SHEARING STRENGTH OF PARTIALLY CONSOLIDATED COHESIVE SOILS

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BY

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CERTIFICATE

CERTIFIED that the thesis entitled "SHEARING STRENGTH OF PARTIALLY CONSOLIDATED COHESIVE SOILS" which is being submitted by Sri Satish Chandra Sharda in partial fulfilment for the award of the Degree of Master of Engineering in Civil Engineering (Soil Mechanics and Foundation Engineering) of University of Roorkee is a record of student's own work carried out by him under my supervision and guidance. The matter embodied in this thesis has not been submitted for the award of any other Degree or Diploma.

This is further to certify that he has worked for a period of Five Months from 16th April 1981 to 16th September 1981 for preparing thesis for Master of Engineering Degree at the University.

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NOTATIONS

C = Cohesion.

$\bar{C}$ = True Cohesion.

C_w = Cohesion at Water Content w.

σ = Total normal pressure on plane of failure.

$\bar{\sigma}$ = Effective Normal pressure " " "

p = Total Normal Pressure on failure plane due to over burden in field.

S = Shear Strength.

S_T = Shear Strength after a partial consolidation to time T only.

$S_{initial}$ = Shearing Strength to the normal pressure existing over the soil in the natural condition.

S_p = Shearing Strength due to p after complete consolidation.

S_{pw} = Shearing Strength at p and water content w.

T = Time for consolidation.

μ = Coefficient of friction between soil grains.

η = Coefficient having value 0 to 1. depending upon T.

U = Pore Water pressure in the soil.

ϕ = Angle of internal friction.

$\bar{\phi}$ = True angle of internal friction.

ϕ_w = Angle of internal friction at water content w.

SYNOPSIS

Attempt has been made to find the shearing strength characterstics of partially consolidated cohesive soils. The importance of the problem lies in calculation of shear strength of soils used in earthen dams and soils below them after a certain time of consolidation. Present methods for finding shearing strength for a partially consolidated soil do not take into account the time of consolidation because the pore water pressures are difficult to measure. The values taken are usually conservative values from the usual shear strength. This method takes into account the time by taking soil density at failure i.e. the water content at failure after partial consolidation. The characterestics found here are for 3 disturbed samples of soils from borrow pits with vicinity of proposed Malin dam in Bijnor District U.P.(India). These are classified as M.L., C.L., and C.L. according to A Casagrande's chart. To get shear strength v.s. water content curves, various loads with various timings of consolidation have been applied on soil samples which are then sheared and water content of failure is found out. Other graphs have been developed from these curves only.. The results show that shearing strength for the soils used can be represented at a particular water content only approximately by a Coulomb's straight line equation. However, all results verify the present concepts about shearing strength of soils.

CHAPTER I

INTRODUCTION

1.1. The most important property of soil which affects all types of engineering structures is the shearing strength. This property has been under investigation for a more than a century and still research go on to estimate it as accurately as possible. The earliest attempt that was made in the right direction to find the shearing resistance was that by Coulomb in 1773, as a result of which a useful empirical relation is known to us viz $S = C + \sigma \,\mathrm{Tan}\, \phi$. Another modified form put forward by Hvorslev in early part of this century was $S = \bar{C} + (\sigma - U)\,\mathrm{Tan}\,\bar{\phi}$.

Attempts have been made in past and are being made to calculate the parameters "C" and "Tan ϕ", as accurately as possible, both in field and in laboratory. But soil being a heterogeneous material these parameters depend on many variables some of which might be still unknown to soil engineers. The usual tests to find shearing strength in laboratory are performed by simulating field conditions in the laboratory. But certain conditions such as the shear strength of soils after partial consolidation i.e. when the consolidation is not complete requires more attention. Present practice in such problems is to use conservative values from the usual tests for shear strength.

It is known that a definite relationship exists between shear strength and normal load and shear strength and water content. It is difficult to calculate effective pressure in partially consolidated soils. Thus a method in which water content is related to shear strength is a better one since it avoids the pore water pressure calculations.

The work presented here deals with finding out characteristics of shearing strength with changing water content and under different loads after a state of partial consolidation. Such a curve can be useful for finding shear strength for all consolidation periods in an earthen embankment or in the foundation of dams.

