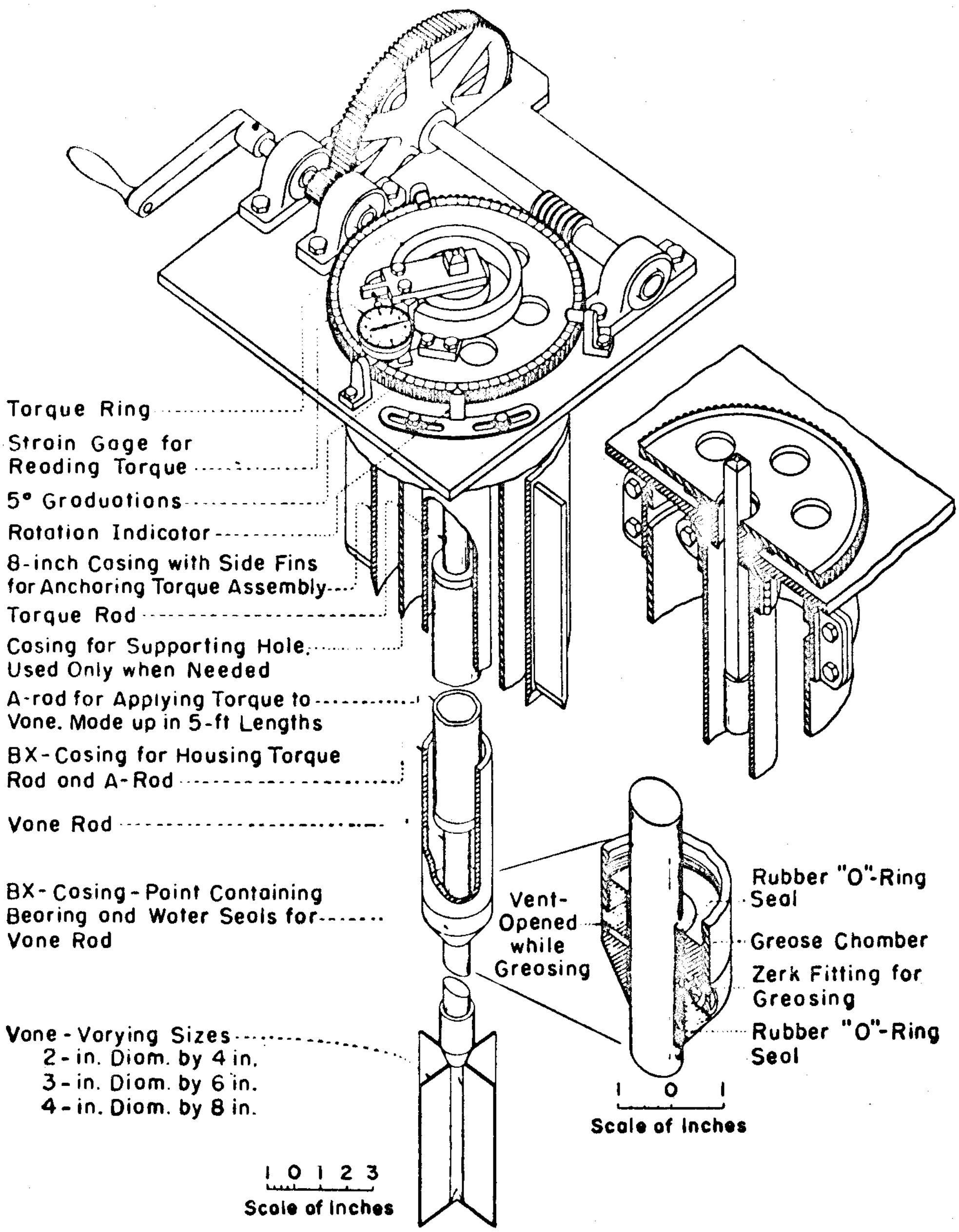
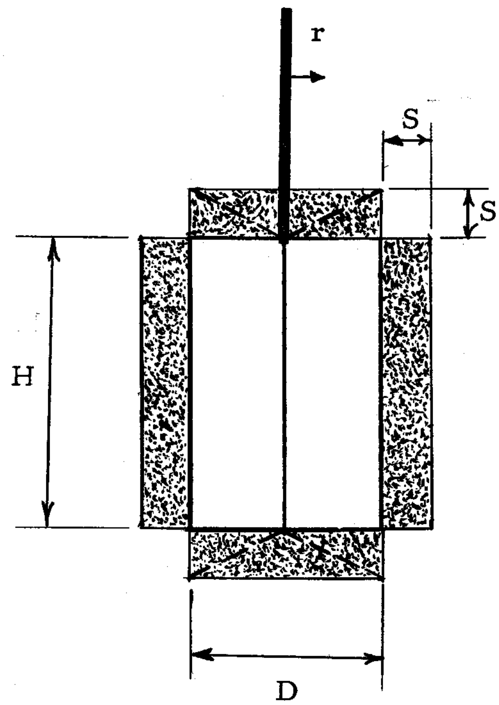


Figure 1. The field vane apparatus (Gibbs, 1956).

crank with a worm gear. This mechanism allows for a more accurate control of the rate of rotation than can be obtained with either the spring balance or the torque wrench while it simplifies the measurement of the torque. The torque applying apparatus is normally mounted on a plate which can be anchored either to the upper portion of the boring casing or to a platform which is in turn anchored to the soil with stakes. Regardless of the particular method used, the application of the torque is continued at a constant rate of rotation until a maximum torque is reached. This torque is then related to the shear strength of the soil by Equation 1. This equation assumes that the vane is rectangular in shape and that the full ultimate shear strength is developed on all portions of the cylindrical failure surface at failure as indicated by the shaded areas of Figure 2. Skempton (1948) pointed out that on the ends of the failure cylinder the shear resistance would not be fully mobilized at failure because of the difference in the amount of strain applied to an element at the tip of the blade and an element at the center of the vane. However, even if the shear resistance is assumed to vary linearly from zero at the center of the vane to a maximum at the outside edge of the vane as shown with the dashed lines in Figure 2, the difference between the resulting shear strength and the one calculated from Equation 1 for a ratio of the vane height to diameter of two amounts to only 3.6 percent.



$$dT = (S) r \, dA$$

$$T = (S)\left(\frac{D}{2}\right)(\pi D H) + 2(S) \int_0^{D/2} (r)(2\pi r dr)$$

$$T = \frac{S\pi D^2 H}{2} + 2(S)\left[\frac{r^3}{3}\right]_0^{D/2}$$

$$T = \left[\frac{\pi}{2} D^2 H + \frac{\pi}{6} D^3\right](S) \tag{1}$$

Figure 2. The assumed distribution of shear resistance at failure and derivation of torque strength relationship.

The rate of rotation of the vane has been shown to affect the maximum torque obtained (Carlson, 1948), but only at very high rates of rotation. A rate of rotation of 6°/min was chosen by both Carlson and Skempton (1948) as being representative of the normal laboratory undrained failure conditions where failure is reached in three to ten minutes. Carlson also pointed out that at rates lower than 6°/min the resulting maximum torques were practically the same as the one found for 6°/min. Housel (1956) has criticized both the vane test results and their comparison with unconfined compression test results because the strain rate is inadequately defined. He suggests (as did Carlson) that the only valid undrained shear strength produced by the vane test would be the shear strength which is obtained by extrapolating back to a rate of zero deformation. Housel based his criticism on his experience with an experiment involving the induced consolidation of a stratified deposit of lacustrine clay by electro-osmosis. As the consolidation took place, samples were taken for shear strength determinations and field vane tests were performed. The vane test did not reveal any "significant" changes in shear strength, but the unconfined compression tests did show a shear strength increase. Housel further explained that since the soils in question were of a stratified deposit, the increase in shear strength was "revealed most clearly by transverse shear in the ring shear test." Therefore, the shear strength of stratified clay was basically being increased on the horizontal plane, while the vane measures basically the shear strength on the vertical plane. (This fact is proven later in this section.) Although this is a valid case against the use of the field vane without additional shear strength tests, the author feels that the reason given (the indefinite rate of loading) is unfounded. It may be of some interest to note that Andresen and Sollie (1965) used the field vane to indicate, quite adequately, the effect electro-osmosis had on the undrained shear strength of a very soft quick clay.

The major shortcoming of the field vane is the fact that it forces the main resistance to failure to occur on a vertical plane. This fact can be easily proved for the ordinary vane with a vane height to diameter ratio of two. Since Equation 1 was obtained by adding the torque developed on the horizontal and vertical surfaces of the cylindrical section failed by the vane, Equation 1 can be re-written in the form of Equation 2.

$$T = [\frac{\pi}{2} D^2 H + \frac{\pi}{6} D^3]\, S \tag{1}$$

$$T = [\frac{\pi}{2} D^2 H]\, S_V + [\frac{\pi D^3}{6}]\, S_H \tag{2}$$

for $H/D = 2$

$$T = \pi D^3 \, [S_V + \frac{1}{6} S_H] \tag{2a}$$

S_H = shear strength on the horizontal plane (lb/ft^2)

S_V = shear strength on the vertical plane (lb/ft^2)

Equation 2a shows that the vertical shear strength is given six times the weight of the horizontal shear strength in the calculation of the apparent shear strength. This multiplication factor would be increased to a value of eight if the horizontal shear resistance was assumed to be developed linearly from zero at the center of the vane to S_H at the outside edge of the vane. Clearly then, the evaluation of the field vane shear strength determination involves the consideration of the magnitude and type of anisotropic properties of a soil (Osterberg, 1956). Aas (1965) conducted an investigation of the ratio of the undrained shear strength on the horizontal plane to the undrained shear strength on the vertical plane, which he defined as the anisotropy ratio. He used seven vanes with different height to diameter ratios to determine the anisotropy ratio. The anisotropy ratio of a slightly overconsolidated clay proved to be approximately 1. 1 while the anisotropy ratios of three normally consolidated clays varied from 1. 5 to 2. 0. Aas also noted that the measured anisotropy ratio is comparable with the probable ratio of *in situ* vertical to horizontal effective stresses. Thus, the vertical shear strength is usually the critical (minimum) shear strength to be determined in homogeneous normally consolidated and slightly overconsolidated soil, but in highly overconsolidated soils and many stratified soils the horizontal shear strength can be the critical shear strength.

The determination of the sensitivity of a soil with the field vane apparatus was originally demonstrated by Skempton (1948). Sensitivity is defined as the ratio of the undisturbed shear resistance to the shear resistance of the soil remolded at constant water content. Skempton obtained the same remolded shear strength from both the field vane tests and the laboratory unconfined compression tests. He, therefore, concluded that the field vane test provided an efficient method of obtaining the sensitivity of a soil. Fenske (1956) also concluded that the field vane test gave a good evaluation of the remolded _in situ_ shear strength. While Eden and Hamilton (1956) found that the field vane tests were not a reliable indication of the sensitivity, they found that the laboratory vane results compared quite well with Bjerrum's (1954) correlation of the sensitivity and the liquidity index (Figure 3). The major difference between the investigations was the type of vane used. Both Skempton (1948) and Fenske (1956) used the Swedish vane, which has protective casing for both the vane and the torque rods. This apparatus reduces the rod friction to a negligible amount during the vane shear test. Eden and Hamilton (1956) used an unprotected vane for a depth, which varied from one foot to ten feet, before the next section of casing was placed. They assumed that the rod friction of the extra sensitive Leda clay was insignificant.

