DEPARTMENT OF SOILS

Use of Vane Borer on Foundation Investigation of Fill

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THIS paper describes use of the vane borer on the foundation investigation for a fill across an arm of Lake Pend O'Rielle near Sandpoint, Idaho. The tests were made from a barge at various depths ranging from 18 to 90 ft. below the bottom of the lake. The vane borer was driven by churning and washing, and it was not retracted from the boring after each shearing strength determination. The vane was driven to the desired depth, and the driving apparatus was removed; the torque apparatus was attached, and a test was made. After the test, the torque apparatus was removed, the driving apparatus attached, and the vane was driven to the next desired depth, with this procedure being repeated for each new boring.

The vane borer is essentially a four-wing vane connected to the end of a shaft. The test was made by applying a torque at the upper end of the shaft, causing the vane to rotate in the soil. The shearing strength was determined by the maximum torque applied and assuming the shearing surface to be that of a cylinder whose dimensions are equal to the dimensions of the vane.

Undisturbed samples were taken for unconfined compression tests and consolidated-quick triaxial compression tests (Q_c). These samples were taken from a boring at a distance of approximately 20 ft. from the vane borer boring and at the same elevations at which vane tests were made. The results of the laboratory tests and the vane borer tests show that a good correlation exists between them and that the vane borer is an accurate, economical tool for determining the shearing strength of fine-grained soils. The time required to make tests with the vane borer is approximately equal to the time required to take undisturbed samples and the results of the vane tests are readily computed in the field, saving many hours of work in the laboratory.

● CONSTRUCTION of a new crossing of the outlet of Lake Pend O'Reille at Sandpoint, Idaho, has been contemplated for many years. The present structure is an untreated timber pile structure over 10,000 ft. long and constructed approximately 20 years ago. An aerial view of the present highway and railway crossings are shown in Figure 1. The highway bridge is the upper-right crossing.

The Materials Division was given the responsibility of determining the feasibility of constructing a fill into the lake to reduce the length of the new structure. The water is quite shallow on the north side of the lake, and the entire area has a sand cover eight to 10 ft. thick underlain by a varved clay to a depth exceeding 140 ft.

In 1945 an investigation was made along Route A which lies between the present highway bridge and the Northern Pacific Railroad bridge via Sandy Point. Disturbed samples were taken along this route and submitted to the testing laboratory at Boise for routine tests. The results of these tests indicated the material would make a poor foundation for a fill or bridge. It was concluded from these tests to make further studies along Route A to determine the bearing, shear and consolidation characteristics of the foundation material.

During the summer of 1950, numerous undisturbed samples were taken and tested in the laboratory at the University of Idaho. Triaxial-compression and unconfined-compression tests were made to determine the shear strength of the clay. The results of these tests varied from a shearing strength of 400 lb. per sq. ft. to 600 lb. per sq. ft., and they showed practically no correlation of shearing strength with increased depth. Preliminary calculations showed that to provide for a stable fill 38 ft. high, if the shear strength of the clay foundation material was 400 lb. per sq. ft., approximately 38,000 cu. yd. of fill material per station would be required. If the shear strength was 600 lb. per sq. ft., the required yardage was reduced to approximately 13,000 cu. yd. or approximately one-third of that required in the first instance.

The conclusions of these tests and computations were that a more-detailed investigation should be made to determine more accurately the range of the shear strength of the clay. It was decided that vane shear tests on the material in place showed promise of verifying laboratory tests and might give more reliable results than laboratory tests on this material. Due to the fragile nature of the soil structure it was felt that a test giving the least manipulation and disturbance would be most desirable. Though no record could be found of the vane borer being used as a design tool in the United States, it was decided that it offered good chances for success in solving this problem.

Figure 1. Highway bridge and railroad bridge and fill across arm of Lake Pend O'Reille.

DESIGN OF THE VANE BORER

Prior to attempting the design of a vane borer all of the recent published literature was thoroughly studied. In this country the only known work with the vane was done by Vey and Schlesinger of the Illinois Institute of Technology (*2*). Their investigation included vane tests in three holes and correlation with laboratory tests of undisturbed samples from these holes. The Royal Swedish Geotechnical Institute has done considerable work with the vane, and in 1950 they published an excellent report *The Vane Borer* written by Lyman Cadling and Sten Odenstad (*1*). Included in this report were results of studies made to determine the size of vane, the number of wings for each vane, and the speed at which the test should be conducted. Odenstad and Cadling concluded that a vane with four wings gave consistent results with a minimum of disturbance, and that the height of the vane