The soils used are with varying clay content. The soils consolidated nearly 100% in about 20 hrs., under a maximum load of 5 Kg. / Cm^2.

1.2. Historical Review of Shearing Strength

The most important and a unique step towards the problem of the shear strength of soil was taken by Coulomb in 1773, while dealing with earth pressure theories he introduced an equation to determine the resistance of soil to shear. The equation,

$$S = C + \sigma \operatorname{Tan} \text{Ø} \qquad (1)$$

states that the total shear resistance of a soil can be considered a sum of (a) cohesive resistance, a constant and (b) friction, a figure increasing proportionally to normal pressure acting on the plane of failure. Coulomb Law has served as the basis for shearing strength problem, eversince it appeared.

Although the form of the equation is not complicated a determination of the soil coefficients introduced by coulomb creat a number of difficulties, especially in case of cohesive soils. To determine the coefficients a knowledge of physical properties of soils and an understanding of the phenomena which determine their strength is required. Thus the progress in this direction took place only with the advent of the science of soil Mechanics.

Terzaghi in 1925 published his "Erdbaumechanik" to give us the fundamental basis for the new science of soil Engineering. This was based on the investigation Swedish State Railways Embankment's Slides in 1922. Thus the question of shear strength was soon taken up. A number of publications appeared in which attempts were made to explain the phenomen-a and to give rules for the application of shear strength in practice. But important factors such as pore pressure were often ignored.

Systematic investigations carried out at Terzaghi's laboratory at Vienna were important in clearing up important questions concerning shear-strengths. During the years 1934-37 Hvorslev carried out a great number of direct shear tests.

The results confirmed in principle the law of coulomb but as cohesion was found to be a function of water content Hvorslev (1937) extended Coulomb's equation as

$$s = \bar{c} + \bar{\sigma} \tan \bar{\phi} \qquad (2)$$

in which $\bar{\sigma}$ is the effective normal stress on the failure plane, $\bar{\phi}$ is the "true angle of internal friction" and $\bar{c}$ is the "true cohesion" which was found to depend on the water content.

The significance of the result of Hvorslev's investigations of 1937 is very profound. The two main results, dependency of the cohesion on water content alone, and recognition of the angle of internal friction as a soil characteristic which determines the strength of even rather plastic clays, were most important steps in the advancement of knowledge of the fundamental strength properties of soils. Terzaghi in some paper supplemented Hvorslev's investigations by pointing out failure conditions and the effect of porewater pressure in 1938.

Rendulic continuing investigation of the fundamental strength properties of soil after Hvorslev made first attempt to measure porewater pressure in a triaxial test (1937). He pointed out the relationship between porewater pressures and deformation properties and utilising this relationship he carried out an experimental investigation of the volume change of sample submitted in a triaxial apparatus.

While research on the shear strength was on in Europe, research in U.S.A. by Jurgenson in 1934 made an important contribution to the understanding of shear strength problems. This paper gave the first triaxial apparatus and tests. A.Casagrande in discussing Jurgenson's paper made the first attempt to analyse the effective stresses acting on a specimen in a triaxial shear at constant volume. These consideration which were further extended give an analysis of the undrained shear test and demonstrate the relation between undrained shear strength and deformation properties.

The years just before the Second World War marked the improvement of the shear apparatus. The ring shear machine was developed (Hvorslev 1936, Tiedemann 1937, Gruner, Haefeli 1934, Haefeli 1938 and Hvorslev 1939), Also the triaxial shear apparatus was improved. In 1939 the Corps of Engineers, U.S.Army, founded a Soil Mechanics fact finding Survey. They with the help of some leading technical institution of America worked for triaxial machine, its testing technique and method of interpreting shear test result. Rutledge in 1944, prepared a report on the findings and published it in 1947 in United States War Department, Corps of Engineers. In this programme, two hypotheses for an interpretation of triaxial shear tests with clays were also investigated. The first one was the above mentioned working hypothesis put forward by A. Casagrande. Rutledge describing the second considers it from an entirely new point of view. The tests carried out indicate that the strength of a saturated clay depends only on the water content at failure, and is independent of either the minor principal stress or pore water pressure, or the method of testing. Consequently Rutledge proposed the use of a diagram in which the logarithm of the compressive strength is plotted against the water content. The resulting curve runs roughly parallel to the semi log pressure- water content curve, plotted from a standard consolidation test. Thus the magnitudes of pro water pressure need not be considered in practical application and in addition the question of internal friction is avoided.

