VANE SHEAR TESTS IN
LAKE AGASSIZ CLAYS

A Thesis

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by

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ABSTRACT

A great deal of testing with vane shear equipment in homogeneous clay soils has been reported throughout the world. This thesis deals with the results of vane shear tests in the varved clay deposits of glacial Lake Agassiz in Manitoba.

The soil shearing resistance determined by a vane was compared, in most cases, to one-half of the unconfined compression test result for tube samples. The vane results from one test hole were compared to one-half the deviator stress as determined from triaxial tests.

In addition to the determination of the reliability of vane test results in varved clays, the suitability of the vane test equipment was also studied.

The test results indicate that the vane shear test will in general yield shear strengths in excess of those determined by unconfined compression tests on tube samples. On the basis of the test data presented in this thesis, the vane shear test appears to be a suitable means of determining the shear strength of varved clays although some refinements may be necessary in the testing equipment.

P R E F A C E

This thesis is an investigation of the suitability and reliability of the vane shear test in Lake Agassiz varved clays.

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CHAPTER I

INTRODUCTION

The main purpose of this thesis was to determine the reliability of shear strength values obtained by the use of vane shear apparatus in the varved clays deposited in Manitoba by glacial Lake Agassiz. This thesis presents the results of vane shear tests in Manitoba and a comparison with the results of unconfined compression and triaxial tests performed in the laboratory on samples obtained from the same location as the vane tests.

The determination of in-situ shearing strength of soil has been a subject of considerable interest for many years. Although several pieces of apparatus and methods of test have been used to determine this strength, there are probably only three basic types of tests presently in use. These are Penetration Tests, Pull Shear Tests and the Vane Shear Tests. The most popular of these has been the latter which, essentially, consists of driving a vane, usually having four blades, into the ground and measuring the torque required to rotate the vane and shear a cylinder of soil.

Since the shearing strength of a soil is so important to the Engineer in the design of dams, highway embankments, bridge and building foundations, etc., it is essential that this value be determined as accurately as possible. The accuracy and reliability of future testing methods may enable the designer to reduce safety factors which would reduce cost of many structures. Due to the complex nature of soil it is essential to investigate thoroughly any proposed foundation site. Depending upon the type and purpose of the foundation, dozens of test holes may be necessary. The vane tests provide a relatively quick method of strength determination and therefore can be used to supplement information obtained by sampling and laboratory testing of the soil, with little additional time consumption or expense.

Many papers have been published presenting data related to the vane-shear test, which is in limited use throughout the world. The data indicates that this method of strength determination may be more reliable for certain applications than those heretofore generally accepted.

Because the Lake Agassiz clay deposits are varved, a good correlation of the vane test results versus unconfined compression test results and the vane test results versus triaxial test results may not be possible. The varves consist of different soil types in varying thicknesses. Not only does the type of soil change distinctly from one layer to another but the moisture content can also vary considerably.

This investigation was carried out to determine whether a correlation similar to that obtained elsewhere on non-varved clays was evident in the glacial Lake Agassiz varved deposits.

It was also the purpose of this investigation to evaluate the suitability of the vane apparatus developed by the Manitoba Highways Branch.

CHAPTER II

A History of the Vane Shear Test

First recorded experiments with vane testing were performed by J. Olsson in Sweden in 1928 [1]. There is a German patent dated 1929 on this subject [1]. At the Third International Congress for Applied Mechanics, which was held in Stockholm in 1930, C. Forssel demonstrated a vane tester[1].

In Canada in 1941 a vane apparatus was used by A.W. McLoughlin on the investigations for the Welland Canal. It was used extensively in studies of slides on the Beauharnois Canal in 1942 and again in Sault Ste. Marie in 1943 [2].

In England the Army developed a vane in 1944 for testing the bearing capacity of soft ground in connection with tank mobility studies [3]. Work on the development of practical field equipment began in Sweden in 1947 under the Royal Swedish Geo-technical Society [1], and in England in 1948 under Bishop[3]. A paper describing tests and results of vane shear tests performed in Chicago glacial clays was published in 1949 [4].

Foundation Company of Canada, in 1950-51, developed a stress-strain device for use with a vane [2]. Information supplied by Geocon Limited, describing this latter device, was used in the construction of the equipment which was used to obtain the data for this thesis.

Tests with a vane in the foundation investigation for a fill across an arm of Lake Pend ' Oreille near Sand Point, Idaho, were undertaken in the early 1950's [5]. W. J. Eden and J. J. Hamilton used a field vane apparatus in determining shear strength in Leda clay deposits in Eastern Canada [6]. The results of these latter field tests were compared with those found by the laboratory testing of tube samples.

The work of Eden and Hamilton [6] was presented at the fifty-ninth annual meeting of the American Society for Testing Materials, at Atlantic City, N. J., on June 22nd, 1956. Work done by the Bureau of Reclamation, Denver, Colorado, was reported by Harold J. Gibbs [6] at the same meeting. A report by Carl W. Fenske on deep vane tests in the Gulf of Mexico and a description of a vane shear device developed by the Oregon State Highway Department, by William C. Hill also were presented at the 1956 meeting.

At the present time many provincial, federal, and state departments as well as consulting firms, are using vane shear information to supplement their regular soil test data.

CHAPTER III

GEOTECHNICAL PROPERTIES OF SOILS TESTED

The geology of the Lake Agassiz area in Manitoba is of special interest because dynamic geological processes have been responsible for marked textural variations in the parent material of the soil. As the result of glaciation during the Pleistocene period, a great deal of the area was covered by glacial drift or boulder till. The boulder till deposits ranged in thickness from less than 20 to over 200 feet [7].

From the beginning of the Pleistocene period when the ice began to melt, to the beginning of the formation of the existing soil, the glacial till was modified by geological agencies other than ice. In the Lake Agassiz basin, the original till was modified by the waters of the lake, and in addition, detritus from the higher lying regions was carried into the lowlands as a result of erosion and stream transportation.

When glacial Lake Agassiz was at its greatest height, the water in the vicinity of Winnipeg must have been between 550 to 600 feet in depth [8]. The waves of the lake caused the erosion of the drift and till along the shore lines and in the shallows. The finer materials removed by the water were transported and deposited to the deeper sections of the lake bottom. The accumulation of these sediments resulted in stratification of the lacustrine deposits in many sections of the basin, and varves of colloidal clay here occur with thin layers of coarse clay and silt or very fine sand. As the lake receded and its depth decreased the materials stirred and transported were the finer fractions consisting mainly of fine sand, silt and clay. When the lake had receded to the central lowlands, very fine materials were brought in by streams and tributary lagoons and deposited in the quiet waters of the lake basin.

As the lake reached its final stages, it began to receive alluvial sediments. These sediments are found spread over large areas of the lake bed. The thickness of these water-laid sediments differs greatly. The superficial deltaic and lacustrine materials in the central lowlands range up to 60 or more feet thick. The fine sand and silty layers outside of the central basin range in thickness from several inches to more than ten feet, and rest either on unassorted till or upon earlier lake deposits which are usually finer in texture than the surface deposits.

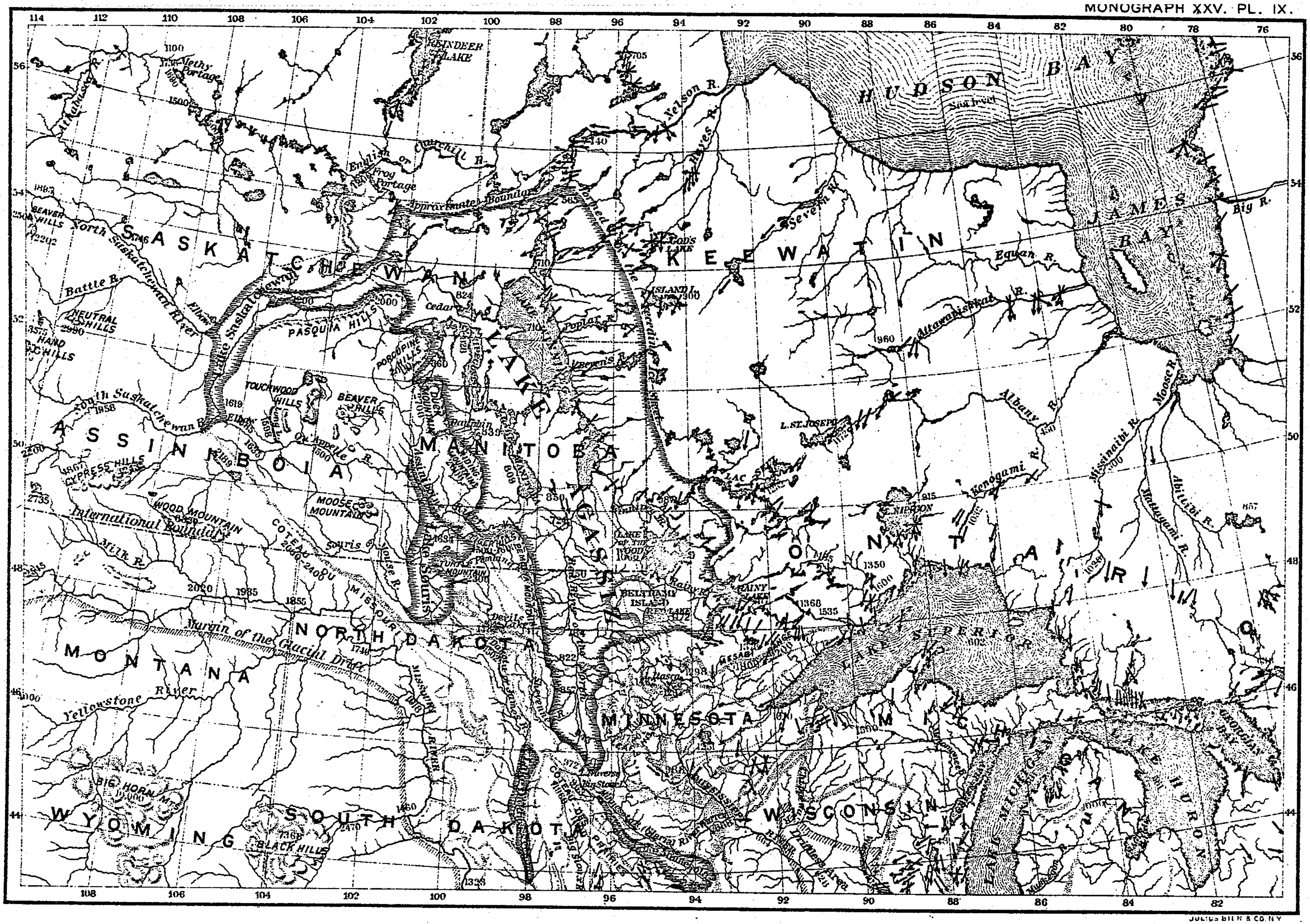
When the beaches were being formed along the western and southern shorelines of Lake Agassiz, glacial ice formed its eastern and northern boundaries. This ice sheet was not continuously retreating but rather oscillated backward and forward. In some cases as the ice advanced into Lake Agassiz the varved clays at the bottom of the lake were foliated and distorted [8]. This helps to explain why although the varves are generally found to be horizontal they are occasionally found in a vertical direction or at some angle between vertical and horizontal.

Figure 1 shows the approximate position and size of glacial Lake Agassiz. The area within which all the tests contained in this thesis were done is shown in Figure 2. This area is referred to as the Red River plain and represents only a small portion of the Lake Agassiz basin [9].

The Lake Agassiz clays generally fall into the classification of A-7-6 or A-7-5 clay soils and generally have a group index of 20. However, occasionally the soil encountered will have a group index slightly lower due to the presence of a higher percentage of silt than normally encountered.

With increasing depth the soil color generally varies from brown to olive or grey. The depth of the layers varies considerably. Because of the method of deposition of these various soil layers there is very little uniformity of layer thickness or soil type. The soil making up these different colored layers generally varies from silt or silty clay to clay with the silt fraction decreasing with increasing depth [10].

The varves themselves vary in thickness from 1/4 inch to as little as 1/64 of an inch. The varves generally consist of clay layers with intermediate silt or silty clay layers. It has been determined in other areas that considerable variations



MAP WITH ALTITUDES OF LAKE AGASSIZ AND ADJOINING COUNTRY.

Scale, about 165 miles to an inch.

Lake Agassiz and associated Glacial Lakes

Altitudes in feet above the sea 710

FIGURE 1. Map of Lake Agassiz.

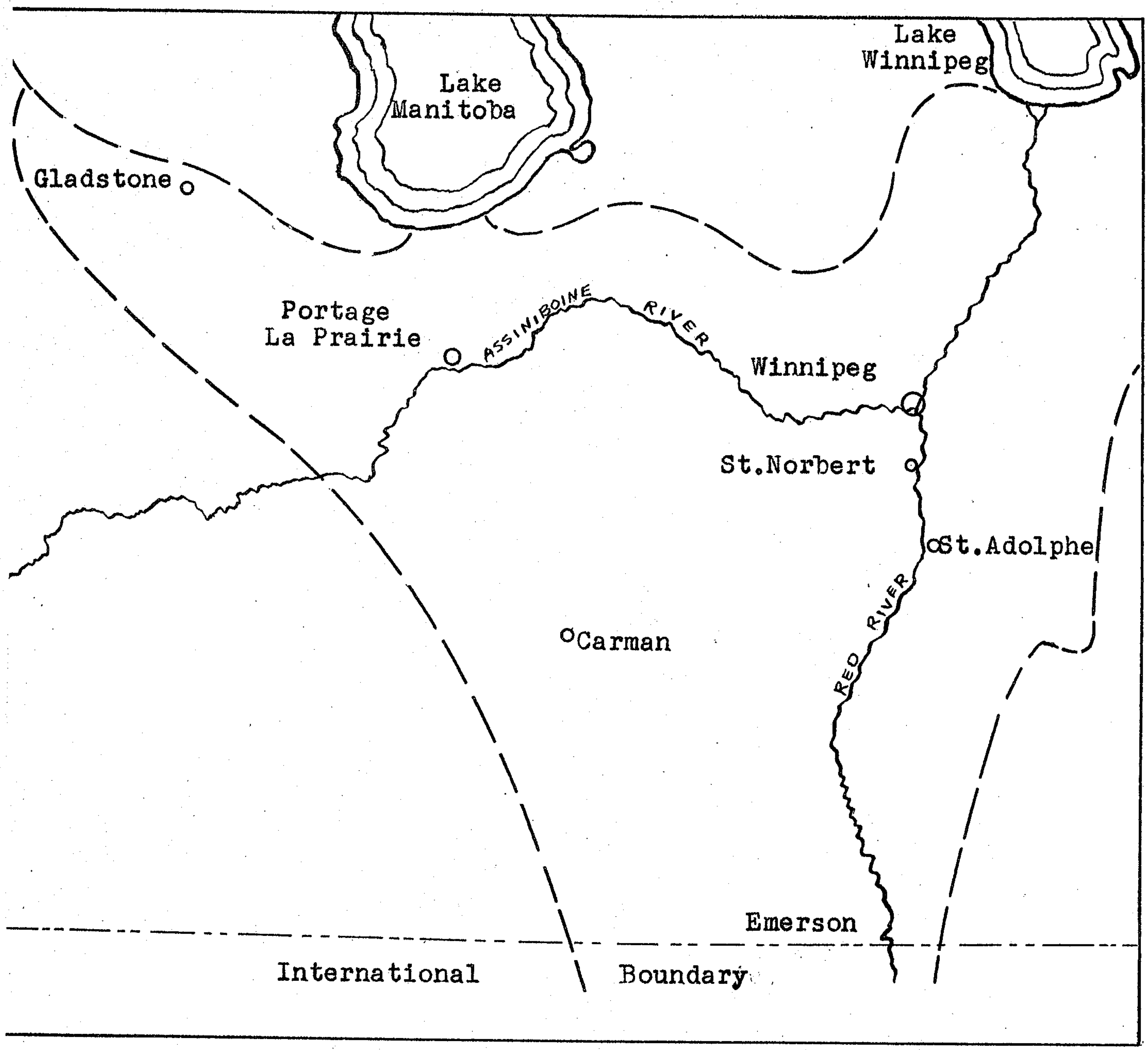


FIGURE 2. Approximate boundaries of the Red River Valley plain

in moisture content are possible within very small depth changes in varved clays. The dry density of the varved clays generally varies between 50 and 100 lbs. per cubic foot. The moisture content of the soil has been found to vary from 27% to 63% based on the dry weight of the soil sample. The degree of saturation of most of the Lake Agassiz varved clay ranges between 86 and 100%. Unconfined compression tests on a large number of varved clays in the vicinity of Winnipeg have yielded strengths averaging 2100 lbs. per square foot approximately [11].

The liquid limit of these clays generally varies between 37 and 117 with plasticity indexes ranging from 20 to 88. The clay is generally found to have low permeability, between 10^{-9} and 10^{-11} cm. per second. Because of the low permeability, ground water conditions have been very difficult to determine. However, the soil below a depth of 6 to 12 feet is generally found to be saturated [11].

The soil below this saturation level at the greater depths, is said to be normally consolidated or slightly over consolidated. That is, the soil has not been subject to greater over-burden pressures than those which presently exist. The upper layer of soil is generally considered to have been pre-compressed. This pre-compression of the soil is the effect of desiccation.

