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The effect of grain size on shearing strength of sand

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THE EFFECT OF GRAIN SIZE

ON SHEARING STRENGTH

OF SAND

BY

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INTRODUCTION

One of the main problems of a structural engineer is to secure the essential requirements of a good foundation. In any kind of structural design, it is essential that the structure rests on a firm sub-structure. The failure of a foundation causes the collapse of super-structure no matter how safely it is designed. The requirements of a satisfactory foundation are mainly these:

(1) It must be safe against breaking into the ground. In other words, the load distribution on the soil should not exceed the shearing strength of the soil.

(2) It must not settle enough to disfigure or ruin the structure.

It is evident from the requirements stated above that the design of a safe foundation involves the complete knowledge about the properties of soil on which the foundation rests so that one can determine the safe design load and adjust the settlement accordingly.

In structural engineering the number of cases in which the load is limited by deformation is small on account of the great rigidity of common structural materials. However, for soils the opposite is generally true. Dr. A. Casagrande has conducted quite an extensive program of research on this subject. In one of his articles which was published in the Journal of the Boston Society of Civil Engineers, he states that soil is so much more compressible, and its deformations are so much larger than those of our common construction materials, that in the great majority of foundation problems the loading of the soil must be limited to a value which is much below the safe bearing load of the soil. Those cases where danger of actual rupture of the soil exists are found in engineering structures, such as dams, dikes, quay walls, in railway and highway engineering, in cuts and fills, retaining walls, etc. In other words, structures with unsymmetrical load distribution on the soil are apt to fail by rupture, and their analysis must be based on the shearing strength of soil. However, in the field of building construction the possibility of actual rupture of the soil is very remote. Even though the settlements may amount to many feet, the actual rupture or failure of soil does not occur. The settlement is, nevertheless, due to deformation and compression of soil.

The above discussion of factors which should be considered in foundation engineering emphasizes two principles: First, the importance of soil studies in foundation engineering; Second, in foundation engineering, the settlement is equally important as the shearing strength in design considerations.

Several decades ago, soil mechanics originated as a science under the pressure of necessity in foundation engineering. As compared with the structural engineer, the foundation engineer was, and still is, in a much less favorable condition. The analysis of stress distribution in soil masses is a complicated problem requiring a thorough knowledge of advanced higher mathematics and of theories of elasticity and plasticity. At first, the scientific tools and the methods of testing were lacking. Continuous efforts in this field remedied the situation.

Unfortunately, as Dr. Terzaghi pointed out in his book, the investigations in this field started the application of mathematics to problems in earthwork engineering. As a consequence, more and more emphasis has been placed on refinements in sampling and testing and on those very few problems that can be solved with accuracy. As a result of this, many engineers started to expect too much from the new science.

Then, they find that the results of mathematical derivations contained in its theory, or the results of its laboratory soil tests, cannot always be directly applied to actual practice in a manner similar to structural engineer. It should be remembered that there is no use to assume simplified conditions different from those actually existing in order to permit a mathematical analysis. The consideration of all different factors affecting a foundation problem is essential, and this method requires a considerable amount of experience and skill. As Prof. G. P. Tschebotarioff pointed out, it is an art which involves a strong element of experience and personal skill.

One of the most important fields in soil mechanics is to determine the shearing characteristics of different kinds of soils. If the shearing stress in a body of soil exceeds a certain critical value, the soil fails. The failure may cause a slide, the collapse of a building, or the sinking of a footing into the ground. Since it is the engineer's desire to avoid such accidents, the factors that determine the shearing resistance of soils have received a considerable amount of research and study. Nevertheless, there are still many questions which have no definite answer. One of these questions is: What is the effect of definite grain size on shearing characteristics of sandy soils?

OBJECT AND SCOPE OF INVESTIGATIONS

The object of this study is to determine experimentally the effect of definite grain size on the shearing characteristics of sandy soils. As it will be discussed in detail under the article - The Mechanics of Shear, the shearing characteristics of sandy soils depends on many factors such as grain size and shape, soil structure and relative density, the water content of soil. An attempt is made throughout the entire investigation to determine the effect of definite grain size on the shearing value and the angle of internal friction under the dry and saturated conditions. Furthermore, the results of shear tests on saturated soils as well as dry soils depend to a large extent on the rate at which the shearing force is increased, the dimensions of the specimen, and other details of the testing procedure. Therefore, all the features of the testing procedure described fully in detail to avoid misleading interpretation of the data.

This study by no means is a complete one to determine the shearing characteristics of sandy soils. As mentioned in the preceding article, the shearing characteristics of soils depends on many factors. One of those factors is the grain size; therefore, this study is one of the basic investigations which will lead us to a complete understanding of the behavior of sandy soils. According to Dr. K. Terzaghi: "At present there is no method of predicting the settlement of buildings on sand or gravel foundations and the prospects for discovering such a method are very slight....Current procedures for predicting the buildings on clay foundations are more promising."(1) I am convinced that a com-

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(1) Terzaghi, K. "Settlement of Structures in Europe," Transactions

of the American Society of Civil Engineers, Vol. 103, 1938.

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plete understanding of shear characteristics of sandy soils will be a great help to discover the methods of the settlement estimations.

Also, the definite analysis of shearing values of sand will be helpful in determining the shearing characteristics of mixture of sand and clay. Since soil is an important structural material in highway and railroad construction, the desirable effects of sand on clay mixtures will be a great help to use a sound construction material in highway fills. As F. Akkoseoglu stated in his thesis: "The addition of sand to clays may increase or decrease the shearing strength of clays depending on the amount of sand added and the nature of type of clay."(1) Furthermore, if the voids of clay decreases on addition of sand, the clay may be more useful construction material.

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(1) Akkoseoglu, F. "Effect of definite Grain Sized Sand on Shearing Strength of Clays". Thesis. Missouri School of Mines and Metallurgy. 1950. p. 58.

Finally, this study is a start for the continuous investigations to determine the real value of shearing characteristics of sandy soils as well as to understand the effect of addition of sand to clay mixtures.

MECHANICS OF SHEAR

Basic Concepts

The shearing strength of a soil is separated into two components. One is the cohesion between the soil particles, and the other is the friction between the individual particles of soil. The French engineer Coulomb developed the equation:

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

As it is evident from the above equation, the shearing strength of a soil is dependent on two components - cohesion and internal friction. Cohesion is considered to be that part of the shearing resistance which is independent of all applied normal and tangential forces. The internal friction is assumed equal to the normal pressure multiplied by the coefficient of friction. The coefficient of friction is taken equal to the tangent of an angle ϕ which is called the angle of internal friction.

Before the accurate methods for determining shear values were developed, the angle ϕ for cohesionless soils was determined by a simple method. The natural slope which a mass of soil will assume is called the angle of repose. The angle of repose is taken equal to the angle of internal friction, but it should be clearly understood that the resistance of cohesionless soils to sliding along a plane is composed of sliding and rolling friction and of the interlocking of the soil grains. As it is clearly seen in the above discussion, this method of determining the coefficient of friction may cause an error in foundation designs. Determining the shearing strength of a soil by using the angle of repose may even cause the failure of a structure.

The shearing strength of a soil depends on many factors. As was stated before, important among these factors are grain size and shape, soil structure and density, and moisture content. Also, previous treatment of soil is an important factor which should be considered carefully. These factors complicate the determination of soil shearing characteristics. It is evident from the above discussion that Coulomb's equation expresses the shear problem in a greatly oversimplified form. The shearing strength of a soil depends on a number of other factors not considered by that equation. Also, it is extremely difficult to determine separately the quantitatively correct values of these two components of the shearing strength.

As stated above, the Coulomb's equation is composed of two main parts. First part is cohesion. F. L. Plummer states that some investigators consider cohesion to be merely a special case of internal friction, the pressure being supplied by internal forces rather than by external forces. Cohesion is due partly to the molecular attraction of the grains of soil for each other. This is called true cohesion. The second part of cohesion is due to the binding of the soil mass together by the capillary action of the water in the pores of the soil. This latter part of the cohesive strength is called apparent cohesion. The capillary action of the water in the pores of the soil should be considered very carefully. When water and air are present in soil, a concave surface forms on the pore water where it comes in contact with the air. This is called a film. The surface tension in these films binds the grains together. Also, the pores of soils serve as capillary tubes which cause water to be present in soils above the ground-water level. The height above the water surface to which water is raised by capillary action varies inversely with the diameter of the pores which is a function of the grain size. The grain size of sand is so large that it possesses practically no capillarity.

If we investigate true cohesion, it is evident that the molecular attraction between two bodies varies directly as the product of the masses of the bodies and inversely as the squares of the distances between their centers of gravity. Since the size of sand grains is large, the distance between their centers of gravity is so large that the molecular attraction is negligible, but the grain size of clay is so small that molecular attraction is an important factor in the cohesive strength. As stated in the preceding paragraph, the capillary action in sand is small because of the large size and the large pores of sand grains. While in clay, with its small grains and pores, it is considerably important. Therefore, the shearing resistance of sand is due to internal friction, while clay possesses both internal friction and cohesion. However, both true and apparent cohesion is affected by many factors which are not clearly explained. As G. P. Tschebotorioff believes, true cohesion can sometimes develop even in sands if some cementing agent is present to bind the particles together in a dry, moist, or submerged state.

The second part of Coulomb's equation is dependent on internal friction. Coulomb's law states that if one body slides over another body, the friction force will equal the normal pressure multiplied by a constant f known as the coefficient of friction. f is assumed to be equal to tan ϕ.