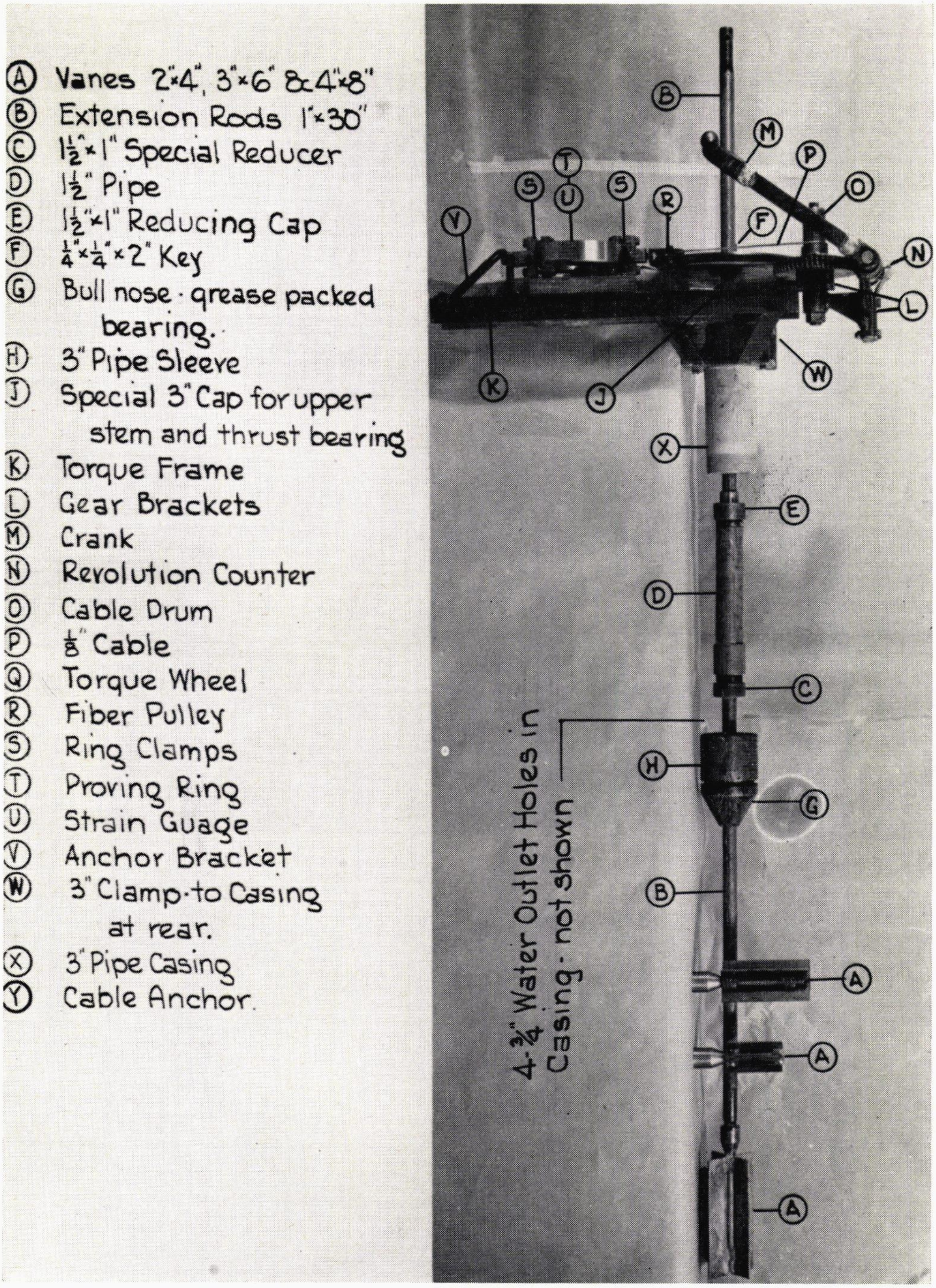

Figure 2. Vane in extended position with portion of 3-in. casing removed.

should be at least twice the diameter to minimize the disturbance at the ends.

Following these recommendations, a vane borer was designed and constructed as shown in Figure 2. Three vanes were made, a 2- by 4-in., 3- by 6-in., and 4- by 8-in. The vane is sections of the 3-in casing. During the driving operation, the torque drum and upper extension rod are removed and a driving head is attached to the 3-in. casing with the vane resting against the bull-nose point. After driving to the desired depth, the driving head is

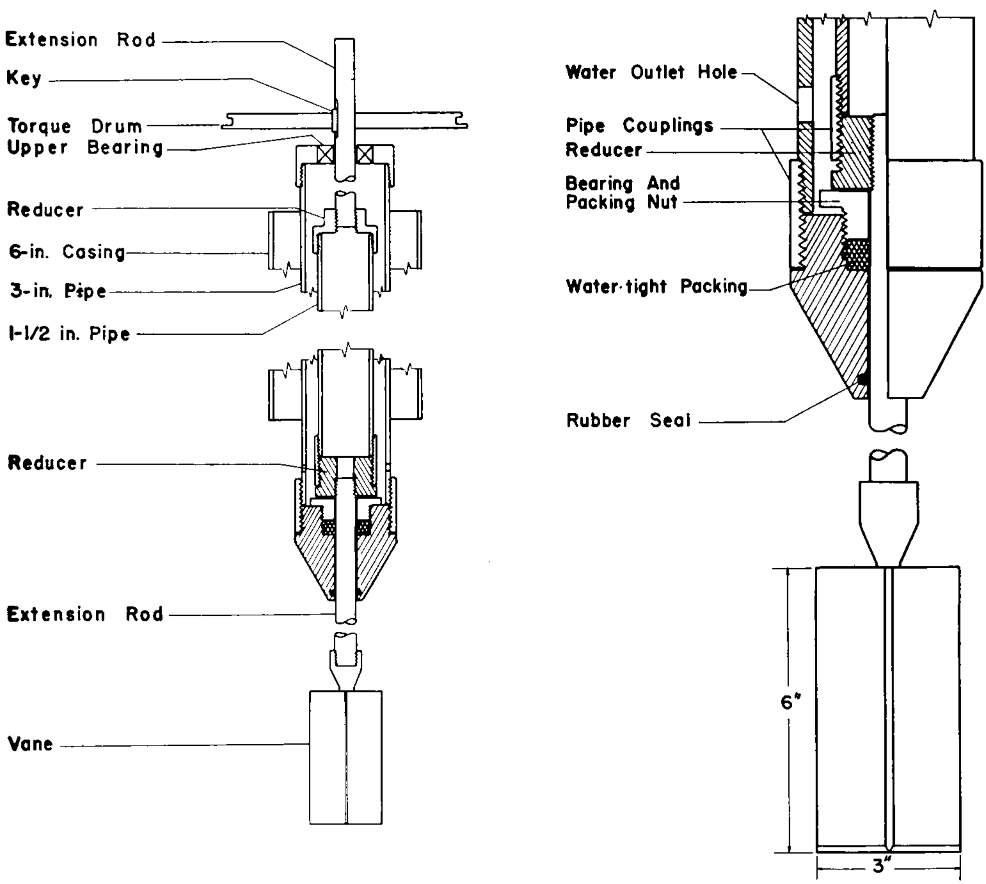

Figure 3. Sectional drawing of the vane borer.

attached to a 1- by 30-in. extension rod (*B*) which goes through a bull nose greased packed bearing (*G*) and attaches to a 1½-in. galvanized iron pipe (*D*). Another 1- by 30-in. extension rod (*B*) which goes through the upper guide bearing and torque drum is attached to the top of the 1½-in. pipe. A 3-in. casing (*X*) attaches to the bull nose and the upper guide bearing enclosing the drive shaft. The vane may be adjusted for different depths by adding sections of the 1½-in. pipe and removed, the upper extension rod is attached to the drive shaft, and the vane is advanced 30 in. beyond the bull nose point into undisturbed material by jacking. The upper guide bearing, the wheel and torque apparatus are attached. A cable is strung from the torque wheel through a pulley attached to a proving ring and thence to a hand winch. Strain is applied at a uniform rate and readings are taken until the test is complete. Figure 3 is a cutaway section of the vane borer. Figure 4 is the plan view of the torque apparatus and drum.