CHAPTER IV

THEORETICAL CONSIDERATIONS OF TEST METHODS

The elements of shear strength are considerably more complicated in cohesive soils such as clays than in cohesionless soils such as sands, because of the more complex nature of cohesive soils. Cohesionless soils are composed of particles which, because of their size and shape, have a small specific surface: that is, ratio of surface area to mass. The mass forces such as gravity, control their behaviour rather than surface forces. Particles of a cohesive soil, however, are flaky giving a large specific surface. Their behaviour is therefore influenced more by surface forces.

The shear strength of a cohesive soil may be considered to consist of a friction component, as in a cohesionless soil, and a cohesion component which is all the strength not due to friction. The exact nature of cohesion forces is not known, but is considered to be due to the natural attraction of solid particles to other solid particles, the presence of capillary forces, and the tenacity of the adsorbed water films. It is considered by some that cohesion is also a function of the inter-granular stresses in the soil. The term "cohesion" is often used loosely for the shear strength of a cohesive soil when tested with no lateral load applied to the specimen.

The portion of a load applied to a cohesive soil which is carried by the soil structure depends on the degree to which the pore water is permitted to drain and thus release its hydrostatic excess pressures. Since the load carried by water is not able to mobilize friction between soil particles, the shear strength of a clay is higher if drainage occurs than if it is prevented. There are therefore two extreme possible shear values which could be obtained for the same soil. The first would be determined if complete pore water drainage was allowed during the test. The second value would be obtained if no drainage of the soil was permitted. The first type of test is referred to as a slow shear test, and the second is referred to as an undrained or quick shear. Intermediate values depend on the percent drainage which has taken place.

There are also two possible shear values which may be obtained for a soil depending upon whether it is undisturbed or remoulded before testing. The Lake Agassiz clays exhibit a sensitivity of 2 [2,12]. The sensitivity is the ratio between undisturbed and remoulded strength of the soil. This means that an undisturbed sample would have double the shear strength found for a remoulded sample.

The shear strength of a soil is usually determined in the laboratory from samples taken from different depths in the ground. The laboratory investigations are generally carried out by means of unconfined compression tests [13]. The results of these tests, as indicated by calculations based on land slides that have occurred, are often smaller than the real strength of the soil [14]. In the unconfined compression test the natural unequal vertical and horizontal state of stress in the soil structure and the important natural restraint conditions are removed by taking a sample from the ground, and are only partially replaced by a uniform state of stress of lower but indeterminate magnitude. The new stress conditions are due to capillary forces which have been brought into action by some slight expansion of the clay soil accompanying relief of stress.

For saturated, normally consolidated clay soils, the capillary forces are probably only slightly changed when the soil is removed from the ground and these forces would be eliminated as soon as the soil was loaded in the test, due to the build up of pore water pressures caused by the prevention of drainage of the sample. Drainage is prevented due to the low permeability of the soil and the fast rate of testing. In the case of a pre-compressed soil, removal from the natural restraint of the surrounding soil causes the sample to swell and creates negative pore pressures in the soil. Because the soil is unsaturated, considerable load must be applied to the sample before pore pressures are increased and become positive.

As well as changes in confining pressure, most common means of extracting a soil sample exert normal pressures and frictional stresses on the sample. These tend to cause a reorientation of the soil particles. It is well known that when soils are disturbed and remoulded a considerable loss in strength results.

If the clay is saturated, the quick shear values are independent of the normal pressure on the clay. On the other hand, if the soil is not saturated the shearing resistance increases with increasing normal stress. The relation between these two values can be expressed approximately by the equation,

$$S = C + P\tan\phi$$

where S equals the shearing resistance, C is the cohesion and P is the normal stress, ϕ is the angle of internal friction. The cohesion depends on the initial consistency of the clay and ϕ on the compressibility and air content. For a completely saturated clay ϕ equals 0, and for a fairly dry clay ϕ is about 30° [15].

For a saturated cohesive soil in undrained loading the shearing resistance of the soil is generally considered equal to one-half of the unconfined compressive strength. It can be seen from the above formula that for a saturated soil the unconfined compressive strength would be dependent entirely upon cohesion whereas for an unsaturated soil the normal stress or load applied to the sample would affect the unconfined compressive strength. This latter could lead to incorrect conclusions being drawn concerning the strength of a soil in-situ should that soil be in an unsaturated condition. This possible incorrect conclusion could be drawn regardless of the type of test being performed on the soil.

The laboratory test which at present provides the best means for determining soil strength is the triaxial compression test. This test consists of compressing the sample to failure, in the same manner as the unconfined compression test, but doing so while maintaining a confining pressure around the sample. The confining pressure used may be equal to the calculated pressure which would have been exerted had the sample remained in-situ. Several confining pressures may be used on sets of samples and by means of Mohr's theory enable one to calculate the soil strength parameters; i.e., the angle of internal friction and the cohesion of the soil.[16,17]

However both the triaxial and unconfined compression tests require that samples be obtained from the soil mass and transported to a laboratory. Indeed this is a necessity for all laboratory test methods.

The vane shear test is performed in the field by forcing a four-bladed vane into the layer of soil to be tested. The vane is then rotated until a cylinder of soil is sheared from the main body of soil and the torque required is recorded. The torque is then translated into lbs. per square foot of area sheared. A similar test can be run in the laboratory on a small scale.[18]

The field vane shear test does not require the soil to be removed from the in-situ confinement. However soil is removed from above the test area by means of an auger. This may result in some change in confinement.

The penetration of the vane into the soil, due to the resulting disturbance, could also have an effect on the strength value obtained. However it has been determined by several authorities that the results of vane shear tests indicated strengths very close to those calculated from slide analyses [6].

In calculating the shear strength, it is generally assumed that the cylindrical surface failed has a diameter and height equal to that of the vane. This was shown to be true by Swedish experiments made on sand and clay by carefully cutting away the soil halfway down the vane and observing the failure surface [1]. It is assumed that the shear stress is uniformly distributed on the cylindrical surface and on the ends of the cylinder [6].

The vane test basically measures shear strength in a vertical direction; no correction is possible to correct for interference of stones, and extreme care must be exercised to reduce the amount of disturbance when the vane enters the test area. The vane test measures the undrained shear strength under in-situ conditions of moisture and existing stresses due to the weight of the overburden.

CHAPTER V

DESCRIPTION OF TEST APPARATUS

The vane test apparatus consisted of a vane, a series of torque rods, a torque wrench and adapter, and the stress-strain device. This apparatus was designed and built by the Manitoba Highways Branch.

The vane had four steel blades, each 1" by 4" by 1/8" thick mounted on a 5/8" diameter steel rod. The rod and fillet welds joining it to the vane blades, were tapered toward the bottom end of the vane. The rod and welds were tapered to minimize the area ratio of the vane while maintaining strength. The area ratio, which is defined as the ratio of the average cross-sectional area of the vane to the cross-sectional area of the cylinder sheared, is 29 percent. The bottom and outside edges of the vane blades are bevelled a t 45^{o}. The ends of the vane blades are square with the sides. It was not necessary to have a conical point as the vane dimensions were small enough in relation to the hole diameter that there would be little or no catching on the casing or the sides of the hole. The vane is illustrated in Figure 3.

An 8" sleeve fits over the 5/8" rod. The contact surface between the rod and sleeve were kept lubricated to eliminate friction as the vane and rod are rotated. Since the vane is driven 8" below the bottom of

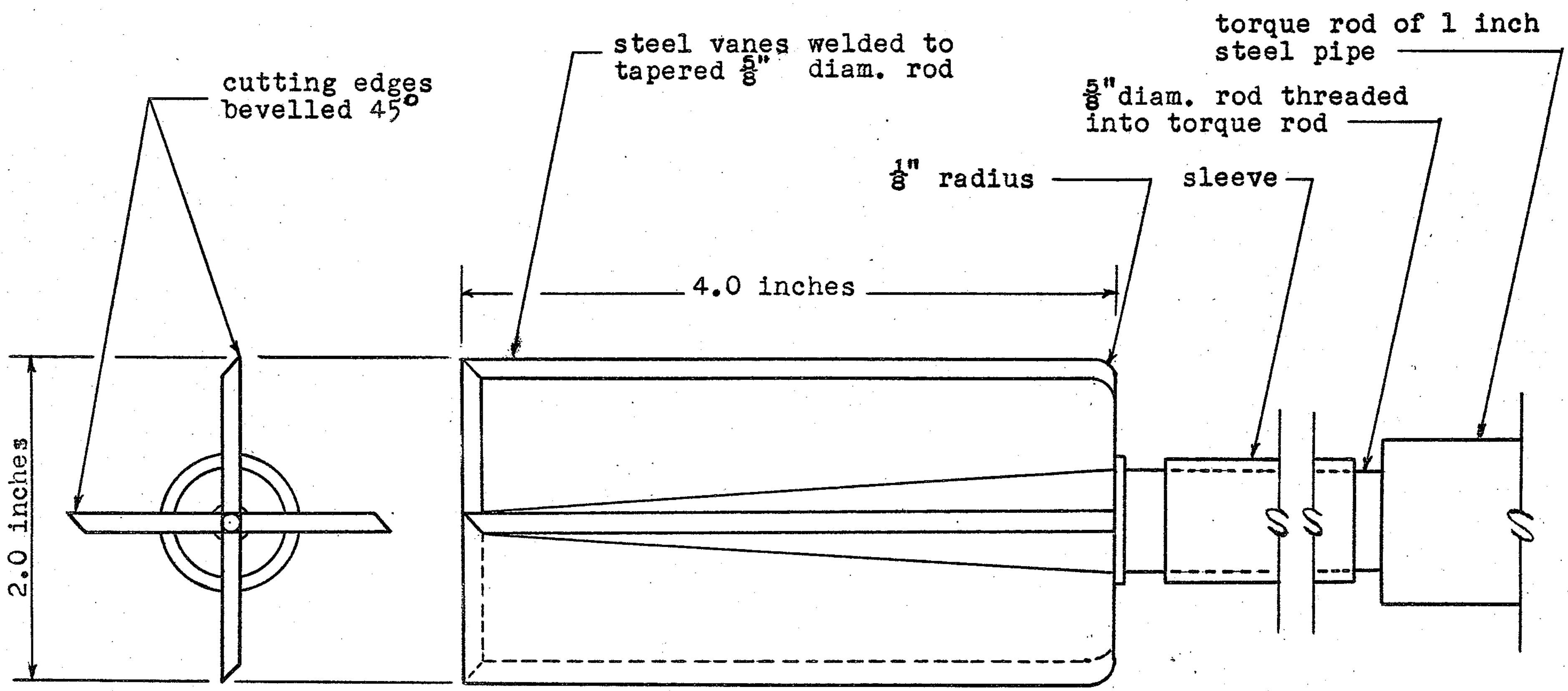


FIGURE 3. The dimensions of the vane.

the hole, the sleeve enables one to neglect the effect of rod friction.

The upper end of the rod was threaded to suit a nut welded into one end of the first torque rod section. The other end of this torque rod was fitted with a steel plug. The plug was riveted in place and projected about one inch. The free end of the plug had a hole through which a 3/16" pin could be placed. Each of the remaining shaft lengths was open at one end and plugged at the other. Both ends of the shaft were drilled for pin connection. The torque shafts were 5'-0" in length and were fabricated from standard 1" I.D. steel pipe.

For use with the torque wrench, a special short section was constructed to fit onto the male shaft end. This short section had a wrench socket at the other end and a loose sleeve by which the shaft could be held while the wrench was rotated.

Figure 4 illustrates the stress-strain device as initially constructed. This device was similar in design and identical in operation to that developed by Foundation Engineering Company [2].

The device consisted of a steel pipe support, a steel mounting for the main steel torsion disc, an aluminum box beam to support the dolley, winch, and pulleys, and an aluminum recording disc driven by a chain connected to the torsion disc.

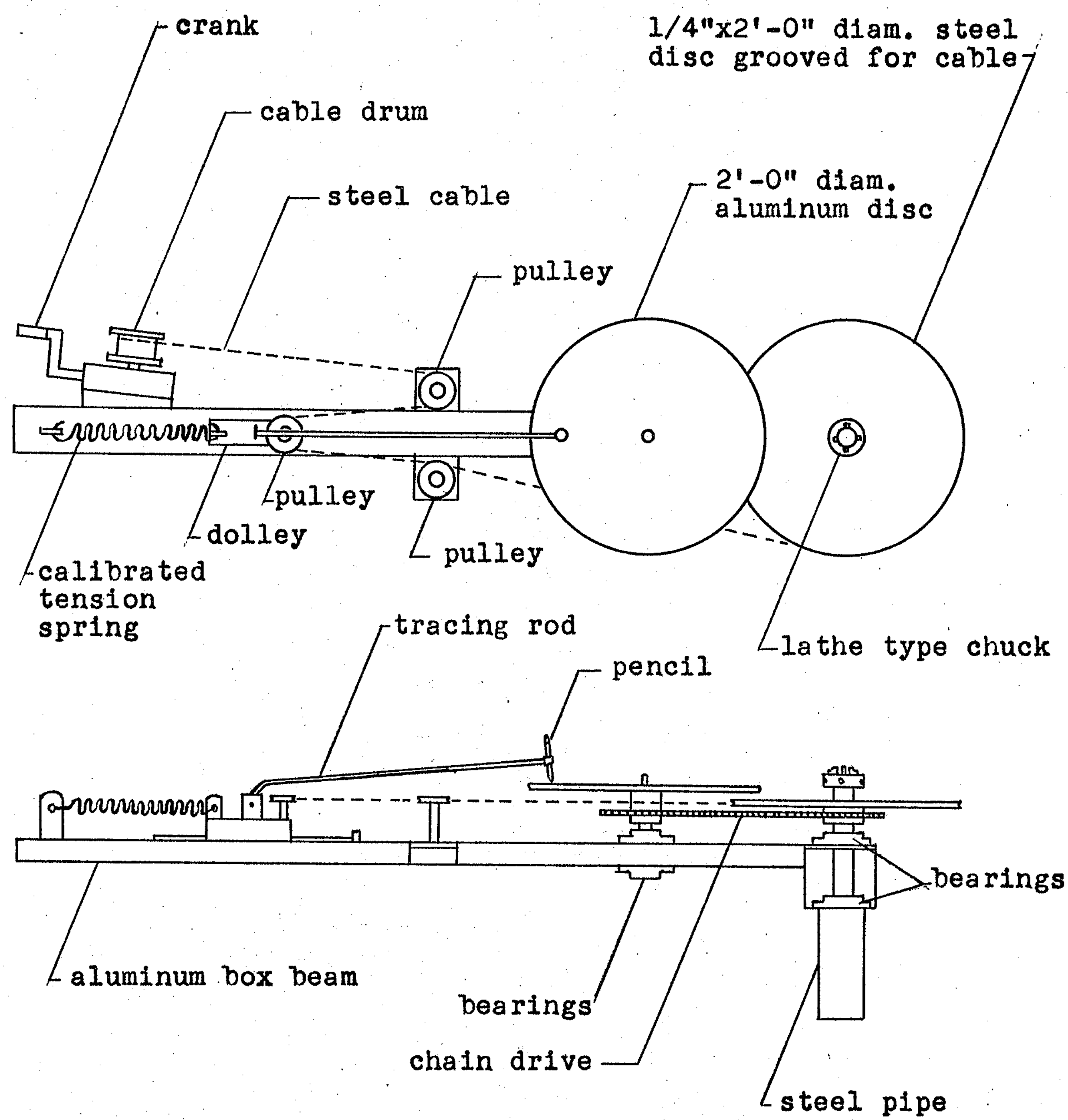


FIGURE 4. Details of the original stress - strain device used in the vane shear tests.

The torque shaft was clamped to the centre of the 1'-0" radius torsion disc by means of a four-jaw lathe type chuck. When the cable was tensioned by the winch the vane tended to rotate and shear the soil. The tension in the cable extended the calibrated spring and moved the dolley along its track. The tracing rod and pencil recorded the movement on the paper on the recording disc. As the soil deformed, the strain was also recorded since the two discs were connected by a chain and sprocket system.

Because of the excessive weight and size of the stress-strain device, it was very difficult to handle. Figure 5 shows the revised apparatus. Most of the steel was removed and the loading disc, now made of aluminum, was attached directly to the recording disc. The operation remained the same as for the original device.

The unconfined compression tests were performed on a motorized testing machine which employed a double proving ring and dial indicators to register stress and strain applied to the soil sample. The load was transmitted directly to the sample by means of gears and chain drive rather than through a hydraulic system.

The triaxial testing was carried out on a motorized machine similar in operation to the unconfined compression test apparatus but employing only a single proving ring to register the applied load. Compressed air was used to apply confining pressures onto the samples.