The shearing resistance of cohesionless materials such as sand is mostly due to internal friction, and the internal friction of cohesionless soils is the result of two effects: (1) The resistance to sliding of grain on grain (2) The interlocking of the particles. The interlocking should be carefully considered especially in dense sands. Furthermore, F. L. Plummer believes that the angle of internal friction for cohesionless soils is not a constant due to the interlocking of irregularities on the surfaces of the interface, and the variability of these irregularities. As some investigators in soil mechanics stated, the internal friction increases with pressure. When sand grains are subjected to pressure with restrictions to lateral flow, each grain becomes more intimately embedded with surrounding grains and the number of points of contact is increased. The pressure effect might increase the shearing strength of the soil. The interlocking of the particles under excessive pressures may effect the shearing strength of a soil very strongly. It is important to note that the interlocking effect plays a more important part in dense materials. Therefore, the coefficient of friction should be carefully adjusted to the actual existing conditions in a soil. It is very clear that the angle of internal friction depends upon how much the mass has been compacted.

Adsorbed films of liquid are also believed to influence the sliding friction strongly. The antilubricating action of water on some substances is proved, for instance on steel or on glass. Terzaghi states that water in soils has an antilubricating effect. "According to the concept that the frictional resistance between soil grains is equal to the shearing strength of the actual contact layer between them, it was assumed that when a film of liquid separated the two solid surfaces, the frictional resistance equaled the shearing strength of the adsorbed film. An increase of normal pressure presumably decreased the thickness and thereby increased the shearing strength of the film."[(1)] Tschebotorioff

(1) G. P. Tschebotorioff, "Soil Mechanics, Foundations, and Earth Structures." McGraw-Hill Book Company, New York, 1951. pp. 122-123

and Welch performed experiments on this subject. They tried to eliminate the interlocking effect by polishing the surfaces of minerals. The experiments proved that a large difference existed between the dry and the moist conditions, and the slightest humidity in the surrounding air could rapidly affect the results, most probably because of the formation of adsorbed moisture films on the surfaces of the minerals.

Tschebotorioff, also, proved that there was a distinct difference between hydrophilic minerals, quartz and calcite, which have an affinity for water, and the two hydrophobic minerals of the talc variety, which are water repellent.

His conclusions were: "In the case of quartz the friction coefficient in a completely dry condition was found to be 4.5 times smaller than in a completely submerged condition. In the case of calcite it was 2.5 times smaller. For both minerals the friction coefficients in a slightly moist and in a completely submerged condition were practically identical. This indicates that the increase of frictional resistance, as compared with the dry condition, was not caused by any surface tension phenomena, but by the changed properties of the water in the adsorbed layer, which had acquired semisolid characteristics. A reverse relationship was observed in the case of the hydrophobic minerals. Water had a slight lubricating effect and decreased the frictional resistance."[2]

(2) ibid., p. 124.

Tests involving a combination of two different minerals were also carried out, and the friction coefficients of such combinations of minerals had values intermediate to the ones obtained from tests in which only one of these two minerals was used.

Tschebotorioff, also, states that the sliding friction between different types of minerals can vary within wide limits and appears to be primarily dependent on the nature of the films absorbed on their surface, and only to a lesser extent on the degree of roughness of the mineral surfaces in contact with each other. Natural soils are composed of mixtures of hydrophilic and of hydrophobic minerals, but the exact proportions have not yet been studied in detail. It is proved that the hydrophilic minerals appear to predominate in most cases.

The effect of soil structure on the shearing characteristics of soils is evident when different kinds of soils are tested. When a cohesionless soil such as sand is subjected to a shearing force under a constant normal pressure, there is a volume change. This indicates that there is a corresponding change in the soil grain structure. If a loose sand is tested under the conditions described above, there will be a decrease in volume when the shearing force increases. However, a volume is reached at which shearing continues with no change in volume. If the same soil is tested after compacted into a dense state, the shearing force increases the volume until failure suddenly takes place. Dr. A. Casagrande has performed an extensive program of research on this subject, and his conclusions are:

1. Every cohesionless soil has a certain critical density, in which state it can undergo any amount of deformation or actual flow without volume change.

2. The density in the loose state of many cohesionless soils,

particularly medium and fine, uniform sand, is considerably above their critical density. Such materials in their loose state tend to reduce their volume if exposed to continuous deformation. If the voids are filled with water and the water cannot escape as quickly as the deformation is produced, then a temporary transfer of load onto the water takes place, and the resulting reduction in friction impairs the stability of the mass, which can lead, in extreme cases, to a flow slide.

3. If a cohesionless soil is below the critical density, then it can stand any disturbance without danger of a flow slide. Whenever there is any tendency for the mass to deform, the water in the voids has a restraining influence.

4. Many coarse-grained and very well graded mixtures of cohesionless soils are in their loose state approximately at the critical density. This fact, combines with their large permeability, renders them relatively stable against any disturbances, even in the loose state.

5. Cohesionless soils in a state above the critical density can be efficiently compacted, and thereby stabilized against any disturbances, by means of special vibration machinery.

The above discussion emphasizes on the fact that many slope failures are due to the structure of soil. The change in volume of soils subjected to shearing stresses may cause the failure of a slide or a retaining wall. F. L. Plummer states that "the volume changes which accompany shearing deformations are of utmost importance. In planning shearing tests and their interpretation we must be prepared to measure this effect and to consider its significance. If we disturb the structure of our soil sample, we cannot be sure that the shearing strength which we measure equals that of the undisturbed sample."(1)

(1) F. L. Plummer, "Soil Mechanics and Foundations", Pitman Publishing Co., New York, 1940. p. 101.

The effect of water on shearing characteristics of a soil is a very important factor which should be considered during experiments. The impossibility of rapid changes in water content due to low coefficients of permeability complicates the experiments to obtain the shear values of fine-grained soils. If a shearing test is made on such a soil under a normal pressure, the complete consolidation under this load will not take place until all excess water has been squeezed out. Since the coefficient of permeability is very low, considerable time will be required for a complete consolidation. If the test is made before the complete consolidation, the part of the load will be resisted by water which has no shearing strength. Many shear tests are made after nearly complete consolidation occurs to eliminate this effect. However, it is evident that the complete consolidation eliminates the error due to the part of the load carried by the water. Also, the same consolidation changes the structure of the soil. Therefore, the measured shear resistance is changed. This proven by the hysteresis loop which occurs when the load during a shearing test is alternately applied and released. As F. L. Plummer stated, "The measured values of the angle ø and the cohesion depend not only on the material itself but also on the past history of that material."[1]

(1) ibid, p. 102.

The water contained in the minute pores of soil is called pore water, and the pressure which is produced is called pore pressure. This pore pressure resists external pressures which tend to compress the soil. Then, by partially holding the soil grains apart, the pore pressure reduces the frictional resistance between grains. This, finally, reduces the shearing strength. Therefore, the tests of the shearing strength of a given soil which do not take into account the pore pressure are not dependable.

As it was evident from the preceding paragraphs, the shearing strength of a soil depends on many factors, and these factors should be considered carefully before a final decision is given. The tests of the shearing strength of a given soil which do not take into account the grain size and shape, soil structure and density, previous treatment of soil, and moisture content are of no value.

SHEARING CHARACTERISTICS OF SANDS

Many tests have been performed by several investigators to determine the shearing resistance of sandy soils. The tests performed mostly investigated the natural soil samples, and the effect of definite grain size was not determined. However, some investigators believed that it was a controlling factor.

The tests performed by A. Casagrande on the direct shear machine, and the values obtained compare well with the results of cylinder tests. One of the conclusions is: The resistance of cemented sands increases with the pressure at a considerably lower rate than that of uncemented sands. Also, sands, consisting of round grains, have a lower resistance but a higher modulus of elasticity than angular sand. Since the contact surfaces are spherical rather than sharp, the resistance to shearing is decreased considerably.

The results of the various shearing tests on sands by Dr. K. Terzaghi are stated in his book. As it is evident from many of his discussions, he rather considers the practical side of soil mechanics. "The angle of internal friction ϕ of a perfectly cohesionless sand in a loose dry state is approximately equal to the angle of repose. Before the angle of repose is determined, the sand should be dried in an oven; otherwise, the values obtained are too high. The value ϕ of a given sand in a thoroughly compacted state under a pressure of less than 2 tons per sq. ft. is 5° or 10° higher than its angle of repose. On the basis of these statements, the value ϕ can be estimated roughly without shear tests. In practice, more accurate values are seldom required. The angle of internal friction ϕs of a completely submerged sand is about 1° or 2° less than the value of ϕ for the same sand at the same relative density, but in a perfectly dry state."[1] As Dr.

(1) K. Terzaghi, "Soil Mechanics in Engineering Practice", John Wiley and Sons, Inc., New York, 1948. p. 92.

Terzaghi's discussion above shows, the application of soil mechanics to many practical problems will considerably increase after the field engineers can determine the approximate shearing values of soils without performing complicated tests.