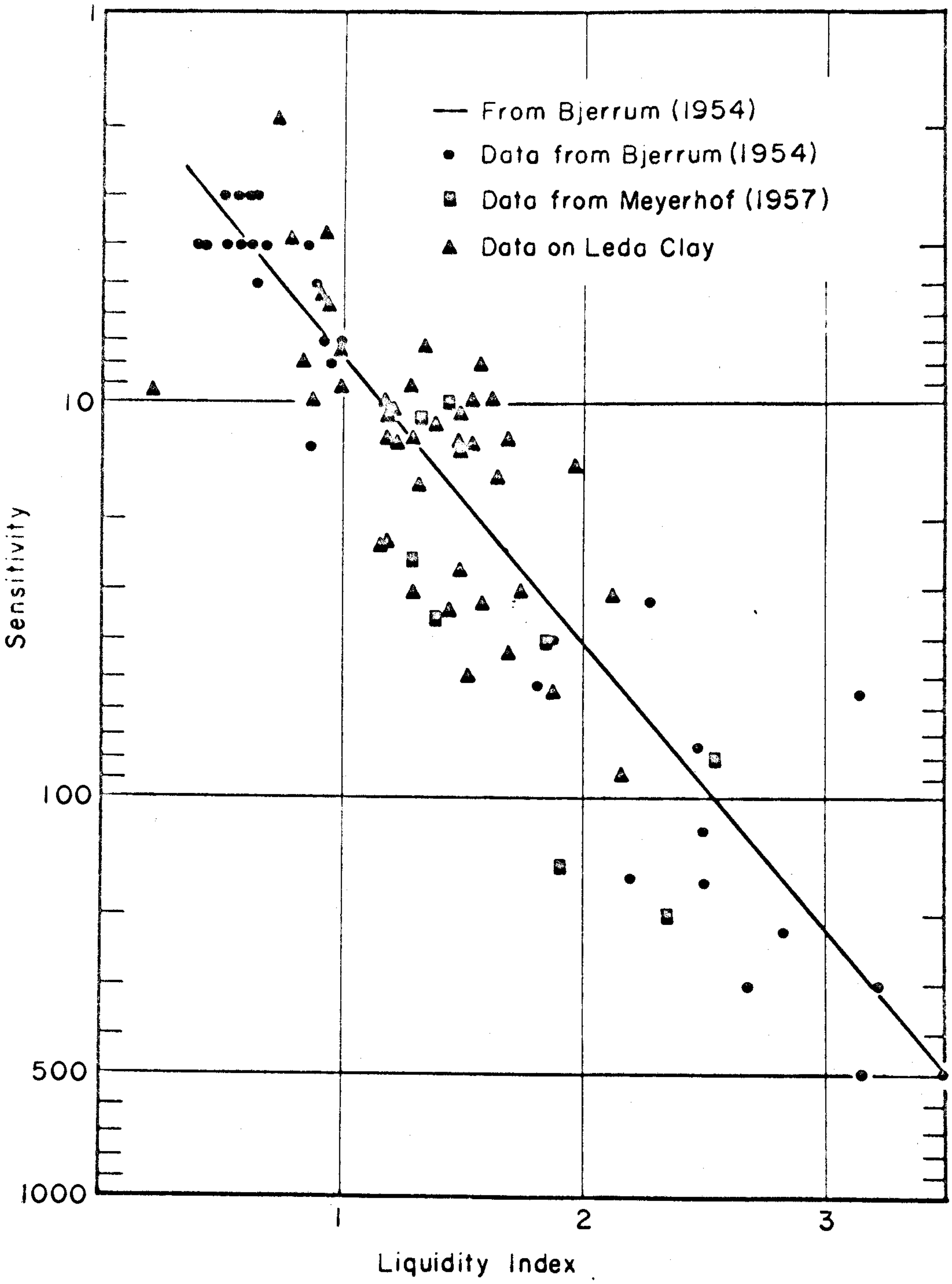


Figure 3. The relationship between sensitivity and liquidity index (Eden and Hamilton, 1956).

This assumption is probably sufficiently correct for undisturbed shear test, but it may be incorrect for the remolded shear test.

Burmister (1956) has criticized the use of the field vane test in place of the confined triaxial compression test. He listed his objections to the validity of the vane test as:

> (1) the plane or surface of shearing failure is forced to follow a prescribed path; (2) shearing stresses are mobilized first and concentrated at the rigid metal cutting edges, and (3) the thickness of the shearing zone is forced to be unnaturally thin.

Point (1) has already been considered, but it should be noted that the confined compression test forces the sample to fail under the applied set of major principal stresses, which are not necessarily those developed in the field at failure (see Figure 4). If the clay layer is highly anisotropic in its shear strength characteristics the confined compression test may also result in the overestimation of the undrained shear strength. Clearly then, the consideration of the type of soil being failed, the type of loading and the rate of loading must be included in the analysis of the results of any shear strength test.

Burmister's second point of criticism notes that in soils that exhibit the type of stress-strain relationship shown in Figure 5 progressive failure will take place along the cylindrical failure surface. This is due to the small amount of strain required to bring the soil to peak strength. Progressive failure results in the underestimation

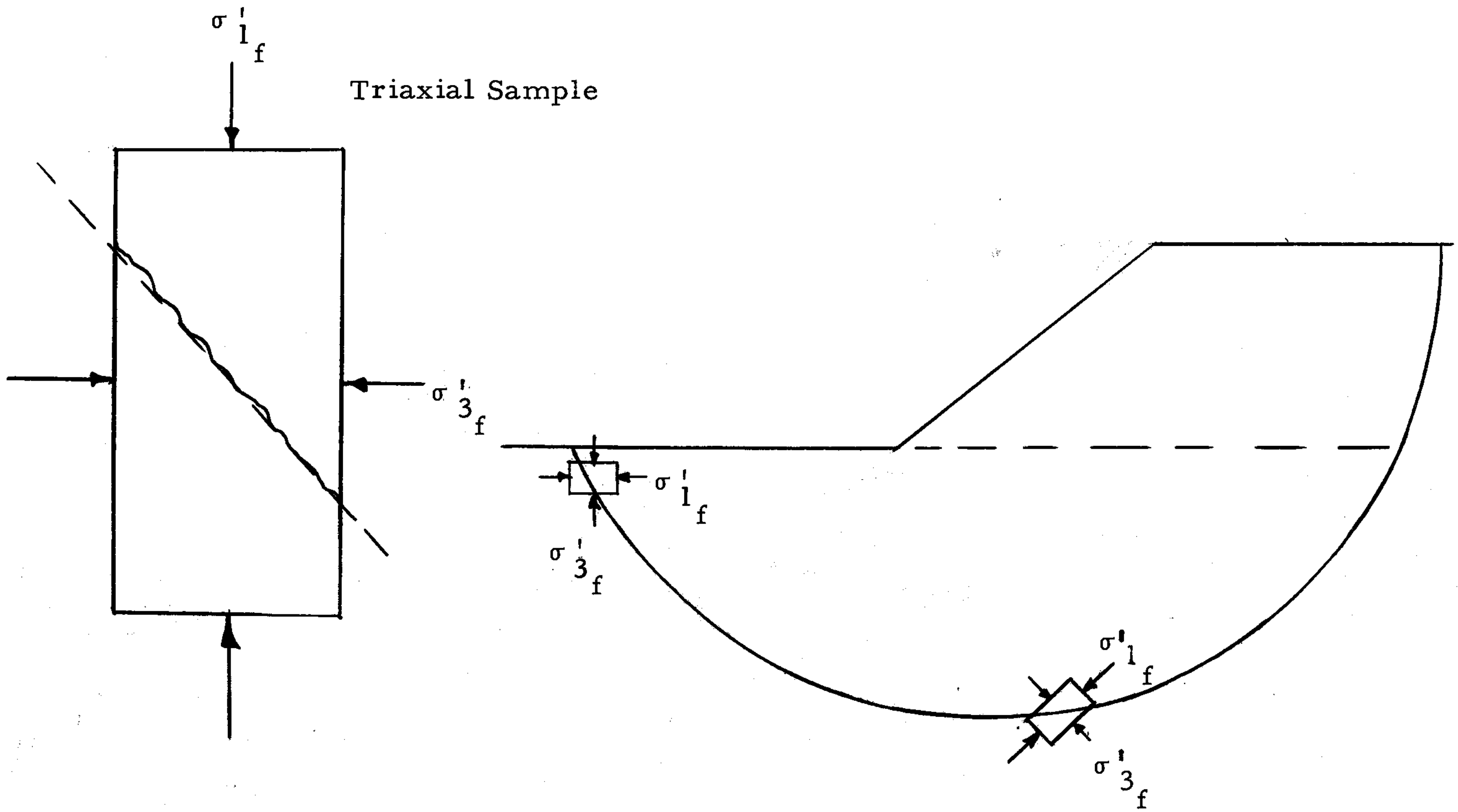


Figure 4. The orientation of the major principal effective stresses at failure for both the triaxial test and the actual embankment failure.

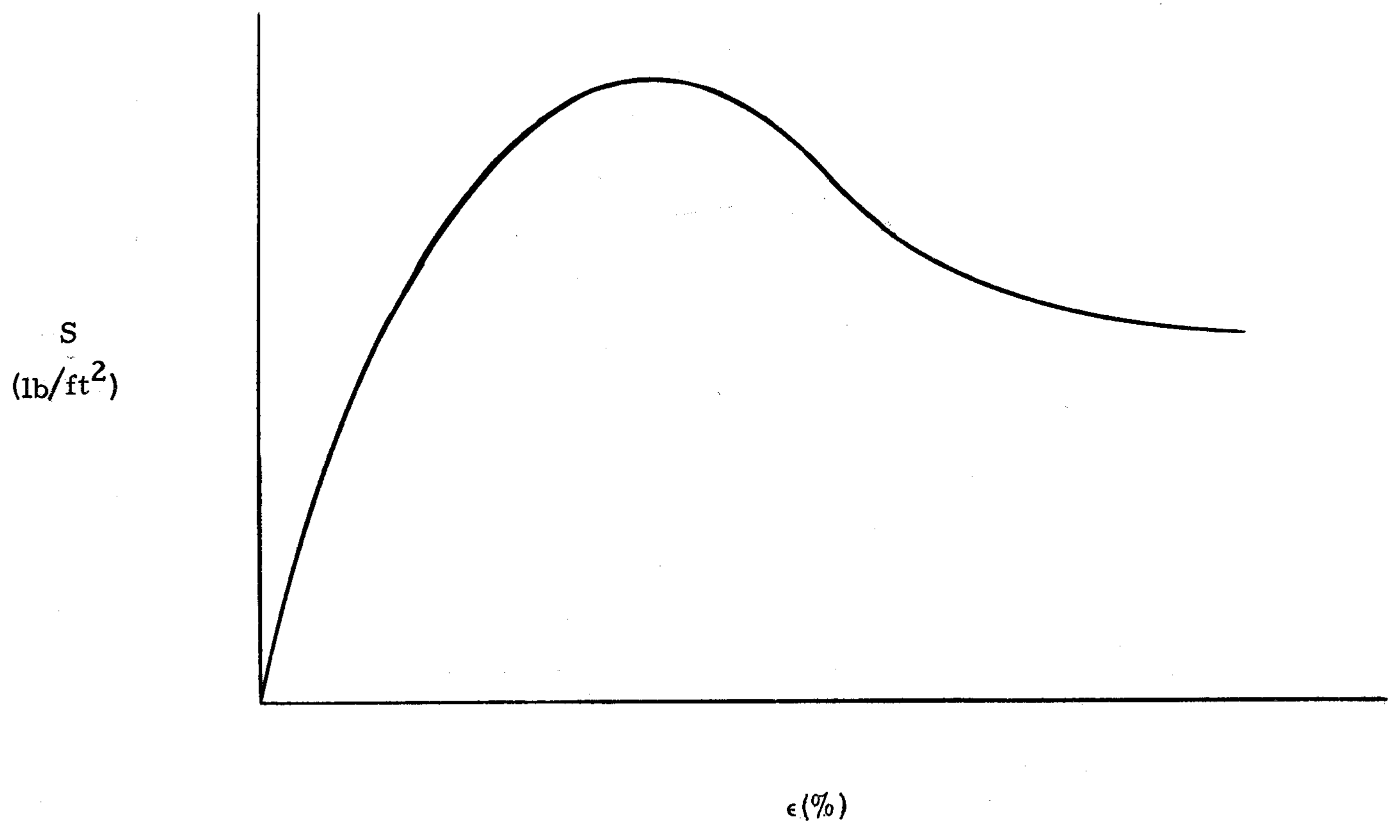


Figure 5. The stress-strain relationship for soils which are characterized by their high initial peak strength and lower ultimate strength.

of the shear strength since the peak shear resistance is never completely mobilized at all points in a sample at any one time. Burmister concluded that in these soils the vane will be too conservative. The two types of saturated clays which conform to the stress-strain relationship shown in Figure 5 are the overconsolidated clays and the sensitive highly flocculated clays of marine origin. It has already been pointed out that the field vane results obtained from highly sensitive marine clays are considered to be far better than the results obtained from the thin-wall piston samples. Eden (1966) concluded from his study of overconsolidated Leda clay, which had been preconsolidated to 4.5 T/ft^2, that field vane results were considerably below the results obtained from large block samples but that the vane characterized the changes in the undrained shear strength of the block samples far better than those taken from a thin-wall piston sampler. The field vane did give higher strengths than those determined from compression tests on tube samples. Eden (1966) also mentioned the fact that at sites where the Leda clay was only overconsolidated by 1 T/ft^2 the field vane strengths compared quite closely with the undrained strength obtained from block samples. Therefore, it would seem that Burmister's criticism related to the progressive failure of the vane shear strength test has not been shown to be a major disadvantage of the test. The major disadvantage of the vane test (as further shown by Eden's investigation, 1966) is that it determines the shear strength on the vertical plane.

As for Burmister's comment on the unnaturally thin shearing zone of the vane, it should be pointed out that thickness of the failure zone in the field failure is usually larger than the entire height of the normal triaxial sample. The triaxial test does not uniformly strain the zone of the sample which is free from end restraint (Coates and McRostie, 1963); therefore, the triaxial test also forces the failure zone to be "unnaturally" thin and results in a certain amount of progressive failure. Each shear strength test imposes a unique set of boundary conditions on the sample and the only "true" analysis of the shear strength is a full scale field failure.

Carlson (1948) used two full scale embankment failures to verify the fact that in sensitive silty clays and in clays which are normally consolidated the undrained shear strength determined by the field vane was superior to the shear strength obtained from the thin-wall piston samples in the laboratory. Eide and Bjerrum (1955) used the field vane to determine the undrained shear strength of sensitive clays of marine origin in the area of the Bekkelagat slide near Oslo, Norway. They calculated the factor of safety for this undrained failure to be 1.09 - 0.97 with the $\phi = 0$ analysis.