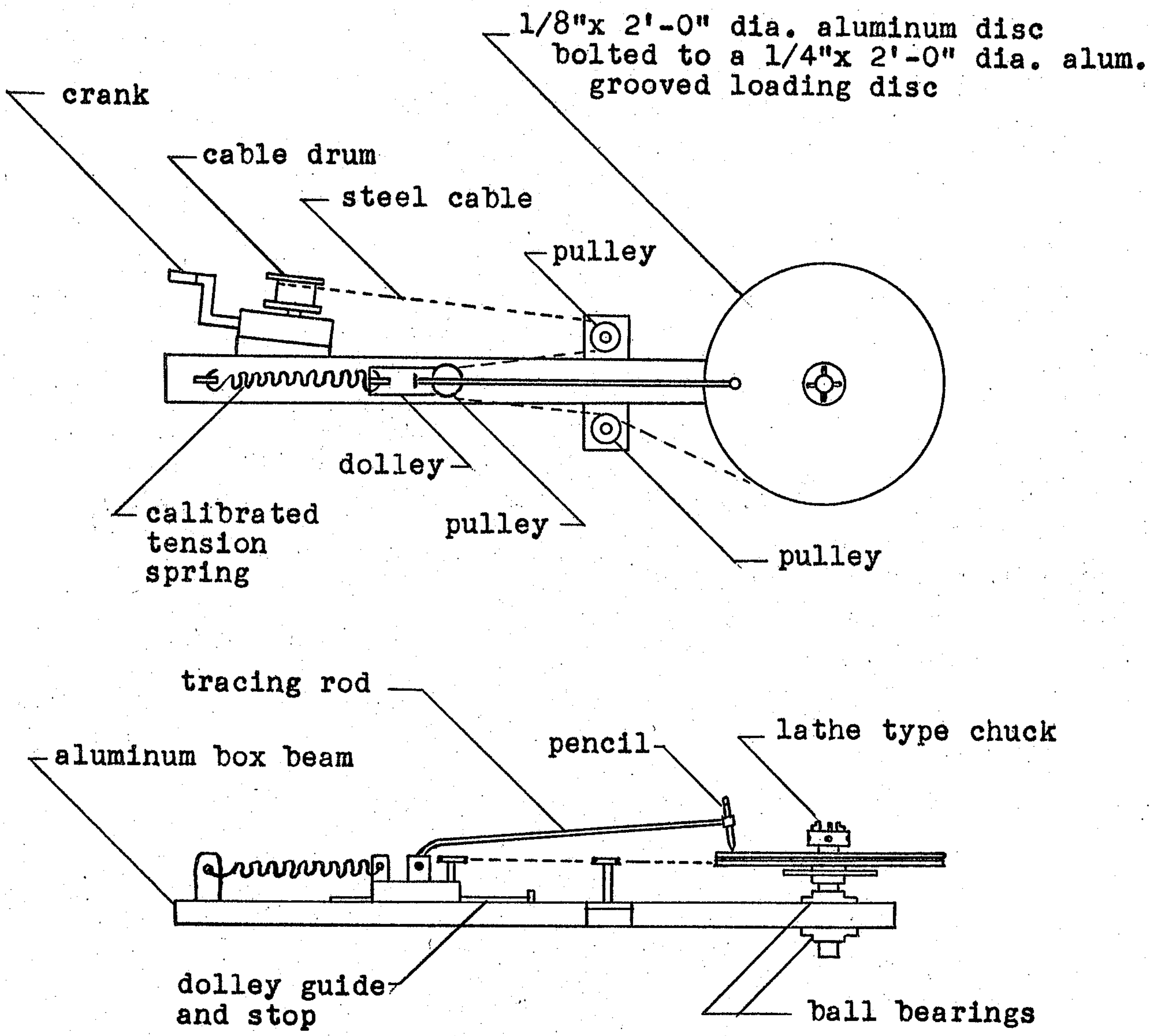


FIGURE 5. The details of the revised stress - strain device.

CHAPTER VI

METHOD OF TEST

When the vane was first put into operation the only loading device was a torque wrench with a range of 0 to 100 foot-pounds. This equipment was used in some thirty tests involving eleven test holes of depths up to 41.5 feet.

The procedure implemented consisted of wet drilling the hole to a depth of 3 to 5 feet and forcing the vane into the soil below the hole bottom. The vane was then rotated until the soil sheared usually within from 30 to 90 seconds. The maximum torque was recorded. The vane was then turned through four complete revolutions, allowed to rest for one minute and the test repeated. The value obtained by this latter test was referred to as the remoulded strength.

The hole was then bored deeper, to remove the soil disturbed by the vane, and a Shelby tube sample was taken. In some cases a vane test was taken both above and below the tube sample.

The stress-strain device was used to load the vane in the same manner as the torque wrench, but was also used in several holes which were drilled by means of a hand auger.

With the stress-strain device, once the torque rod was clamped in place, the winch was turned at a steady rate. The time required for the test was generally about three minutes, which approximates the time involved for the performance of an unconfined compression test. This was important since the vane test results were compared to those obtained from unconfined compression tests.

To perform the remoulded test, the chuck was loosened, the torque rod rotated by means of a pipe wrench, the chuck tightened again and the test repeated.

The location of each test hole was recorded on the plotting paper and each curve was identified as to depth and whether it was an undisturbed test or a remoulded test.

The vane was generally advanced into the soil manually, either by means of a plank on the top of the torque shaft or by using two pipe wrenches. In relatively few cases it was necessary to use the hydraulic head of the drilling rig to advance the vane. The vane was usually retracted manually using two pipe wrenches to grip the shaft.

Two men, in general, were required in the operation of the vane and the torque devices. The most difficult operations were the advancing of the vane in very stiff clay and the retracting of the vane. A cable hoist and tripod were found useful in retracting the vane although they were not essential. A device for advancing the vane is being developed by the Highways Branch, Province of Manitoba.

The samples for unconfined compression and triaxial tests were obtained by means of thin-walled steel Shelby tubing. The tubes were either forced into the soil using the hydraulic head of the drilling rig, or were driven by means of a thirty pound weight dropping 3 feet.

The tubes cut a sample 2" in diameter. The ends of the tube were cleaned and filled with melted paraffin to prevent loss of moisture.

When the tubes arrived at the laboratory the soil sample was pushed out of the tube by a hydraulic ram. The samples were then cut into four-inch lengths. Only one tube was emptied at a time and the samples obtained were either tested immediately or wrapped, sealed and held in a humidity cabinet.

The 2-inch diameter by 4-inch long sample was placed in the unconfined compression machine. The sample dimensions have a length to diameter ratio within the recommended 1.5 to 3.0, and have the advantage of requiring no trimming upon removal from the tube.

The dials on the machine were set to zero and loading was commenced. A constant rate of strain of .058 inches per minute, 1.45% of the sample length, was maintained. The recommended rate of strain is between 1/2% to 2.0% per minute.[13] Readings of the stress, or load, were taken at every .01 inch of vertical deflection. The sample was loaded until the stress began to fall off and the failure cracks were visible.

Immediately after failure, the sample was weighed and placed in an oven at 230° F and dried over night. The next day the sample was again weighed and the moisture content, in percent of dry weight, was determined.

Generally sufficient soil was left over after the 2" by 4" sample was obtained, for the Atterberg limit tests. The liquid and plastic limits of the soil were determined, the plasticity index was calculated, and the grain size distribution determined by the hydrometer method.[19] This information was then used to classify the soil by the U.S. Bureau of Public Roads System.[20]

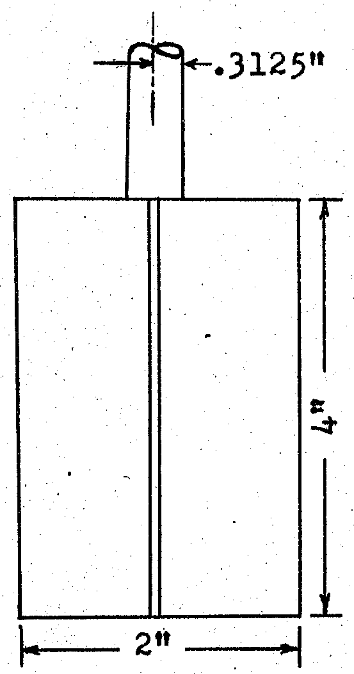
The triaxial tests were carried out in much the same manner as the unconfined compression tests.[13] However, in order to provide the three sets of test data required to determine the soil strength parameters, it was necessary to obtain three test specimens from each tube. Each specimen was tested in compression while supported by a confining pressure applied through an enclosing rubber membrane. The pressures used were 10, 20, and 30 pounds per square inch. The Mohr's circles of stress were plotted and the cohesion value and angle of internal friction were determined.

CHAPTER VII

CALCULATIONS

In order to translate torque into shear stress it is necessary to determine the vane constant. This constant depends upon the size and shape of the vane.

The dimensions of the vane are as follows :



D = diameter of the vane = 2 inches

R_1 = radius of the vane = 1 inch

R_2 = radius of the shaft = .3125 inches

L = length of the vane = 4 inches

The torque applied to the vane shaft, assuming there is no friction along the shaft, must equal the sum of the moments of the soil shear strength times the end and side areas of the cylindrical surface sheared by the vane.

Let M_1 = moment of shear force on cylinder side

M_2 = moment of shear force on cylinder bottom

M_3 = moment of shear force on cylinder top

S = shear stress in the soil

$$M_1 = \pi D \times L \times R \times S$$

$$= 3.1416 \times 2 \times 4 \times 1 \times S$$

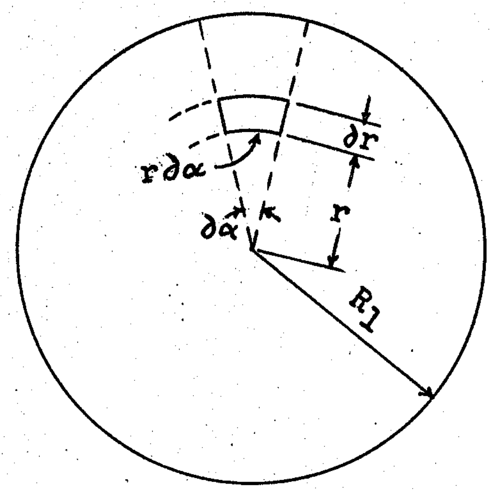
$$= 25.1328\ S$$

To determine the moment of the shear force on the bottom of the cylinder, we must consider the shear force on the element shown.

Area of the element $= \partial A$

Shear force on area $= S\,\partial A$

$= S\,\partial r\, r\partial\alpha$



The moment of $S\,\partial A$ about the center $= r^2\, S\,\partial r\,\partial\alpha$

Therefore :

Total moment $= M_2$

$$= \int_{r=0}^{r=R_1} \int_{\alpha=0}^{\alpha=2\pi} r^2\, S\,\partial r\,\partial\alpha$$

$$= 2\pi \frac{R_1^{\,2}}{3}\, S$$

$$= 2/3\ R_1\ AS$$

$$= 2/3 \times 3.1416\ S$$

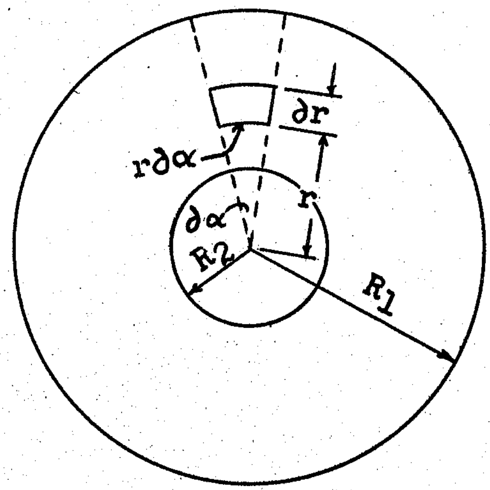
$$M_2 = 2.0944\ S$$

To determine the moment of the shear force on the top of the cylinder, consider the shear force on the element shown in the figure on page 30.

Area of the element $= \partial A$

Shear force on area $= S\,\partial A$

$= S\,\partial r\; r\,\partial\alpha$



The moment of $S\,\partial A$ about the center $= r^2\, S\,\partial r\,\partial\alpha$

Therefore :

Total moment $= M_3$

$$= \int_{r=R_2}^{r=R_1} \int_{\alpha=0}^{\alpha=2\pi} r^2\, S\,\partial r\,\partial\alpha$$

$$= 2/3\,\pi(R_1^{\,3} - R_2^{\,3})\, S$$

$$= (\, 2/3\; R_1 A - 2/3\; R_2 A_1)\; S$$

Where $A =$ area of cylinder bottom

and $A_1 =$ area of shaft

$$M_3 = (\, 2.0944 - .0961\,)\; S$$

$$= 1.9983\; S$$

Therefore :

Total moment for entire cylinder $= M = M_1 + M_2 + M_3$

$$M = (\, 25.1328 + 2.0944 + 1.9983\,)\; S$$

$$= 29.2255\; S$$

Or $\quad S = \dfrac{M}{29.2255}$ where M is in inch - pounds and S is in pounds per square inch.

When the torque, T, is expressed in foot - pounds and S in pounds per square foot, the equation becomes :

$$S = \frac{T \times 12}{29.2255} \times 144 = 59.1 \text{ pounds per square foot}$$

Therefore a torque of 1 foot - pound represents a shear stress of 59.1 pounds per square foot.

When the torsional force is applied by means of the torque wrench, the reading in foot - pounds may be converted to shear strength simply by multiplying by 59.1 .

When the torsional force is applied by means of the stress-strain device, a tension of one pound in the cable is equal to a shear strength of 59.1 pounds per square foot in the soil. The mechanical arrangement of the pulleys, however, causes the calibrated spring to be loaded to double the cable tension. Since the springs were designed and calibrated to extend one inch per unit of tension (5 lbs. and 10 lbs.),the torsional load as plotted on the recording disc must be halved.

CHAPTER VIII

CALIBRATION OF THE VANE APPARATUS

In order to determine the relation between cable tension and spring extension the stress-strain device was set up exactly as it would be in the field with the exception that the cable was carried around the loading disc and over a pulley instead of being fastened to the loading disc. The calibration weights were then fastened to the free end of the cable.

As weights were added, the cable tension increased, rotating the loading disc and extending the spring. The pencil plotted the spring extension on the recording disc. Thus, as the spring was calibrated, the values were automatically compensated for any frictional resistance in the apparatus. The information made it possible to draw the concentric circles on the recording graph as shown in Figure No. 6.

The springs were found to be fairly accurate. That is, the extension per unit of load remained similar regardless of the initial extension, particularly in the range within which most of the vane results were determined. No value was attributed to each concentric circle on the graph because it was frequently necessary to use different springs and combinations of springs. Calibrations were done at various temperatures but, since no appreciable difference was discovered, this factor was neglected. This data is presented in the Appendix.

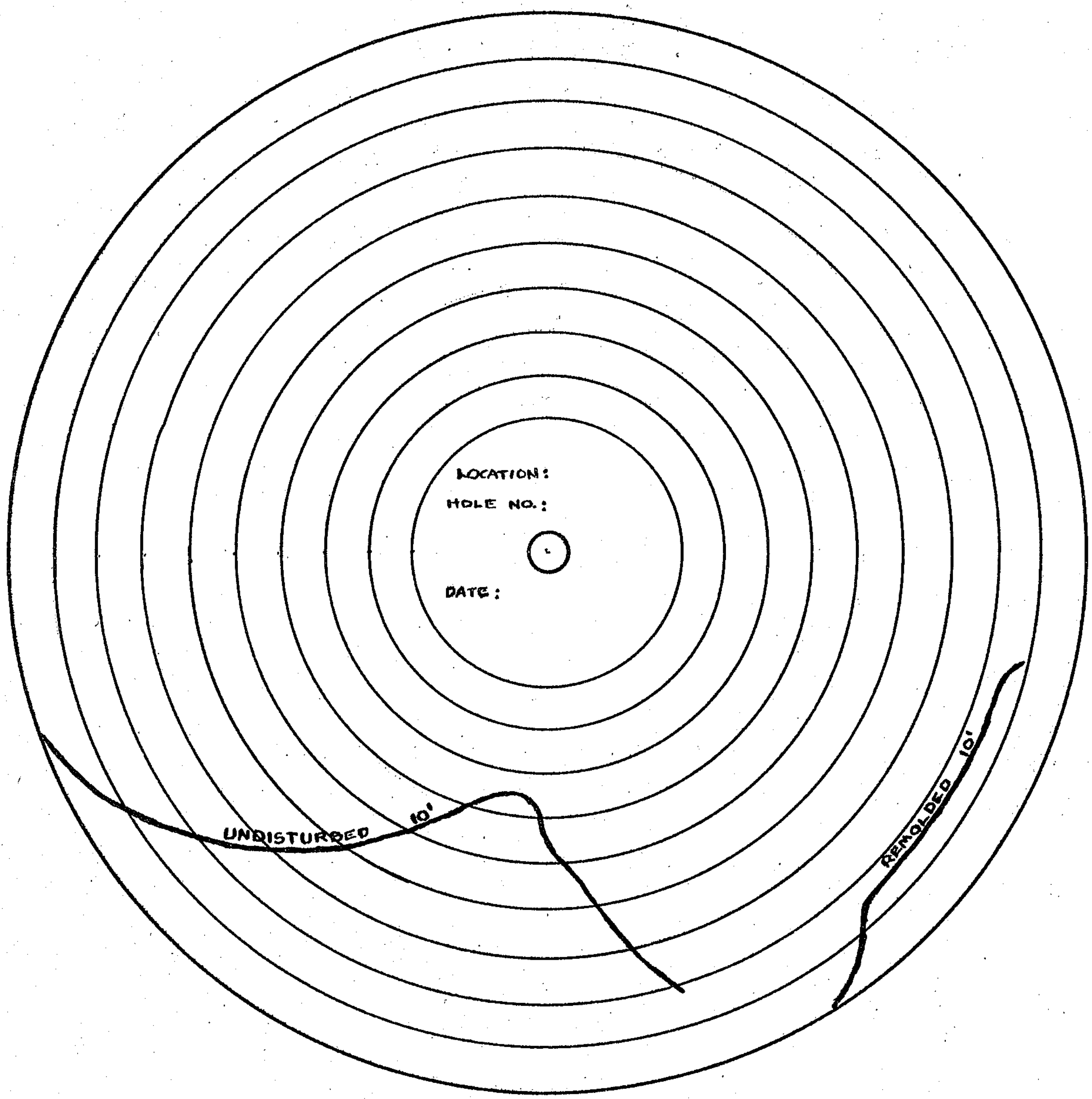


Figure 6. Typical recording graph.