The other important shearing characteristics of sands are: As it was discussed before, dense sands expand during shear. Also, loose sands contract during shear. The critical void ratio and liquefaction phenomena causes the failure of many super-structures as well as the flow slides of embankments or earth dams. The liquefaction of a loose sand as a result of sudden shock may be demonstrated by the laboratory experiments. The contraction and the temporary loss of supporting capacity of a loose submerged sand as a result of sudden shock can be prevented in many cases. The importance of the compaction of sands is very necessary for all cases where earthquakes, sudden shocks due to blasting may decrease its shearing resistance. However, it is difficult to establish an exact numerical value for the minimum densities which are safe. Many investigators believe that no danger of liquefaction can be expected if the sand is so dense that its natural void ratio lies below the value of critical void ratio. As G. P. Tschebotarioff stated, "Full-scale field-test measurements under conditions stimulating shock waves of intensities corresponding to actually possible values are needed to establish possible limit values of the resulting transitory excess pore pressures compatible with stability."(1)

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(1) G. P. Tschebotarioff, opus cited. p. 150.

The direct relationship between the density of sand and its angle of internal friction ø has been established by numerous tests in different localities. It is shown that the angle of internal friction ø of the sand is directly dependent on the sand density and increases with it. As G. P. Tschebotarioff states in his book of foundation engineering, there is no evidence that under static conditions in the field the actual value of friction angle ø would be any different from the laboratory value. The lowest static values of ø to be recorded for any sand by triaxial or direct-box shear tests equal 28°, the highest 45°, depending on the absolute density of the sand. Values up to ø = 60° also obtained by means of double-ring shear tests on sand, but this appears to indicate jamming of grains in this type of apparatus. Generally, the increase in value of friction angle at greater densities is due to better interlocking of grains. G. P. Tschebotarioff also, believes that the angle of internal friction o is essentially the same for a completely dry sand and a fully submerged sand.

METHODS OF SHEAR TESTING

The three common methods of shear testing are direct shear, cylindrical, or triaxial, compression, and torsional shear. In a direct shear test the soil is stressed to failure by moving one part of the soil container relative to another. As it will be described in detail later, the direct shear testing methods were used in this research work.

In a torsional shear a circular column of soil is subjected to a twisting moment. The moment is normally applied through a disk at the top or bottom, and the disk has ribs to prevent slippage between the disk and the soil. A lateral pressure can also be applied during tests. The cylindrical compression test, also called the triaxial test, loads axially a cylinder of soil to test its failure.

As Lambe described in his book, "The main advantage of the torsional shear test over the other two types is that the cross section of the soil remains more nearly constant during shear. In both the other tests, the sample is often badly disturbed at ultimate failure, this distortion causes non uniform stresses and strains within the soil, and often makes it difficult to measure accurately the effective area of the failure surface. The most dependable measure of the ultimate shear strength of a soil, therefore, can probably be obtained from torsional shear tests. This advantage, however, is more than outweighed by the fact that the shear displacement vary as the specimen radius, thus exaggerating the progressive failure. This effect is reduced somewhat if an annular-shaped soil specimen is used rather than a solid cylinder."[1]

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(1) W. Lambe, "Soil Testing for Engineers", John Wiley and Sons, New York, 1951. p. 90.

There are mainly two types of loading conditions under which the shear experiments are performed: controlled-stress and controlled-strain. In the controlled-stress type, the horizontal force is gradually increased until complete failure occurs. The shearing displacements are measured by means of a dial gage. This type of loading is used in this research work.

In the controlled-strain type of test, the shearing displacements are induced and controlled at constant fixed rate. The shearing resistance offered to this displacement by the soil specimen is measured.

Also, the rate of shearing and the conditions of specimen drainage have a considerable effect on the final values. Dr. Terzaghi describes three types of tests: slow tests, consolidated-quick tests, and quick tests. In a slow test both the load and the shearing force are applied so slowly that the water content adapts itself almost completely to the change in stress. In a consolidated-quick test complete consolidation under the vertical load is followed by shear at constant water constant. In a quick test the water content of the soil sample remains practically unchanged during the application of both the vertical load and the shearing force. Usually, slow tests are used in any soil, and the other two methods are mostly for clay.

DIRECT SHEAR TEST METHOD

A direct shear test machine was used to test the effect of definite grain size on the shearing strength of sand. The picture of the testing machine is shown in figure 1.

The apparatus consists of a lower frame that is stationary and an upper one that can be moved in a horizontal direction. The soil sample is located between two porous stones. and they serve as drains during the consolidation of saturated samples. As the surface of contact of the porous stone is grooved, the slippage between sample and stones during the process of shearing is prevented.

The shear area of the shear box (Fig. 2) is 31.2 sq. cm. The upper part of the shear box is attached to the lower frame by means of screws. The lower part of the shear box has a hook which is connected to the loading device. The lower section is attached to the base. There is also a waterpan (Fig. 1) to hold the water for a definite length of time to saturate the samples. Set screws are provided to hold the top and bottom parts of the shear box stationary during the preparation of the samples. The lower part of the shearing box moves on ball bearings to reduce the frictional resistance to a minimum during tests. The remainder of the shear box consists of supports for Ames dials, a ball bearing on the shear box to apply a concentrated normal load, and loading hangers.

During the experiments, a normal load is applied first, and the normal load is kept constant. The shearing force is applied by pulling the lower part of the shear box. The horizontal force increased gradually until the sample fails. The normal force is applied by means of a yoke which rests on a ball bearing (Fig. 1) A loading pan is

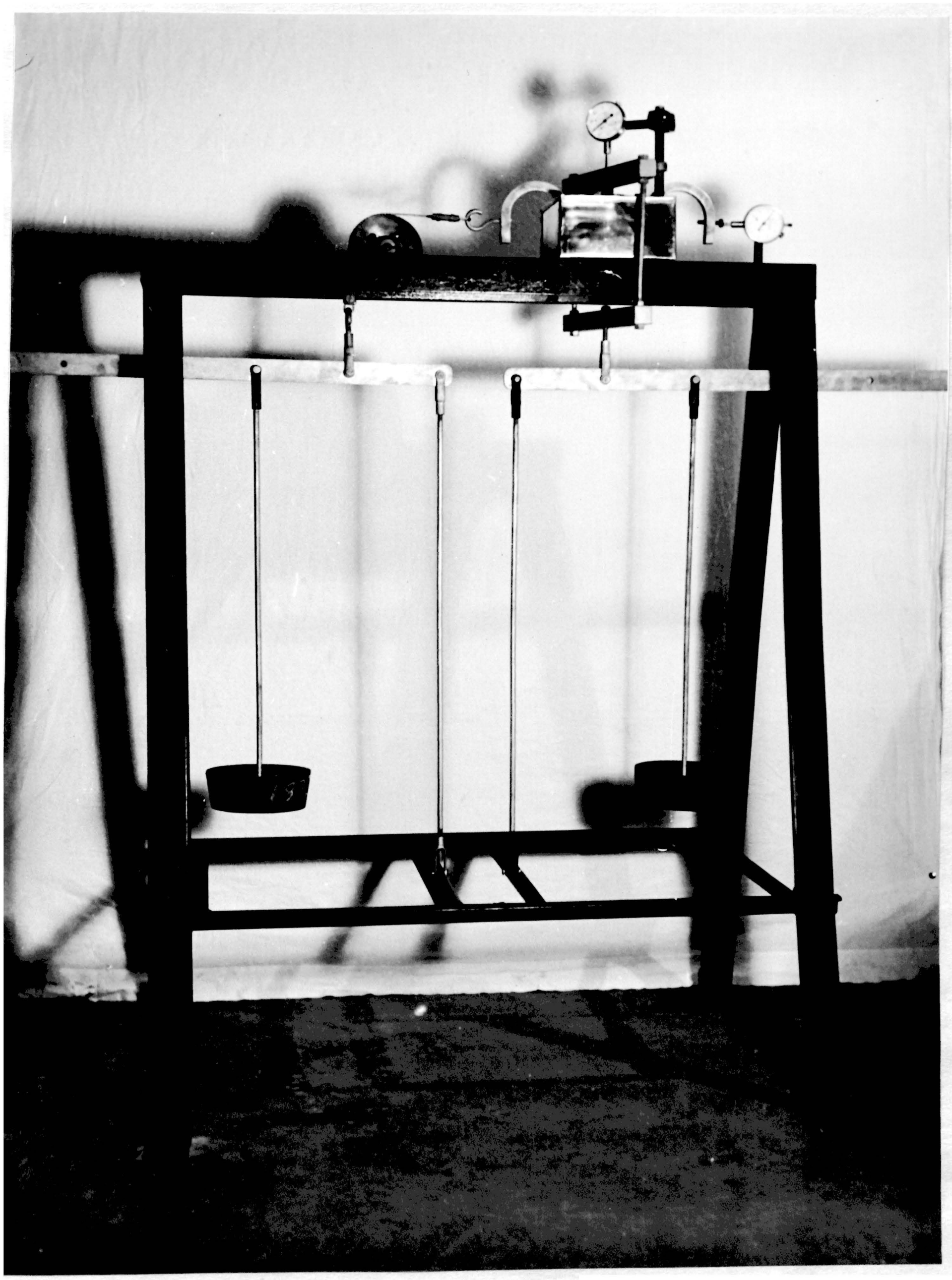

Figure 1.