COMPUTATION OF SHEARING STRENGTH

It is assumed that the shearing surface will be in the form of a cylinder whose dimensions will be equal to that of the vane. This will not always be true since gravel particles and the hardness of the clay will affect the shearing surface. If the unit shearing stress of the soil is S, and it is assumed that the unit shearing stress is uniform over the sides of the cylinder and at each end it varies from S at the edge to zero at the center then the resulting moment will be

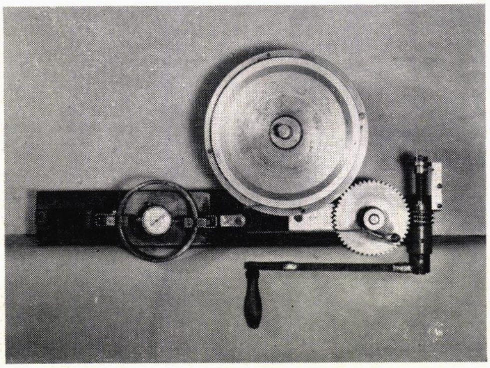

Figure 4. Plan view of the torque apparatus.

$$M = \frac{9}{8}\pi D^3 S$$

If the unit shearing stress is assumed to be uniform over the entire surface and ends of the cylinder the resulting moment will be

$$M = \frac{7}{6}\pi D^3 S$$

and the percent difference, assuming the shearing stress constant over the entire surface as compared to the assumption of variable shear at the ends, is only about 3.6 percent. The percentage difference is very small and in our calculations it was assumed that the shearing stress was uniform over the entire surface of the cylinder.

FIELD OPERATIONS

All tests were made in the lake bed from a barge fashioned from two Army pontoons. An A frame was mounted on the barge, and a Universal Prospector drill rig was used for lifting and churning the vane borer. To obtain penetration of the borer, it was first attempted to drive the entire casing, vane and stem without the use of water for working out displaced material. After several trials, we saw that it was impossible to obtain penetrations over 40 ft. this way, that pulling of casing was difficult, and that equipment and casing would not stand the necessary driving. The procedure adopted and used was to place a 6-in. casing through the sand cover to permit the free return of water to the surface and then to wash and churn down the vane stem and casing, being careful to maintain a continuous return of wash water to the surface. In order to be sure that tests were made in material unaffected by water pressure and disturbance from the churning operation, stem and casing were churned down a short distance after the water was shut off and then pried down 1 to 2 ft., before extending vane. This method permitted relatively easy penetration, and withdrawal of borer to depths of 90 ft. below the ground surface.

After a set procedure had been determined, the tests proceeded very rapidly. The following step-by-step description of a typical testing operation may serve to further describe the borer and its use:

1. With torque-measuring apparatus and upper extension rod removed, the torque pipe is adjusted to extend about 3 in. above the top of the casing when the vane is in a retarded position against the casing cap or bull nose, to permit use of wrenches for tightening.
2. Extend the casing to clear the end of the torque pipe and by means of a suitable head attach water and lifting rope. Churn to the desired depth being careful to shut off the water a sufficient distance above elevation to be tested to permit proper seating of the casing in undisturbed material.
3. Disconnect jetting and churning apparatus, remove temporary casing extension, and pry casing down to seat in undisturbed clay. Attach upper extension rod to torque pipe and carefully pry stem down to extend the vane approximately 30 in. into the undisturbed material ahead of the casing.
4. Slide cap with bearing over the extension rod and screw to the casing. Slide the torque wheel over the upper extension rod.

Clamp the torque-measuring apparatus to the casing and connect the cable to the torque wheel and cable drum. Tighten the cable by turning the crank until tension begins to record on the gage. Release the tension to the zero reading. The equipment is now ready for the test.

5. The test is made by uniformly turning the crank at six revolutions per minute. Readings of the number of turns and the proof ring gage are taken to obtain data for a stress-strain curve. After peak and ultimate values have been obtained, the vane is rotated by hand through five or six revolutions, and readings are then made to determine the disturbed or remolded strength.

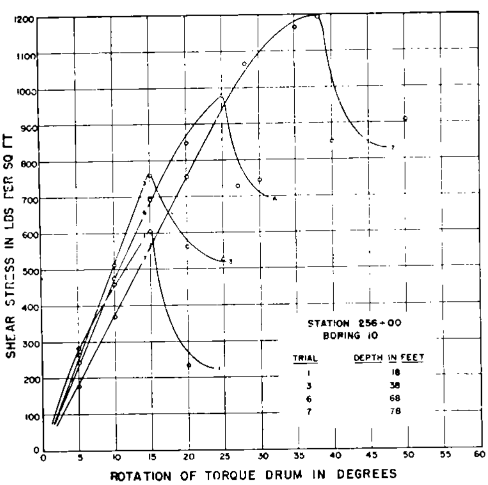

Figure 5. Typical stress-strain curves for the individual vane tests.

6. Apparatus including the upper extension rod is then removed and the stem and casing added for the next penetration, and the procedure is repeated.

Typical stress-strain curves are shown in Figure 5. The rotation of the torque wheel is not exactly the same as the rotation of the vane itself, since it includes the angular rotation of the shaft and any possible slippage in the joints.

The amount of torque required to overcome friction in the vane borer was an item of considerable concern. Figure 6 shows results of tests at various depths made without the vane in place. These values indicate the friction of the entire assembly. As shown on Figure 6, a curve was drawn to be used as a correction for each vane test.