So that comparisons of the stress-strain relationship of the vane shear test and the unconfined compression test could be made, the strain in the torque rods was determined. This was accomplished by setting up the device in the normal manner for calibrating but with a torque rod fastened in the chuck. By preventing the free end of the rod from turning it was possible to obtain a graph showing the stress-strain curve simply by operating the winch. Since all rods were identical it could be assumed that they would each strain an equal amount under the same load.

The information regarding the strain in the torque rods was never used because, as is explained in a subsequent chapter, the stress-strain device does not maintain a constant rate of strain. Since the unconfined compression test does maintain a constant rate of strain, a comparison of the two stress-strain curves would not be applicable.

CHAPTER IX

TEST RESULTS

The results of vane shear tests and the Shelby tube samples for test holes no. 1, 2, and 3 were obtained from the Bridge Office, Highways Branch, Province of Manitoba. The laboratory tests were performed on the tube samples by the Highways Testing Laboratory. Test hole no. 1 was drilled on the Perimeter Highway at the C.P.R. overpass north of Winnipeg. Hole no. 2 was located on Highway no. 4 near Gladstone. Hole no. 3 was located on the Morden-Sprague Road near Letellier.

The vane tests in the above test holes were performed by the drill crew and utilized a torque wrench to measure the torque required to shear the soil.

Test hole no. 4 was located on the upper bank of the Red River near lot 68, Turnbull Drive, south of St. Norbert. The hole was drilled by the same crew,as the previous holes,but the author performed the vane tests. The stress-strain device was employed to measure the shearing resistance of the soil.

Hole no. 5 was drilled by hand and was located near St. Mary's Road north of St. Adolphe. The Water Control and Conservation Branch of the Department of Agriculture, Province of Manitoba, provided personnel and equipment for obtaining Shelby tube samples manually. A drop hammer was used to drive the tubes and a winch and tripod were used to withdraw the tubes. The author performed the vane shear tests implementing the stress-strain device to measure shearing resistance.

Test holes no. 6 and 7 were drilled manually at 1760 Pembina Highway in Fort Garry. The devices used in hole no. 5 were again used to drive and withdraw the Shelby tubes. The author performed the vane shear tests using the stress-strain device to measure shearing resistance.

All the laboratory tests were performed in the Highways Testing Laboratory. The tests were performed by the laboratory staff and the author.

The results of the undisturbed vane tests, the remoulded vane tests and the unconfined tests were plotted for each test hole.

Figure 7 shows the results of tests in hole no. 1. At the five-foot depth the unconfined compression value of shear strength is equal to about two-thirds of the undisturbed vane strength and about double the remoulded vane strength. At the ten and fifteen-foot depths the unconfined compression strengths are only slightly higher than the re-

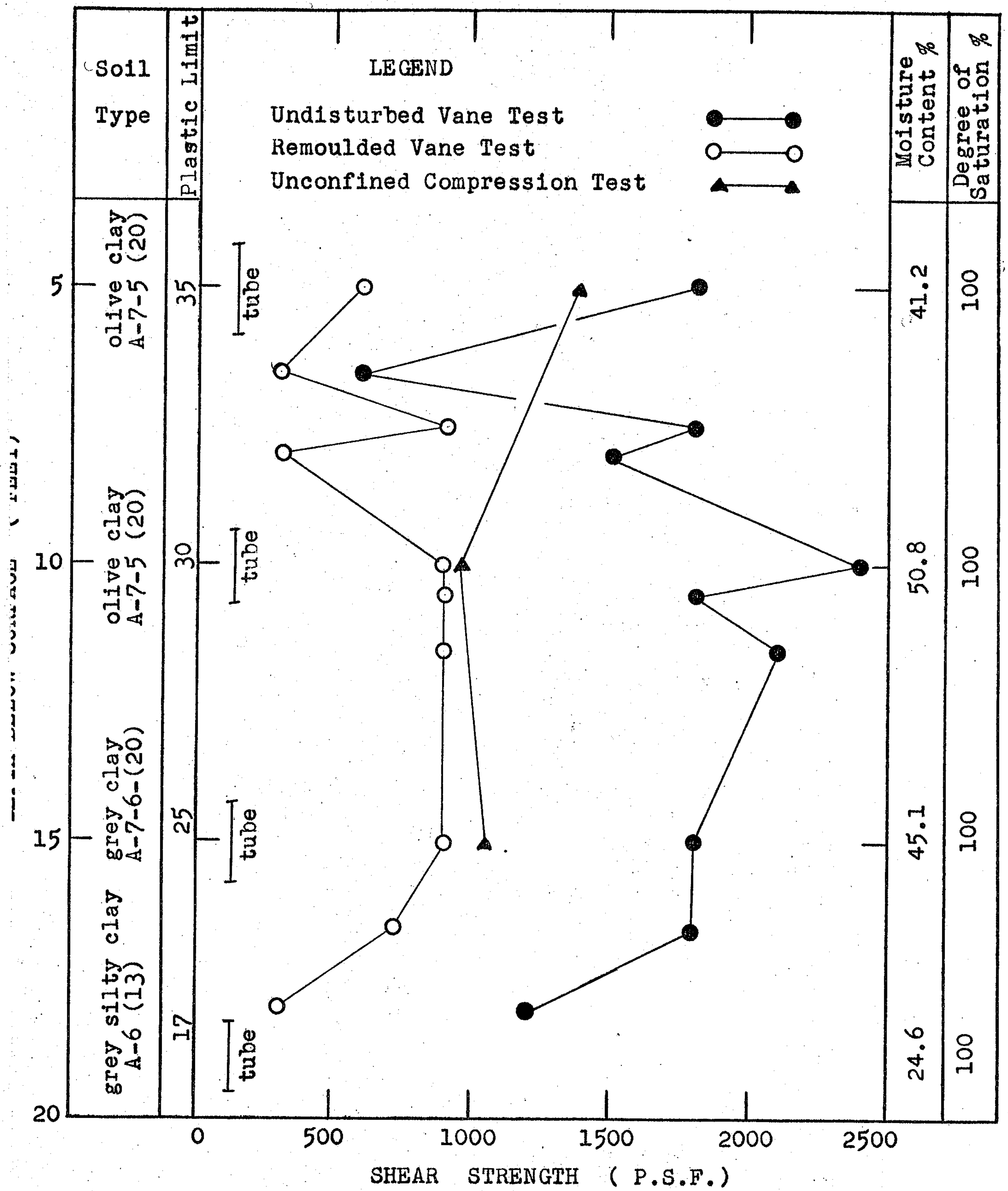


FIGURE 7. The results of tests in Hole no. 1.

moulded vane strengths and equal to only about one-half of the undisturbed vane test results.

Although there was a soil change between ten and fifteen feet, there was little change in the strength values. A considerable decrease in strength was indicated by the vane for the silty clay encountered at eighteen feet. Although a Shelby tube sample was obtained, it was in such a loose state that it was impossible to perform an unconfined compression test.

All the samples tested were saturated and had moisture contents in excess of the plastic limit. The soils, with the exception of the silty clay, were varved clays.

The results of tests in hole no. 2 are illustrated in Figure 8. The unconfined compression test values are of the same order as the remoulded vane values with the exception of the shear strength of the clay loam which is only about two-thirds of the remoulded vane strength. The undisturbed vane strength at thirty-five foot depth is roughly three times as great as the unconfined compression value. The average unconfined compression value for the samples from forty and forty-two foot depths equals roughly one-half of the undisturbed vane shear strength.

The sample from thirty-five feet was 98.7% saturated and the other two were 100% saturated. The change in soil type at forty-two foot depth resulted in an unconfined compressive strength drop of 50%. The vane tests did

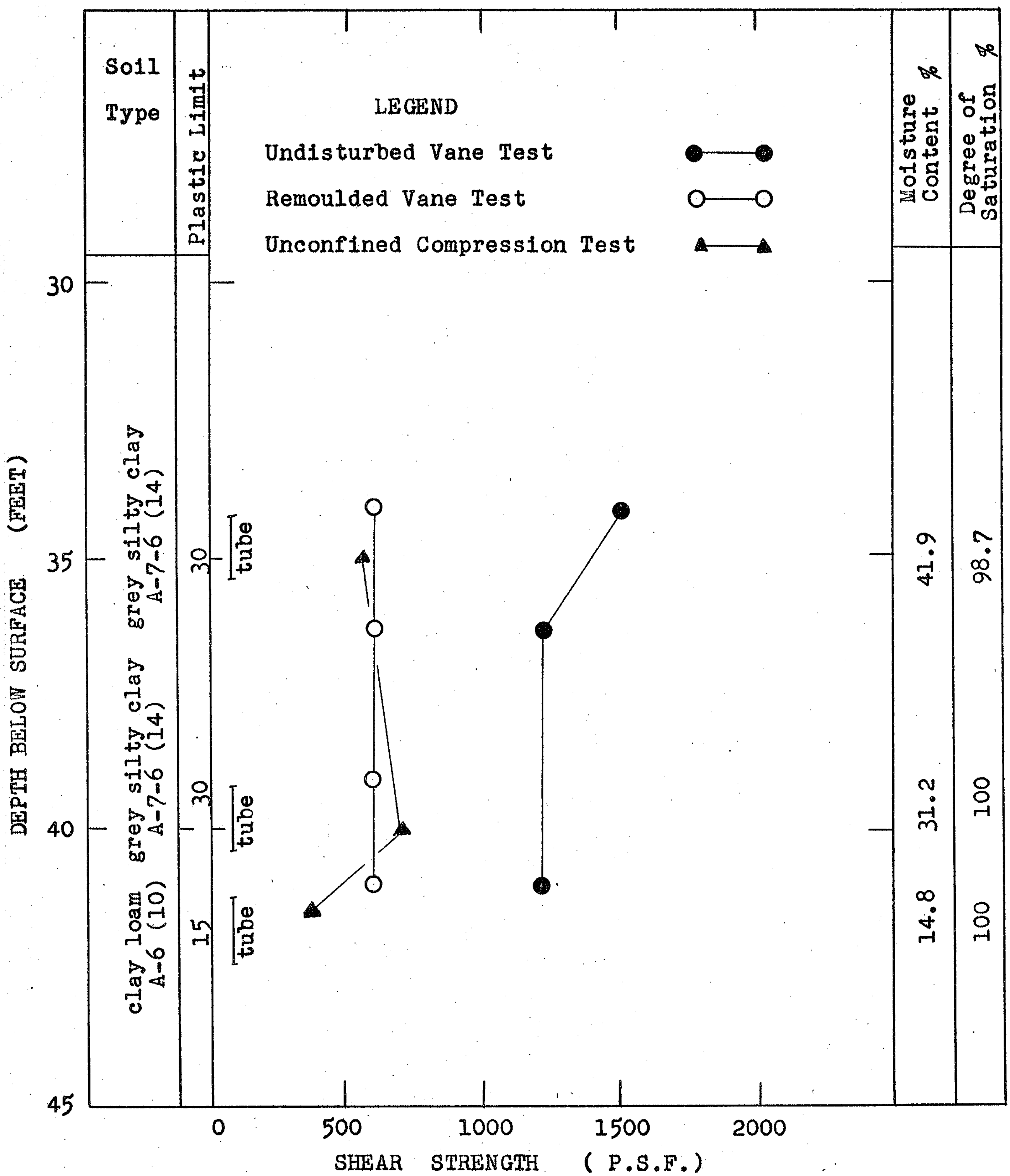


FIGURE 8. The results of tests in Hole no. 2.

not indicate any reduction in strength.

Figure 9 shows the results of tests performed in hole no. 3. At depths of fifteen and twenty-five feet the unconfined compression test shear strength values equal the remoulded vane strength and are roughly equal to 40% of the undisturbed vane strength. The soil at fifteen feet was an unsaturated silty clay and the soil at twenty-five feet was saturated clay. Both samples were varved.

At the twenty-foot depth the unconfined compression shear value decreased from that at fifteen feet while the moisture content increased. The degree of saturation remained the same and the soil type changed from silty clay to clay.

From twenty-five to thirty-five feet, the unconfined compression test strength decreased from 1100 p.s.f. to about 600 p.s.f., and the moisture content increased from 51.3% to 57.6%. The three samples were, to all intents and purposes, in a saturated condition.

The four samples of clay consistently showed higher strengths for lower moisture contents.

The vane results indicated a sensitivity of 2.2 for each test depth. The undisturbed vane test values were in excess of double the unconfined compression values at the same depths.

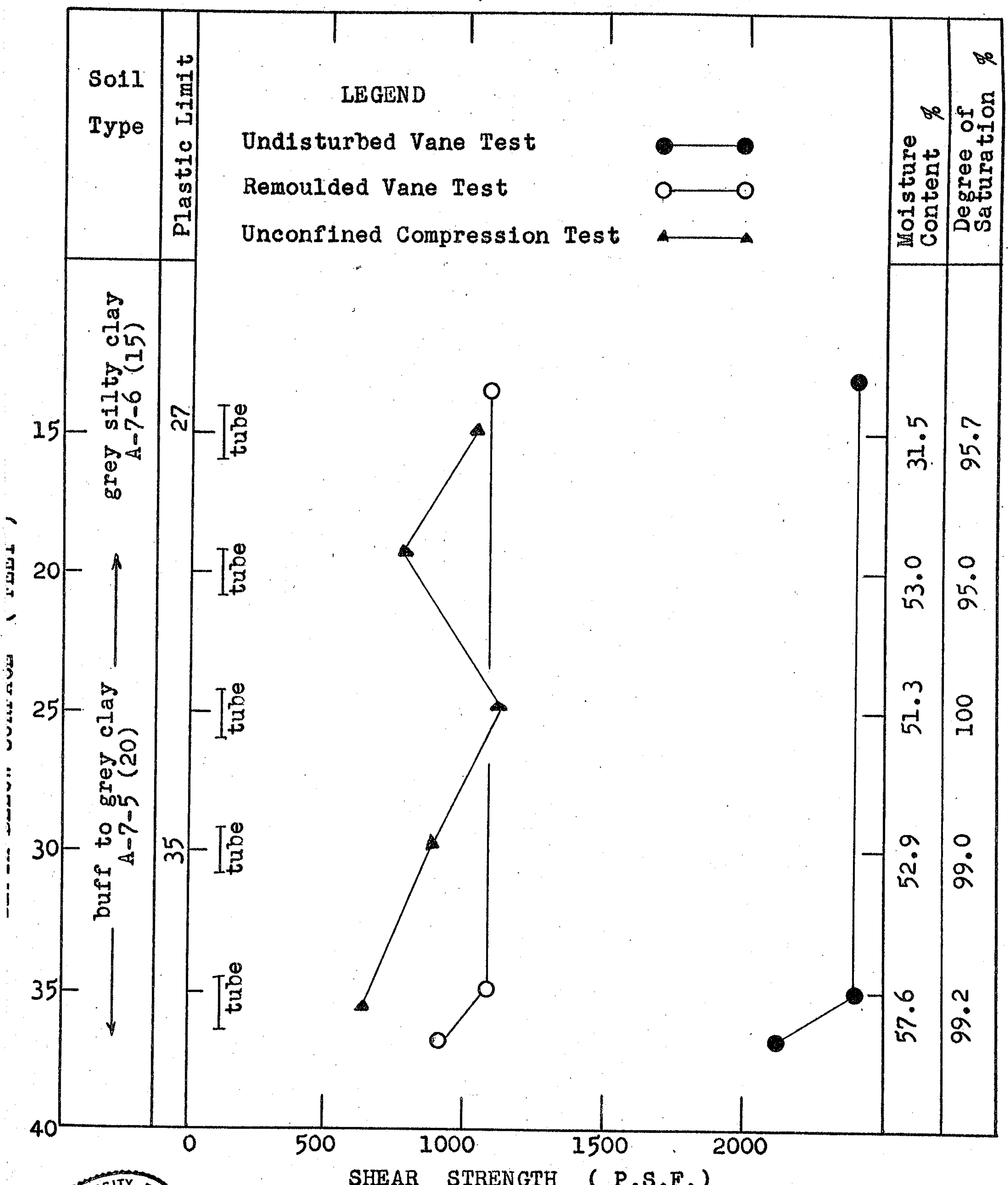


FIGURE 9. The results of tests in Hole no. 3.

For the soils tested in holes no. 1, 2, and 3, the vane results in undisturbed soil were higher than the unconfined compression test shear strength values with only one exception. Also, with the exception of the value at five foot depth in hole no. 1, the unconfined compression test results were of the same order as the remoulded vane strength. The average sensitivity of the A-7-5(20) clay was about 2 and that of the A-7-6(14-15) silty clay was about 2.5.

Figure 10 shows the comparison between the undisturbed vane test results and the results of unconfined compression tests, related to hole no. 4. The remoulded vane shear values were all smaller than the unconfined compression test shear strength. The soil sensitivity as determined by the vane was about 4 at every level.