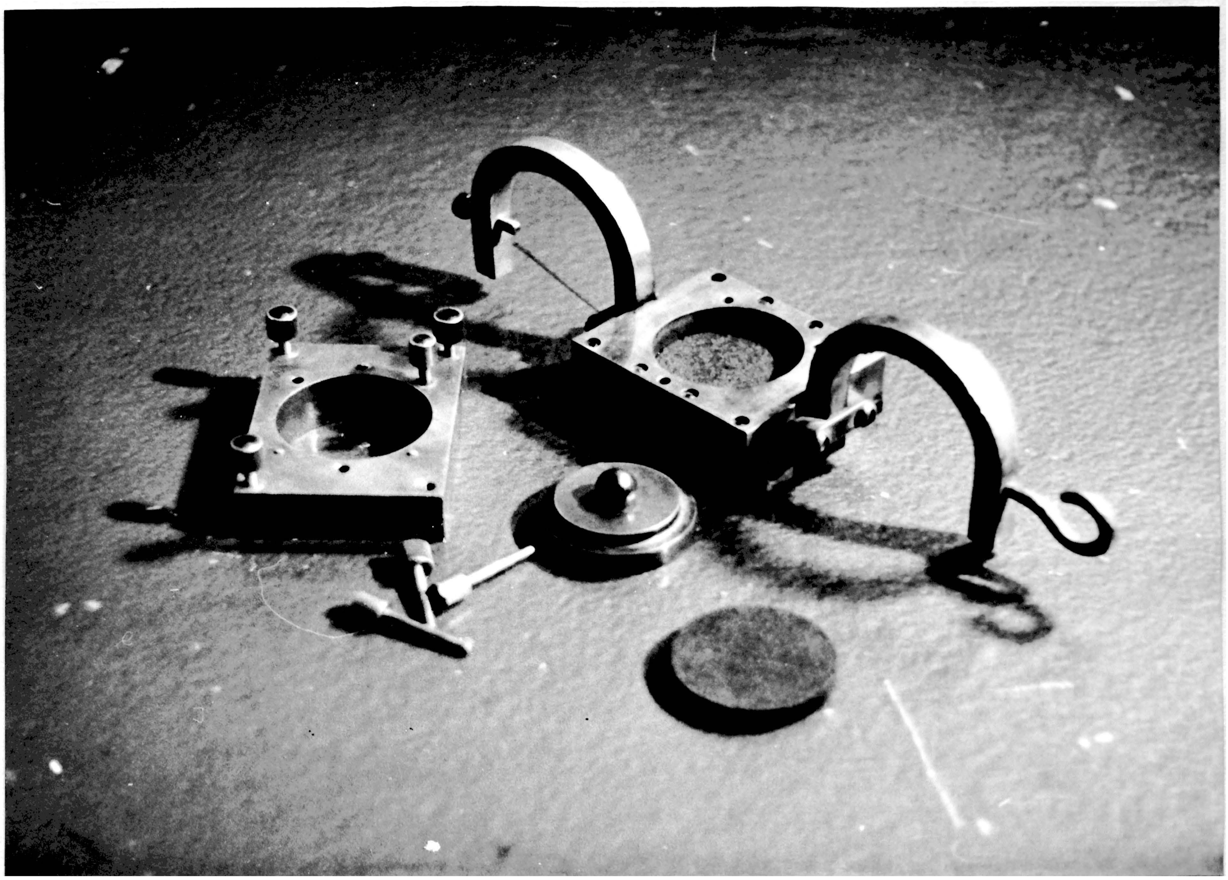

Figure 2.

attached to the yoke (Fig. 1.) The horizontal load is applied by means of a flexible cable, through a pulley. One end of the cable is attached to the lower part of the shear box, and the other end is connected to a beam which has a loading hanger.

The soil sample to be tested by the direct shear test method is first placed in the shear box. At the same time a normal load is applied, and it is kept constant during the experiment. The shearing load is applied gradually. The movement of the lower frame relative to the upper frame is shown on Ames dial which is attached to the movable frame. When the sample failed, the shearing force recorded. The angle of internal friction (tan o) was determined by the methods explained in the article - Mechanics of Shear. Tan (ϕ) equals the shearing load divided by the normal load.

Each sample is tested under two different conditions: Dry and saturated. In the first case, enough dry sample to fill the shear box is dumped into it. The lower box and the frame of the upper box were held together with set screws. The sample tapped slightly to keep the porous stones firmly in contact with the sample. Also, tamping would decrease the large voids. Each sample is tested under four different normal load conditions. After the normal load is applied, the set screws holding the upper part are released, and the horizontal loading started with increments of 500 gms. until the soil failed.

In the second case, after the shear box is filled with the sample, the waterpan is filled with water, and the normal load is applied. The sample usually left there for at least one hour.

After the sample is fully saturated, the set screws holding the upper frame are released, and the horizontal loading started again with the same amount of increments as in the first case. Also, in this case four different kinds of normal loading is used.

For each case of normal loading, the sample is tested at least two times. Sometimes, it is tested more than two times to get closely agreeing values.

Three different kinds of definite grain size were used during tests: First, the size retained on No. 60 mesh sieve and passed No. 40 mesh sieve; Second, the size retained on No. 140 mesh sieve and passed No. 60 mesh sieve; Third, the size retained on No. 200 mesh sieve and passed No. 140 mesh sieve. Finally, according to the obtained values, the graphs are plotted showing the relation between the horizontal force and the normal load, and the angle of internal friction is determined.

REVIEW OF LITERATURE

As the investigation of the proceedings of the second International Conference on Soil Mechanics and Foundation Engineering showed, there is no research on the effect of definite grain size on shearing strength of sand. However, several investigators tried to improve the testing methods of shearing strength.

One of the investigators tried to establish some definite rules for determining soil friction coefficient as the water content varies. Prof. P. Ariano stated the water contents of minima and maxima of shearing strength respectively: For sandy soils from 7.5 to 6.5 per cent (maxima) and from 10.0 to 9.5 per cent. (maxima)

Prof. R. Haefeli, Prof. of Soil Mechanics of the Swiss Federal Institute of Technology, stated that the possibilities of influencing the internal friction and the shearing strength of fine-grained kinds of soil by altering the water content are as varied as the nature of the water itself. In his paper, he discussed the shearing strength of unsaturated loose sediments as a function of the water content. He believed that, in practice, it is often necessary to determine the shearing strength and internal friction of moist, non-saturated loose sediments when endeavoring to control earth slides. Prof. R. Haefeli performed his experiments on clay. He used three different kinds of clay-Kaolinite, alumina, and quartz clay. However, his results are not applied to sandy soils. He concluded that the dependence of the shearing strength on the water content is to a large extent conditioned by the grain distribution and mineralogical composition. He, also, stated that the shearing operation, in the saturated state, causes a consolidation. His research work was one of the few attempts which considered the practical aspects of soil mechanics.

Prof. Takeo Mogami, Prof. of Soil Mechanics at Tokyo University had a research work on the law of friction of sand. He pointed out that the friction angle between sand layers varies as the compactness changes.

As stated above, the discussion of several papers showed that there was not a direct approach to the grain size investigation. However, it is evident from the above discussion that some investigators started to realize the practical importance of soil mechanics in foundation engineering. We believe that their attempt will increase considerably the practical value of soil mechanics.

Size - Retained on 60 Mesh Sieve; Passing 40 Mesh Sieve; Dry

Run No.	Normal Load	Horizontal Load
1	(3160)	(7265)
2	(3160)	(7625)
1	(6320)	(12020)
2	(6320)	(11510)
1	(9480)	(15280)
2	(9480)	(15280)
1	(12640)	(17320)
2	(12640)	(17870)

Size - Retained on 60 Mesh Sieve; Passing 40 Mesh Sieve; Wet

1	(3160)	(4200)
2	(3160)	(4000)
1	(6320)	(7815)
2	(6320)	(9935)
1	(9480)	(13240)
2	(9480)	(12010)
1	(12640)	(16200)
2	(12640)	(17075)

The horizontal and the normal loads are in gms.

Size - Retained on 140 Mesh Sieve; Passing 60 Mesh Sieve; Dry

Run. No.	Normal Load	Horizontal Load
1	(3160)	(6595)
2	(3160)	(7265)
1	(6320)	(9745)
2	(6320)	(9255)
1	(9480)	(12565)
2	(9480)	(13015)
1	(12640)	(15790)
2	(12640)	(14940)

Size - Retained on 140 Mesh Sieve; Passing 60 Mesh Sieve; Wet

1	(3160)	(5685)
2	(3160)	(5665)
1	(6320)	(11240)
2	(6320)	(11155)
1	(9480)	(15520)
2	(9480)	(14600)
1	(12640)	(17575)
2	(12640)	(17155)

The horizontal and the normal loads are in gms.

Size - Retained on 200 Mesh Sieve; Passing 140 Mesh Sieve; Dry

Run No.	Normal Load	Horizontal Load
1	(3160)	(6625)
2	(3160)	(6160)
1	(6320)	(10460)
2	(6320)	(10560)
1	(9480)	(13080)
2	(9480)	(12700)
1	(12640)	(16630)
2	(12640)	(15855)

Size - Retained on 200 Mesh Sieve; Passing 140 Mesh Sieve; Wet

1	(3160)	(3870)
2	(3160)	(2875)
1	(6320)	(7715)
2	(6320)	(7715)
1	(9480)	(13030)
2	(9480)	(12630)
1	(12640)	(15905)
2	(12640)	(16665)

The horizontal and the normal loads are in gms.