Golder and Palmer (1955) used both the vane and the triaxial tests to analyze the Scrapsgate, England embankment failure. They found that the vane undrained shear strength was greater than the undrained shear strength of the unconfined and confined compression tests. The vane test indicated that the shear strength increased progressively with depth while the triaxial tests indicated a more constant shear strength with depth in the soft grey organic silty clay stratum 22 feet thick. The embankment which failed had been constructed over an old embankment. The zone directly beneath the old embankment was tested in the same manner as was the adjacent material. Both the vane tests and the triaxial tests gave higher shear strength results under the old embankment which indicated the affect of consolidation. The same relative strength distributions with depth were noted in both zones. Careful evaluation (the author's) of the test results taken beneath the old embankment showed that the vane definitely indicated a disproportional amount of increased shear strength in the zone nearest the ground surface while the triaxial test did not. Golder and Palmer found that the vane test results defined a factor of safety of 1. 3 while the triaxial test results defined a factor of safety of 1. 0. Since the factor of safety at failure must have been 1. 0, they concluded that the vane test results were too high. They also showed that a plastic analysis of the problem indicated that a large section beneath the toe of the embankment was over stressed or in a plastic state. The failure took place three days after the final embankment height was reached. Since the $\phi = 0$ slip-circle analysis can make no allowance for the possible progressive failure of this sensitive clay, the validity of the slip-circle analysis is questionable. Skempton and Henkel (1953) investigated a similar site only ten miles from Golder and Palmer's (1955) embankment site. In the investigation performed by Skempton and Henkel the unconfined compression tests and the field vane tests indicated approximately the same values of undrained shear strength, and both tests attested to the increase in shear strength with depth. It is, therefore, this author's opinion that the vane tests were indicating the correct shear strength of the Scrapsgate subsoils. It also appears that the factor of safety (as defined $\phi = 0$ analysis) of 1.3 is correct if no progressive failure or large plastic zones are developed. Since failure did take place, it is believed that the large plastic zone at the toe of the embankment worked its way back to the base of the embankment, and the fill material cracked because it was unable to withstand the tensile stress. Embankment stability designs (normally determined by the $\phi = 0$ analysis) must be protected against possible progressive failures, which cannot be accounted for by any other means than increasing the factor of safety to which the embankment is designed.

The vast majority of the experience with the field vane test has proved it to be an inexpensive and efficient method of obtaining the *in situ* vertical undrained shear strength of normally consolidated and slightly overconsolidated soils. When the field vane test results are compared with the unconfined and confined compression test results, an insight into the shear strength characteristics of the substrata can be quickly obtained. Since the field vane essentially determines the vertical shear strength, the comparison between the vane and the triaxial compression test results indicates the relative amount of shear strength anisotropy of a given soil. The field vane can be used to determine the change in undrained shear strength during and after consolidation has taken place under a new overburden stress. The vane test should always be considered an undrained shear strength test and should never be used on soils which will not fail in an undrained manner. Wilson (1963) showed that the maximum torque obtained with laboratory vanes in silts depended upon the negative pore pressure developed during shear. This effect was caused by the dilatancy of these silt samples, but this author questions the validity of using the vane to determine the undrained shear strength (which includes the negative pore pressure) of a soil which will probably be partially drained (therefore, lose the negative pressure) in an actual field failure.

It is evident after considering the past experience with the field vane test that this test requires careful interpretation. The field vane shear strength determinations can be very misleading if no logical evaluation is made of them. A logical evaluation of field vane results requires an adjacent logged boring from which some undisturbed samples are obtained for either the simple unconfined or confined compression tests.

Since the subsoils of this investigation are highly organic, a brief review of the characteristics of organic soils is required for the evaluation of this embankment failure. The classification of organic soils is usually no more elaborate than noting that organic material is present or that the soil is a peat. At the present time, no quantitative method of classifying peats is available, and the methods of classification which are available are not universally accepted. The general basis for the quantitative classification of peats includes the determination of the water content, the organic (or ash) content and the specific gravity of the soil solids (Mac Farlane and Allen, 1963). Identification of the organic material is improved if terms such as woody, non-woody, fibrous, non-fibrous or amorphous are used to describe it, but this is not always done. At present, it is very difficult to compare the characteristics of organic soils due to the lack of accepted standard methods for classification.

The general properties of organic soils have been observed by many investigators. Organic soils are associated with large settlements, high secondary consolidation coefficients, low undrained shear strengths and low ratios of the horizontal effective stress to the vertical effective stress. The determination of the shear strength of an organic soil is complicated by the nonhomogeneous nature of these soils. Factors such as the direction of the major principal stresses at failure, the direction of organic bedding (if any), the amount and type of organic material present, and the type of non-organic material must be considered. Therefore, the shear strength tests of small sections of organic subsoils can easily misrepresent the "average" shear strength developed upon loading.

POST-FAILURE SITE INVESTIGATION

After the Glen Aiken Creek failure occurred on June 1, 1966, some 15 days after the embankment had been brought to grade, the Oregon State Highway Department decided to make a post-failure investigation of this site. The investigation was carried on in an area which was unaffected by the failure, but adjacent to the failure site. This investigation included the use of the field vane to determine the undrained shear strength distribution with depth at four locations. These results were averaged and found to indicate a factor of safety of 2.6. Since the factor of safety had to be 1.0 at failure, undisturbed samples were obtained from borings P-21 (station 175+00, 300 feet right) and P-22 (station 177+00, 300 feet right) for the further investigation of the substrata.

The samples taken from borings P-21 and P-22 were obtained with a modified Dames and Moore split spoon sampler. The inside liner of this sampler is composed of thin brass rings that have an inside diameter of 2.5 inches and are either 1 inch or 5 inches in length. Normally, the central 12 inch portion of the 18 inch sample was saved inside these rings. The rest of the material was saved in jars and must be considered disturbed material. The undisturbed 12 inch portion incased in the brass rings was wrapped in cellophane and placed in a metal tube container. The cap of the metal container was greased and then fit tightly over the container. These containers were placed in a humidity room until portions were required for testing purposes. The portions of the sample which were not used or left over after a certain test was performed were rewrapped in cellophane and returned to the humidity room.

The last portion of the site investigation was accomplished with the use of a modified field vane. This vane differed from the original one only by the fact that it had a slip-joint connection located two feet above the vane. This allowed the torque rods to develop their full frictional resistance with the soil before initiating vane movement. It was felt that this mechanism would provide a better determination of the rod-soil frictional resistance than the original "dummy" rod method. Originally, the "dummy" rod had been positioned adjacent to the vane and extended down to the depth at which the vane was located and rotated to determine the rod-soil friction. The results of these tests (only performed at station 175+00, 193 feet right) show a definite reduction from the previous investigations (see Figure 6 and Figure 7).

A summary chart of the subsoils found at the site is presented in Figure 8. This chart was determined from one boring (P-21), but it is generally consistant with the information obtained from several other borings. The portions of this boring which are not consistant with those of the surrounding borings are the two zones that contain

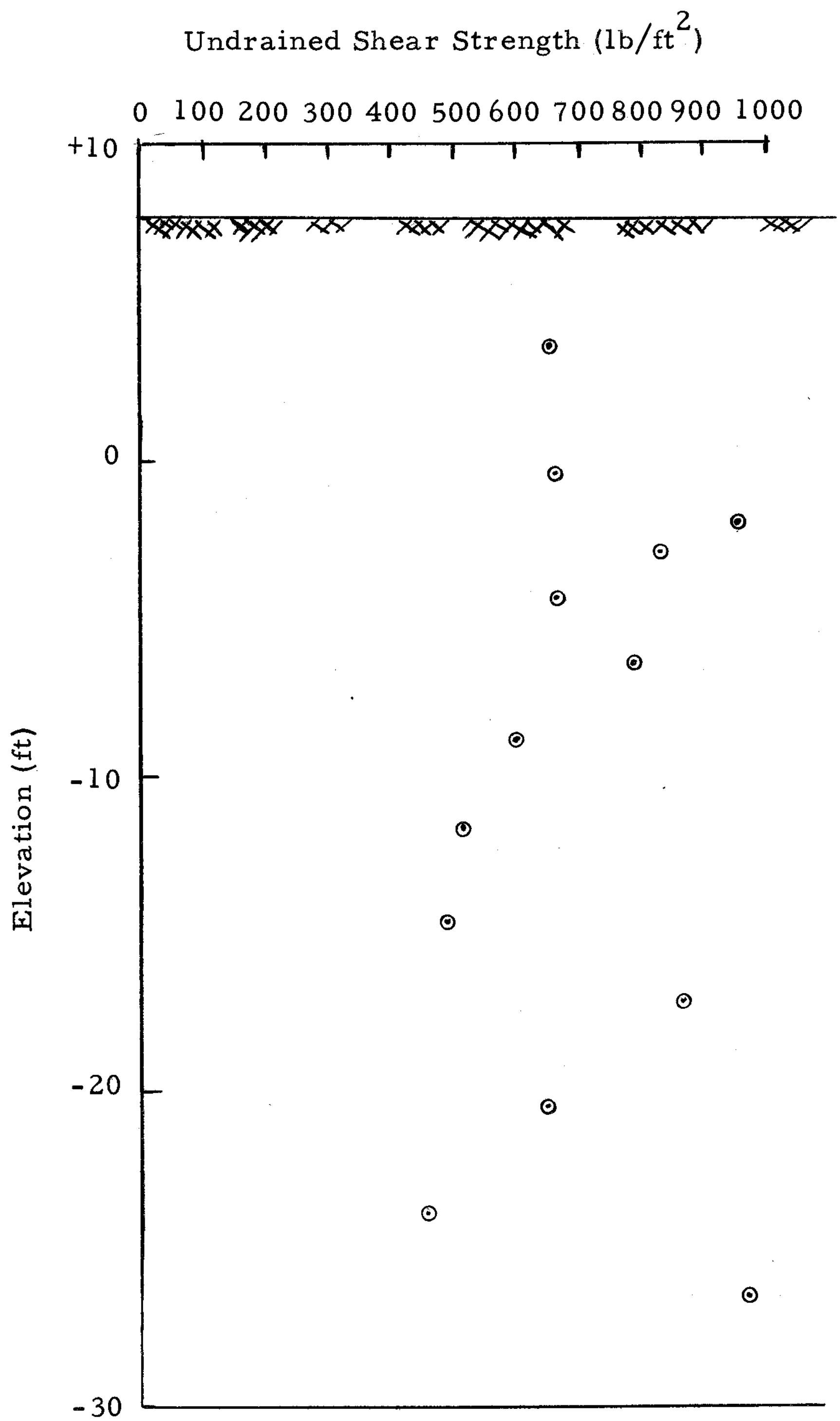


Figure 6. Typical shear strength distribution as determined by the field vane.

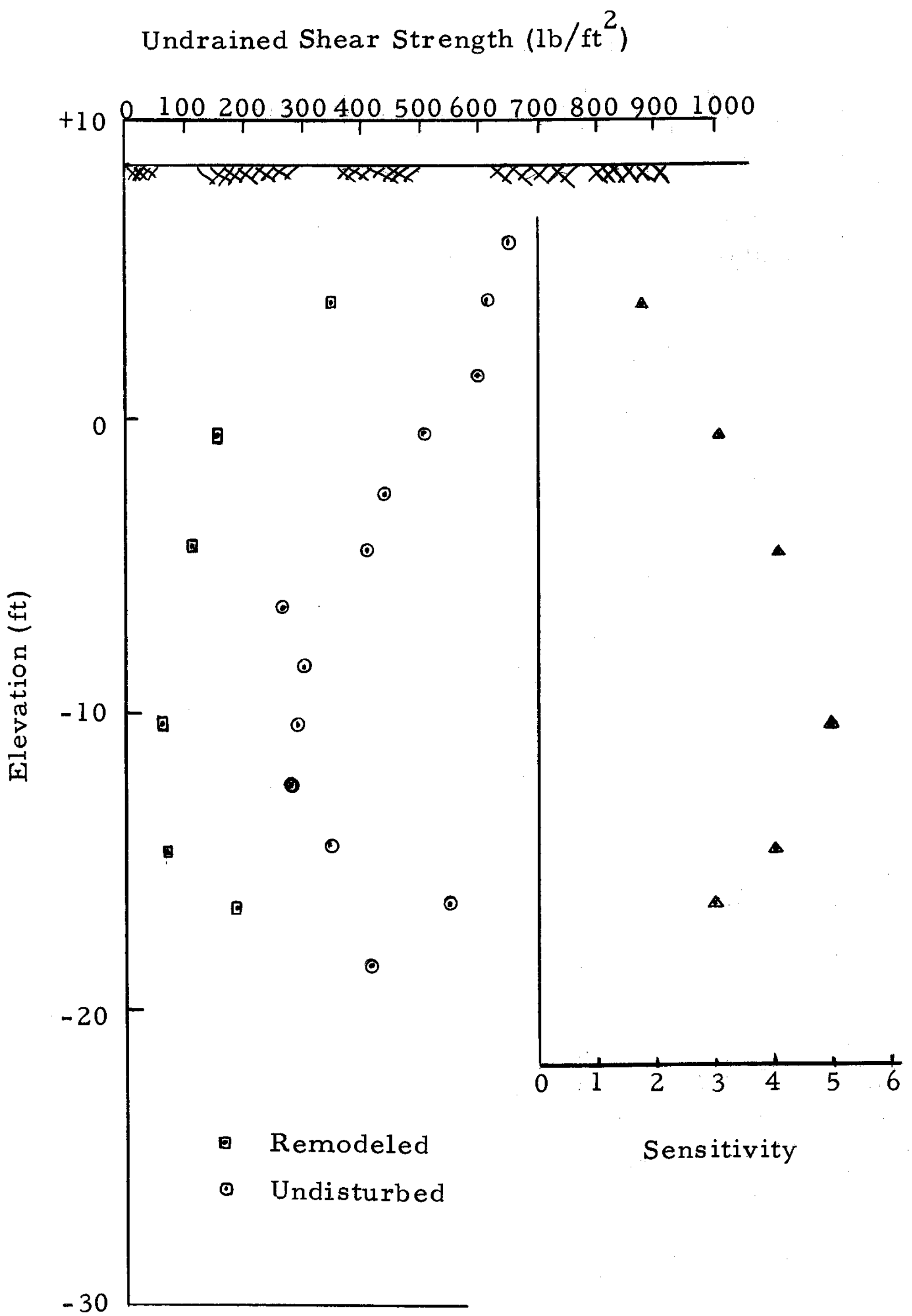


Figure 7. Typical shear strength and sensitivity distribution as determined by the modified slip-joint field vane.