Although this programme helped much in improving testing devices no final conclusion were drawn and the problem of the fundamental properties of clay remained unsolved.

Rutledge (United States War Department Corps of Engineers 1947) expresses the opinion that it is doubtful if saturated cohesive soils can be considered to have an angle of internal friction, a point of view which is not shared by Taylor (1948) who ascribes the total strength of normally consolidated clay to friction.

Skempton in 1948 published, an outstanding paper for shearing strength of clays, in Geotechnic Skempton uses the definition of cohesion and friction as given by Coulomb - Hvorslev, and through an introduction of the Coefficients of Elasticity of compression and expansion, he succeeds in obtaining an expression for the compressive strength found by undrained triaxial tests. Skempton has elaborated this analysis method later (Skempton 1948, Skempton and Golder, 1948).

At the Second International Conference of Soil Mechanics held in Rotterdam in 1948, there was a great difference of opinion regarding the nature of shear strength of clay. Some research concluded the shear strength in clay is due to cohesion only and some contributed it to friction only. The water content strength diagram was opposed to Mohr's diagram. The $\phi = 0$ analysis for practical work however deserves special notice. (Golder, Skempton 1948).

At the fourth International Conference of Soil Mechanics held in London in 1957 research from many well known workers concludes that in general, the shear strength expressed in terms of effective stress was the most reliable solution stability analysis irrespective of type of soils. The undrained shear strength is considered to be approximately valid for the stability conditions immediately after loading changes have taken place. An analysis based on undrained shear strength is very simple and economical. However - further efforts are required for a better development. Geological factors study is also emphasised. Jakobson concluded that the angle of internal friction depends to a great extent upon intermediate principle stress. However the conclusion is that the relative merits of different apparatus failure criteria testing techniques and interpretation procedure are far from being sufficiently explored.

CHAPTER II

PRESENT CONCEPTIONS OF SHEAR STRENGTH

2.1. <u>Criteria for Failure</u> :-

On any plane[a] through a point in a stressed soilmass a direct stress σ_a normal to the plane and a shear stress τ_a will in general be acting on the plane. The magnitude of the normal and the shear stresses can be determined by the well known equilibrium equation.

A failure occurs in a soil element if on any plane the shear stress τ_a exceeds the shear strength of the plane s_a . If the shear strength is the same on all planes through the same point, a failure takes place in the planes of maximum shearing stress. The shear strengths of the different planes through a point of a stressed soil mass are not equal but the shear strength depends on the normal stress σ_a which acts on the plane.

Coulomb was the first one to express a failure criterion for soil giving the well known equation.

$$s_a = c + \sigma_a \mu$$

i.e. shear strength is a sum of a constant cohesion C and a friction which increases in proportion to normal pressure.

Hvorslev's experiments during 1934-1937 confirmed the Coulomb Law in general, and coefficient of friction μ was found to be a constant.

thus $\mu = \tan \phi$

and was considered to a characterstic of shear strength.

Hvorslev proved that cohesion was not constant as assumed by Coulomb but varies with density i.e. the water content of a saturated soil. The cohesion can be expressed as a function of water content alone and that this was independent of a number of factors which otherwise affect total shear strength. Hvorslev's experiments varified the results of Terzaghi that the coulomb equation can be generally valid if σ_a is effective stress.

Thus it may be stated that the Coulomb - Hvorslev failure criterion may be accepted as a basis for strength analysis for all types of homogeneous soil and that the shear strength in an arbitrary plane through a point of a stressed soil mass can in general be expressed by the equation

$$S_a = \overline{C} + \overline{\sigma}_\alpha \tan \bar{\phi}$$

where S_a = shear strength in the plane "a"

$\overline{C}$ = Cohesion corresponding to the water content.