The average unconfined compression test shear strength values at five, ten and fifteen foot depths, are roughly equal to one-half of the undisturbed vane shear strengths. At the twenty foot level the average shear strength of the tube sample was equal to about two-thirds of the undisturbed vane shear strength taken above the tube sample and very nearly equal to the value obtained just below the tube sample. At each test level, the sample which had the higher moisture content exhibited the lower shear strength, regardless of the degree of saturation. All samples had moisture contents in excess of their plastic

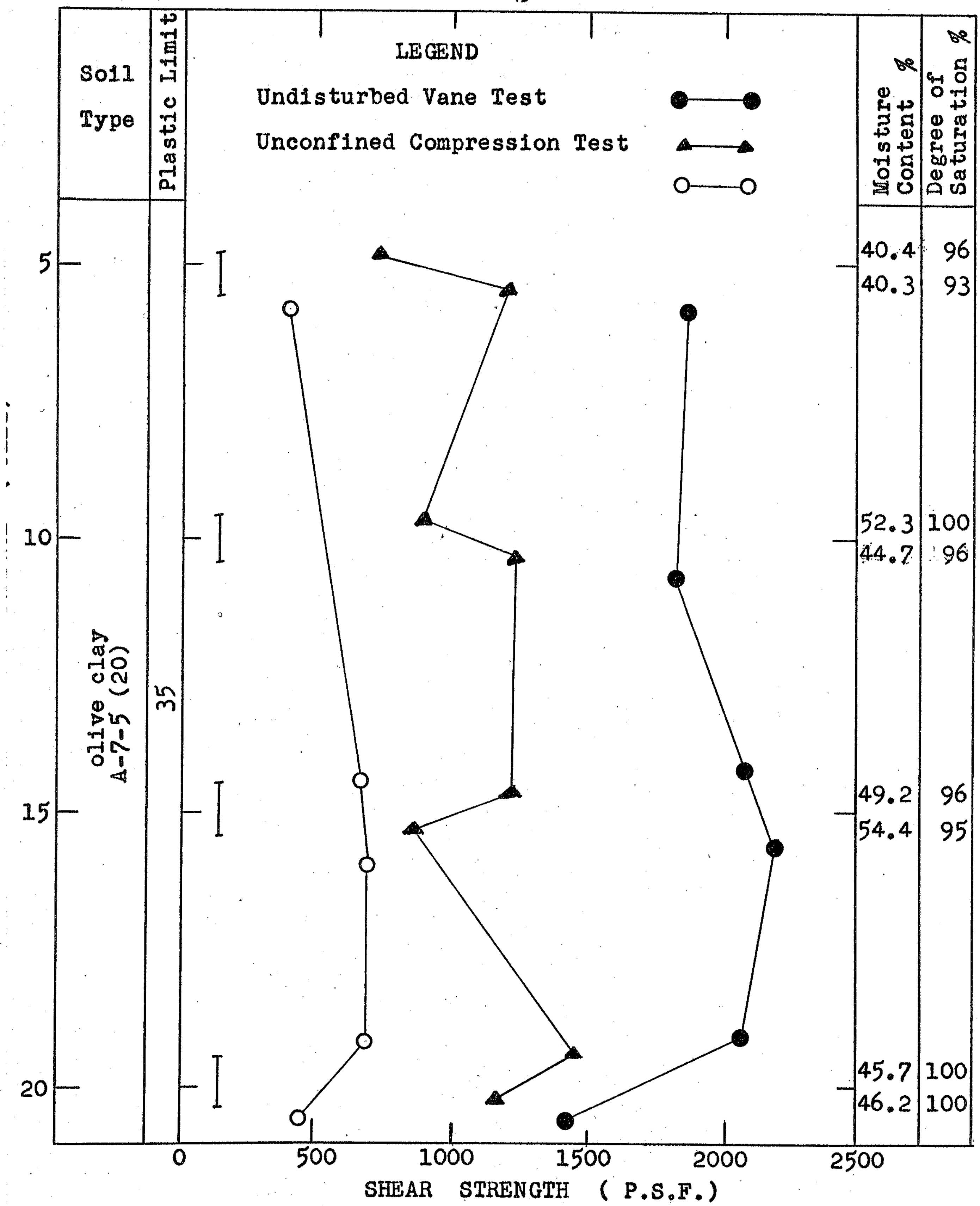


FIGURE 10. The results of tests in Hole no. 4.

limits and all were varved clay soils.

The shear strengths obtained with the undisturbed vane test were all of the same order of magnitude with the exception of the value below the twenty foot depth. The decrease in strength at this latter depth may have been due to a sudden change in moisture content or soil type. A similar strength decrease was recorded at the deepest level in holes no. 1 and 3. The strength loss may also have been due to disturbance of the soil by the sampling tube. In hole no. 1 at depths of five, and ten feet there was an appreciable decrease in vane shear value a foot below the depth from which the sample was obtained. This same effect was recorded at a depth of thirty-six feet in hole no. 3.

The results of tests in hole no. 5 are plotted in Figure 11. The shear strength value determined by the unconfined compression test on a Shelby tube sample from a depth of eight feet, exceeded both the undisturbed and remoulded vane strengths which were determined at a depth of ten feet. The soil sensitivity as determined by the vane tests varied from 3.0 at ten feet to 1.5 at eleven feet.

At a depth of fourteen feet the soil sensitivity as determined by the vane tests was slightly greater than 3 as was the case at the nineteen foot depth.

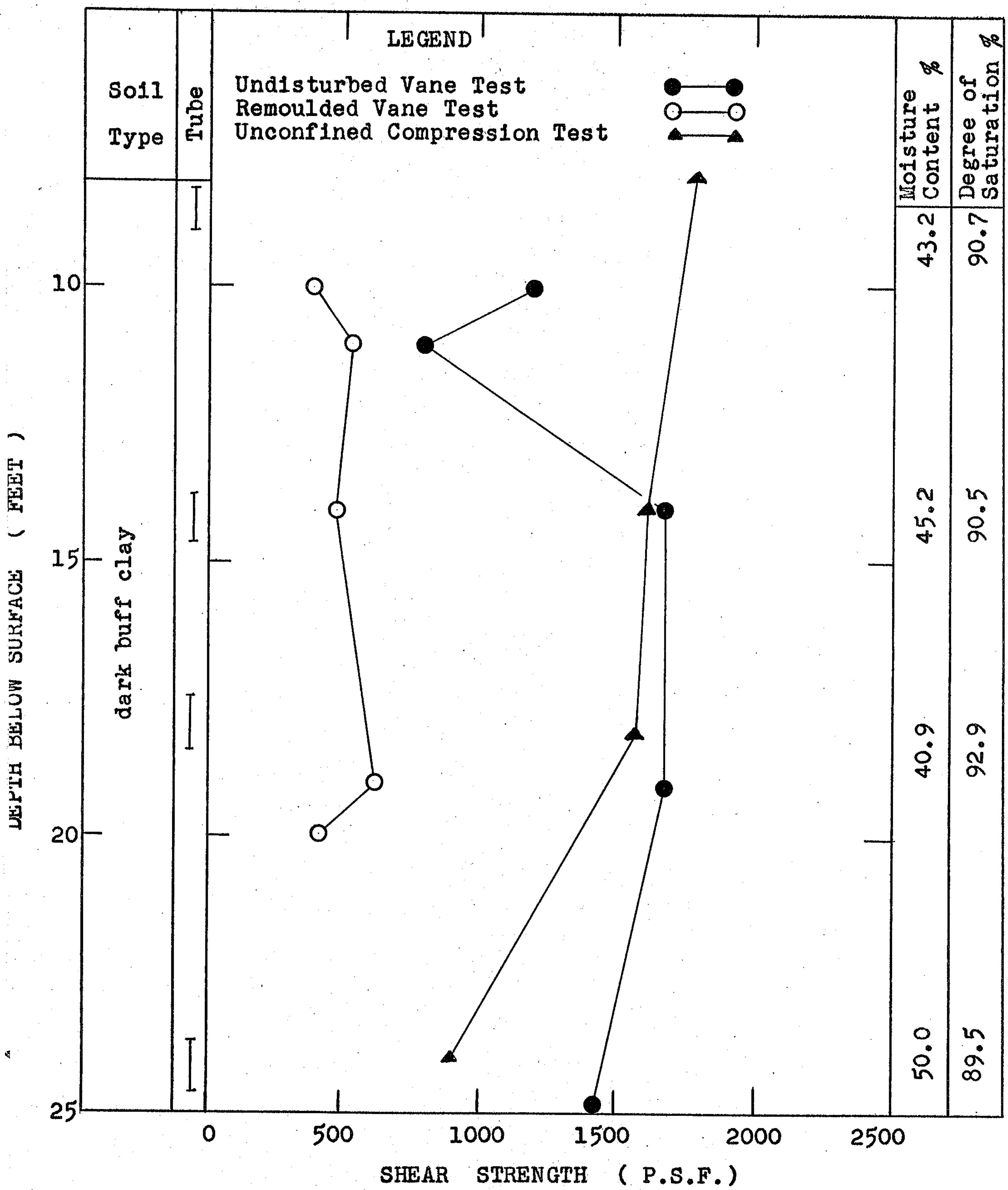


FIGURE 11. The results of tests in Hole no. 5.

The unconfined compression test shear value was approximately equal to the undisturbed vane shear strength at both the fourteen and nineteen foot levels.

At a depth of twenty-four feet the undisturbed vane strength was greater than the unconfined compression value by about fifty percent.

None of the samples tested was saturated but all were similar varved clay soils. The increase in moisture content at the twenty-four foot level may account for the decrease in both undisturbed vane shear strength and the value obtained by the unconfined compression test.

Figure 12 shows the results of tests in hole no. 6. At a depth of five feet the unconfined compression test value was roughly three times the shear strength from the undisturbed vane test. At a ten foot depth the undisturbed vane value was equal to two-thirds of the unconfined compression test shear strength value. At fifteen feet the unconfined compression test value was greater than double the undisturbed vane strength.

At a depth of twenty feet the undisturbed vane strength was greater than the unconfined test value by fifty percent.

The increasing moisture content with increased depth was accompanied by decreasing unconfined compression strength. Although the undisturbed vane strength also decreased from the ten to fifteen foot levels, this strength

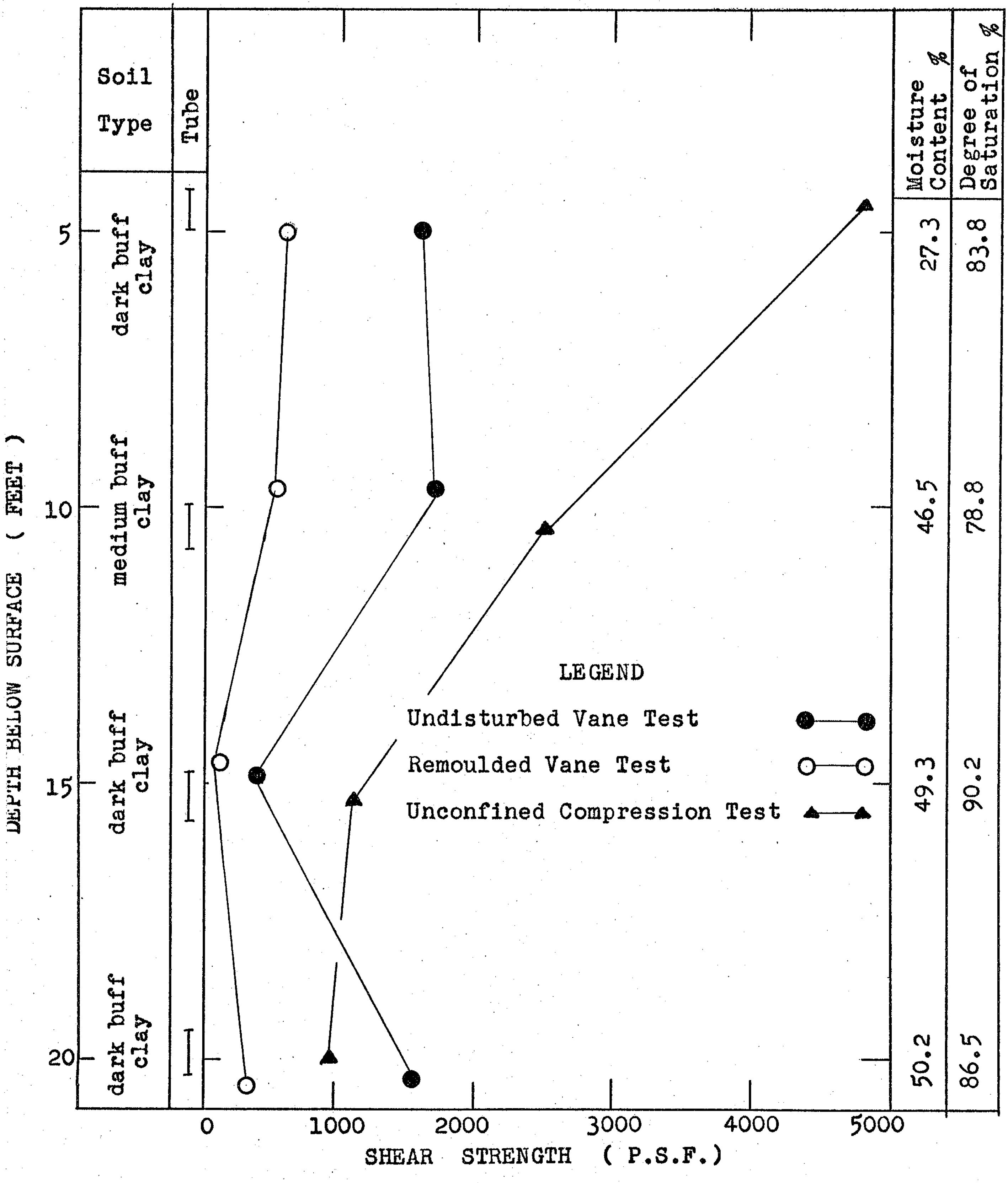


FIGURE 12. The results of tests in Hole no. 6.

value increased slightly from the five to ten foot levels and increased over three times its magnitude from the fifteen to the twenty foot levels.

The soil sensitivity as determined by the vane tests was, approximately, 2.5 at the five foot depth, 3 at ten feet, 4 at fifteen feet, and 4 at the twenty foot level. The soils tested were all unsaturated varved clays and, with the exception of the sample at the five foot level, had moisture contents in excess of the plastic limit.

The results of several tests performed by the Bridge Office, Highways Branch, Province of Manitoba, were not plotted since only one or two tests in any one of the holes were performed in varved clay soil. However this information was plotted in Figures 13, 14 and 18 and was used in compiling Tables No. 1, 2 and 3.

Figure 13 illustrates the relationship between unconfined compression test values for shear strength and those obtained by the undisturbed vane test where the vane tests employed a torque wrench to measure the soil shear strength. The soil types are coded and the depth at which each test was performed is noted. The saturated soils are also differentiated from those which were unsaturated.

In only one case the unconfined compression test value was found to be equal to the undisturbed vane test value. All other results from these tests show the undis-

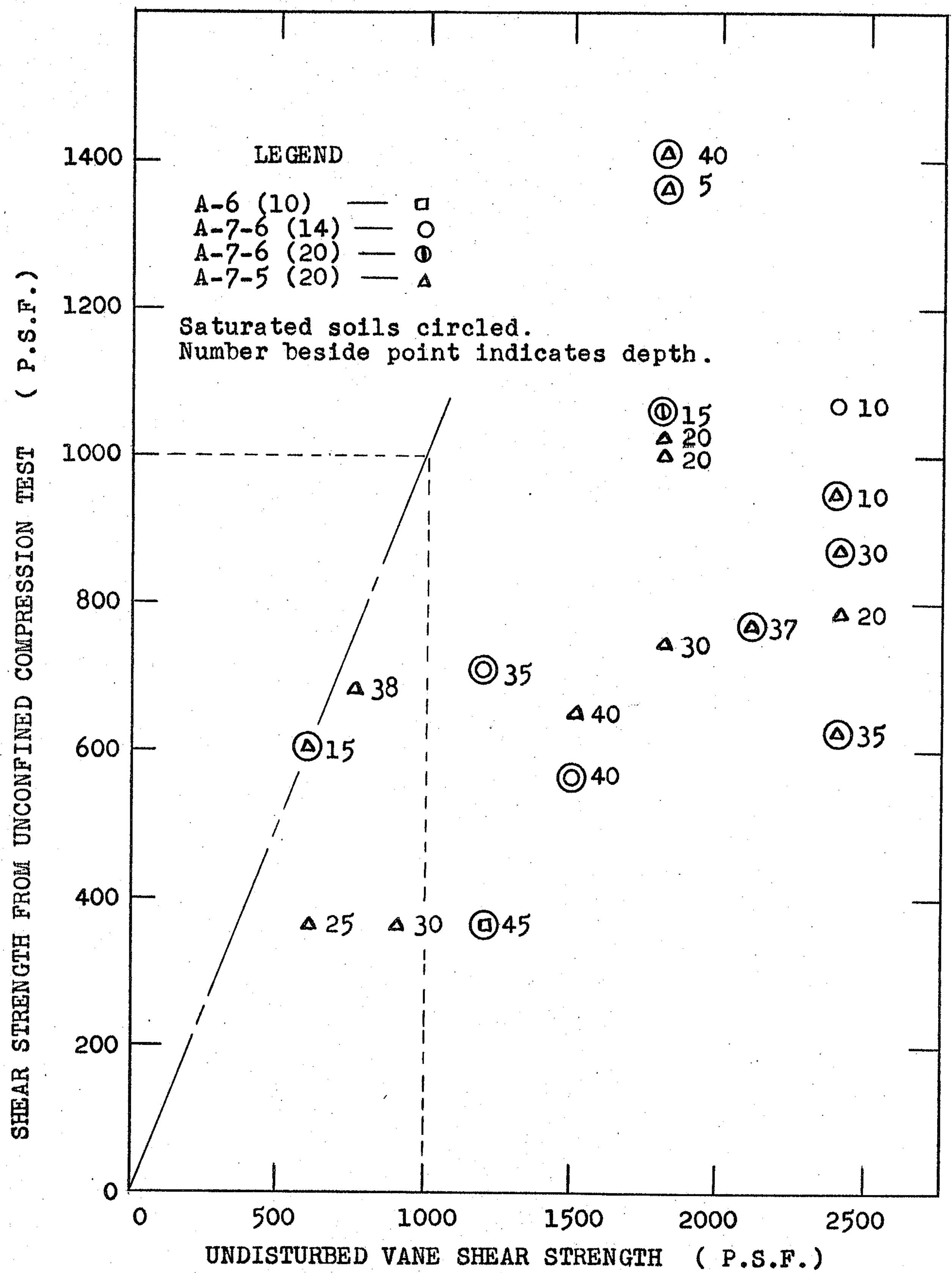


FIGURE 13. Comparison of the results of undisturbed vane shear tests using the torque wrench, with the related shear strehgths from unconfined compression tests on tube samples.

turbed vane test values to be greater.