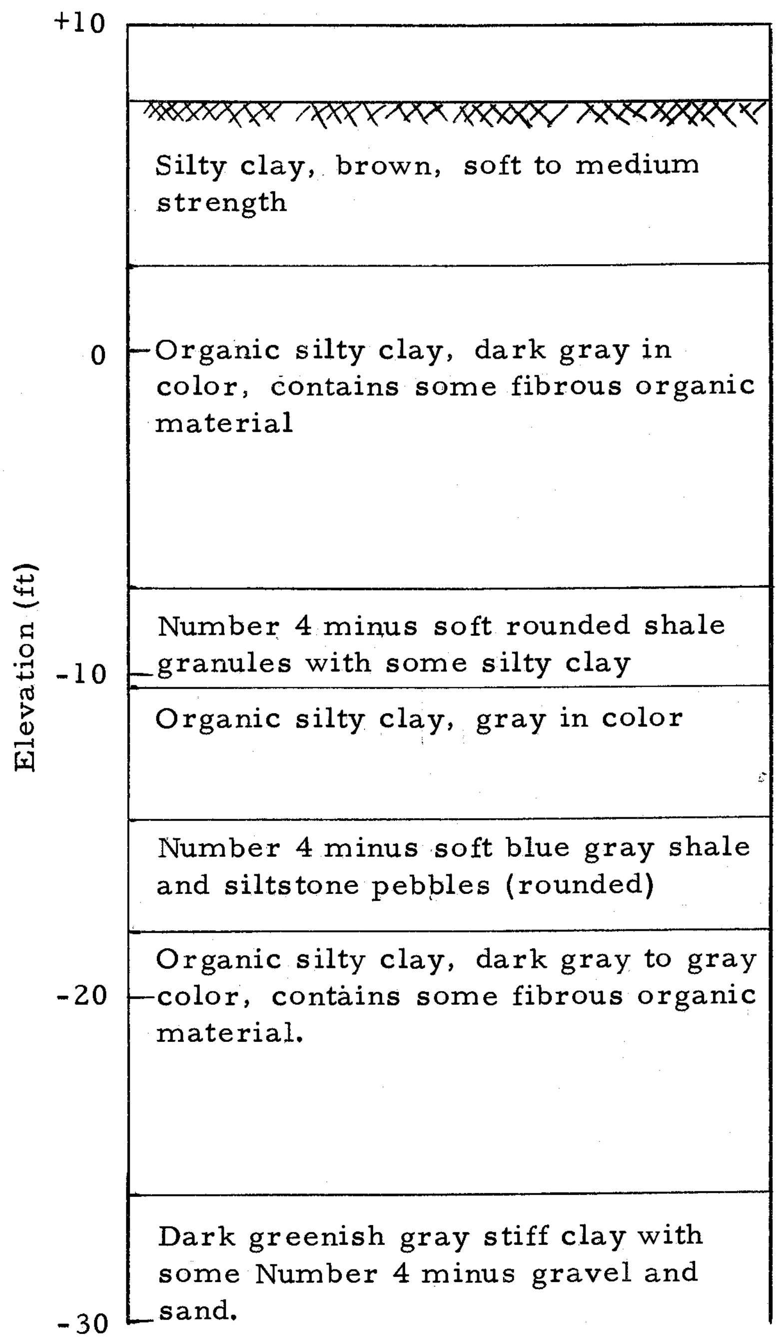


Figure 8. The distribution of soils found at the Glen-Aiken Creek embankment site.

the fine gravel nodules with the rusty silty clay binder. Neither the adjacent borings nor the undrained shear strength of the samples obtained from this boring indicate that these gravelly zones offer effective drainage. The shear strength of these gravelly layers can, therefore, be assumed to be that of the silty clay matrix material.

The soils encountered in this investigation classify as CH, CL, OH and Pt. Although these soils contain appreciable amounts of organics (loss on ignition ratio ranged from 0.067 to 0.206), the term fibrous can only be applied to a portion of the samples taken. Most of the organic matter is of the amorphous type and is distributed in a matrix of silty clay, but the shear strength determination of small samples of this material must be expected to feel the effect of the organic inconsistancies. The weakest material ($S = 100 - 200\ lb/ft^2$) found at this site was a highly sensitive silty clay (PI-63 to 42). The liquidity index of this silty clay ranged from 1.29 to 1.67. The organic material contained in this silty clay was small in comparison with the amount contained in samples taken from other depths.

FIELD OBSERVATIONS TAKEN DURING CONSTRUCTION

During construction of the embankment the Oregon State Highway Department set up a series of stations at which both settlement and piezometer records were kept. One of these special recording stations was located approximately 200 feet from the edge of the embankment failure. This station was Number 5, and its location was 172+00. Although complete failure did not occur at this station, two partial failures are indicated from data of the settlement and piezometer records of this station.

The settlement records of station 172+00 show that during the eight month period when the height of the fill was ten feet a total settlement of 1.7 feet occurred. In an attempt to analyze the settlement at this station, a plot was made of the fill height and settlement versus the time on a logarithmic scale. This plot was used to estimate the primary settlement under each new increment of fill. The resulting primary settlements were divided by the thickness of the soft silty clay zone (estimated to be 32 feet from boring P-21 at station 175+00) and were recorded as a percentage of this thickness. These final percentages were plotted against the fill height on a logarithmic scale in Figure 9. This method of presenting the settlement data is analogous to the void ratio versus consolidation stress (e-log p curve) plot, which is normally used to investigate the

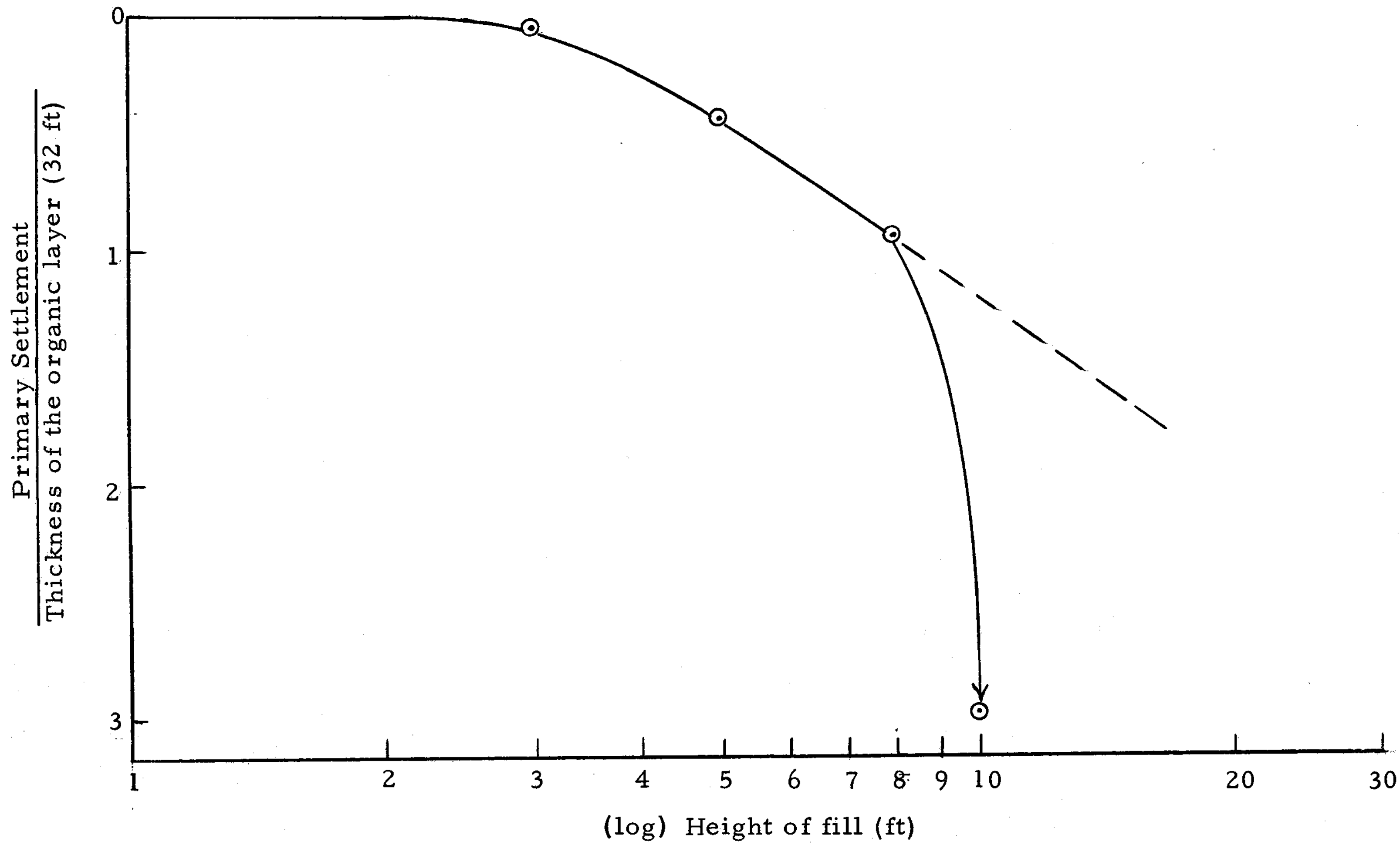


Figure 9. Settlement analysis of the embankment.

consolidation properties of soil samples. Figure 9 indicates that for a fill height less than eight feet the plot is linear, but as the load is increased by an additional two feet of fill the settlement is no longer a linear function. Since Figure 9 is analogous to the normal e-log p plot, it must be concluded that the nonlinear portion of the curve represents plastic flow or partial failure conditions.

The piezometers at this site were formed by making a 25 foot boring and then backfilling the boring with a clean sand to a depth of six to eight feet from the ground surface. The piezometer tube was installed ten feet below the ground surface while the boring was being backfilled with sand. Finally, the last six to eight feet was filled with a clay material to provide a seal between the ground surface and the piezometer tip. The piezometer record indicates that during the eight month period of constant overburden pressure (ten feet of fill) the pore pressure remained quite constant. Exactly how constant it remained is impossible to determine since the water surface was above the top of the piezometer for about six of the eight months. At the end of this inundation period the average pore pressure was slightly lower than it had been originally. This reduction could have been due to consolidation or to an ineffective seal at the top of the boring.

When the overburden stress was increased, the piezometer pressure increased at a slower rate than it originally did. This seemed to indicate that the piezometer was not as effective as it originally was. Four days after the fill had been brought to grade (17 feet at station 172+00), the piezometer record shows a marked decrease in pressure with time. It is impossible to state whether the decrease was actually sudden or whether the decrease was progressive since the piezometer seemed to have a longer response time. In either case, though, it can be said that this reduction in pore pressure required a volume change. Either the clay seal broke or some lateral movement allowed this pressure to dissipate.

LABORATORY INVESTIGATION

The results of the classification tests on samples from this site are presented in Figure 10. These soils generally classified as CH, CL, OH and Pt. Some of the samples were fibrous, but the majority of the samples contained amorphous organic material in a matrix of sensitive silty clay.