The possibility that there was considerable lateral deflection in the vane stem to permit friction between the stem and inside the casing was considered, and it was decided to raise the vane 1 in. after it had been pried into testing position. This would cause the vane and stem to be supported at the top bearing. Tests without a vane were made with the 1-in. raise and without the 1-in. raise to determine the friction in the borer. The friction in the vane borer was little affected by raising the vane and stem 1 in. before each test; therefore, this procedure was not used. On shear tests where the vane was raised 1 in. the increased disturbance due to raising the vane reduced the shear strength values materially.

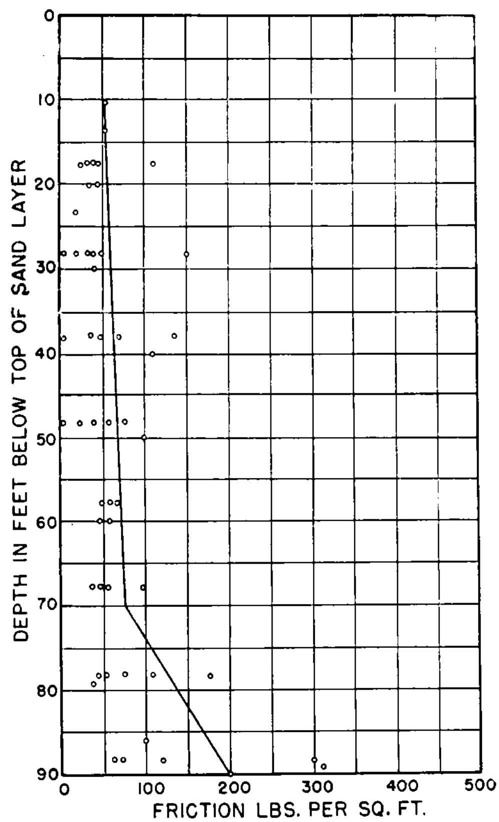

Figure 6. Determination of friction in vane borer.

Undisturbed samples were taken adjacent to the vane test holes so that correlation could be made with laboratory tests. The undisturbed samples were taken by churning a 4-in. casing down with a continuous flow of water, i.e., water forced into the casing returned to the surface along the outside of the casing carrying the disturbed clay material. When the desired depth was reached with the casing, churning was stopped and the casing was cleaned by allowing water to run until it was nearly clear. The sampler was then lowered to the bottom of the casing and forced down 6 to 8 ft. The sampler was then extracted from the hole, and a sample of approximately 3 ft. in length was cut from the aluminum sample tubing and prepared for shipment to the laboratory.

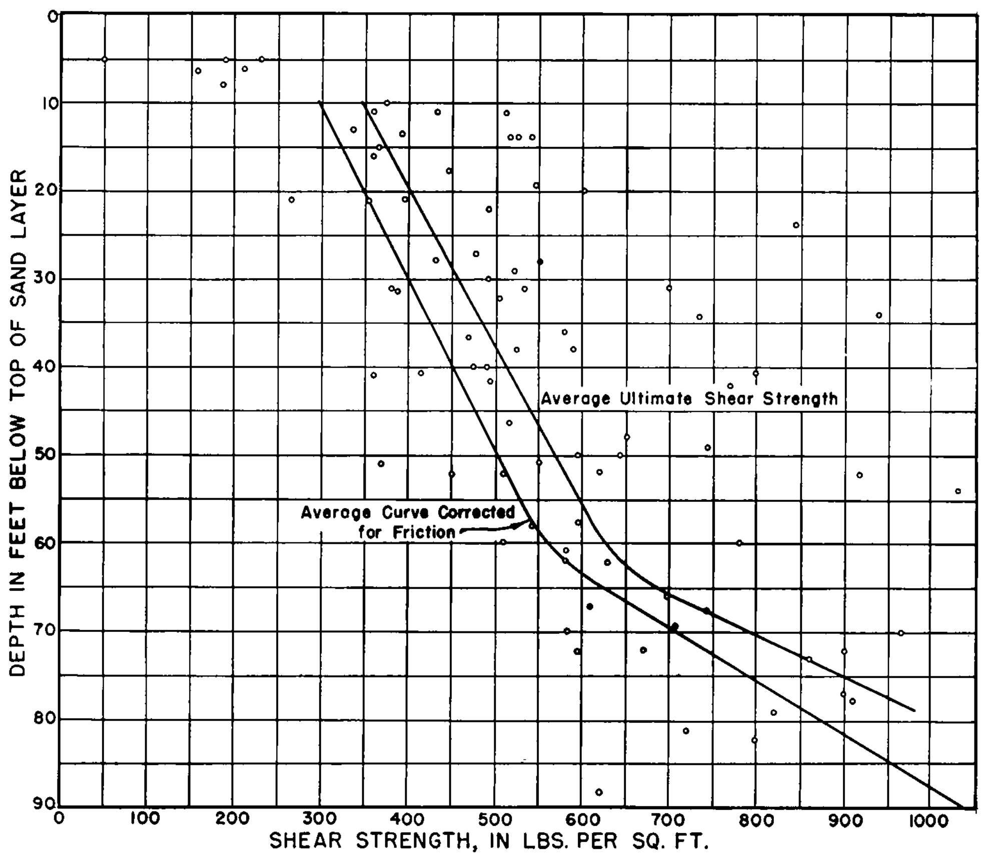

Figure 7. Ultimate shear strengths, vane borer.

DISCUSSION OF VANE BORER RESULTS

The data for each vane test was plotted as shown in Figure 5, and a stress-strain curve was drawn. The rotation of the torque drum in degrees was used along the strain axis, since it was easy to measure and is proportional to strain. The torque drum rotation includes the angular rotation of the drive shaft due to the applied moment. Including this angular rotation has no effect on the shear values, and since the purpose of the tests was to determine the shear strength of the soil, it was felt that the correction for the angular rotation of the drive shaft need not be made. The shear strength was picked from these stress-strain curves.

There was considerable discussion on the subject of using the peak-point shear strength values or the ultimate shear strength values for design. An attempt was made to select the ultimate shear strength, although in most cases few data were taken beyond the peak point. Figure 7 is a plot of ultimate shear strength versus depth for several borings. An average curve was drawn which shows the ultimate shear strength to increase with depth. The variations of the shear strength can be attributed to the variation in the clay layers and the increase in density with increased depth.