CALCULATIONS

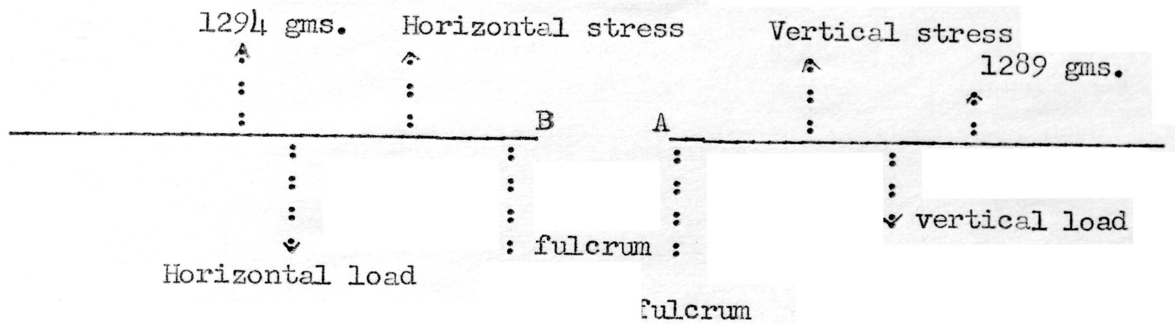

Figure 3.

Distance between the centers of holes - 12.7 cm.
Total weight of left beam - 1294 gms.
Total weight of right beam - 1289 gms.
Weight of yoke, brass plate, and ball bearing (should be added to normal load) - 3075 gms.
Weight of left loading pan (should be added to Horizontal load) - 1875 gms.
Weight of right loading pan (should be added to normal load) - 1922 gms.
Shear Area - 31.2 sq. cm.

CONSTANT FOR VERTICAL STRESS (NORMAL)

Taking sum of the moments around A.

(Vertical Stress) (12.7) = 1289 (31.75) + 1922 (25.4)

Vertical Stress = 7064 + 3075 = 10139

Constant for Normal Stress = 10139 divided by 31.2 K_N = .326 kgs. per sq.cm.

CONSTANT FOR HORIZONTAL STRESS (SHEARING LOAD)

Taking sum of the moments around B

(Horizontal Stress) (12.7) = (1294) (31.75) + (1875) (25.4)

Horizontal Stress = 6990 gms.

Constant for Horizontal Stress = 6990 divided by 31.2

K_H = .224 kgs. per sq.cm.

Size - Retained on 60 Mesh Sieve; Passing 40 Mesh Sieve; Dry

Run No.	Normal Load		Horizontal Load	
	2N	$\frac{2N}{31.2} + K_N$ kgs/cm^2	2H	$\frac{2H}{31.2} + K_H$ kgs/cm^2
1	2(3160)	.529=(.203+.326)	2(7265)	.690=(.466+.224)
2	2(3160)	.529=(.203+.326)	2(7625)	.713=(.483+.224)
Average			2(7465)	.702=(.478+.224)
1	2(6320)	.731=(.405+.326)	2(12020)	.995=(.771+.224)
2	2(6320)	.731=(.404+.326)	2(11510)	.962=(.738+.224)
Average			2(11765)	.978=(.754+.224)
1	2(9480)	.934=(.608+.326)	2(15280)	1.202=.978+.224
2	2(9480)	.934=(.608+.326)	2(15280)	1.202=.978+.224
Average			2(15280)	1.202=.978+.224
1	2(12640)	1.137=.811+.326	2(17320)	1.335=1.111+.224
2	2(12640)	1.137=.811+.326	2(17870)	1.368=1.114+.224
Average			2(17595)	1.353=1.129+.224

K_N=.326 kgs/cm^2

K_H=.224 kgs/cm^2

Size - Retained on 60 Mesh Sieve; Passing 40 Mesh Sieve; Wet

Run No.	Normal Load		Horizontal Load	
	2N	$\frac{2N}{31.2} + K_N$ kgs/cm^2	2H	$\frac{2H}{31.2} + K_H$ kgs/cm^2
1	2(3160)	.529=(.203+.326)	2(4200)	.493=(.269+.224)
2	2(3160)	.529=(.203+.326)	2(4000)	.481=(.257+.224)
Average			2(4100)	.487=(.263-.224)
1	2(6320)	.731=(.405+.326)	2(7815)	.725=(.501+.224)
2	2(6320)	.731=(.405+.326)	2(9935)	.861=(.637+.224)
Average			2(8875)	.793=(.569-.224)
1	2(9480)	.934=(.608+.326)	2(13240)	1.073=.849+.224
2	2(9480)	.934=(.608+.326)	2(12010)	.995=(.771+.224)
Average			2(12625)	1.035=.811+.224
1	2(12640)	1.137=.811+.326	2(16200)	1.263=1.039+.224
2	2(12640)	1.137=.811+.326	2(17075)	1.318=1.094+.224
Average			2(16637)	1.293=1.069+.224

K_N=.326 kgs/cm^2

K_H=.224 kgs/cm^2

Size - Retained on 140 Mesh Sieve; Passing 60 Mesh Sieve; Dry

Run No.	Normal Load		Horizontal Load	
	2N	$\frac{2N}{31.2} + K_N$ kgs/cm^2	2H	$\frac{2H}{31.2} + K_H$ kgs/cm^2
1	2(3160)	.529=(.203+.326)	2(6595)	.647=(.423+.224)
2	2(3160)	.529=(.203+.326)	2(7265)	.690=(.466+.224)
Average			2(6930)	.668=(.444+.224)
1	2(6320)	.731=(.405+.326)	2(9745)	.849=(.625+.224)
2	2(6320)	.731=(.405+.326)	2(9255)	.815=(.591+.224)
Average			2(9500)	.833=(.609-.224)
1	2(9480)	.934=(.608+.326)	2(12565)	1.029=.805+.224
2	2(9480)	.934=(.608+.326)	2(13015)	1.060=.836+.224
Average			2(12790)	1.044=.820+.224
1	2(12640)	1.137=.811+.326	2(15790)	1.236=1.012+.224
2	2(12640)	1.137=.811+.326	2(14940)	1.183=.959+.224
Average			2(15500)	1.219=.995+.224

K_N=.326 kgs/cm^2

K_H=.224 kgs/cm^2

Size - Retained on 140 Mesh Sieve; Passing 60 Mesh Sieve; Wet

Run No.	Normal Load		Horizontal Load	
	2N	$\frac{2N}{31.2} + K_N$ kgs/cm^2	2H	$\frac{2H}{31.2} + K_H$ kgs/cm^2
1	2(3160)	.529=(.203+.326)	2(5685)	.589=(.365+.224)
2	2(3160)	.529=(.203+.326)	2(5665)	.588=(.364+.224)
Average			2(5675)	.588=(.364-.224)
1	2(6320)	.731=(.405+.326)	2(11240)	.945=(.721+.224)
2	2(6320)	.731=(.405+.326)	2(11155)	.938=(.714+.224)
Average			2(11198)	.942=(.718-.224)
1	2(9480)	.934=(.608+.326)	2(15520)	1.218=.994+.224
2	2(9480)	.934=(.608+.326)	2(14600)	1.160=.234+.936
Average			2(15060)	1.189=.224-.965
1	2(12640)	1.137=.811+.326	2(17575)	1.349=1.125+.224
2	2(12640)	1.137=.811+.326	2(17155)	1.324=1.100+.224
Average			2(17365)	1.337=1.113+.224

K_N=.326 kgs/cm^2

K_H=.224 kgs/cm^2

Size - Retained on 200 Mesh Sieve; Passing 140 Mesh Sieve; Dry

Run No.	Normal Load 2N	Normal Load $\frac{2N}{31.2} + K_N$ kgs/cm^2	Horizontal Load 2H	Horizontal Load $\frac{2H}{31.2} + K_H$ kgs/cm^2
1	2(3160)	.529=(.203+.326)	2(6625)	.649=(.425+.224)
2	2(3160)	.529=(.203+.326)	2(6160)	.619=(.395+.224)
Average			2(6393)	.634=(.410+.224)
1	2(6320)	.731=(.405+.326)	2(10460)	.895=(.671+.224)
2	2(6320)	.731=(.405+.326)	2(10560)	.900=(.676+.224)
Average			2(10510)	.897=(.673+.224)
1	2(9480)	.934=(.608+.326)	2(13080)	1.062=.838+.224
2	2(9480)	.934=(.608+.326)	2(12700)	1.038=.814+.224
Average			2(12890)	1.049=.825+.224
1	2(12640)	1.137=.811+.326	2(16630)	1.293=1.069+.224
2	2(12640)	1.137=.811+.326	2(15855)	1.241=1.017+.224
Average			2(16243)	1.265=1.041+.224

K_N=.326 kgs/cm^2

K_H=.224 kgs/cm^2

Size - Retained on 200 Mesh Sieve; Passing 140 Mesh Sieve; Wet

Run No.	Normal Load		Horizontal Load	
	2N	$\frac{2N}{31.2} + K_N$ kgs/cm^2	2H	$\frac{2H}{31.2} + K_H$ kgs/cm^2
1	2(3160)	.529=(.203+.326)	2(3870)	.472=(.248+.224)
2	2(3160)	.529=(.203+.326)	2(2875)	.409=(.185+.224)
Average			2(3373)	.451=(.227+.224)
1	2(6320)	.731=(.405+.326)	2(7715)	.720=(.496+.224)
2	2(6320)	.731=(.405+.326)	2(7715)	.720=(.496+.224)
Average			2(7715)	.720=(.496+.224)
1	2(9480)	.934=(.608+.326)	2(13030)	1.061=.837+.224
2	2(9480)	.934=(.608+.326)	2(12630)	1.035=.811+.224
Average			2(12830)	1.048=.824-.224
1	2(12640)	1.137=.811+.326	2(15905)	1.244=1.020+.224
2	2(12640)	1.137=.811+.326	2(16665)	1.293=1.069+.224
Average			2(16285)	1.269=1.045+.224