Figure 14 shows a comparison between remoulded vane test results and the unconfined compression test values. The torque wrench was employed to determine the remoulded vane shear strength.

The majority of the remoulded vane shear strength values exceeded the unconfined compression test results. Whether the soil was in a saturated condition or not appears to have little effect on the test results.

Figure 15 shows the comparison of undisturbed vane test results versus unconfined compression test values for shear strength in buff clay and olive clay. The soils tested were varved but only three samples were saturated. The load required to shear the soil with the vane was measured by means of the stress-strain device.

The unconfined compression test values for the olive clay appear to be independent of depth or degree of saturation. However the unconfined test values for the unsaturated buff clay generally are lower for greater depths. None of the olive clay samples exhibited vane shear strengths lower than those obtained from the unconfined compression test. Four of the vane test values for the buff clay were greater than the unconfined test result and four values were lower.

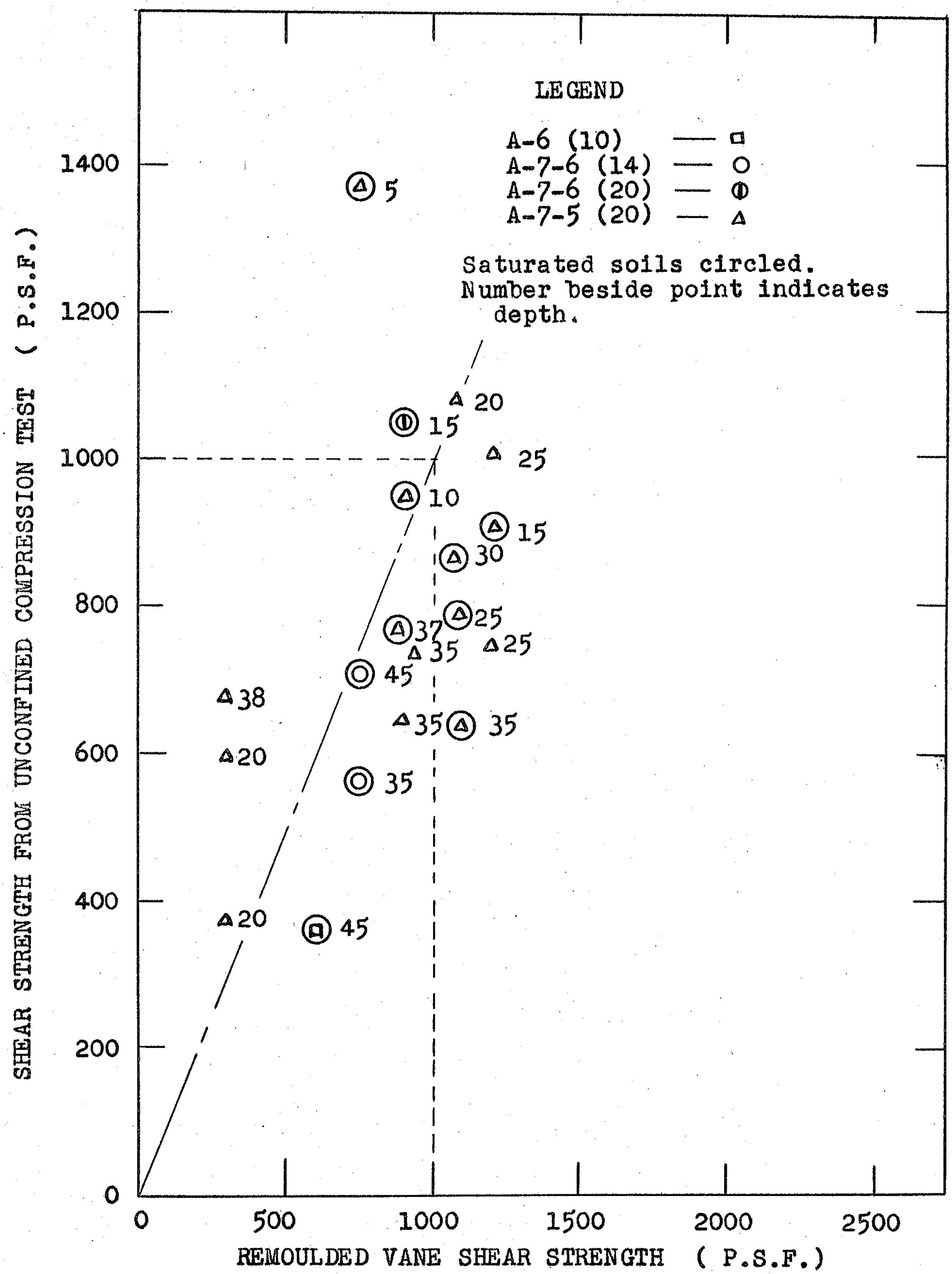


FIGURE 14. Comparison of the results of remoulded vane shear tests using the torque wrench, with the related shear strengths from unconfined compression tests on tube samples.

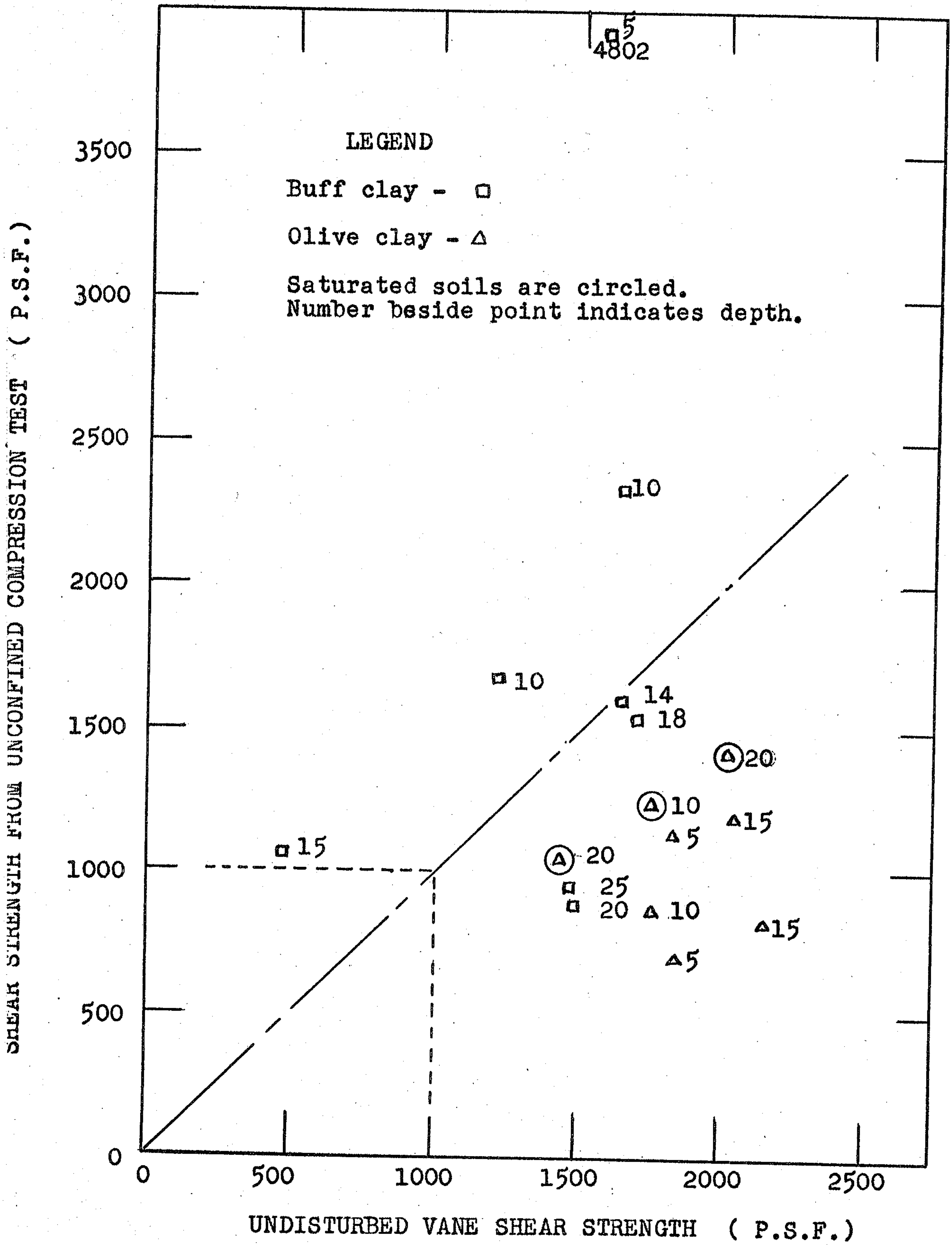


FIGURE 15. Comparison of the results of undisturbed vane tests using the stress-strain device, with the related shear strengths from unconfined compression tests on tube samples.

The results of remoulded vane shear tests versus unconfined compression test values are shown in Figure 16. The stress-strain device was used in these tests to determine the load required for the vane to shear the remoulded soil.

In every case the unconfined test value exceeded the remoulded vane shear strength. The unconfined test values were generally higher for the buff soil than for the olive soil.

In order to evaluate the test results in the broadest terms, all the shear strength values available were used to compile Table No. 1. No distinction was made as to soil type, moisture content or degree of saturation. The data was recorded for each depth at which tests were taken and average values were used where more than one test hole was involved. Because it was apparent that the unconfined compression test values of shear strength were considerably higher in relation to vane shear strengths in cases where the stress-strain device was used to measure vane shear values, these results were recorded separate from the results of tests employing the torque wrench.

The remoulded vane test shear strengths measured by means of the torque wrench were considerably higher than those where the stress-strain device was used with the vane. The average undisturbed vane test

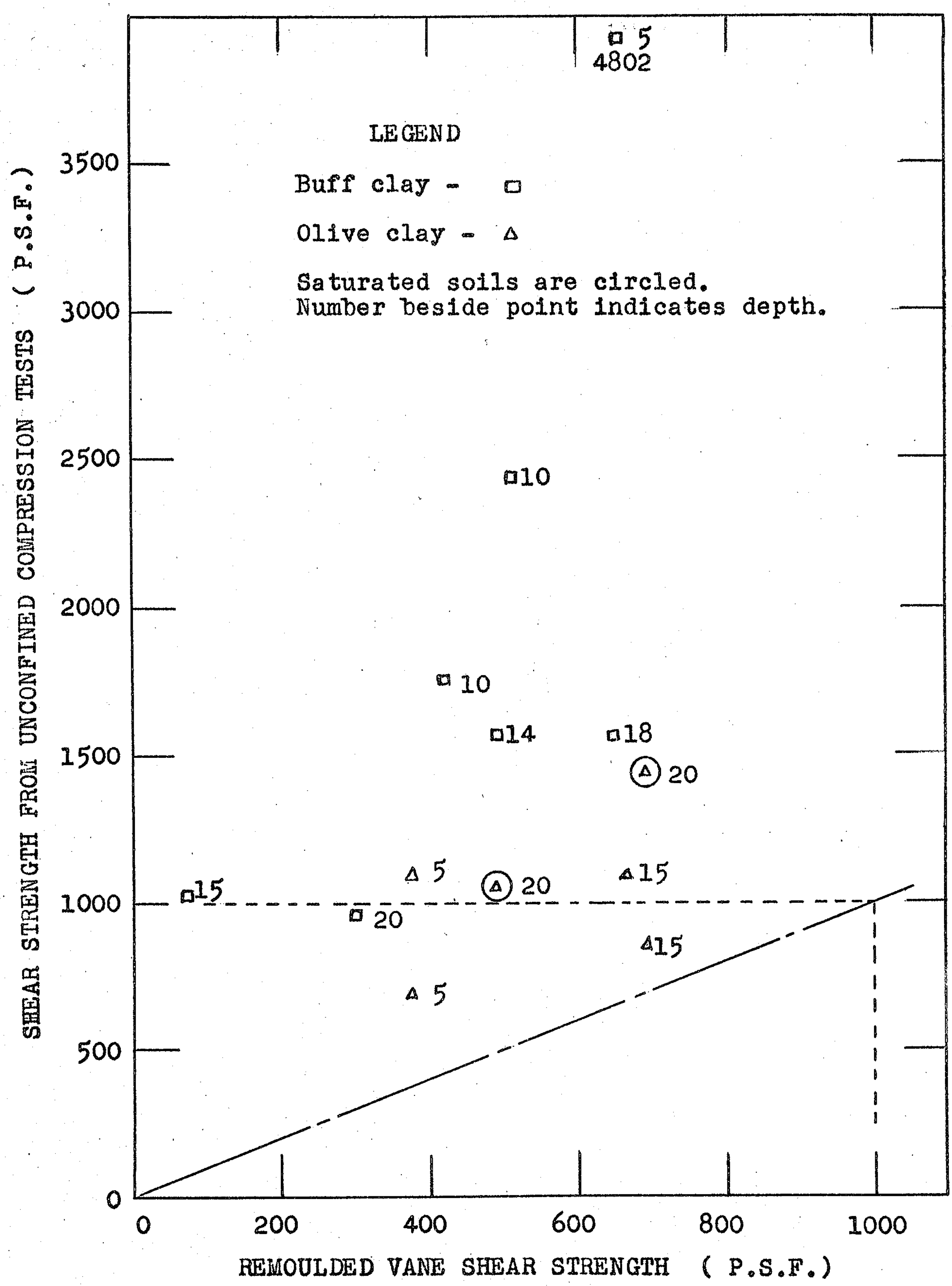


FIGURE 16. Comparison of the results of remoulded vane tests using the stress-strain device, with the related shear strengths from unconfined compression tests on tube samples.

Apparatus Used to Load the Vane	Depth feet	Average Remoulded Vane Shear Strength (P.S.F.)	Average Undisturbed Vane Shear Strength (P.S.F.)	Average Sensitivity	Average Unconfined Compression Test Shear Strength (P.S.F.)	% U.C. is of Undist. Vane
Torque Wrench	5	600	1800	3.0	1375	76
	10	900	2400	2.7	950	40
	15	1020	2025	2.0	973	48
	20	900	1500	1.8	737	49
	25	1200	1800	1.5	881	49
	30	1200	1800	1.5	1217	67
	35	780	1700	2.2	618	36
	40	500	1000	2.0	584	58
Average		887	1753	2.0	917	52
Stress Strain Device	5	470	1753	3.7	2238	127
	10	457	1612	3.5	1593	99
	15	484	1568	3.3	1189	76
	20	472	1654	3.5	1274	77
Average		471	1647	3.5	1573	96
Average from both test methods		679	1700	2.7	1245	74

TABLE NO. 1 - Average strength results obtained for all tests.

strengths were in close agreement for both measuring devices. The soil sensitivity determined by the vane and torque wrench combination averaged 2.0 while the vane and stress-strain device combination indicated an average sensitivity of 3.5.

The average shear strength of all the soils tested, as determined by the undisturbed vane test, was 1700 pounds per square foot. Lea and Benedict determined the sensitivity of the clay in the Winnipeg area to be 2.0 and the shear strength to be 1900 pounds per square foot. The values obtained by Lea and Benedict were determined by using a stress-strain device with a vane of different shape to that used by the author.

The unconfined compression test results which were obtained for samples taken from the test holes in which vane strengths were measured by the torque wrench are noticeably lower than the results for the other test holes. The latter results averaged 96% of the undisturbed vane strengths while the former test results averaged only 52% of the undisturbed vane strength.