The peak-point shear strength can be taken from the stress-strain curve very easily, and the data is sufficient to accurately determine this value. Figure 8 is a plot of the peak-point shear strength versus depth for the borings along Route A. An average curve was drawn through these points; also a curve was drawn through the minimum peak-point shear strengths. These curves are corrected for friction by subtracting the amount of friction at different depths as determined from Figure 6. It was felt that the peak-point shear strength results could be used for the basis of a design by using the proper safety factor.

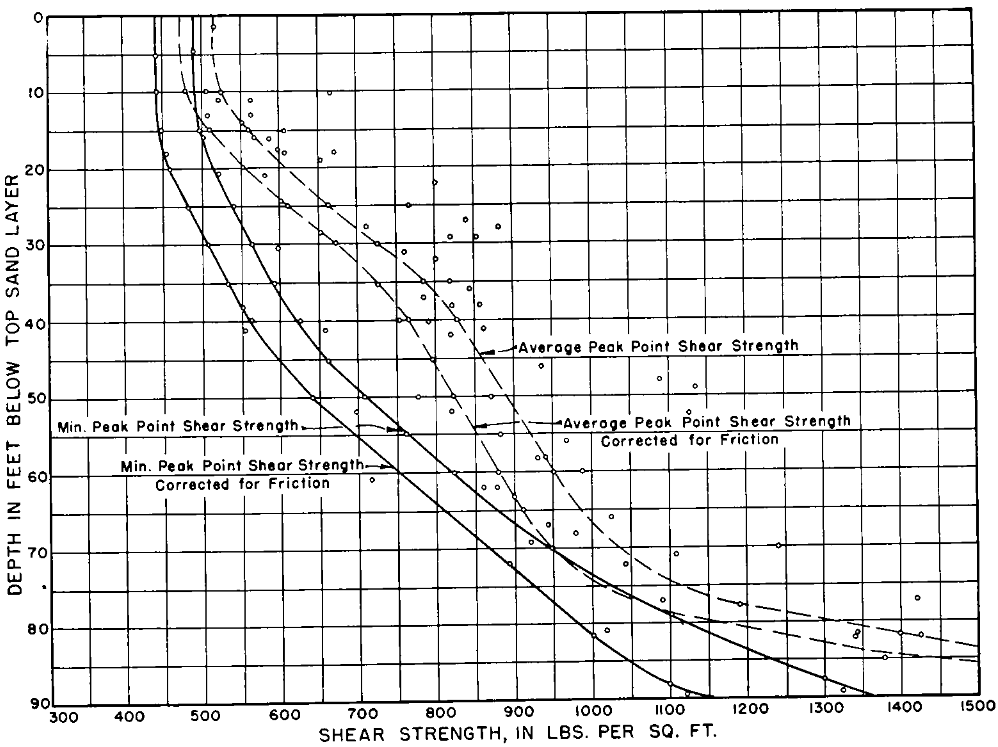

Figure 8. Peak-point shear strengths, vane borer.

LABORATORY TESTS

Undisturbed samples were taken to the laboratory and stored in a moist room until time of use. This moist room had a temperature range of 65 to 75 F. and a relative humidity of 100 percent.

Routine Tests

The liquid limit and plastic limit were determined on only a small number of the samples to show that the results were consistent with the tests from the 1950 investigation. The liquid limit was determined by using a specimen at natural moisture content, thoroughly working the specimen until the particle structure was destroyed, and a number of trials were made in the liquid limit device as the specimen lost moisture. The liquid limit, plastic limit, and natural moisture content are plotted versus depth in Figure 9 for boring No. 10. The curves indicate that the natural moisture content is greater than the liquid limit, and this is true only when the particle structure is completely destroyed. If, when determining the liquid limit, the specimen is worked just enough to place it in the liquid-limit device, the liquid limit is equal to the natural moisture content, which indicates that the amount of working or remolding of the specimen has a definite effect on the liquid limit value.

Shear Tests

Quick triaxial shear tests (Q) performed during the 1950 investigation indicated that the angle of internal friction was very near zero. The minor principal stress had no effect on the compressive strength of the specimen, and also the compressive strength of the samples did not seem to have any correlation with depth. The most probable reason for this was that the material is a clay interspersed with silt lenses of varying thickness, and these silt lenses, depending on the number in each specimen, affect the compressive strength. Unconfined compression tests (U) were made, and the results were identical to the results of the Q tests.

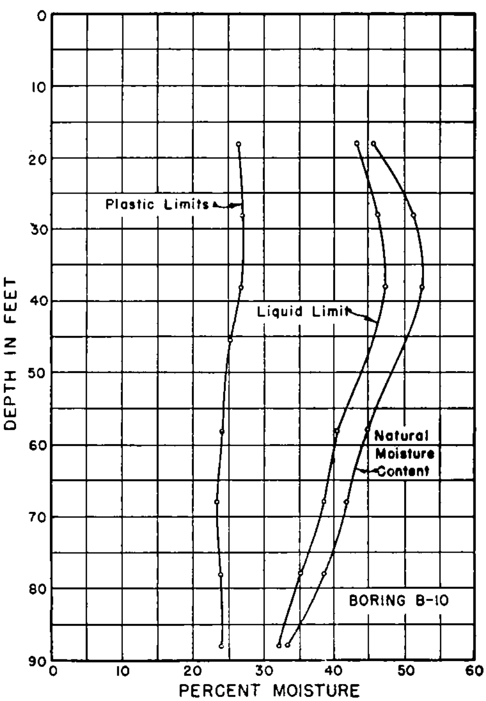

Figure 9. Variation in moisture equivalents with depth

Quick-consolidated triaxial shear tests (Q_c) were run on the samples from Borings 6 through 13 from the 1951 investigation. Each specimen was carefully placed in the chamber of the triax machine; the chamber was filled with glycerin to the top of the sample; all air bubbles were removed from the lines to the manometer; and the minor principal stress was applied by air pressure to the top of the glycerin. The sample was allowed to consolidate a sufficient length of time to reach approximately 100 percent consolidation before the Q_c test was run. Change of volume versus time data was taken during the consolidation process and the percent consolidation time was determined by the square root of time fitting method. Three specimens from each sample were consolidated at different values of minor principal stress. The results of the Q_c tests again indicated the angle of internal friction to be near zero, and the minor principal stress had little effect on the compressive strength of the specimen. It was decided that only one specimen from each sample need be tested, and this specimen was consolidated to the computed intergranular pressure. The shearing strength was taken as half the compressive strength ($\frac{1}{2}[\sigma_1 - \sigma_3]$). The compressive strength was not corrected for the resistance offered by the rubber membrane.