$\overline{\sigma}_\alpha$ = effective normal stress acting on the considered plane.

$\tan \bar{\phi}$ = Coefficient of internal friction.

$\bar{\phi}$ = angle of internal friction.

The above equation thus clearly shows that the shearing strength in a plane through a point in "a" soil mass consists of two parts.

(1) Cohesion

(2) Friction resistance.

Cohesion is characterised by the fact that its magnitude is the same in all planes through the considered point. This magnitude varies with water content in a saturated soil.

Friction resistance is that, the magnitude of which is proportional to the effective normal stress on the considered plane. If, in the considered point, a hydrostatic pore water pressure exists, the effective stress will be determinative for friction resistance. Thus the above said equation will be changed to

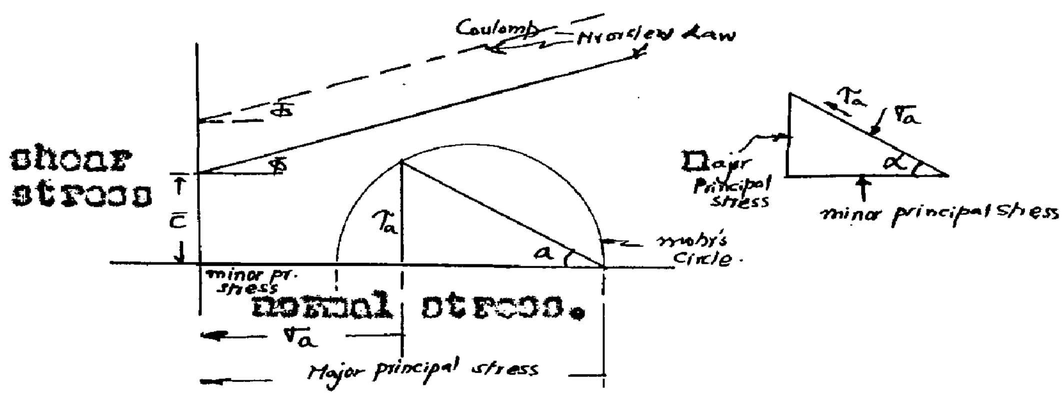
$$S_a = C + (\sigma_a - u) \tan \phi$$

where u = is the pore water pressure.

It is most important to maintain the above established definitions of shear strength, cohesion and friction. Also cohesion has nothing to do with the dhear strength of the unloaded soil element. The cohesion is constant for all planes through a point and is independent of normal stress. Any change in cohesion will be brought about by such conditions which change the water content of the soil.

3.3. The Shear Strength Diagram :-

It is known that, Mohr's stress circle method is used to know the shearing strength characterstics graphically. The diagram used for determination of the stresses is commonly used for an analysis of shear strengths of soils. In this case the abscissas represent the normal stresses and on the ordinates are plotted the strengths. Thus a direct comparison is possible between the normal stresses and shear strength. If in such a diagram the failure condition expressed by the Coulomb - Hvorslev Law is plotted, a straight line would appear which is a strength line. This line cuts $\bar{C}$ on the ordinate axis and forms the angle ϕ with the abscissa axis as in figure :



A failure line in Mohr's diagram is, thus a graphical representation of the shear strength in the different planes through a point of a soil mass, and gives the strength as a function of the normal stresses on the different planes.

For a cohesive soil the strength line will be the same for all points of the soil if the cohesion is the same in all points. This is the case if the water contents are equal at all points. In general however, the water content varies from point to point and hence the cohesion is different at different points. Therefore in the analysis of shear strength by this diagram a number of failure lines will appear, all of them running parallel to the inclination $\tan \bar{\Phi}$ and cutting different $\bar{C}$ on ordinate axis as shown in the diagram by dotted and solid line.

Thus for cohesive soils the failure conditions cannot be represented by a single line in the strength diagram as assumed by coulomb and Mohr. The lines of rupture from a system of parallel lines with the cohesion as parameter rising at an angle of $\bar{\Phi}$ with the horizontal axis. It is not, thus possible through static considerations alone to determine the magnitude of shear strength of a cohesive soil. The cohesion is a function of water content and it is, therefore desirable to try to establish the relationship between cohesion and water content in the shear strength diagram.