Table No. 2 was compiled from most of the test results. The test depth, undisturbed vane strengths, unconfined compression test shear strengths, moisture content, degree of saturation and soil type, were tabulated. The method of determining the vane shear strength, whether by

Depth (Ft.)	Soil Type	Undisturbed Vane Shear Strength (P.S.F.)	Unconfined Compression Shear Strength (P.S.F.)	Moisture Content (%)	Degree of Saturation (%)
5	A-7-5	1843 S*	1194	40.3	93
5	A-7-5	1843 S	717	40.4	96
5	A-7-6	1590 S	4802	27.3	84
5	A-7-6	no test	2585	43.2	91
5	A-7-5	1800 T**	1375	41.2	100
10	A-7-5	1785 S	885	52.3	100
10	A-7-5	1785 S	1231	44.7	96
10	A-7-6	1680 S	2462	46.5	79
10	A-7-5	2400 T	950	50.8	100
15	A-7-5	2073 S	1209	49.2	96
15	A-7-5	2160 S	828	54.4	95
15	A-7-6	1650 S	1634	45.2	90
15	A-7-6	390 S	1087	49.3	90
15	A-7-6	1800 T	1055	45.1	100
15	A-7-6!	2400 T	929	31.5	96
15	A-7-5	2100 T	897	36.6	100
20	A-7-5	2042 S	1440	45.7	100
20	A-7-5	1410 S	1150	46.2	100
20	A-7-6	1485 S	921	50.2	86
20	A-7-5	600 T	597	58.1	94
20	A-7-5	600 T	375	68.9	95
20	A-7-5	2400 T	1123	43.5	98
20	A-7-5	2400 T	856	52.2	86
25	A-7-6	1440 S	904	50.0	89
25	A-7-5	1800 T	1008	57.7	98
25	A-7-5	1800 T	754	52.0	96
30	A-7-5	1800 T	1030	53.8	84
30	A-7-5	1800 T	1404	49.2	98
35	A-7-6!	1500 T	561	41.9	99
35	A-7-5	2400 T	871	52.9	100
35	A-7-5	2400 T	626	57.6	99

! - Group index equals 15.
All other samples have group index of 20.

* - Vane shear values measured by stress-strain device.

** - Vane shear values measured by torque wrench.

TABLE NO. 2. - Shear strength values, moisture contents and degree of saturation obtained from vane tests and laboratory tests on tube samples.

means of the torque wrench or the stress-strain device, was also noted.

Depth, moisture content and degree of saturation individually had no apparent effect on the shear strengths. However it may be seen in several cases that at the same depth and similar moisture content, an increase in degree of saturation was associated with a decrease in shear strength. One such case occurred at the five, ten, fifteen and twenty-five foot levels.

Also in many cases at depths of fifteen and twenty feet for a similar degree of saturation and the same depth, a decrease in shear strength was generally associated with an increase in moisture content.

Table No. 3 shows the typical analysis of the soils tested. It may be seen from the table that although the soils were classified similarly there were considerable differences in their make-up.

A comparison of shear strength versus moisture content for several of the samples tested is shown in Figure 17. The shear strengths were obtained for tube samples by means of the unconfined compression test.

Both soils show a trend of decreasing shear strength with increasing moisture content. The figure indicates that for A-7-6(20) soil, very small increases

C.A.	C.S.	F.S.	Silt	Clay	L.L.	P.L.	P.I.	Class	Type
0	0	6	29	65	61	25	36	A-7-6 (20)	clay
0	0	4	49	47	48	20	28	A-7-6 (17)	clay
0	0	1	8	91	77	31	36	A-7-5 (20)	clay
0	0	1	0	99	104	37	67	A-7-5 (20)	clay
0	0	1	6	93	115	35	80	A-7-5 (20)	clay

Legend: C.A. - coarse aggregate - % retained on a no. 10 sieve.

C.S. - coarse sand - % passing no. 10, retained on no. 40 sieve.

F.S. - fine sand - % passing no. 40, retained on no. 200 sieve.

Silt - particle size from .074 mm to .005 mm.

Clay - particle size finer than .005 mm.

L.L. - Liquid Limit - percent moisture.

P.L. - Plastic Limit - percent moisture.

P.I. - Plasticity Index - percent moisture.

Class - U.S. Bureau of Public Roads Classification, the number in parenthesis denotes Group Index.

Table No. 3 - Typical analyses of the soils tested.

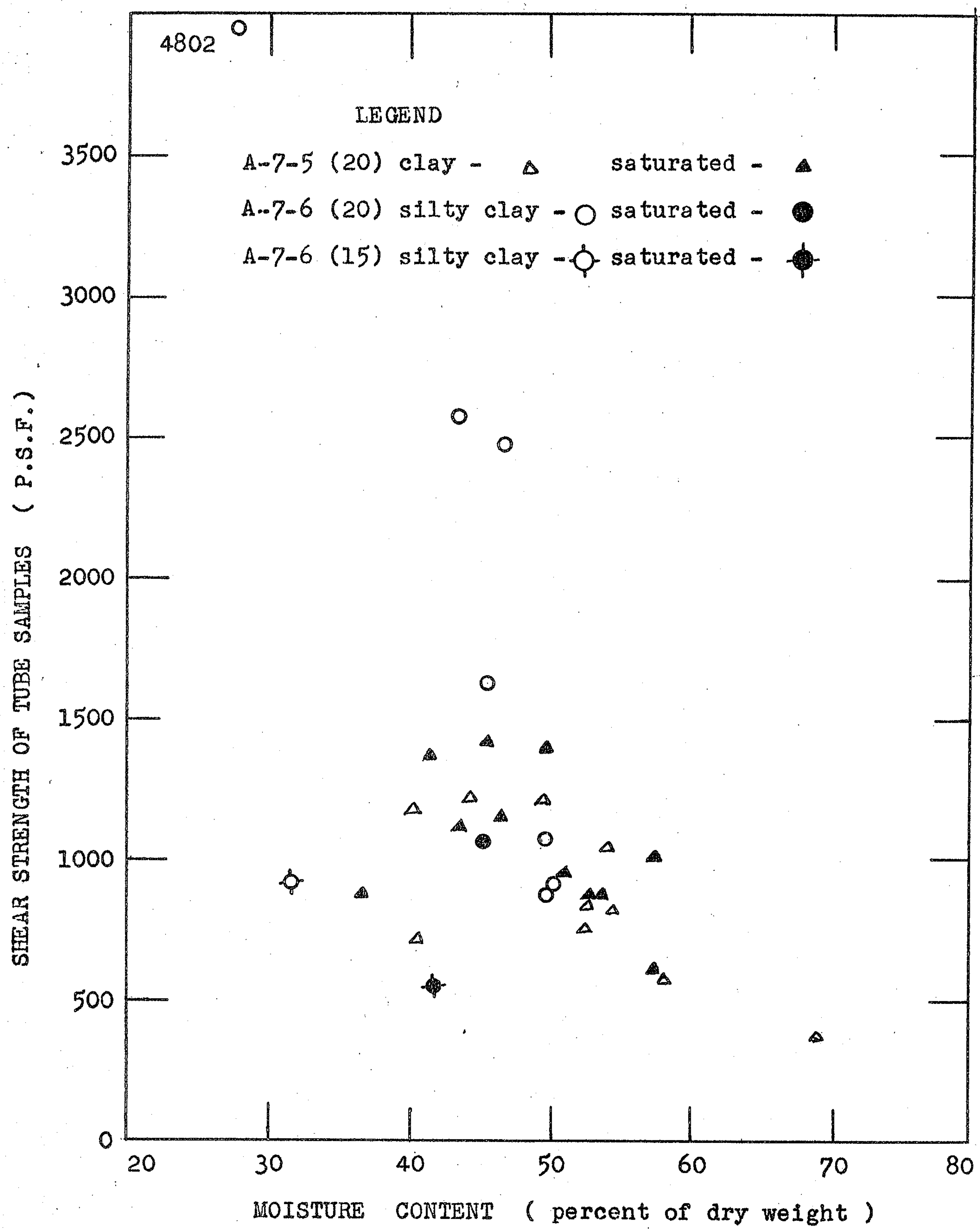


FIGURE 17. Relationship between moisture content and the shear strength of tube samples.

in moisture content result in a considerable loss in strength. The A-7-5(20) soil behaved in a similar manner but required a greater increase in moisture content to achieve the same loss in shear strength observed for the A-7-6(20) soil.

Whether the A-7-5 soil was saturated or not has little apparent effect on the shear strength - moisture content relationship.

Figure 18 shows a plot of undisturbed vane shear test strengths versus moisture content of tube samples obtained at approximately the same depth as the vane shear values. There is no apparent relationship between the vane shear values and moisture content regardless of degree of saturation of the soil tested.

Figure 17 did show a decrease in shear strength with increase in moisture content. This relationship did not appear in Figure 18, possibly due to the fact that the moisture contents were not determined for the actual soil tested with the vane.

The average values obtained at each depth are recorded in Table No. 4. The values of shear strength obtained with the vane remained comparatively uniform. The maximum variation from the average vane shear strength was 298 pounds per square foot.

The average values from unconfined compression tests were, except at the five-foot level, lower than the

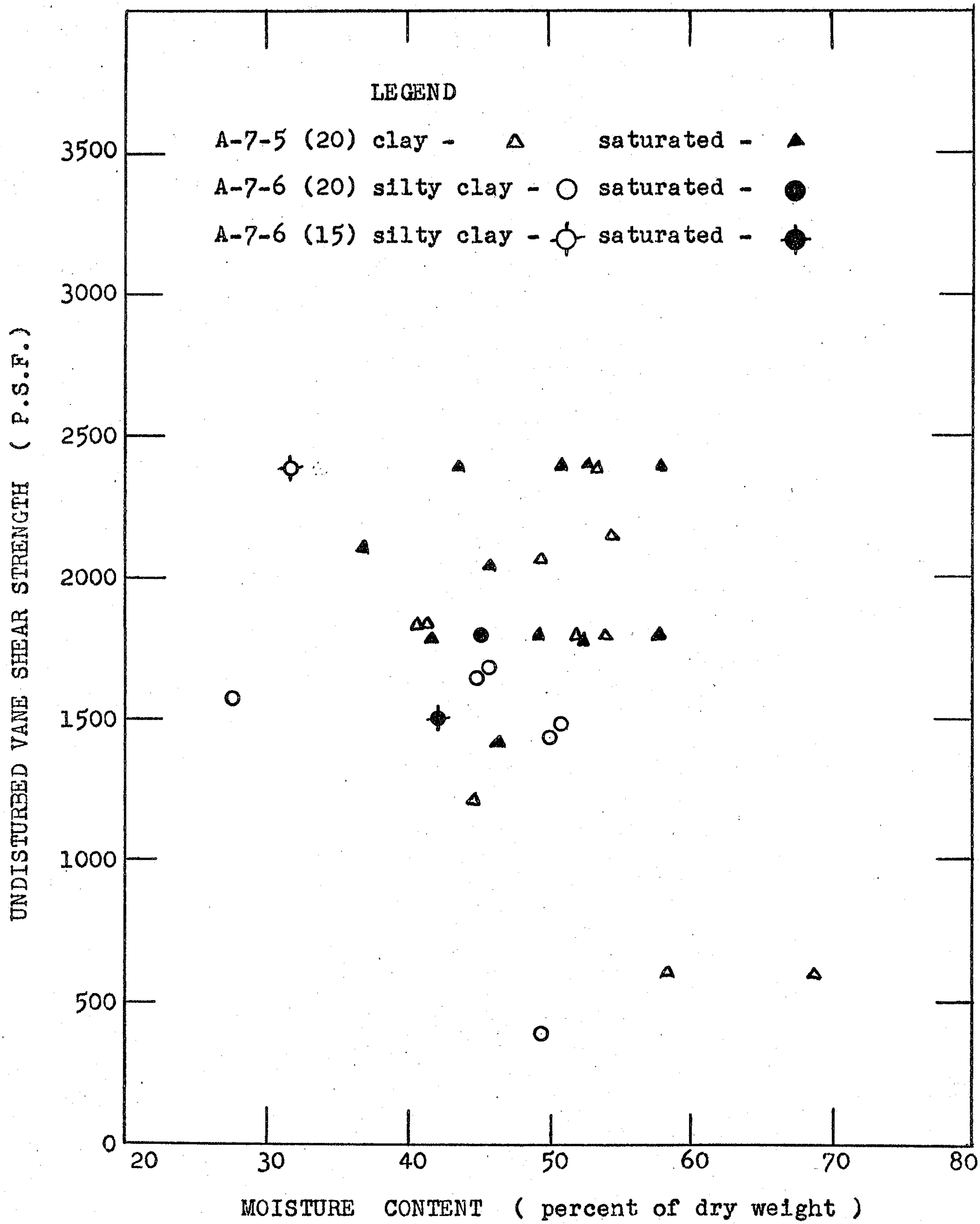


FIGURE 18. Relationship between moisture content and the shear strength from undisturbed vane tests.

Depth (Ft.)	Average Undisturbed Vane Shear Strength (P.S.F.)	Average Unconfined Compression Shear Strength (P.S.F.)	Average Moisture Content (%)	Average Degree of Saturation (%)
5	1769	2134	38.5	93
10	1912	1382	48.6	94
15	1796	1091	44.5	95
20	1562	923	52.1	94
25	1680	888	53.2	94
30	1800	1217	51.5	91
35	2100	686	50.8	99
Average	1802	1188	48.4	94

TABLE NO. 4 Average values of shear strength, moisture content and degree of saturation from all test holes.

average vane shear strengths. There was one particular test result which contributed to the high average shear strength value at the five-foot depth. If this high value was excluded from the averages the comparison would change considerably. The average vane shear strength would be 1828 p.s.f., the unconfined compression test value would be 1468 p.s.f., the average moisture content would be 41.3 percent and the average degree of saturation would be 95 percent.

The average shear strengths determined by the unconfined compression test vary considerably. The maximum variation from the average of the values shown in Table No. 4 was 946 p.s.f. The vane shear values appear to be much more consistent than those obtained from the unconfined compression test.

The results of the tests on tube samples obtained from holes no. 5 and 6 are shown in Table No. 5. There are no apparent trends in any of the test results except that the unconfined compression strength decreased with depth.

The results of tests in hole no. 7 are shown in Figure 19 and Table No. 6. The data recorded in Table No. 6 indicated increasing density with increasing depth and in general there was an accompanying increase in moisture content. The vane shear strength increased with depth

Hole No.	Depth (Ft.)	Unconfined Compression Strength (P.S.F.)	Shear Strength (P.S.F.)	Moisture Content (%)	Void Ratio	Deg. of Sat. (%)	Dry Density (P.C.F.)	Wet Density (P.C.F.)	Vane Shear Strength (P.S.F.)
5	5	5170	2585	44.6	1.323	90.7	72.3	104.5	no test
5	8	3586	1793	43.2	1.289	90.5	73.6	105.4	1200
5	14	3269	1634	45.2	1.315	92.3	72.3	105.1	1650
5	18	3168	1584	40.9	1.191	92.9	77.0	108.5	1680
5	24	1814	907	50.0	1.508	89.5	67.1	100.7	1440
6	5	9605	4802	27.3	0.883	83.8	89.9	114.5	1590
6	10	4925	2462	46.5	1.604	78.8	71.9	105.4	1680
6	15	2174	1087	49.3	1.482	90.2	68.2	101.8	390
6	20	1843	921	50.3	1.575	86.5	65.8	98.9	1485

TABLE No. 5 - Results of tests on tube samples from Holes no. 5 and 6.

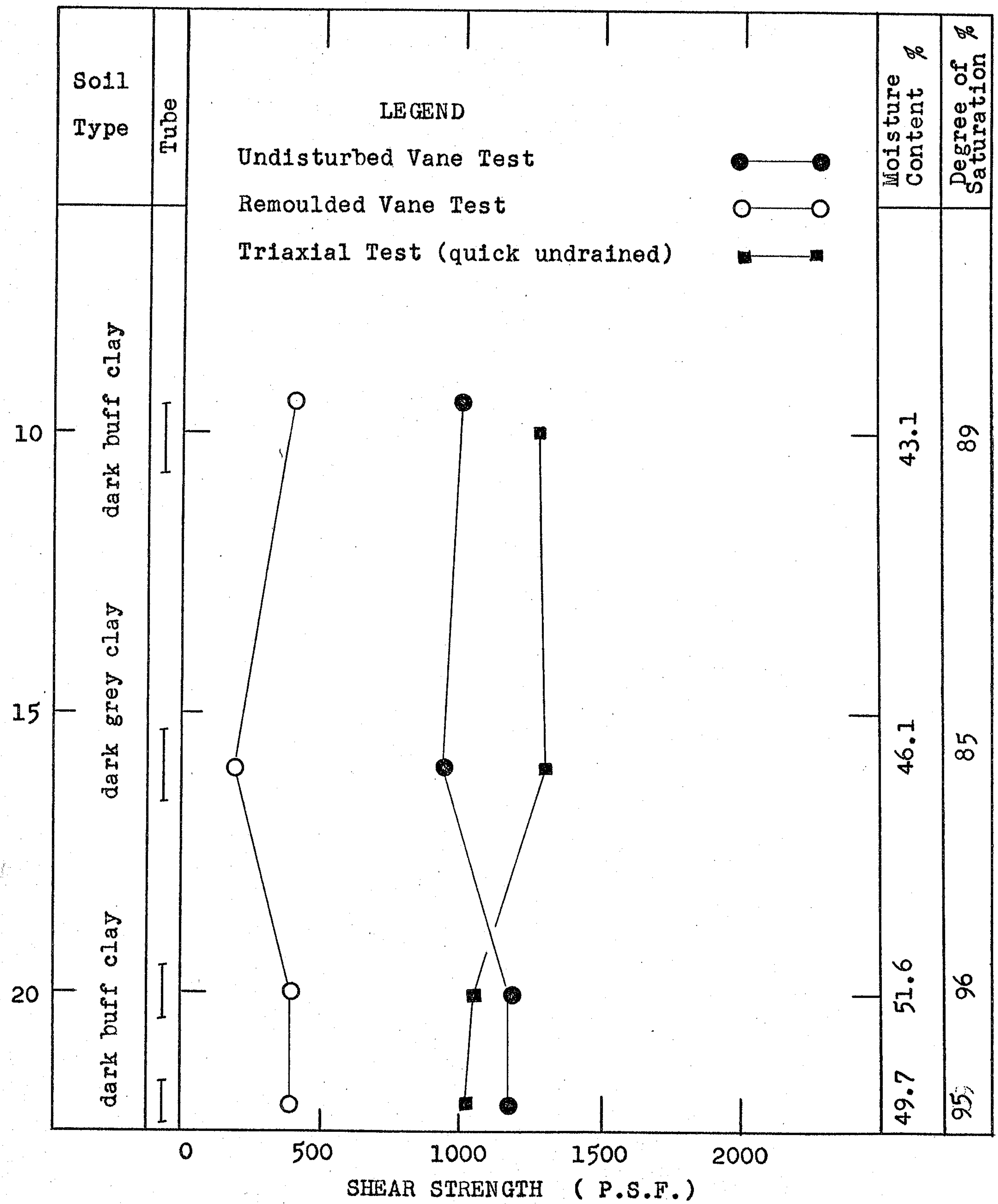


FIGURE 19. Results of vane shear tests performed in Hole no. 7 using the stress-strain device, and results of quick undrained triaxial tests on tube samples.