The first phase of the laboratory investigation was concerned with the affect which the rate of rotation of the vane had upon the measured shear strength. If the undrained shear strength was found to be a function of the rate of rotation, the rate of rotation of the vane that produced results which correlated with the results of the unconfined undrained tests would have to be determined. This investigation consisted of three laboratory vane shear tests conducted at three different rates of rotation. The Wykeham Farrance laboratory vane was used in this investigation. The three tests had to be performed on the same layer of soil to insure against the possibility of obtaining different shear strength determinations for somewhat different soils since these samples were not homogeneous. Even within the same layer of the 2. 5 inch diameter sample the inconsistancies of the organic matter caused difficulties in the interpretation of the results obtained. Since three tests had to be fitted into the 2. 5 inch inside diameter sample tube, it was somewhat of a problem

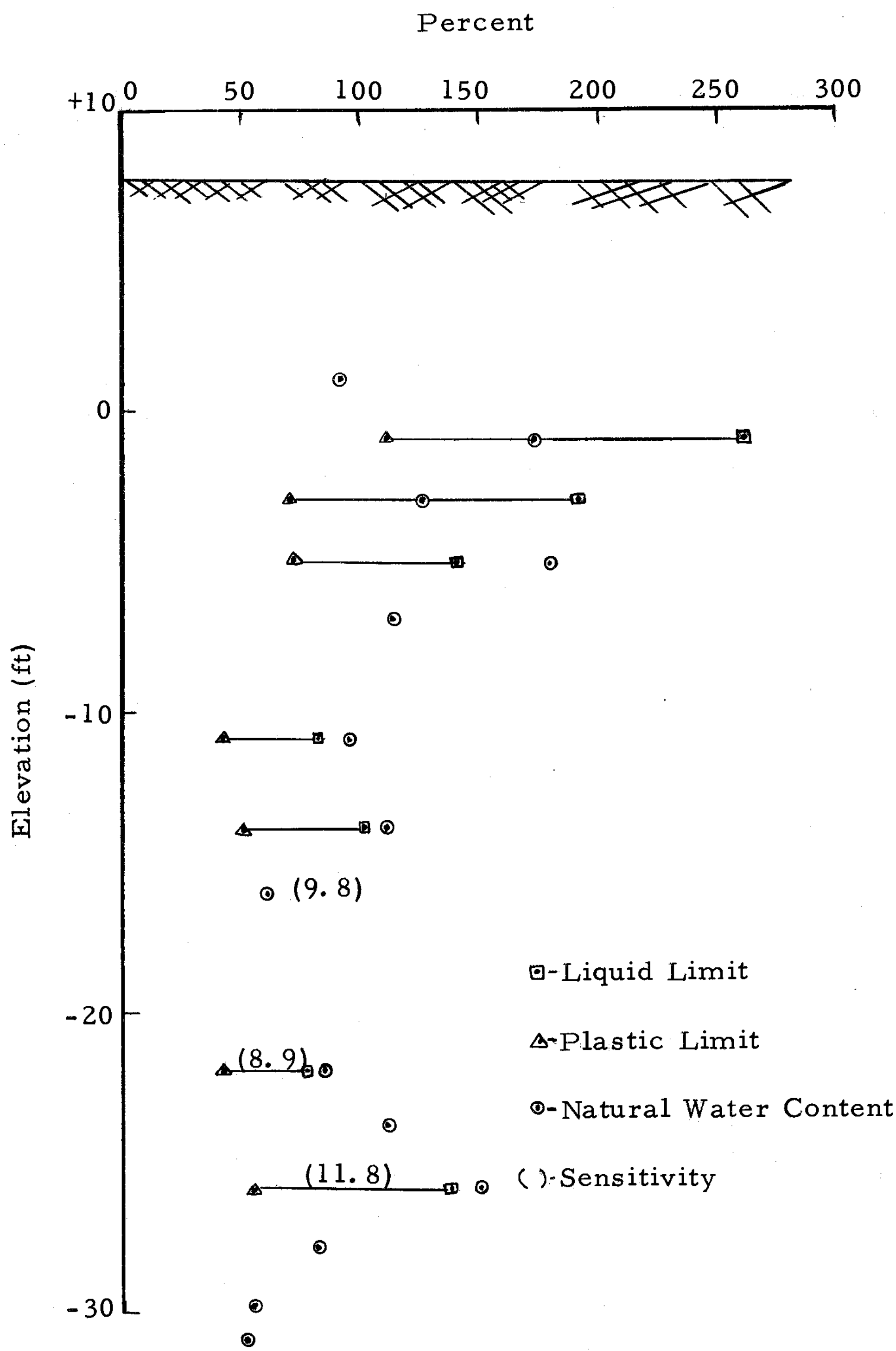


Figure 10. The results of the classfication test performed on the Glen-Aiken Creek embankment subsoils.

to prevent the disturbance of the rest of the sample while performing the first and second tests. The blades of the vane had the dimensions of one-half inch in diameter and one-half inch in height. Therefore, the vane could be inserted in three sections of the sample sufficiently separated as to reduce the disturbance caused by each shear strength test to a negligible amount on the other sections. The greatest disturbance was caused by the removal of the vane after failure since the vane was always inserted to a depth one-half inch above the top of the vane. This top layer would rip out as the vane was being removed. The solution to this technical problem was simply to hold a one-half inch inside diameter washer on the surface of the sample while the vane was brought up through it. The three rates of rotation were 1°/min, 3°/min and 30°/min. The approximate times required for the occurrence of failure at these rates were respectfully 30 minutes to 80 minutes, 5 minutes to 15 minutes and 1 minute to 3 minutes. The results of these tests are presented in Table 1.

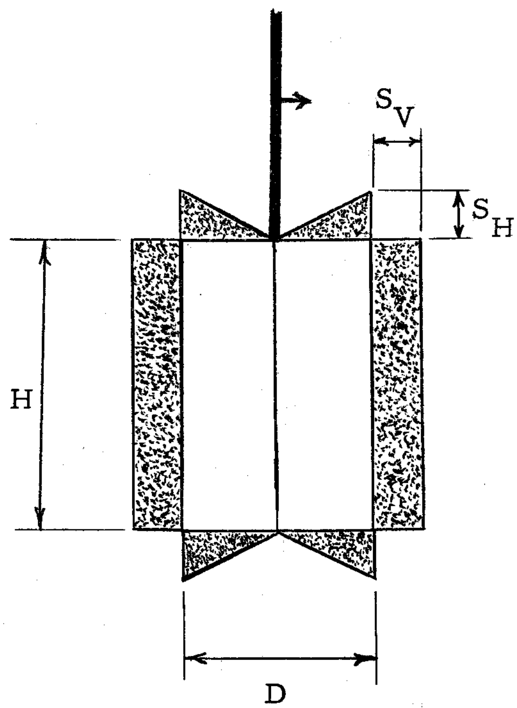
Table 1 indicates no consistant difference between the shear strength determinations of the laboratory vane at the different rates of rotation. Since the three tests were performed on different elements of the same layer of soil, the accuracy with which each test could reproduce the results of the others is questionable. Therefore, the erratic distribution of the results in Table 1 must be attributed to the inconsistancy caused by the erratic distribution of the

Table 1. The change in shear strength accompanying the change in rate of rotation.

Sample No.		Portion	Shear Strength T/ft^2 (30°/min)	Ratio of the Shear Strength Determined to that which was Obtained at 30°/min (1°/min)	(3°/min)
P-21	2	Top	0.26	1.23	1.11
	"	Bottom	0.20	1.10	1.35
	3	Top	0.24	0.83	0.92
	"	Middle	0.31	1.19	1.10
	4	Top	0.18	0.82	0.82
	"	Middle	0.09	0.89	1.11
	7	Bottom	0.065	0.92	0.62
	"	Middle	0.10	1.10	1.00
	8	Top	0.22	0.82	0.96
	"	Middle	0.16	1.12	0.75
	11	-	0.28	0.93	0.96
	13	-	0.37	1.05	1.19
P-22	17	Top	0.29	0.82	0.86
	"	Middle	0.36	0.89	1.11
	"	Bottom	0.31	0.87	0.97
	18	Top	0.28	0.68	1.00
	"	Middle	0.45	0.98	0.98
	19	Top	0.28	0.96	0.82
	"	Middle	0.27	1.04	0.96
	20	Top	0.23	1.13	0.96
	"	Middle	0.32	0.97	1.00
	21	Top	0.16	1.06	1.06
	"	Bottom	0.19	1.05	0.74
	22	Top	0.23	1.30	1.09
	"	Bottom	0.25	1.12	1.12
	23	Top	0.25	0.92	1.05
	"	Bottom	0.22	1.09	1.14
	24	Top	0.25	1.04	1.00
	"	Bottom	0.27	1.00	0.93
	25	Top	0.18	1.11	1.05
	"	Bottom	0.18	1.11	1.05
	26	Top	0.26	1.38	1.35
	"	Bottom	0.41	0.90	1.24
	27	Top	0.34	1.18	1.06
	"	Bottom	0.34	1.20	1.23
	28	Top	0.32	1.18	1.06
	"	Bottom	0.35	1.20	1.23

organic material.

The second phase of the laboratory investigation was involved in the study of the possible anisotropic shear strength characteristics of the organic sensitive silty clays. This study was carried out with two vanes of different dimensions. The investigation was limited by the fact that only two tests were possible on the same layer of soil in the 2.5 inch diameter sample tube due to the dimensions of the vanes used. The formula used to separate the torque components developed on the vertical and horizontal planes was derived from the assumed shear resistances at failure shown in Figure 11. The first term in Equation 3a is four times the value of the second term if S_V is assumed to be equal to S_H for a laboratory vane whose dimensions are $D = 1/2$ inch and $H = 1/2$ inch. Since the purpose of this investigation was to determine the relative magnitude of S_V and S_H, the dimensions of the second vane were chosen for the purpose of increasing the multiplication factor of the second term in Equation 3. The practical limit placed upon the selection of the diameter was that it be no greater than half the inside diameter of the sample tube. The dimensions finally used were $D = 1$ inch and $H = 1/2$ inch. Formulas 4 and 5 are derived from Equation 3 by the substitution of the particular dimensions of each vane for the terms H and D. The expressions which were used to determine S_V and S_H were obtained by solving Equations 4 and 5 simultaneously.



$$T = \frac{D}{2}(\pi D)(H)S_V + 2S_H \int_0^{D/2} [\frac{r}{D/2}] (2\pi r) r \, dr$$

$$T = \frac{\pi D^2}{2} H(S_V) + \frac{\pi D^3}{8} (S_H) \qquad (3)$$

$$\text{If} = \frac{H}{D} = 1$$

$$T = D^3 [\frac{\pi}{2} S_V + \frac{\pi}{8} S_H] \qquad (3a)$$

Figure 11. The assumed distribution of shear resistances at failure and derivation of the relationship between them and the torque measured at failure.

If $H = 1/2$ inch and $D = 1/2$ inch

$$T_1 = \frac{\pi}{16} S_V + \frac{\pi}{64} S_H \qquad (4)$$

If $H = 1/2$ inch and $D = 1$ inch

$$T_2 = \frac{\pi}{4} S_V + \frac{\pi}{8} S_H \qquad (5)$$

Therefore,

$$S_H = \frac{16}{\pi} [T_2 - 4T_1] \qquad (6)$$

$$S_V = \frac{4}{\pi} [8T_1 - T_2] \qquad (7)$$

Thus, the determination of both S_V and S_H could be obtained by measuring the torque required to fail a section of the sample with each vane.

The problems of determining the horizontal and vertical shear strengths were greater than those involved in the study of the effects of the rate of rotation. Problems such as the variability of the organic material, both in terms of quantity and amount of weathering, and the accuracy with which the results could be duplicated were still present. However, the importance of the exact duplication of the results was far more crucial than before. Assume that $\pm \delta$ represents the standard deviation of each torque reading from the actual torque required for failure. Then Equation 6 has a standard deviation of $\pm 80\,\delta/\pi$, and Equation 7 has a standard deviation of

$36\,\delta/\pi$. Thus, it is evident that the errors involved in the calculation of S_H are approximately twice those involved in the calculation of S_V and that the errors involved in the calculation of S_H are approximately five times as great as those involved in the calculation of the average shear strength.

The results of the horizontal and vertical shear strength determinations were too erratic to justify definite conclusions. A generalization of Table 2 would simply note that half of the samples tested indicated that the horizontal shear strength was appreciably lower than the vertical shear strength.

Table 2. The calculated values of horizontal and vertical shear strengths as determined by the laboratory vane tests.

Sample No.	Part	T_1 (lb/in)	T_2 (lb/in)	S_V lb/ft^2	S_H lb/ft^2	S_{ave}
2	Bottom	0.85	3.63	580	170	500
3	Middle	0.64	3.33	330	560	375
7	Middle	0.44	1.88	300	87	260
17	Top	0.60	3.51	240	810	350
18	Middle	1.55	6.04	1,160	Negative value	910
21	Top	0.34	1.96	140	370	200
23	Top	0.63	3.19	340	490	370
25	Top	0.77	1.88	970	Negative value	450

The third phase of the laboratory investigation was concerned with the consolidated undrained shear strength of the subsoils found at the embankment failure site. The fact that the field vane tests performed in the undisturbed area adjacent to the failure site indicated that the embankment was stable, while the soil which participated in the failure had been pre-loaded with approximately ten feet of fill for eight months, was quite puzzling. It seemed that either consolidation was not taking place or that the error resulting from the use of the field vane was even greater than (F.S.= 2.60) was originally estimated. A second reason for performing the triaxial undrained test was to obtain the magnitude and sign of the pore pressure developed at failure. A convenient form to present the effect of the pore pressure is the ratio of the pore pressure to the deviator stress developed at failure for a constant confining pressure. This ratio is Bishop's (1954) "A_f" parameter for a saturated soil. The sign of the "A_f" parameter will indicate whether or not the sample dilates during shear.