3.3. <u>Factors Affecting the shearing strength:-</u>

As has been shown the shearing resistance of a soil is given by the equation

$$S = \bar{C} + \bar{\sigma} \tan \bar{\Phi}$$

– The main components being cohesion and internal friction and the characteristics being cohesion and angle of internal friction. Any property which affects the main constituents of shearing strength i.e. c and $\sigma \tan \phi$ must directly affect the shearing strength.

According to Taylor [5]* " No out line of the factors on which shearing strength depends can be set up which is both simple and in all respects satisfactory". The following may be some of the known factors which affect the cohesion and angle of internal friction.

Fundamental Factors :-

a. Intergranular pressure.

b. Intermediate principle stress.

c. Structural strength.

d. Speed of shear.

e. Colloidal phenomena.

f. Degree of progressive action.

Alternate or Inter-related factors.

g. Pressure history.

h. Drainage conditions.

i. Air content and water content

* Reference no. 3

Fundamental Factors:

a. <u>Intergranular Pressure</u>:- In an ideal expression for Mohr's envelope,

$$s = \bar{\sigma} \tan \bar{\phi}$$

as $\bar{\sigma}$ will increase shear strength will also increase. Also shearing strength will be affected by the value of $\bar{\phi}$ which can be either the angle of peak point or ultimate failure, generally $\bar{\phi}_{peak}$ is greater than "$\bar{\phi}$ ultimate". Cohesion is not affected by this factor.

b. <u>Intermediate Principle Stress</u>:- According to idealised Mohr's strength theory, the shear strength is independent of the value of intermediate principle stress. But it is generally believed that the shearing strength actually depends somewhat on this stress. The value of ϕ being greater when this stress is near to major principle stress in magnitude than that to minor one.

c. <u>Structural Strength</u>:- The structure of a soil is an independent variable. The true nature of structural strength is not known. But the general envelope applies to soil whether or not structural strength is there. The envelope has larger ordinate i.e. cohesion if structural strength is high; It is of great importance. The structural strength is destroyed due to large amount of strains or remoulding.

d. Speed of Shear :- Available data on this factor effect on shear are less, however some experiment show that the effect of speed of shear are not large in case of clays. The increase is due to the variation in plastic resistance which depends upon the intergrannular pressure also.

e. Colloidal Phenomena :- It has a great effect on shearing strength of cohesive soils. It is due to the particles of colloidal size. It is explained by the adsorbed water films around the soil particle. The structural strength is lost due to shearing strain is explainable by this phenomena. The "thixotropic regain" is also due to this factor.

f. Degree of Progressive Action :- Progressive action takes place in all types of failure of soil mass. The conditions on the plane of failure are seldom the same. This is due to non-uniform shear strains occuring at different places on failure surface. The effect of this is that the peak point strength is smaller for the average condition of failure than that from the ideal condition of failure where strain must be same throughout the soil mass. Thus the peak point friction angle is less than that from ideal case and varies depending upon the degree of progressive action. If it is more the angle is less and vice-versa.

<u>Alternate or Inter-related Factors</u>

g. <u>Pressure history</u> :- If the sample is preloaded to a greater load than that of which it is being sheared there will be an increase in cohesion with respect to a sample which was normally load. The cause is the density of specimen. The cohesion is due to intrinsic pressure caused due to excess compression.

h. <u>Drainage Conditions</u> :- If no drainage occurs and after the application of load during shear, the shearing strength is equal to cohesion and does not depend upon the normal load. In case the soil is consolidated under the load first and their sheared under no water content change the angle ϕ will be smaller than that for a case of complete drainage during shearing. Thus shear strength will be maximum if the drainage is allowed to occur during shear testing.

1. Water Content and Air content :- The effect of a small air content at a given value of water content, in a soil sample is that the shearing strength is reduced as compared to a full saturation case. At smaller normal pressures the difference in shearing strength between a completely saturated case and partially saturated case is more due to more space covered by air bubbles at higher loads however this difference is smaller.

Water content also affects the cohesion and internal friction. With reduced water content both increase upto a certain limit.