K_N=.326 kgs/cm^2

K_H=.224 kgs/cm^2

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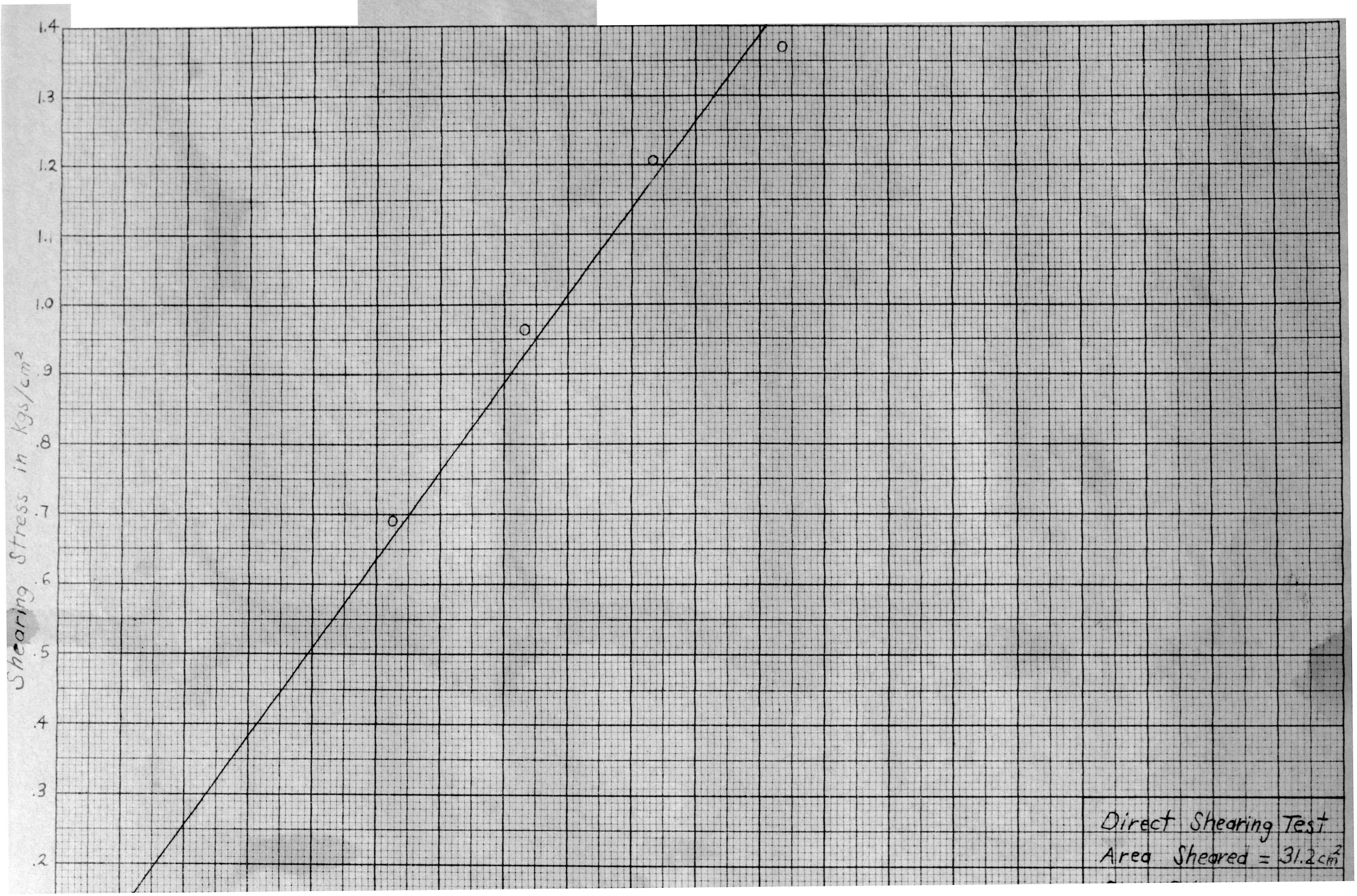