Samples Number 3 and 7 were used for the series of consolidated undrained shear strength tests with pore pressure measurements. The Geonor triaxial equipment was used for the measurement of both the shear resistance and the pore pressure. Sample Number 3 represented the soft organic silty clay while sample Number 7 represented the sensitive somewhat organic silty clay. These samples represented different sections of the first 20 feet of the subsoils. Each sample was consolidated to 1 T/ft^2 and then brought to failure in the triaxial cell. Since the failure deviator stress is approximately constant over large strains, the failure conditions were only just developed and then the loading was stopped. The samples were then consolidated to 2 T/ft^2, and the same procedure was repeated. Finally, the samples were consolidated to 4 T/ft^2 and failed. This process of shearing and then reconsolidating the sample is not recommended for homogeneous clays, but in a nonhomogeneous deposit the comparison of the consolidated undrained shear strength of samples from different depths may be far more complicated to interpret than the results of sheared and then reconsolidated samples. Except for the modification just mentioned, the procedures used in these triaxial tests are well described by Bishop and Henkel (1962).

Sample Number 3 indicated an apparent angle of internal friction of 17° for the series of consolidated undrained tests. The value of the "A_f" parameter was + 0.9 for sample Number 3. This value of "A_f" indicated that the sample does not dilate during shear and is representative of a normally consolidated clay. The coefficient of consolidation (C_V) of sample Number 3 was found to be approximately 1.76×10^{-4} in^2/min. The procedures used to calculate C_V are presented in Bishop and Henkel (1962). This value was far lower than was originally expected for an organic soil, but since the organic matter is contained in a matrix of silty clay, the permeability of the soil can be completely controlled by the properties of the silty clay.

Sample Number 7 had an apparent angle of internal friction of 13° and an "A_f" parameter of + 1.0. Neither series of consolidated undrained tests indicated the presence of an apparent cohesion parameter. Sample Number 7 was found to have a coefficient of consolidation of 2.93×10^{-5} in.^{2}/min. Therefore, the time required for the effective consolidation of the uppermost ten feet would be measured in terms of years. If it is assumed that C_V of sample Number 3 is representative of the uppermost ten foot layer, the time required for ten percent of the ultimate consolidation to take place would be 1.22 years. Thus, although ten feet of fill was left on the site for a period of eight months, the shear strength of the soil was not affected because no appreciable consolidation occurred.

The last phase of this investigation was concerned with the sensitivity of the silty clays. Sensitivity is defined as the ratio of the undisturbed shear strength to the remolded shear strength at constant water content. The modified field vane determination of the sensitivity is shown in Figure 7. Sensitivity, as determined by the field vane, was only a fraction of the value estimated from Bjerrum's (1954) plot (Figure 3) of sensitivity versus the liquidity index. Since the liquidity index of most of the soils below the first 12 feet is 1.0 or greater, it was felt that a few of the lower samples could be used to test the correlation between the liquidity index and sensitivity of these soils. Samples Number 9, 11 and 13 were used for the sensitivity tests. The Wykeham Farrance laboratory vane (H = 1/2 inch, D = 1/2 inch) was used to determine the sensitivity. The vane first determined the undisturbed shear strength and then it was rotated in place four complete revolutions. The vane was then allowed to set for two minutes before the remolded shear strength was determined. The values of sensitivity obtained were respectively 9. 8, 8. 9 and 11. 8. These values correlated well with Bjerrum's (1954) plot of liquidity index versus sensitivity (Figure 3).

ANALYSIS OF THE FAILURE

The first stability analysis was developed from the information obtained from the modified slip-joint field vane. Figure 12 gives a summary of the undrained shear strength tests of one of the borings at station 175+00, 193 feet right. The final slip-circle and the assumed soil parameters are shown in Figure 13. No tension crack was assumed in the first solution of the embankment stability problem since this solution was only intended to locate the most critical slip-circle. The factor of safety of 1. 05 was found to be the minimum for the given analysis. This indicated that a tension crack in the embankment of three feet would reduce the factor of safety to 0. 996. The maximum tension crack which could be developed for the assumed conditions was 16. 7 feet. Therefore, only a fraction of the possible tension crack height was required to initiate the failure of the embankment. Since a tension crack requires a reasonable amount of time to develop, a failure of this type could possibly require several days or weeks to occur.

The second portion of the stability analysis of this highway embankment was concerned with the partial failure, which occurred under a fill height of ten feet eight months before the final failure. Again, the information obtained from the modified field vane was used (Figure 12) for the smaller embankment. A tension crack was

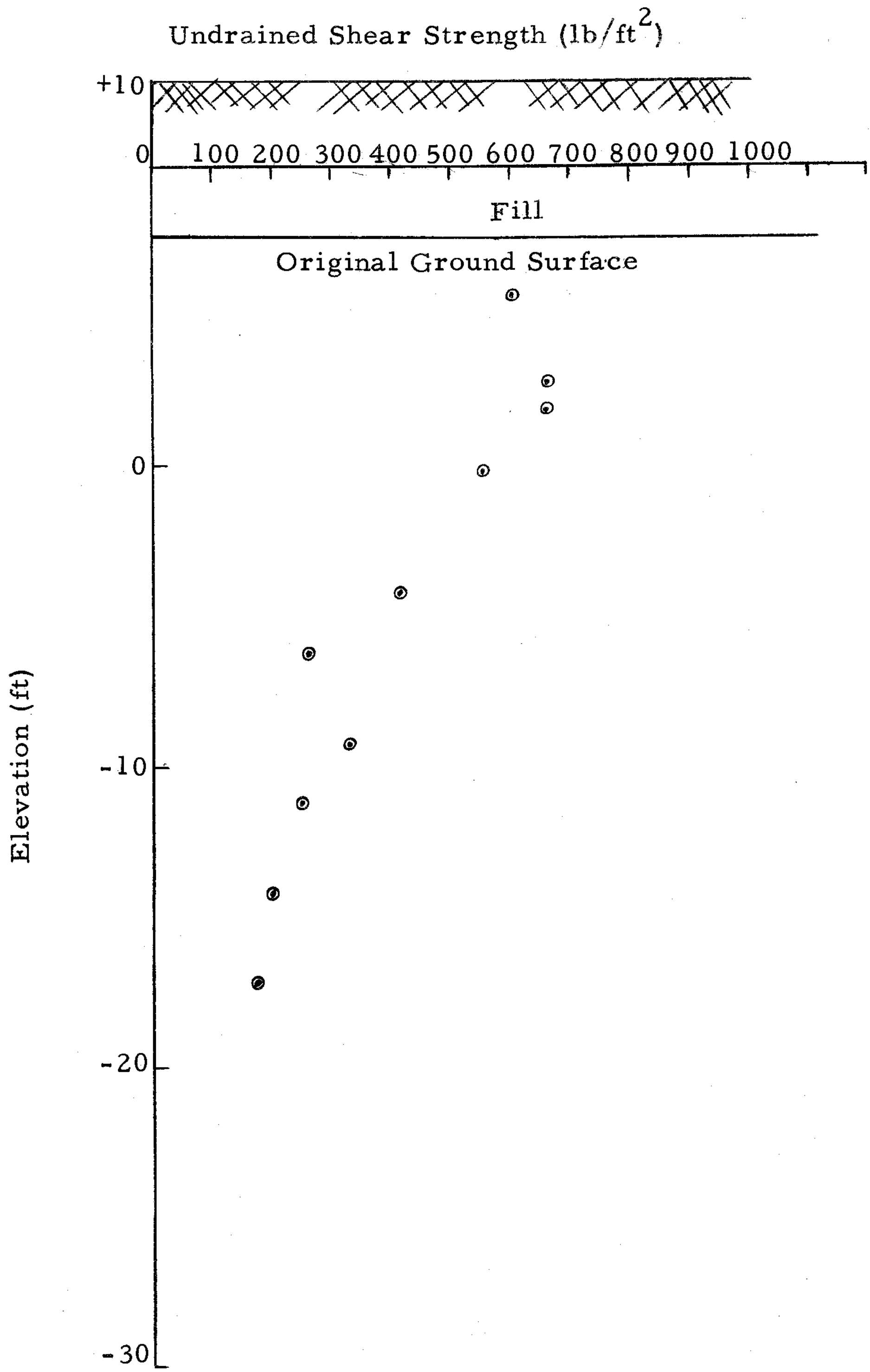


Figure 12. The distribution of the undrained shear strength of the Glen-Aiken Creek subsoil as determined by the modified slip-joint field vane.

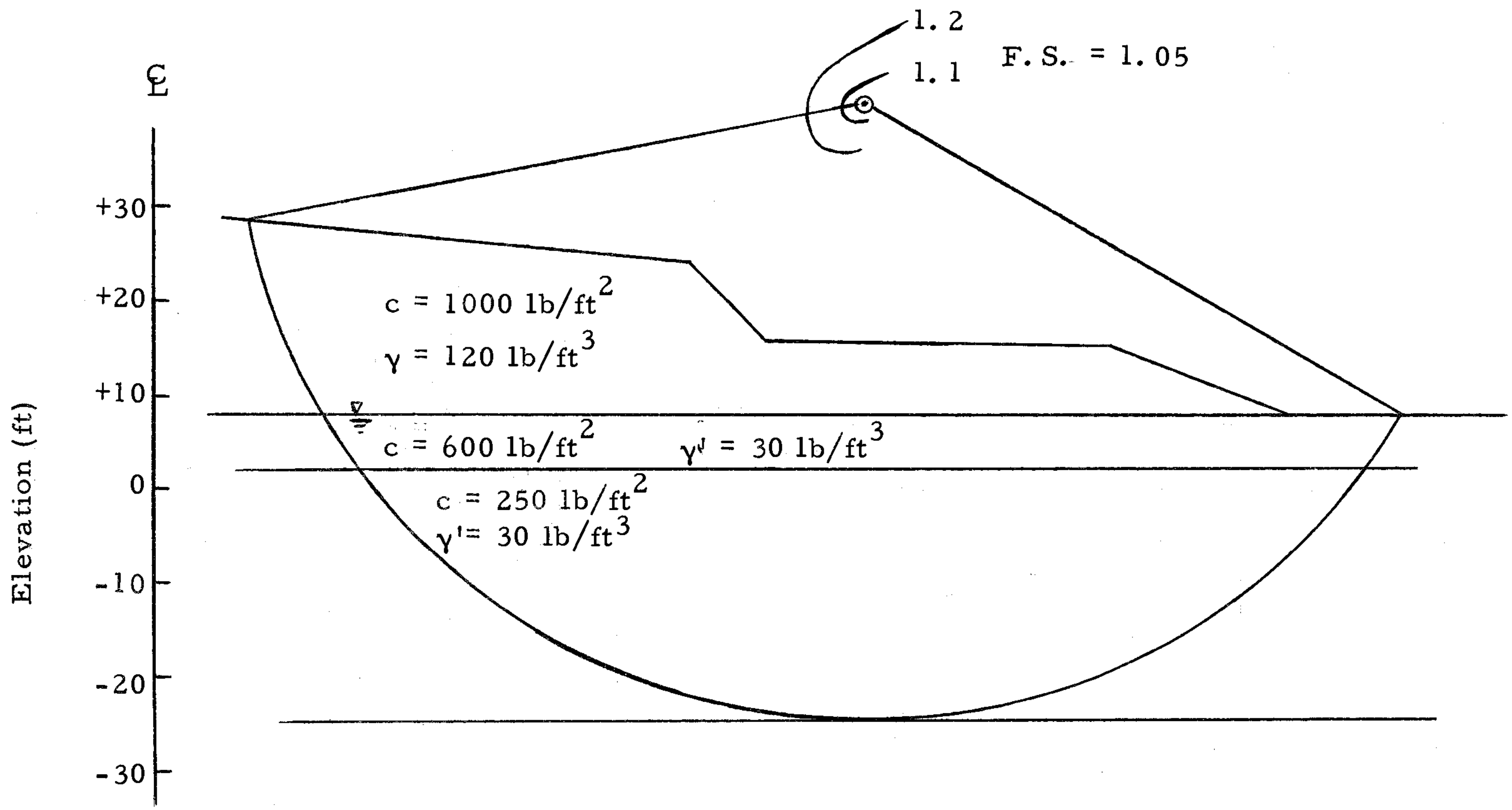


Figure 13. The $\phi = 0$ slip-circle analysis of the 22 foot high highway embankment based on the shear strength determinations of the modified field vane.

assumed to be developed to the full height of the fill (normally seven to eight feet near the toe of the fill). The results of this analysis are shown in Figure 14. The factor of safety as determined by the $\phi = 0$ analysis was found to be 1.55. Since this analysis indicated that the embankment was reasonably stable and the settlement record indicated that the embankment experienced a partial failure, the laboratory determined shear strength distribution offered a logical check of this analysis.

The combined data obtained from both the laboratory vane and the unconfined compression tests (Figure 15) were used to define the distribution of the undrained shear strength with depth. Figure 16 contains the results of the slip-circle analysis and the assumed soil properties used in the analysis. A minimum of 1.41 was found for the factor of safety of this ten foot embankment. Since a factor of safety of 1.41 seems too high to indicate that a partial failure occurred, it appears that the $\phi = 0$ slip-circle analysis may not represent the critical failure mechanism.