2.4. Physical Explanation of the Factors Contributing to the Shear Strength:-

The description dealt about the shear strength in section 2.3. gives only an interpretation of the shear strength of soils as it results from a statistical treatment of shear tests. It can be concluded easily that the total strength consists of internal friction, in creasing proportionally to the effective normal pressure and varies with water content.

A physical explanation of the behaviour of soil for shear strength is difficult, however the following qualitative and hypothetical built on observations and published theory by Casagrande 1932, Terzaghi 1941, Terzaghi & Peck 1948, Taylor 1948 tries to give us some possible behaviour.

<u>Internal Friction</u> [1] The present conception of free friction is that it is due to resistance to mutual movement of grains in direct contact. In actuality when two grains touch, the area of the contact point is very small. Where the grains touch the molecules are in intimate contact. The molecular bonds in the contact, thereby exist in a state of attraction which is similar to that occuring within the individual grains. Each atom in the contact area is in this way tied to lattice work of its own grain, but is also subjected to forces of attraction from neighbouring atoms of the other grain.

If the considered grains are subjected to forces which tend to cause a sliding in the contact area, the contacting atoms will be pulled a little out of the original position, but for forces smaller than the sum of the attractive forces. The mutual movement will not be very great. If, however, the forces are -

increased, a failure will take place for a certain value of the average shear stress in the contact area indicating the number of atoms which has to be liberated by a failure, and the average value of the attractive forces between the atoms determine the resistance to sliding. Assuming that the average attractive force of the atoms is equal at all points of contact, the total shear resistance is determined by the total area of the contact points only.

Investigations by Bishop and Eldin 1950 about the size of contact area and its dependence on the pressure transmitted through the contact point show that the contact between two solids consists in reality of a number of small contact points, and the total area of these contact points increases proportional to a transmitted stress. This means that at each point of contact the solid material is loaded until a failure occurs, and that the contact pressure is approximately constant.

This it is concluded that the area of a single contact point i.e. the resistance to sliding is linearly proportion to the normal pressure transmitted through the contact point. Thus true friction is built up of the total resistance of the contact points of the mineral grains to sliding and the friction is therefore assumed to increase proportionally to the total contact area. Assuming that a mineral grain can bear a constant load, the total area increases proportional to the effective normal stress on a considered plane, thus

$$S = \mu \overline{\sigma_a}$$

μ being constant for the soil. Thus μ is the average value of the ratio between the resistance to sliding and the load that all contact points can sustain per unit area.

In case of soils with true cohesion only a point of stresses will be transmitted through grains directly in contact. With such soil the coefficient of true internal friction " μ " will depend not only on the properties of the mineral grains, but will also be function of to what extent the stresses are transmitted through grains in direct contact with eachother.

True Cohesion:- True cohesion may be explained as that particular point of the shear strength which is due to those forces of attraction which exist between the clay mineral or that point of the clay ..

minerals which are not directly in contact with each-other. Thus this point of contribution is probably depends upon the property of mineral grains and on the form and surface activity of the grains and the nature of the water which separates the clay particles.

Very little is known about the bond between the clay particles. Terzaghi 1941, assumes that the water shell which surrounds a clay particle is so firmly tied by electro-chemical forces that it is almost solid near the particle surfaces. In the contact points these water shells merge into each other and so a certain resistance against mineral displacement is established, which gives the soil its cohesive property. This theory has, however successfully explained the behaviour of cohesive soils due to disturbance and remoulding. Also the theory accounts for the fact that the cohesion increases with decreasing water content.

Again, according, to above theory the shear stresses in clays must be transmitted through the water shells separating the single grains. The cohesion is in this way determined by the strength of adsorbed water. It is however uncertain whether or not water can be solidified, at anyrate it is rather doubtful that a firm tying of the adsorbed water really endows it with a true strength. According to (Haefeli 1934) water may have high viscousity, possesses no real strength under ordinary conditions of occurance.

These considerations suggest the possibility that the cohesive resistance may be a function of time i.e. the rate of loading. If true cohesion is caused by an extremely high viscousity rather than real strength its value may be expected to decrease with an increase of the load period. Investigations on the effect of rate of loading by Casagrande and Wilson (1951) show that the strength of saturated brittle clays and clay shoels for constant water content is actually reduced under sustained loads.