Depth (Ft.)	Angle of Internal Friction (Degrees)	Cohesion (P.S.F.)	Moisture Content (%)	Void Ratio	Deg. of Sat.	Dry Density (P.S.F.)	Wet Density (P.S.F.)	Vane Shear Strength (P.S.F.)	Sensitivity
10	0	1267	44.1	1.383	86.6	61.0	87.6	990	2.68
15	0	1281	46.0	1.457	85.2	68.6	100.1	937	4.80
20	0	1036	51.6	1.452	96.5	69.1	104.7	1162	2.98
22	0	1008	49.7	1.407	95.7	70.2	105.1	1162	2.96
Avg.	0	1148	47.8	1.425	91.0	67.2	99.2	1063	3.35

TABLE NO. 6 - Results of consolidated quick (undrained) triaxial tests on the tube samples from test hole No. 7.

while the cohesion value of the soil decreased. The confining pressures used to determine the soil strength parameters in the quick undrained triaxial test were 10, 20 and 30 pounds per square inch.

The cohesion value averaged 1148 pounds per square foot and the shear strength from the vane test averaged 1063 pounds per square foot. The cohesion value exceeded the vane shear strength by only 8 percent.

The soils tested in hole no. 7 were unsaturated A-7-5(20) varved clays. The sensitivity of these soils as determined by the vane tests averaged 3.35. The values of the sensitivity are shown in Table No. 6.

CHAPTER X

DISCUSSION OF RESULTS

It is obvious from the test results that the shear strengths obtained with the vane were fairly consistent whether measured by means of the torque wrench or the stress-strain device. This may be seen in Table No. 1. However, the variation in strength results was greater when the torque wrench was used.

The torque wrench has the advantage of being readily obtainable, economical, simple to operate and easy to handle. There are the disadvantages that the rates of stress and strain are dependent upon the operator and there can be some difficulty in rotating the wrench and reading the torque at the same time. The wrench used in these tests could be read only to the nearest five foot pounds of torque which limited the accuracy of the test results to within 300 pounds per square foot.

The stress-strain device yielded fairly uniform results and enabled the operator to maintain reasonably consistent rates of strain regardless of the soil strength. Since the spring extension was plotted on a graph, the torsional load was measured with a ruler. This measurement could be done very accurately. Also, because of the design and operation of the apparatus, the spring load was equal to twice the cable tension. This means that if, for example, the spring extension was only accurate within plus or minus 1/4 inch or 2.5 pounds for the larger spring, the calculated shear strength would be accurate within plus or minus 75 pounds per square foot. The calibration of the spring as shown in Appendix I indicates that the accuracy would be within a smaller range than the 75 pounds per square foot.

The disadvantages of the stress-strain device are its weight and large size and the fact that the operator must be well trained in its use because of the somewhat complicated nature of the equipment.

The vane itself was relatively simple and its operation was straight-forward. In general there was little difficulty in driving the vane or in retracting it, however mechanical equipment which could perform these operations would greatly reduce the physical effort presently required. The vane was found to be capable of withstanding load under test conditions and could withstand rough usage.

The torsion rods were sufficiently strong but the type of connection between the rods should be revised. The joints were flexible to a degree and allowed the torsion rods to flex. In some cases, while driving the vane, the rods flexed enough to come in contact with the walls of the hole. It is impossible to determine the effect on the results of the friction between the rods and the hole.

The author would recommend that the torsion rods be revised to consist of an inner rod and an outer tube. The rod would pass through frictionless bearings inside the tube and be connected to the vane. The outer tube could be equipped with a shield into which the vane could be retracted during the driving operation. This type of vane equipment has been used in Norway and Sweden with a great deal of success.[6]

By far the greatest variation in test results was achieved in unconfined compression tests on samples obtained by means of Shelby tubing. The strengths varied considerably, not only from hole to hole and from one depth to another, but also from one end of a tube to the other. This change in strength in a tube sample was also inconsistent in that the top of one may have yielded the higher result whereas in another case the bottom may yield the higher strength. These variations were independent of soil type.

In Table No. 2 there is some evidence that the shear strength of the soil tested was dependent, to some degree, on moisture content and degree of saturation. An attempt was made to determine whether there was a relation between shear strength and the ratio between moisture content and degree of saturation. No relationship was found and there was insufficient data available to establish whether the introduction of a constant would have yielded a relationship between these values.

It was the intention of this thesis to compare the results of vane shear tests in Lake Agassiz clays with the results of unconfined compression tests on tube samples obtained from the same location. It is apparent that, due to the variation in unconfined compression test results, no direct comparison is possible.

Tube samples, such as those used in unconfined compression and triaxial tests, are widely referred to as undisturbed samples. The author has carefully avoided this terminology for many reasons. The soil is disturbed by advancing the tube [20] and therefore the soil must again be disturbed upon the removal of the sample from the tube. It is impossible to handle all tubes and the samples they contain in exactly the same manner, which introduces the possibility of some samples being disturbed more than others. The samples are also removed from overburden loads and the inherent stresses in the soil are released. Presently accepted theory, as outlined in Chapter IV, indicates that for a saturated soil, the undrained shear strength is independent of confining pressure. However, the shear strength of an unsaturated soil is dependent upon the confinement.

The unconfined compression test does not incorporate any means of artificially replacing the natural conditions under which an in-situ soil would be loaded. The triaxial test is used to simulate confined conditions but is performed on at least a partially disturbed tube sample. The size and shape of the sample, the method of loading and the restriction of the end bearing plates, cause most soils to fail along a definite shear plane. In the soil mass, however, conditions may cause the soil to fail along certain planes of weakness entirely divorced from the failure plane of a small sample.

The results of all the unconfined compression tests showed considerable variation in shear strength values. Figures 17 and 18 reveal that, for saturated varved clays, the range of values from unconfined compression tests is no broader than the range of values determined with the vane although the latter values are of a greater magnitude. This relationship is also true of the unsaturated varved clays but does not appear to hold for the silty clays. The silty clays exhibited extremely high unconfined compression shear strengths in some cases, possibly due to the effect of the sample drying slightly before or during the test. Small changes in moisture content affect the shear strength of silty soils to a much greater degree than for clay soils.

The vane shear test yielded average shear strengths in excess of the average from unconfined compression tests as shown in Table No. 1. This relationship agrees with the findings of several authorities.[1,2,3,4,5,6,13,21] However, tests reported by Bjerrum [22] indicate that when the greatest possible care is taken in obtaining, preparing and testing samples, it is possible to obtain shear strengths from the unconfined test, in excess of the vane test values. This may explain the results of tests in hole no. 6 as shown in Figure 12.

The average shear strength obtained with the vane and torque wrench was 106 pounds per square foot greater than the average value obtained with the vane and stress-strain device. The vane tests using the torque wrench were generally performed in less than two minutes while the vane and stress-strain device tests generally required from two to three minutes. It is generally accepted that higher rates of strain result in greater shear strengths which could account for the difference in the average values for the above vane tests.[1, 6] This reasoning could also be applied to the results of the unconfined compression tests as compared to the results of the vane tests. Since the type of loading to determine shear strength is different for the vane and unconfined compression tests it is difficult to compare their respective rates of strain. In the triaxial and unconfined compression tests all applied stresses are normal, and the complete state of stress is known; in the vane test the applied shear stress is known and the complete state of stress is unknown. In the vane test the strain is angular in nature while the triaxial and unconfined compression tests subject the sample to axial strains.

The time required to perform an unconfined compression test was roughly the same as the time required to perform an undisturbed vane test using the stress-strain device. The rate of strain in the unconfined compression test remained constant throughout the test. This was not the case when the vane and stress-strain device were operated. When the stress-strain device had been set up and the operator began to turn the winch, the cable tightened. As the winch continued to turn, the cable tended to rotate the discs. As the cable tension increased and the soil began to resist the rotation of the vane, the spring began to extend. It was possible to continue winding the cable onto the drum as the spring extended, without appreciable rotation of the disc. At this point the rate of strain approached zero. The torsional force built up in the torque rods causing them to strain, until the torque was great enough to shear the soil. At the point of failure there was a sudden release of load and an extremely high rate of strain. The actual rate of strain in the soil could not be measured using the stress-strain device. The device only measured the spring extension and the rotation of the disc. The strain in the torque rods and that in the soil was not measured. For these reasons it was not possible to compare the curves plotted by the stress-strain device with those which could be plotted from the results of unconfined compression tests.

The variable nature of the rotation of the loading disc on the stress-strain device may have produced an effective rate of strain greater than that of the unconfined compression test and possibly equal to that of the vane test using the torque wrench. This could explain the similarity in vane test results even though the torque wrench tests required only about two-thirds the time required by the stress-strain device. The difference in rate of strain for the unconfined compression test as compared to the vane tests is also a possible explanation for the lower results obtained from the unconfined test.

The fact that the two sets of unconfined compression test results shown in Table No. 1 are dissimilar cannot be readily explained. Identical Shelby tubes were used to obtain the samples for testing. The first set, which yielded an average shear strength of 917 pounds per square foot, was obtained from tubes which were forced into the soil by hydraulic means which provided a slow and steady tube penetration. The second set, with the exception of hole no. 4, was obtained using a drop hammer and yielded an average shear strength of 1573 pounds per square foot. The use of two different sampling techniques could have had some effect on the result. The results in hole no. 4 indicate that the hydraulic driving of the Shelby tubes may result in values from the unconfined compression test of lower magnitude than those from the vane test. The values of shear strength for the tube samples showed a marked increase for the tests which implemented the drop hammer for the driving of the tubes.

J. D. Parsons presented a paper to the Second International Conference on Soil Mechanics and Foundation Engineering which dealt with sampling disturbance.[20] However this paper did not describe the effects of different methods of driving the same type of sampling tube. Photographs accompanying the paper clearly illustrate that Shelby tubes do disturb the soil sample on its peripheral surfaces. The Shelby tubes were driven by a steady, controlled force which, according to Parsons disturbs the soil to a lesser degree than the hammer method. When a tube sample is to be tested in a laboratory it is generally trimmed before testing to remove the disturbed soil. This procedure has not been adopted by the Manitoba Highways Testing Laboratory and since some of the data collected had originated in this laboratory the author decided to maintain the same laboratory test procedures. This may have introduced some errors but does not explain the difference in the values obtained by the two methods of driving the tubes.

Another factor which may have had some effect on the samples which exhibited the lower strengths, with the exception of those from hole no. 4, is the fact that in general these tubes were handled more often and were transported a greater distance than those which yielded higher strengths.

The varves of all the samples tested were approx-

-imately horizontal. The thickness and composition of varves were not determined although, from visual observation, the lighter colored varves were of a more silty soil. In a soil with varves with a ϕ value the apparent shear values would be too high. However, it is believed the soils tested have varves with $\phi = 0$. Therefore, the shear values are valid. The moisture content, which in varved clays has been found to vary considerably within very small depth changes, may have had some effect on the strength results.[23,24]

The triaxial test results yielded shear strengths of the same order as the vane shear strengths. This indicates that, in so far as unsaturated varved clays are concerned, the confinement of the soil results in higher strength values. The introduction of the vane into the soil, the removal of the column of soil above the vane and the fact that the soil failure is directed by the vane rotation would appear to balance the disturbance of the soil by tube sampling. However on the basis of area ratios, 11% for Shelby tubes, and 29% for the vane, the converse would appear true. The results of the few triaxial tests which were performed cannot be used to form any conclusions. No tests were performed in locations where shear strengths could be determined theoretically, such as in the vicinity of a landslide, but it has been found that for non-varved clays the vane results do agree closely with the shear strengths calculated in such cases.[6]

There were a large number of variables in the tests performed which could influence the results. There were two methods of measuring vane shear strength and two methods of driving Shelby tubes. Although the soils tested were generally A-7-5(20) varved clays, other soil types were encountered. Moisture content and degree of saturation were both variable. Depending upon the level of the water table at each test site, the soil could have been normally consolidated or pre-compressed by desiccation. The thickness and composition of the varves may also have varied. Another important factor which should be considered is the performance of the vane test at levels above or below the depth from which Shelby tube samples were obtained. The vane used in these tests was in the development stage and had not been used extensively previous to the tests reported herein, and therefore experience in its operation was limited.

The purpose of this thesis was the determination of the reliability of shear strength values obtained by the use of vane shear apparatus in the varved clays deposited in Manitoba by Glacial Lake Agassiz. The test results indicate that the vane shear test can yield consistent results in excess of those determined by unconfined compression tests and roughly equal to the results of triaxial tests for varved clay soils. It is possible to obtain satisfactory vane shear results using a simple torque wrench for the strength measurement. However, it must be stressed that variations in technique can have a marked effect. For example,the test results of unconfined compression tests performed by the author and those by others show a difference, the author's results being consistently higher. No comparison of calculated shear strengths in landslide areas with vane shear strengths was made due to a lack of information concerning landslides in the test area.

Although the vane test is widely used, further research is required, particularly concerning varved soils. The author would recommend that, although the unconfined compression test is generally accepted as a means to determine soil shear strength, future research should implement the triaxial test for comparison with the vane shear test. The triaxial test would, to some degree, because of confining action, simulate field conditions.

The vane test, as it is normally performed, is an undrained test and thus is strictly applicable only in cases where the undrained strength is representative of the soil strength under the actual condition of loading. Interpretation and use of vane shear test results must also consider whether the soil tested is precompressed or normally consolidated.

The test results were presented in an effort to evaluate the vane shear test in Lake Agassiz varved clays, using an accepted test, the unconfined compression test, as a yardstick or standard. It is indicated that a similar relationship between vane shear strength values and shear strengths determined by the unconfined compression test may exist for local varved clays as well as for non-varved clays. However, due to the scatter of the results, it may be necessary to employ a higher safety factor when using vane shear test strength values.

The author would recommend that future work be directed toward the determination of the effect on shear strength of samples obtained by different types of samplers and methods of sampling. The pertinent factors to be investigated include area ratio, size of vane and its shape and variations in stress-strain measurement. Once the influence of these factors has been determined, a study of the results of laboratory tests on varved clays could be made so that the most suitable test could be chosen for the evaluation of vane shear test results. The influence of moisture content, depth, degree of saturation and the soil type could each become the topic for future research projects. The thickness of varves, their individual composition and their effect on soil behaviour are also extremely important and deserving of some study. Future vane shear tests could be performed in holes adjacent to those from which tube samples are obtained to determine the effect of sampling disturbance by Shelby tubes on the vane shear results. The performance of laboratory vane tests and comparison with unconfined compression tests could yield useful information.

It is essential to studies of this nature that sampling and testing techniques and equipment employed are carefully controlled to ensure the reproducibility of results. The author would also suggest that future vanes used should have area ratios of the order of the area ratio of a Shelby tube. As more data is collected a proper statistical analysis could be performed and some conclusions could be drawn as to the comparison between vane and unconfined compression tests.

APPENDIX

Spring Calibration Data

Temp (°F)	Applied Load (lbs.)	Spring Extension (ins.)	Calculated Load (lbs.)	Error in Calc. Load (percent)	Error in Vane Shear Strength (psf)
72	10	0.93	9.3	7.0	-21
72	20	2.11	21.1	6.5	+33
72	40	3.80	38.0	5.0	-60
72	60	5.76	57.6	3.0	-72
72	80	7.68	76.8	4.0	-96
24	10	0.94	9.4	6.0	-18
24	20	2.03	20.3	1.5	+ 9
24	40	3.98	39.8	0.5	- 6
24	60	5.95	59.5	0.8	-15
24	80	7.82	78.2	0.2	-54
16	10	0.96	9.6	4.0	-12
16	20	2.05	20.5	2.5	+15
16	40	4.05	40.5	1.2	+15
16	60	5.97	59.7	0.5	- 9
16	80	7.87	78.7	1.6	-39
-10	10	0.89	8.9	11.0	-33
-10	20	1.91	19.1	4.5	-27
-10	40	3.86	38.6	1.0	-42
-10	60	5.80	58.0	3.0	-60
-10	80	7.72	77.2	3.0	-84

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