The possibility of the failure occurring along the very soft silty clay layer ($c = 110\ lb/ft^2$) was investigated. It was found that the factor of safety of this type of failure was 1.65. Therefore, the sliding wedge analysis of this embankment's stability is farther from reality than is the slip-circle analysis.

Since the organic silty clay is a very sensitive soil, it is

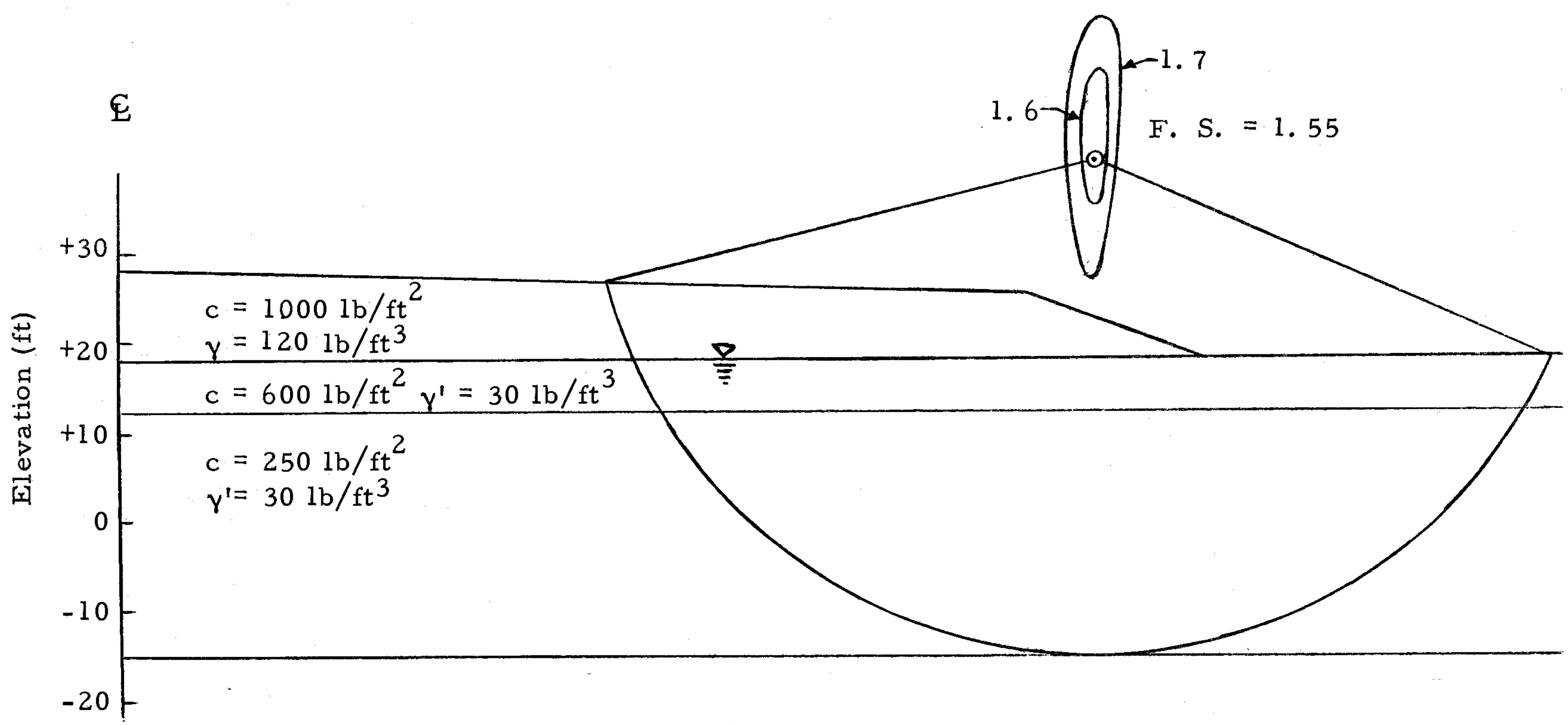


Figure 14. The $\phi = 0$ slip-circle analysis of the ten foot high embankment based on the shear strength determinations of the modified field vane.

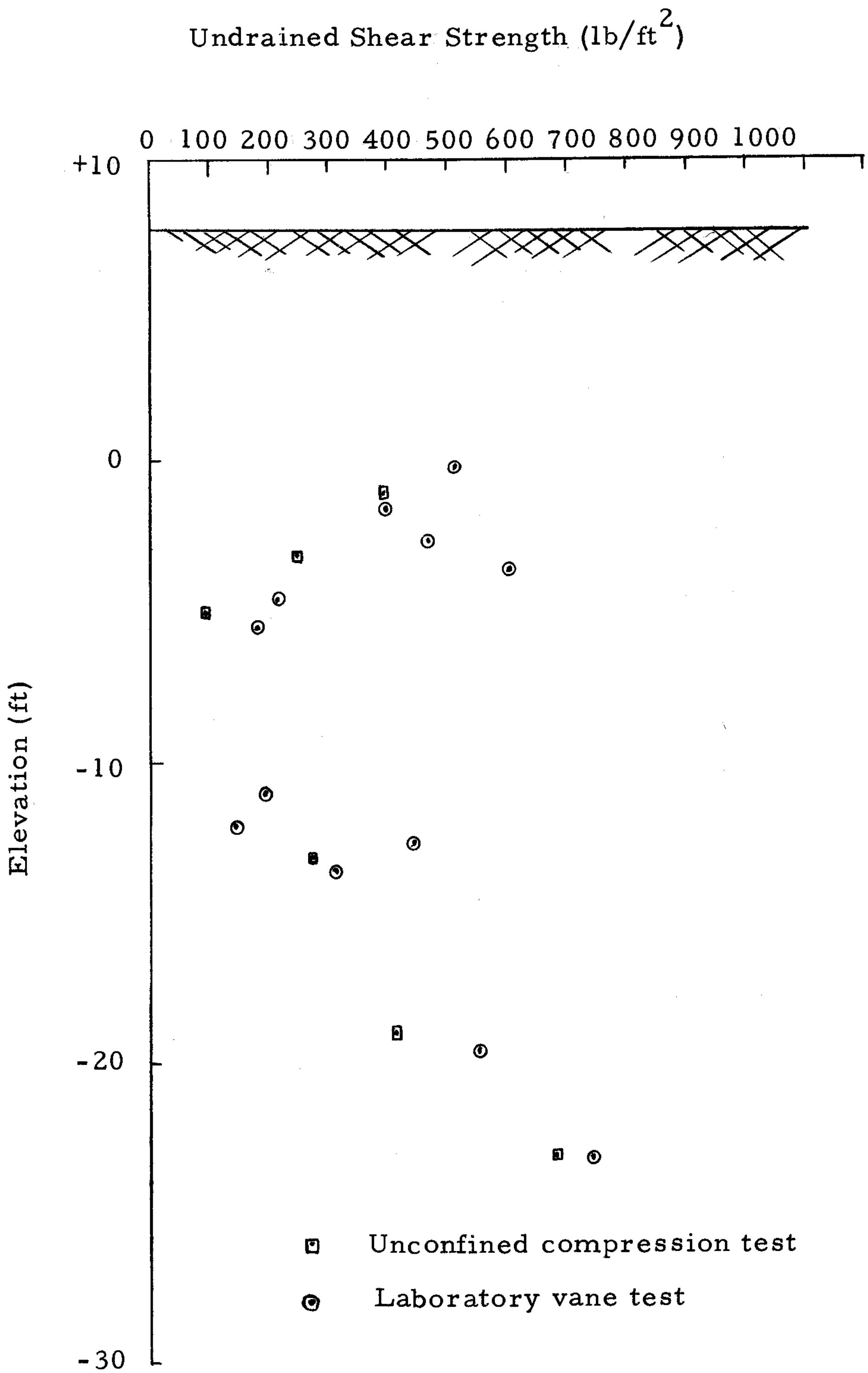


Figure 15. The combined results of the shear strength test performed in the laboratory.

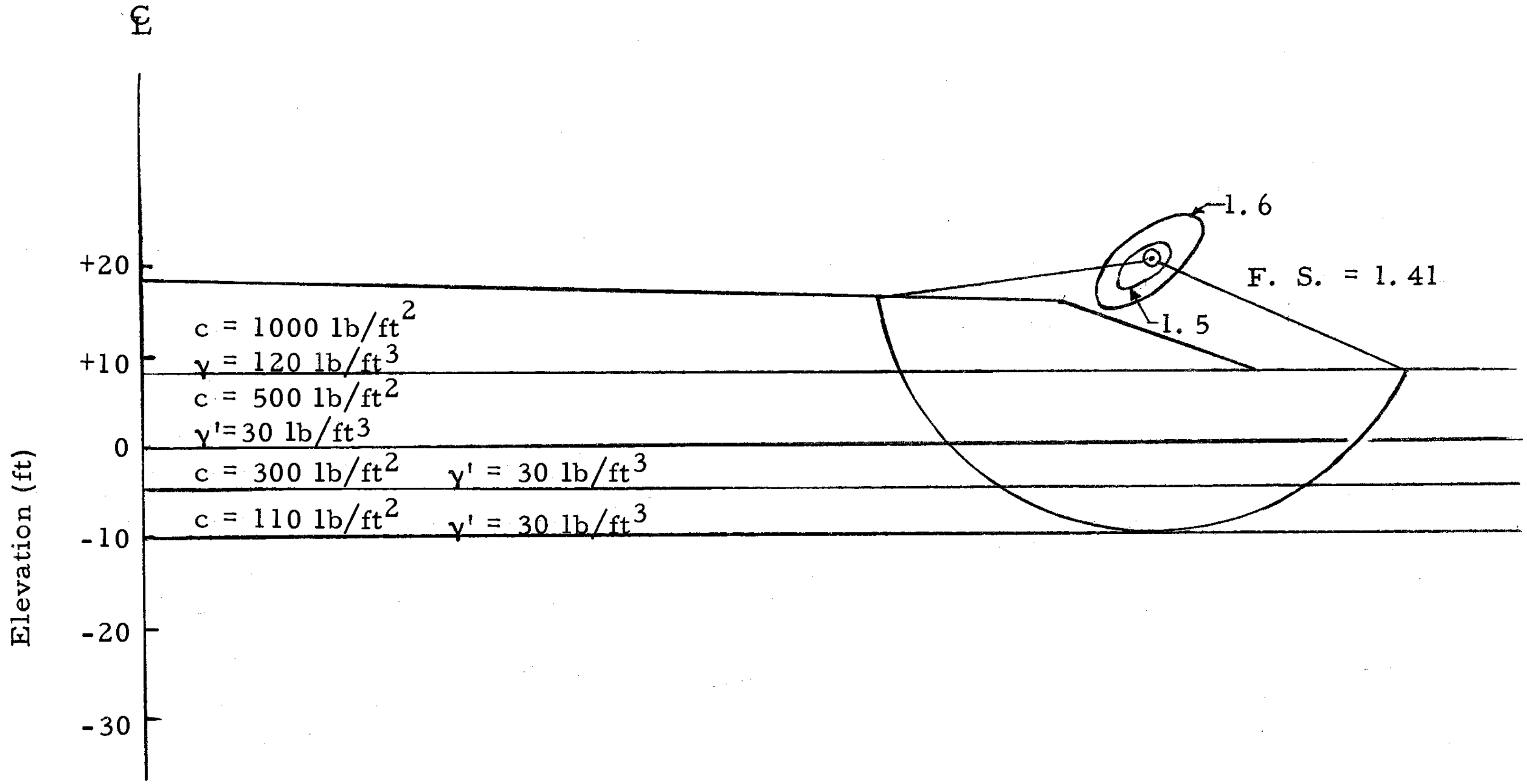


Figure 16. The $\phi = 0$ slip-circle analysis of the ten foot high embankment based on the shear strength distribution obtained from the combined laboratory data.

possible that a plastic analysis (Jurgenson, 1934) would be the critical criterion for the stability of this embankment. A large plastic zone was found to exist between the depths of 11.5 feet and 20.5 feet if the laboratory shear strengths were assumed. Figure 17 presents the results of the plastic analysis. It should be pointed out that the plastic zone of Figure 17 is completely in the zone of the very sensitive soils. Although the soil between the plastic zone and the fill interface is only moderately sensitive, it contains the greatest amount of organic material. With the plastic zone developed so extensively in the horizontal direction, it seems quite possible that the plastic zone could have relieved itself by developing a plastic deformation of the upper layer or partial failure. It is also very possible that this sensitive soil lost part of its shear strength after being deformed in a plastic state for an appreciable period of time. This could have caused a partial failure of the embankment. Neither the nonuniform properties of the organic matter nor the reduction of shear strength due to the existence of a large plastic zone in a sensitive clay can be directly accounted for in the $\phi = 0$ slip-circle analysis, but they can be provided for in the factor of safety.