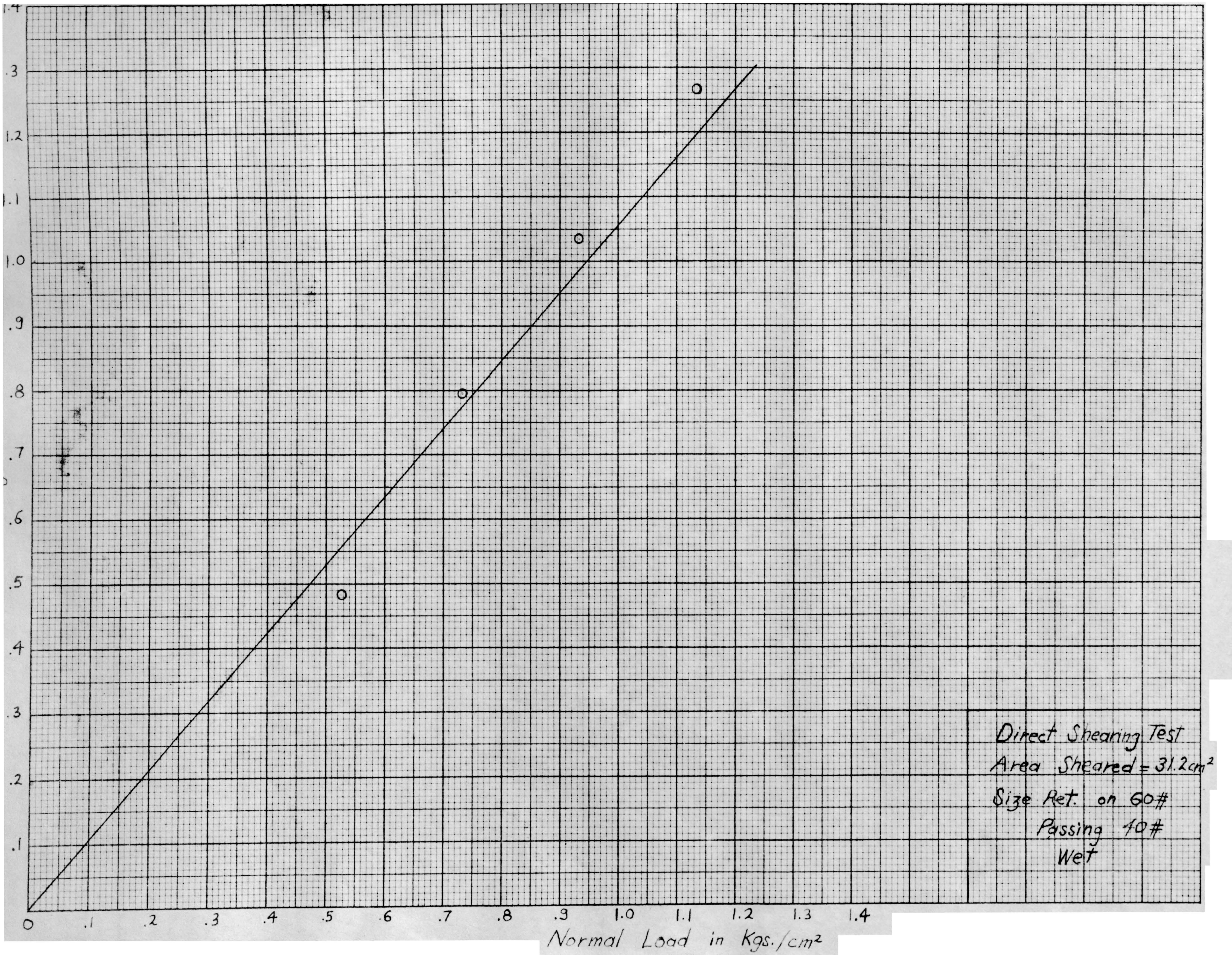

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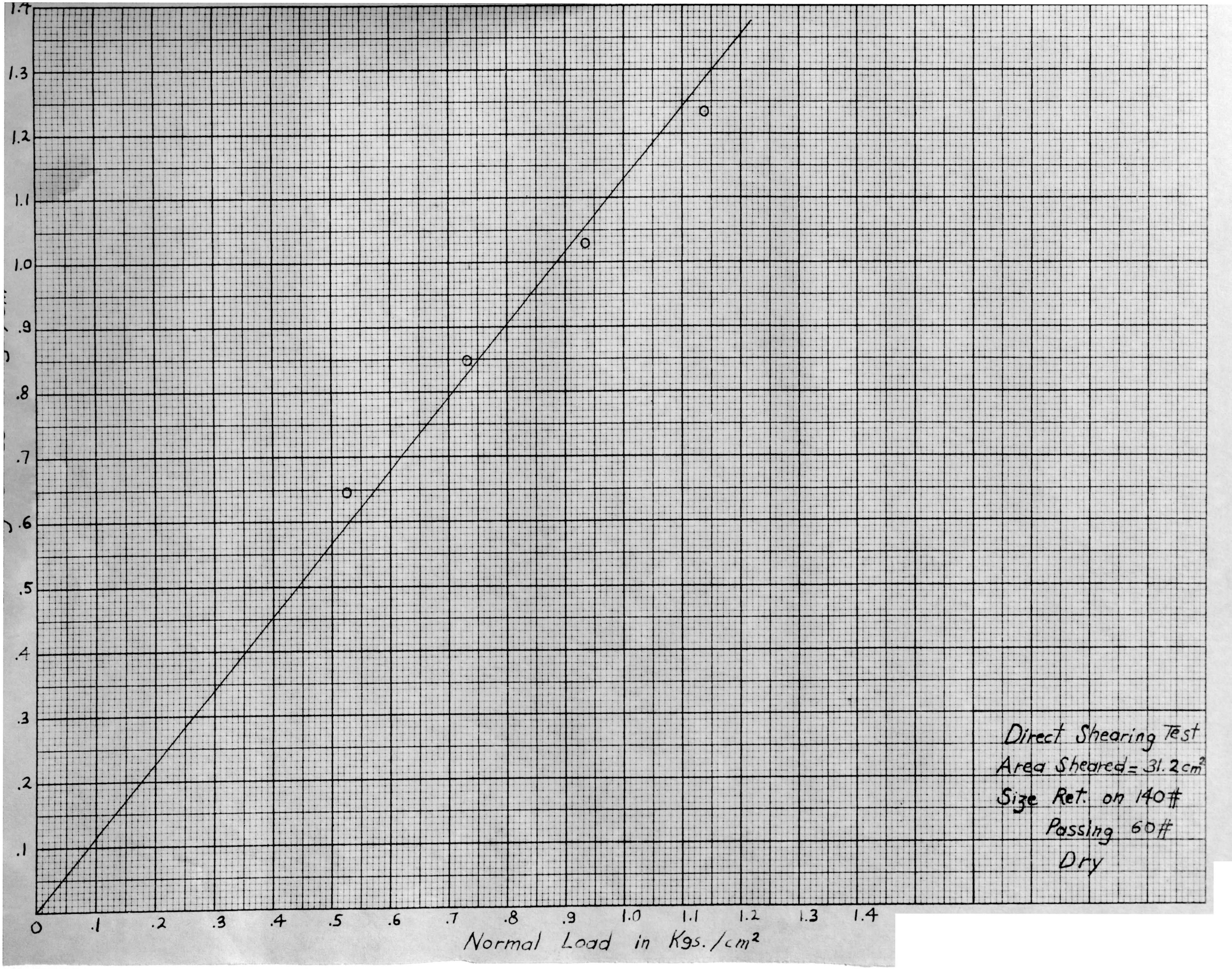
1.4
Normal Load in Kgs./cm²
Direct Shearing Test
Area Sheared = 31.2cm²
Size Ret. on 60#
Passing 40#
Wet
Table II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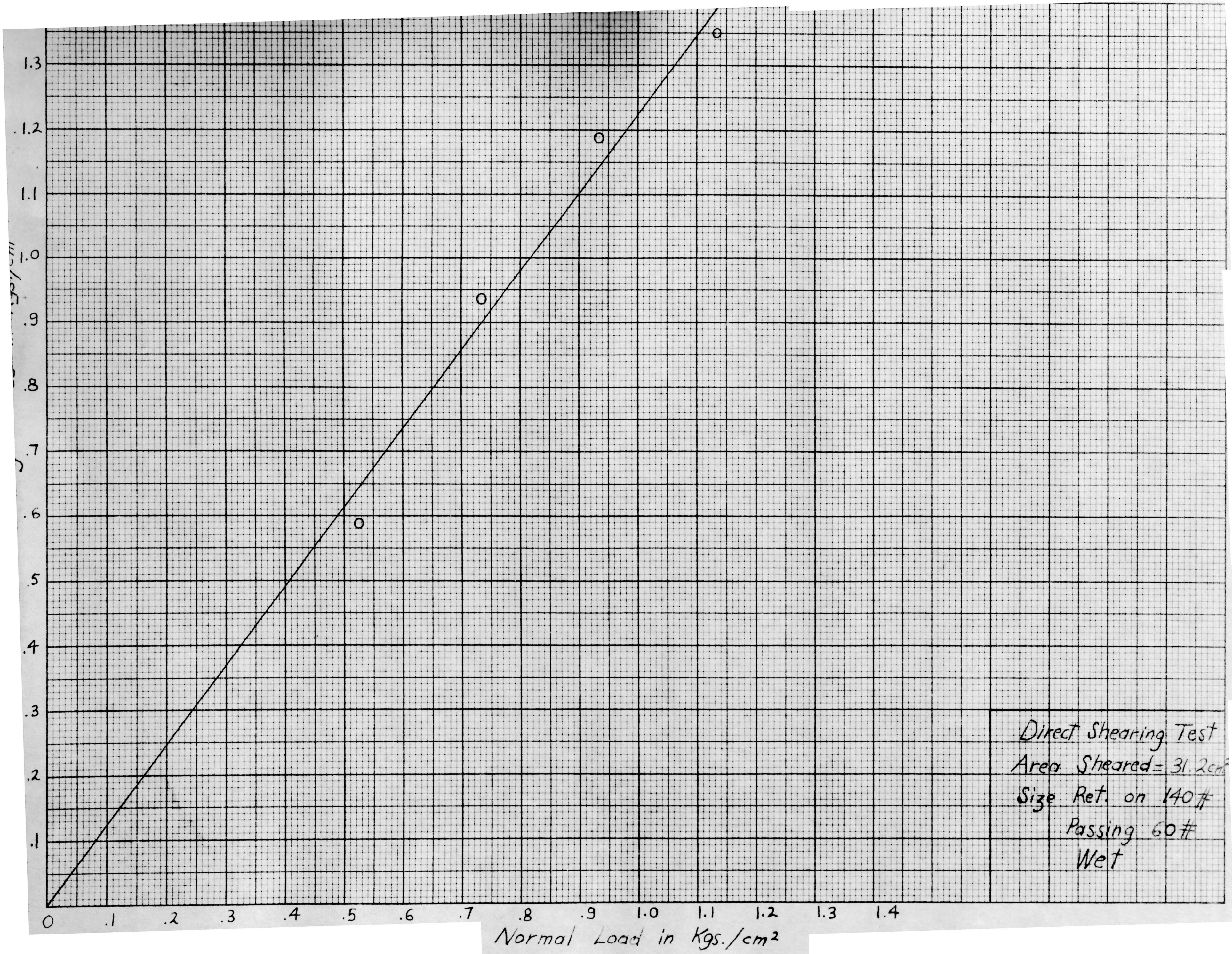
1.0
1.1
1.2
1.3
1.4
Normal Load in Kgs./cm²
Direct Shearing Test
Area Sheared = 31.2 cm²
Size Ret. on 140#
Passing 60#
Dry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1.0
1.1
1.2
1.3
1.4
Normal Load in Kgs./cm²
Direct Shearing Test
Area Sheared = 31.2cm²
Size Ret. on 140#
Passing 60#
Wet

Table XI

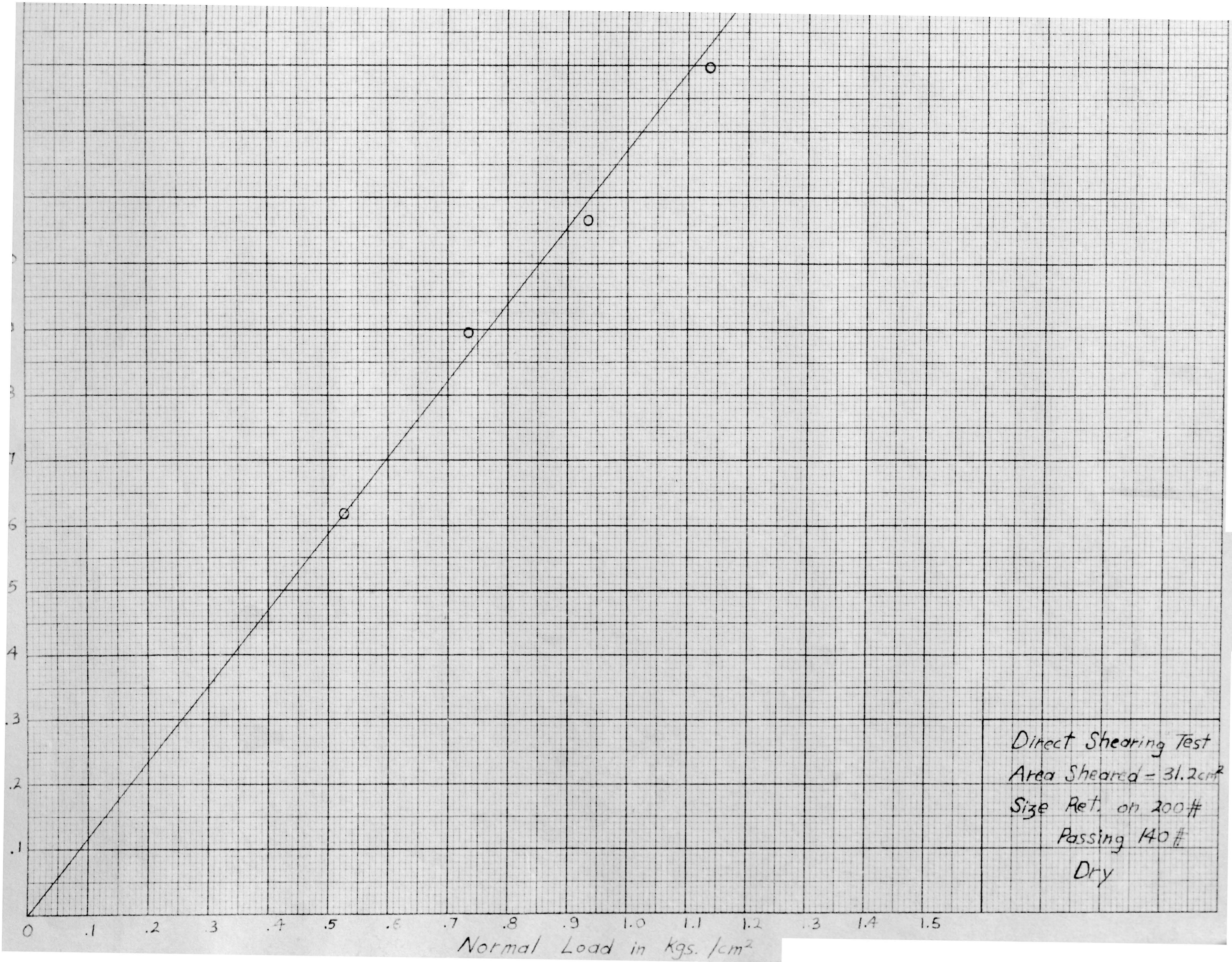
Direct Shearing Test
Area Sheared = 31.2 cm²
Size Ret. on 200#
Passing 140#
Dry
0 .1 .2 .3 .4 .5 .6 .7 .8 .9 1.0 1.1 1.2 1.3 1.4 1.5
.1 .2 .3 .4 .5 .6 .7 .8
Normal Load in kgs. /cm²