The shearing-strength profile for the Q_c tests is shown in Figure 10 for Borings 6 through 13. The shearing strength increases with depth. Many of the points deviate from the average curve and a probable reason for this is that the silt layers increased in number and thickness with depth. The density also increased with depth which is another reason for increased shear strength. The samples were all found to be 100 percent saturated, which was natural since they were taken from the bottom of the lake.

COMPARISON OF LABORATORY AND FIELD TESTS

The shear strength profile for vane tests of Borings 6 through 13 is compared with triax (Q_c) tests in Figure 10. The vane results are somewhat less than the Q_c results, but the curves are similar. It must be kept in mind that the Q_c test results are not corrected for the resistance of the rubber membrane.

The shear strength profile for the vane and Q_c tests was started at 18 ft. below the bottom of the lake. A 10-ft. layer of sand and 8 ft. of the clay are above this. The profile shows a gradual increase in shearing strength from 20 to 80 ft., and below 80 ft. the shearing strength increases quite rapidly. This agrees with the boring log and data which show the clay to

80 ft. were 6 in. thick. The results of the Q_c tests and the vane tests show a very good correlation.

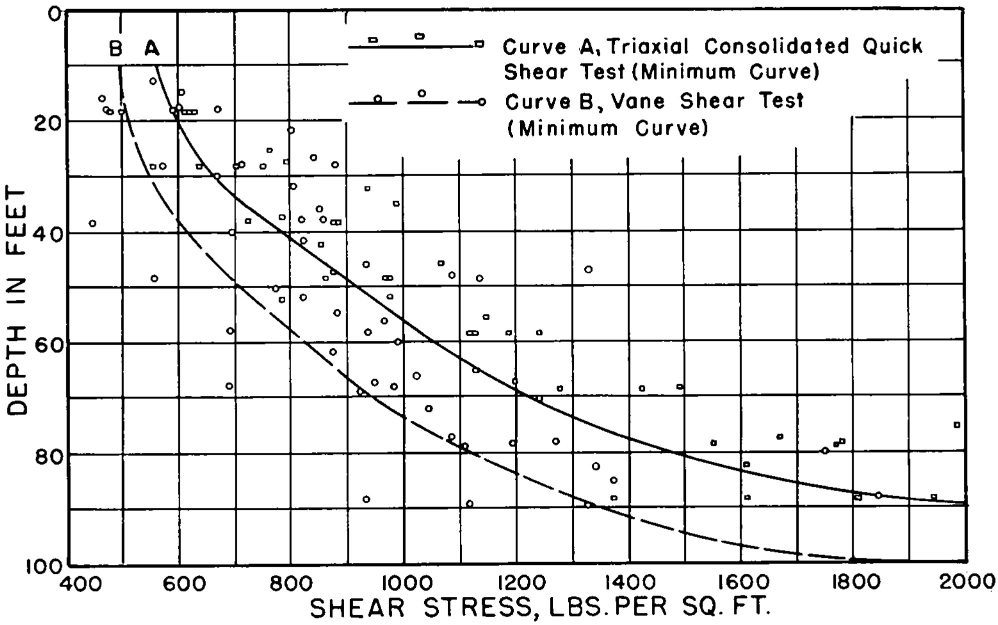

Figure 10. Shear strength versus depth, vane and triax.

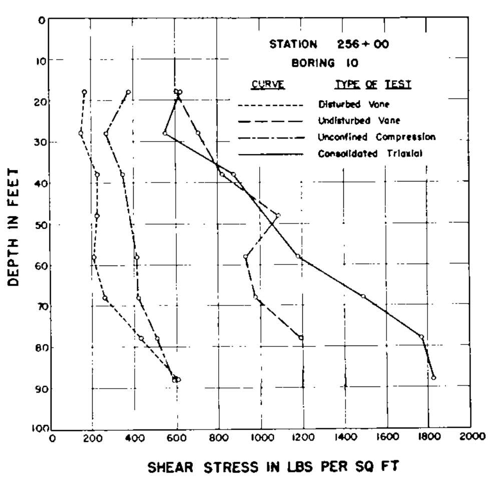

Figure 11. Comparison of shear strengths as determined by vane and laboratory tests, Boring 10.

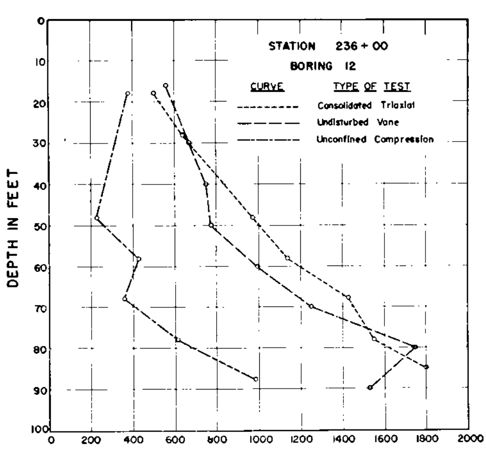

Figure 12. Comparison of shear strengths as determined by vane and laboratory tests, Boring 12.

increase in density with depth, and at a depth of 80 ft. the silt layers began to increase in thickness. Some silt layers at depths below

Unconfined-compression tests were run on samples from Borings 10 and 12. Figures 11 and 12 show the results of these tests compared to the Q_c and vane tests. The *U* test followed a curve very similar to the disturbed vane results. Q_c and undisturbed vane results were much higher than the *U* results. Figures 11 and 12 also show typical correlation of vane test and consolidated quick triaxial test data.

CONCLUSIONS

The results of the vane tests indicate that the vane borer is an accurate, economical tool for determining the shear strength of clay soil. The time required to make a test with the vane is about the same as that required to take undisturbed samples. The results of the vane tests are readily computed in the field which saves many hours of work in the laboratory. In soil investigations of stability problems, the vane borer will economically allow a much more extensive testing program than conventional methods involving undisturbed samples and laboratory testing.