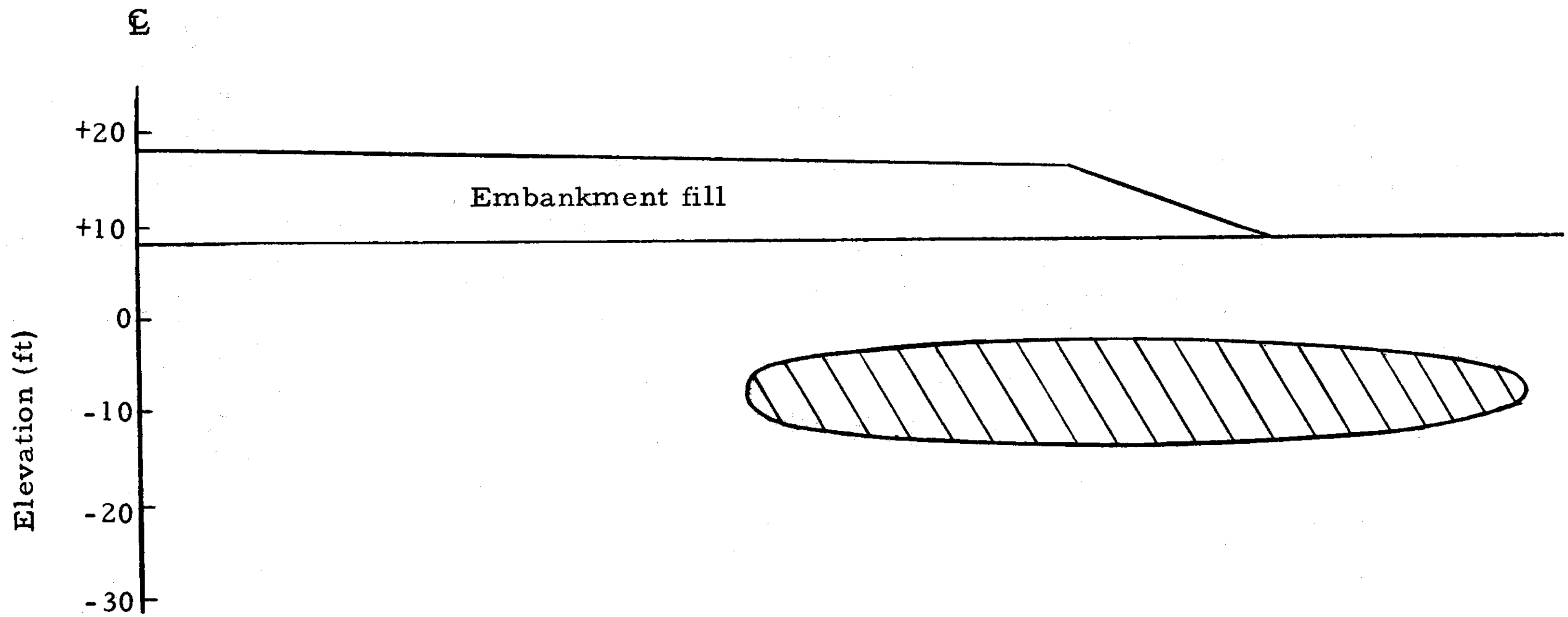


Figure 17. The plastic zone developed under the embankment

DISCUSSION OF THE INVESTIGATIONS

The preliminary field vane test results simply did not represent the actual undrained shear strength of the subsoils at the Glen Aiken Creek embankment failure site. Had these field vane test results been used for the design of this embankment, the 22 foot high embankment would have appeared to be adequately designed. The possibility that the rate of rotation was affecting the vane shear strength determination was investigated and found to have only a negligible affect on the resulting shear strengths. The standard rate of 6°/min (Carlson, 1948) was originally used and is recommended for reasons of efficiency and convenience. Since the vane shear strength is basically a measurement of the shear strength on the vertical plane, the relative magnitudes of the vertical and horizontal shear strengths were investigated with the laboratory vane. The results of this investigation were too erratic for any sound conclusions, but Aas (1965) has shown that the shear resistance on the vertical plane is the minimum shear resistance for normally consolidated soils. Organic soils, though, are usually bedded horizontally and the plane of minimum shear resistance is normally along the bedding plane, if there is a bedding plane. Samples taken at this site did not indicate any consistant beding plane of the organic matter, and, therefore, the shear strength determined from a vertical failure surface would be representative of the minimum shear resistance of the soil. The possibility of the soil expanding during the undrained shear strength test was eliminated after obtaining an A_f of $+0.9$ in the undrained triaxial test. Since all results of this investigation seem to indicate that the field vane test should have produced a representative shear strength, the explanation of the actual results must have involved the mechanics of the method used for the measurement of the shear resistance.

The method used to determine the net torque developed on the vane included two separate torque measurements. The torque required to develop the maximum resistance of the soil against the movement of the vane and its torque rod was measured first. A rod which had the same dimensions as the torque rod was then forced to the same depth at which the vane was located. This rod ("dummy" rod) was then rotated several times before the torque was measured. The difference between the torque developed with the vane and the torque developed with the "dummy" rod was used to determine the shear strength as shown in Equation 1. Figure 18 shows a very simplified picture of what was probably being measured. The solid line represents the stress versus strain plot of the soil adjacent to the vane and its torque rod. The dashed line represents the stress versus strain plot of the remolded soil adjacent to the "dummy" rod. It is evident from Figure 18 that the remolded shear resistance

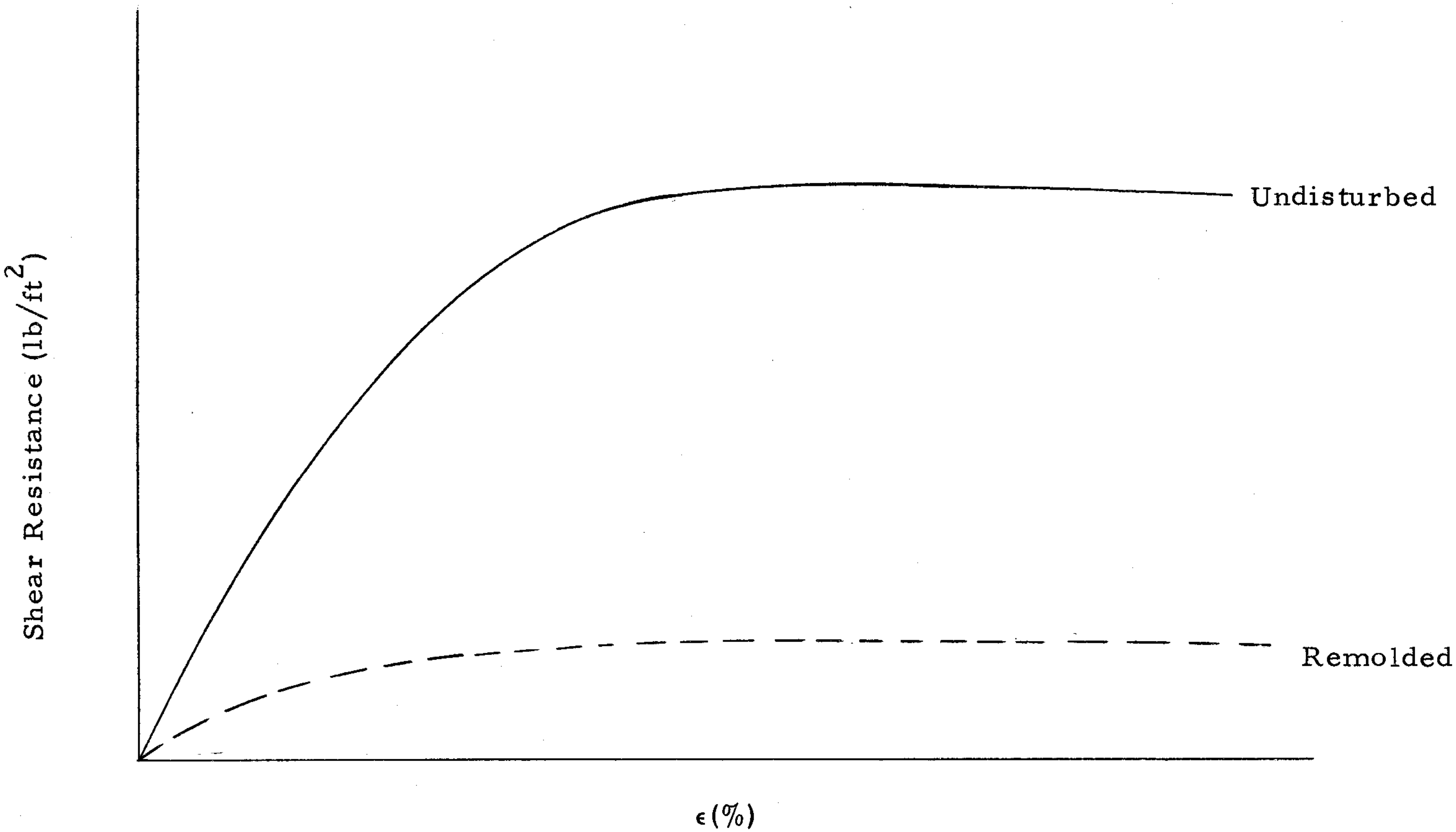


Figure 18. The simplified relationship between the stress-strain plot of undisturbed and remolded soil.

measured with the "dummy" rod is not representative of the shear resistance developed by the torque rod. Since these soils were sensitive, the remolded shear resistance was only a fraction of the undisturbed shear resistance. Therefore, the net torque (the total torque minus the torque of the torque rod) was being consistantly overestimated by the field vane.

In an attempt to reduce the uncertainties and the misrepresentation of the shear resistances measured with the field vane, a modification of the normal field vane apparatus was made. This modification consisted of using a slip-joint located two feet above the vane in the torque rod. This slip-joint allowed the torque rod to develop the full undisturbed shear resistance in the soil adjacent to it before the vane was coupled with the torque applying apparatus. Therefore, the actual magnitude of the rod friction was measured up to the point at which the vane began to rotate. The difference between the total torque measured and the rod friction torque measured by this modified field vane was then used to determine the shear resistance as shown in Equation 1.

The second series of field vane tests performed at this site was done with the modified slip-joint field vane. The results of this investigation indicated that the 22 foot high embankment was definitely unstable. The modified slip-joint field vane shear strength results were about half the magnitude of those originally determined with the field vane. It is, therefore, recommended that the torque rod friction either be eliminated or measured "directly." The effective elimination of the torque rod friction is obtained with the Swedish vane apparatus (Fenske, 1956).

The modified slip-joint field vane will not produce an accurate measurement of the sensitivity of a soil. This is due to the fact that the modified field vane must measure both the total resistance and the torque rod frictional resistance to determine the net shear resistance of the vane. Since the remolded shear resistance of sensitive soils is normally less than the torque rod frictional resistance, the magnitude of the possible errors involved in the determination of the two resistances as compared to the magnitude of the remolded shear resistance makes it impossible to obtain an accurate value of the sensitivity. The Swedish vane has been successfully used to determine the sensitivity of a soil (Skempton, 1948 and Fenske, 1956), but it must be remembered that the Swedish vane apparatus effectively eliminates the torque rod friction.

Since the settlement records at station 172+00 indicated a partial failure occurred under the fill height of ten feet, an analysis of the stability of the ten foot high embankment was made with the shear strength results of the modified field vane. A factor of safety of 1.55 was obtained for the $\phi = 0$ slip-circle analysis of the embankment. A factor of safety of 1.41 was obtained for the $\phi = 0$

slip-circle analysis of the embankment by combining the results of both the laboratory vane tests and the unconfined compression tests. Actually, the factor of safety of this ten foot embankment was approximately 1.0 as was indicated by the field settlement records. Since both the field vane and the laboratory data indicate approximately the same degree of stability, it must be concluded that the limitations of the $\phi = 0$ slip-circle analysis are responsible for the discrepancy.

The field vane test has been shown to be an inexpensive and efficient test, but it should not be used without the knowledge of the type of soil being sheared. The knowledge of the magnitude of the permeability, the type of soil, the amount of stratification, the direction of stratification, the sensitivity, the amount of organic matter present, the type of organic matter present and the location of the drainage layers is required for the logical analysis of the stability of a given site under a given set of loading conditions. The amount and quality of the tests performed at a given site depends entirely upon the importance of the project, but the "minimum amount of knowledge required for the interpretation of the field vane test results would require disturbed samples of the substrata for soil identification and classification purposes.

CONCLUSIONS

1. The practice of using the remolded frictional resistance measured with the "dummy" rod for the determination of the rod friction of the field vane is both inaccurate and misrepresentative. It is, therefore, recommended that the torque rod friction either be eliminated or measured directly with the modified slip-joint field vane. The modified slip-joint field vane yields an accurate determination of the undrained strength of soft soils.

2. At sites where the subsoils are organic or where the subsoils are notably sensitive, the factor of safety obtained from the stability analysis should be 2.0 or greater. This increase in the minimum factor of safety is required to compensate for a possible progressive failure.

3. The sensitivity of a soil cannot be accurately determined with the modified slip-joint field vane. Only by eliminating the torque rod friction, can an accurate <u>in situ</u> sensitivity measurement be made.

4. Classification tests on samples from a boring adjacent to the point at which the field vane is advanced into the subsoils are the minimum additional tests necessary for the rational application of field vane data to stability calculations.

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