Table X

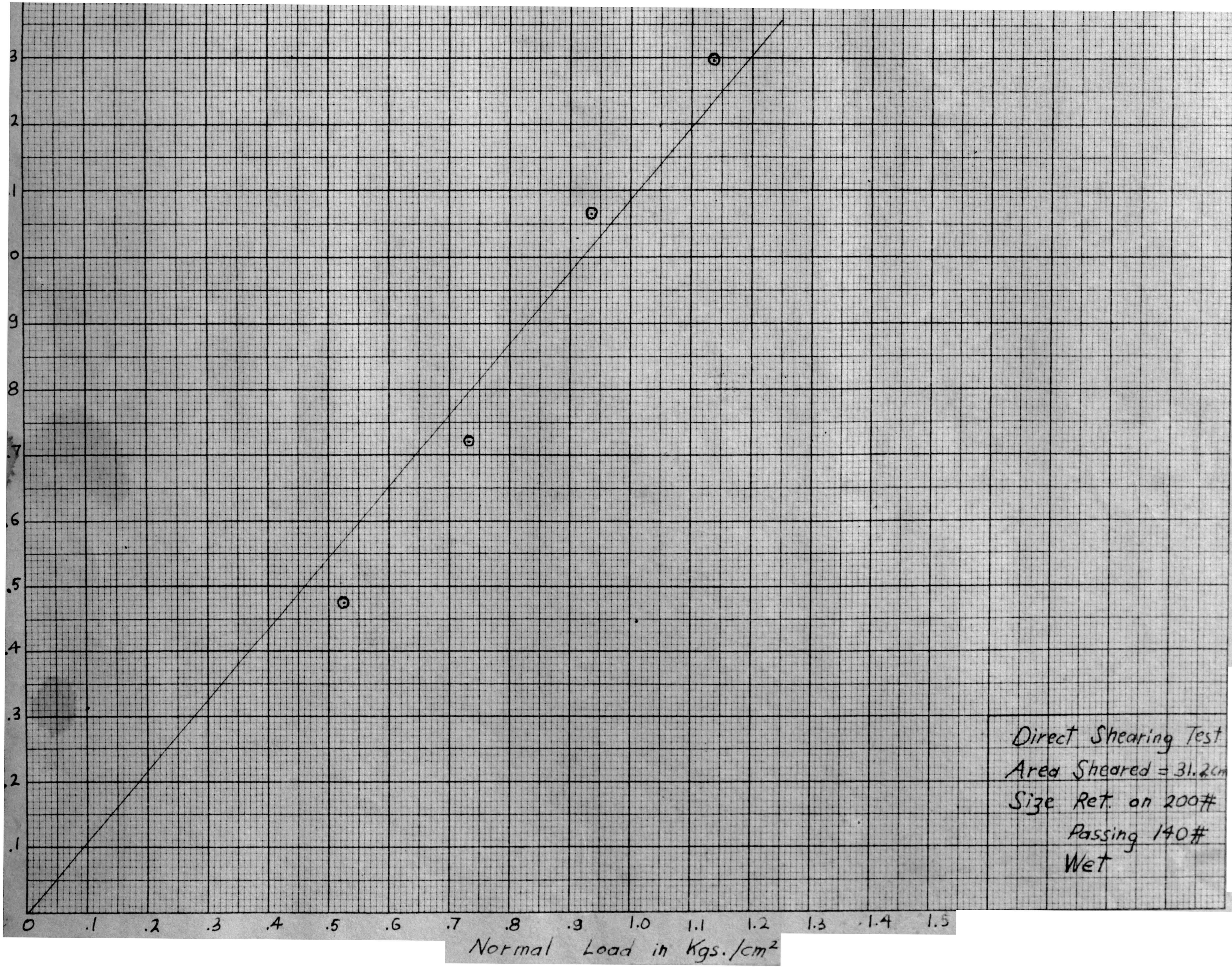
Direct Shearing Test
Area Sheared = 31.2cm
Size Ret. on 200#
Passing 140#
Wet
0
.1
.2
.3
.4
.5
.6
.7
.8
.9
1.0
1.1
1.2
1.3
1.4
1.5
Normal Load in Kgs./cm²

Table VI

THE INTERNAL FRICTION ANGLES DETERMINED FROM GRAPHS

Size retained on 60 mesh sieve; passing 40 mesh sieve

Dry - tan ϕ = 52°

Wet - tan ϕ = 47°

Size retained on 140 mesh sieve; passing 60 mesh sieve

Dry - tan ϕ = 49°

Wet - tan ϕ = 51°

Size retained on 200 mesh sieve; passing 140 mesh sieve

Dry - tan ϕ = 49° 30'

Wet - tan ϕ = 47° 30'

CONCLUSIONS

As stated in the title - the object and scope of investigations, the writer's purpose was to determine the effect of grain size on the shearing characteristics of sand. The results of experiments proved that the grain size has a definite effect on the shearing strength of a sand. A sand composed of three different groupings of grain sizes was investigated in this research work, the internal friction angles of each size was determined after each grain size was tested by the direct shear machine and the graphs were plotted. The internal friction angles obtained were different for each size. Therefore, the effect of grain size was evident on the shearing strength of sand. As the results of experiments determined the conclusion reached above, the writer observed other facts which are mentioned below:

(1) The maximum internal friction angle obtained in the dry tests was 52° for the size retained on 60 mesh sieve. The values obtained for sandy soils tested by the direct shear machine were several degrees less than 52°. This proves that as sandy soils are usually composed of different grain sizes, the internal friction angle is considerably decreased. Also, the compactness of soil was different for a given location while the experiments in this work were performed in a rather compact state.

(2) The minimum angle obtained in the dry tests was $49^{\circ} 30'$ for the size retained on 200 mesh sieve. The experiments showed that as the grain size decreases, the angle of internal friction also decreases. This might be due to the effect of interlocking in the larger grain sizes.

(3) The results obtained in the wet state showed that the value of the internal friction angle is a few degrees less than the values obtained in the dry state. However, for coarser particles, the difference between

the two cases was greater. Therefore, the lubricating action of water was more pronounced in coarser particle sizes.

(4) When the size retained on 140 mesh sieve was tested, the results determined showed that the internal friction angle for the wet state was 1° or 2° greater than the value for the dry state. This shows that there is an intermediate size where the water increases the internal friction angle.

(5) The investigation of each particle size with a microscope showed that the particles of sand were angular. As the coarser particle sizes were investigated, several rounded particles were seen. Before a definite decision is given, the other sands consisting of rounded particles should be investigated. However, as the results on sandy soils showed, the internal friction angles were less for the soils composed of rounded particles. This is expected to be the same for the case of definite grain size.

The results obtained in these tests should be checked on a large scale for different kinds of sands, and these results should also be tested by the triaxial compression methods. After the final decision on the effect of grain size is determined, the other factors influencing the shearing characteristics of sands should be carefully investigated. One of the most important effects is the moisture content. The general procedure of future experiments on this subject should be: After the effect of one factor such as grain size is determined, the effect of other factors such as the moisture content, the relative density, and the structure of the soil should be determined. This could be performed by holding the other factors constant and varying the factor which is to be investigated. One of the most important ideas which should be kept in mind is to per-form the tests on many kinds of soils so that a general solution can be determined.

Finally, the writer believes that several individuals, working on the same subject and investigating some part of the problem, can solve the entire problem of the shearing characteristics of sands. The importance of soils as a highway subgrade material is evident, and the future development on the shearing characteristics will enable the highway engineer to select the best material for his job. Further study would furnish the best possible material for highway construction, and it will be a great help to increase the practical value of soil mechanics in foundation engineering.

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VITA

The writer of this research work was born on December 19, 1929, in Istanbul, Turkey. After completing his education in the grade school, he was enrolled in the Turkish State School for three years. In the 1944-1945 school year, he was in the Robert Academy of American Robert College in Istanbul, Turkey. Following the completion of his work in Exact Science division, he entered the Engineering School of American Robert College in the Fall term of 1947-1948.

He received his B. S. degree in Civil Engineering at the end of the 1950-1951 school year.

During the fall term of the 1951-1952 school year, he was enrolled in Missouri School of Mines and Metallurgy, Rolla, Missouri, for the purpose of continuing his studies to receive his M. S. degree in Civil Engineering.