Our investigation indicated that results of the vane are lower than the results of the Q_c test, but are higher than the unconfined-compression tests. Cadling and Odenstad investigated 11 different slides making laboratory tests on extracted samples, vane borer tests, and calculating shear strengths developed by the slides. In all cases the vane results compared very closely to the calculated shear strength while the unconfined compression results were much lower. No comparison was made with Q_c test results. The Cadling and Odenstad investigation indicates that the results of the vane tests are accurate and valid.

The Idaho Department of Highways is using the data from the vane tests for the stability analysis of the fill on the Sandpoint Bridge relocation project. The use of the vane borer on this project has allowed a more extensive testing program. Undisturbed samples were taken from eight borings with an average of six samples per boring for laboratory testing to correlate with vane results. The time required to do the testing in the laboratory was much greater than the time required for an inexperienced crew to make vane tests in 30 borings with approximately 10 determinations per boring. Although no actual cost data is available, it may be concluded that the vane borer is a very economical method of determining shearing strengths of fine-grained soils. We estimate that at least three vane tests can be made in the time equivalent to one undisturbed sample taken and tested in the laboratory.

It is felt that a portable unit, perhaps mounted on a light truck or power wagon, would be extremely effective in investigating marsh conditions. The University of Idaho is cooperating with the Idaho Department of Highways on a special research project for developing such equipment. It is anticipated that this unit may become a routine tool for foundation investigations of fine grained soft materials. Also, tests will be made with different shapes of vanes to determine the shape of vane that will cause the least disturbance when being driven into the soil.

Work was started on this research project this past summer. The equipment is now in the fabrication stage. It is expected that testing will get under way this next summer, and the equipment will be ready to use on routine foundation investigations in the near future.

REFERENCES

1. Cadling, Lyman, and Sten Odenstad, "The Vane Borer," *Royal Swedish Geotechnical Institute Proceedings*, No. 2, Stockholm, 1950.
2. Vey, E., and L. Schlesinger, "Soil Shear Tests by Means of Rotating Vanes," Twenty-Ninth Annual Meeting of the Highway Research Board, Proceedings, Vol. 29, 1949, pp. 544–552.
3. Skempton, A. W., "Vane Tests in the Alluvial Plain of the River Forth Near Grangemouth," reprinted from *Geotechnique*, Vol. I, No. 2, December, 1948.
4. Evans, Ivor, "The Measurement of the Surface Bearing-Capacity of Soils in the Study of Earth-Crossing Machinery," reprinted from *Geotechnique*, June, 1950.
5. Evans, I., and G. G. Sherratt, "A Simple and Convenient Instrument for Measuring the Shearing Resistance of Clay Soils," reprinted from the *Journal of Scientific Instruments and of Physics in Industry*, Vol. 25, No. 12, December, 1948.
6. Couard, A., "Determination of the Angle of Internal Friction and of the Cohesion of Soils in Place by Means of Dynamic Drillings," *Le Genie Civil*, Vol. 125, No. 24, pp. 472–474, December 15, 1948.
7. Waterways Experiment Station, "Triaxial Shear Research and Pressure Distribution Studies on Soils," *Soil Mechanics Fact Finding Survey Progress Report*, War Department, Corps of Engineers, Mississippi River Commission, Vicksburg, Mississippi, April, 1947.

DISCUSSION

Harl P. Aldrich, Jr., *Assistant Professor of Soil Mechanics, Massachusetts Institute of Technology*—This writer has read with especial interest the authors' paper, not only because he is currently involved in a soil investigation where vane tests have been run, but because he is an alumnus of the University of Idaho and knows the Sandpoint area quite well.

The authors have pointed out that the vane borer, a tool used to determine the shearing strength of a cohesive soil *in situ*, has had but limited use in the United States. The authors' paper and the one by Vey and Schlesinger of the University of Illinois are the only published accounts, to the writer's knowledge, of the use of the vane borer in this country. The writer knows, however, of the following other instances of its use: (1) Leo Casagrande at the Fore River Bridge site at Portland, Maine in 1952; (2) Hamilton Gray at the Cousin's River Bridge site at Freeport, Maine in 1952; and (3) Donald W. Taylor at the Fore River Bridge site at Portland, Maine in 1952. In addition considerable laboratory work has been done at Harvard University utilizing a miniature vane borer. Table A summarizes briefly the results of these investigations.

TABLE A

Case	Description of Soil	Natural water Content	Atterberg Limits		Shearing Strength Comments
			L_w	P_w	
		%	%	%	
(a)	Very soft uniform dark gray clay	70	57	25	Vane tests gave a strength of about 260 psf. while the shearing strength from unconfined tests averaged 100 psf.
(b)	Soft blue-gray varved clay and silt	40	40	17	Vane tests and unconfined tests agreed closely and averaged about 700 psf.
	Blue-gray silty clay	38	30	21	Vane tests averaged 1,300 while unconfined tests gave about 600 psf. for shearing strength.
(c)	Soft blue-gray silty clay	32	40	22	Vane tests averaged about 950 and unconfined tests 700 psf. Data from a few tests indicated that the vane results were about midway between those from triaxial Q and Q_c tests.

Bennett and Mecham found that the vane test results are only slightly lower than those from Q_c tests but that unconfined tests and Q tests gave results only one third or half as large as vane tests.

There isn't space enough to name and discuss the many variables affecting results from various types of tests used to determine the *in situ* shearing strength. It is apparent, however, that the sensitivity of the soil, a measure of which is given by the relative values of the natural water content and the liquid limit, is a very important factor.

The vane borer is potentially a valuable tool for determining the *in situ* shearing strength of soft cohesive soil. Often the *in situ* strength is of primary importance in a practical problem. However, a most important consideration in many engineering projects, for example, where deep highway or railway fills are involved, is the added strength gained as a result of consolidation from the fill load. Furthermore, it is essential that the investigator know the speed at which this shearing strength builds up, in other words, the speed at which consolidation takes place. To determine these properties undisturbed samples may be consolidated in the laboratory to pressures representing different heights of fill and then tested in direct shear or cylindrical compression apparatus.

The authors report that they consolidated samples from several tubes to various values of minor principle stress, and found from the results of Q_c tests that the internal friction angle was nearly zero; in other words, there was little effect of the consolidation pressure on the compressive strength of the sample. This would be most unusual unless the varved clay stratum were highly precompressed which does not appear to be the case. It can be seen from Figures 11 and 12 of the authors' paper that the shearing strength of the soil as determined from Q_c tests varies almost directly with depth. In fact, if the submerged unit weight of the soil were assumed to be equal to 60 lb. per cu. ft., then the equation $S = \bar{\sigma} \tan 20°$ would very closely represent the shear strength envelope determined from Q_c tests. From this alone one would expect that if samples of the varved clay were consolidated to pressures, for example, twice as large as the overburden intergranular pressure, then the resulting shearing strength would be about double.

It is a most challenging soil engineering problem to determine the maximum rate at which a fill, to be constructed on a soft cohesive soil, can be placed. The problem becomes one of comparing the available shearing strength of the soil to the required shearing strength to give an adequate factor of safety. As the fill height is increased, the required strength of the underlying compressible material obviously increases. In addition, the available shearing strength is increasing nonuniformly as the soil consolidates under the fill load. The maximum rate at which fill can be placed may be determined by a combination of stability, consolidation, and shearing strength analyses. In many instances this maximum rate is greater than the rate at which the fill can actually be placed.

From preliminary studies, the authors found that 38,000 cu. yd. of fill were required per 100 ft. of abutment if the shearing strength was 400 psf. which is nearly three times the quantity required if the shearing strength of the foundation material was 600 psf. This difference in required yardage appears to the writer to be unusually large. Nevertheless, it would seem possible that the varved clay might consolidate fast enough as the fill was placed to make up the 200-psf. deficit in shearing strength. If not, the use of vertical sand drains to speed up the consolidation may be a promising solution. Sand drains are ideal for use in soft, varved clays, since the horizontal permeability is large compared to the vertical. The writer believes that sand drains would compete economically with the added yardage in this case.

In closing, this writer would like to make a few comments regarding the use of the vane borer for the determination of the *in situ* strength: First of all, the shearing strength determined by the vane depends to a certain extent on the speed at which the vane is rotated. Accompanying the rotation are increased pore water pressures, and consolidation will occur as water flows from the vane area to the surrounding soil. This effect is difficult to account for quantitatively. It is most pronounced where the horizontal permeability is high and when the speed of rotation is slow.

The second comment regards friction in the vane assembly. By running tests with the vane removed, the authors estimated that the effect of friction was to give shearing strengths 50 to 100 psf. greater than the actual value. This approach has been used by other investigators. Since the vane was removed the writer wonders if this gives a true picture of the friction which would occur if the vane were in place. Because of the nonhomogeneity of soil, in many cases bending will occur in the vane stem as the borer is pushed into the soil, and additional friction and binding may result at the bearings.

This writer does not expect the vane to ever replace conventional methods of obtaining undisturbed samples in a soil investigation. To begin with, there are too many cases where the determination of the *in situ* shearing strength comprises but a small part of the investigation. Secondly, soil engineers should not be satisfied unless they can look at the soil and study it in the laboratory. Finally, it is recognized that the vane borer has application only to fairly soft plastic soils. Nevertheless, the vane borer will probably become more widely used in the United States. Its use may be particularly warranted in cases (1) where the soil is unusually sensitive to disturbance during sampling and (2) where a clay stratum is unusually non-homogeneous and it is essential to explore extensively to locate soft zones in the deposit.

The authors are to be commended for pioneering in the use of the vane borer.

G. Bryce Bennett and James G. Mecham, *Closure*—We agree with Aldrich that the increase in shearing strength due to consolidation is of primary importance and must be considered in practical problems. Perhaps he has misinterpreted the purpose of this paper, which is to report the results of the vane borer as used on this project; and it does not include the results of all tests performed during this investigation. The results of laboratory tests which appear here are used as a means of comparison of vane-borer results. Other tests have been made on the material to determine the rate of consolidation and the increase in shearing strength as the consolidation takes place. Also, the possibility of using vertical sand drains has been considered, and it was determined that the cost was greater than the cost of the increased yardage required for loading berms.

The discusser points out that the shearing strength determined by the vane depends to a great extent on the speed at which the vane is rotated. It seems that regardless of the type of test, the speed at which the test is performed will affect the moisture movement within the specimen, which will in turn affect the final results. The scope of the use of the vane borer on this project did not include tests to determine the speed at which the test should be run. Cadling and Odenstad (*1*) performed a great number of tests in the study of the vane borer to determine the ratio of the diameter to height of the vanes. From their studies they recommended a speed of 0.1 deg. per sec. and a ratio of diameter to height of 1 to 2. These recommendations were followed in our testing program.

It is possible that friction in the vane assembly could be caused by bending of the vane stem when being pushed into the soil with a vane in place, whereas this bending would not be so pronounced with the vane removed. However, this would depend upon the stiffness of the soil, and in this particular case we believe that the bending of the vane stem with the vane in place would be negligible in the sensitive soils encountered.

We agree with Aldrich that the vane borer will never replace the conventional methods of obtaining undisturbed samples in soil investigations. The statement was made in this paper that the vane borer was considered for use because it might offer more reliable results than the conventional methods of soil testing. This statement was made with this project, and only this particular project, in mind in which the type of soil in question is quite sensitive. It is only logical that the test which will involve the least amount of manipulation and disturbance will give the most reliable results on clays of this type. Skempton (*3*) and Cadling and Odenstad (*1*) compared vane borer results to results obtained from studies of actual slides, and their investigations in these sensitive soils show that the vane borer gave results which compared closely to results which were computed from these slides; the results of conventional tests did not.

It would have been interesting indeed to have compared the vane borer results of this investigation to results of an actual slide. However, since a slide did not exist in this area, the vane results were compared to results of conventional laboratory tests.

The authors wish to thank Aldrich for his discussion and interest in the paper, and they are glad to learn of other instances of the use of the vane borer on foundation investigations.