The effect of electrolytes on true cohesion is also important. Investigations have shown that the strength is influenced by adsorbed ion content under the condition of constant water content.

Dilatancy component:- A third contribution the shear strength which is not included in the Coulomb Hvorslev's failure criterion is caused by the

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interlocking of grains. This is observed in dense soils especially in dense sand and compacted materials. The increase of angle of internal friction i.e. shear resistance is caused by the energy to be provided to break the interlocking. This phenomenon has been analysed by Taylor 1948 and Bishop 1950.

2.4. PRESENT CONCEPTION:-

Summarising our present knowledge of shear strength [1] and the stability of slopes and dams, the most characterstic feature seems to be the sharp differentiation between the treatment of the shear strength in applic tion to practical problems, and the scientific consideration of the fundamental shear strength properties.

Generally the stability calculations are based on the results of a series of shear tests carried out with intact samples. In case of fill problems the samples are compacted with the same initial water content and dry density as used in the field. In each stability case a special shear test is required which simulates the practical condition in field as closely as possible. In each test series the strength..

of a number of identical samples are determined under different pressures. But in order to simplify testing technique the stability cases are, in general, idealised with respect to drainage possibilities, water content etc. General recommendation is that for Drained shear tests and consolidated undrained shear tests.

The results thus obtained are analysed with respect to effective stresses and are plotted in a Mohr's diagram to determine a failure condition.

$$S = C + \sigma \tan \phi \qquad (i)$$

In calculations for stability the shear strength is directly concluded from test results for the case considered. i.e. C and ϕ of the above equation are considered as if they are really true cohesion and the true angle of friction. In general different failure criteria have to be introduced for the different stability cases.

Thus the strength for stability is by judgement from the results obtained by simulating as closely as possible the conditions of field in laboratory tests.

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In a number of stability problem concerning clays in which effective stress changes can be ignored safely due to low permeability, the analysis is done with respect to total stresses according to the "$\phi = 0$" analysis (Skempton 1948). This means that the shear strength of clay is introduced in calculation as found out by compression test, being independent of the total stresses.

The shear strength coefficients used in the above mentioned procedure are empirical constants as their value is dependent on the conditions under which the shear tests are carried out. Thus the coefficients in the equation (i) are known as "apparent cohesion" and "angle of shearing resistance" respectively (-- Skempton 1950) in order to distinguish them from true cohesion and angle of internal friction.

A failure criterion which expresses the shear strength by fundamental shear strength coefficients is not essentially more complicated in form than the equation

$$s = \bar{c} + (\sigma - u) \tan \bar{\Phi} \qquad \text{(ii)}$$

but the determination of the true cohesion $\bar{c}$, dependent ..

on the water content, and of the pore water pressure "(U)" is in practice possible in a few simple cases only. Thus determination of Tan $\bar{\phi}$ and $\bar{C}$ is complicated task. Thus the extention of our knowledge of the fundamental soil properties is, necessary in order to judge the degree of accuracy and the seriousness of possible errors in the more simple methods employed in practice.

2.5. SCOPE OF THE PRESENT EXPERIMENTATION :-

The experimentation is an attempt to find out the shear strength charactorstics for partially consolidated cohesive soils. Shear strength has been compared with water content at failure i.e. density taken into account under different loads with varying times of partial consolidation. This case can be useful in problems as dam foundations and earthendams.

As has been previously stated that the failure criterion which expresses the shear strength by the fundamental shear strength coefficient can be represented by Coulomb's Law,

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

According to Maslov [2] the shearing strengaftor a cortain time of consolidation also can bo roprosented by

$$S_{pw} = C_w + p \tan \phi \qquad (3)$$

S_{pw} boing strongth at load p and water content w and Cw and ϕ_w cohosion and angle of internal friction angle water content "w" at failure, aftor a time T of partial consolidation

Also

$$S_T = S_{initial} + \eta (S_p - S_{initial}) \qquad (4)$$

is shoar strength aftor incomploto consolidation in time "T" "S initial" is shoar strongth due to existing load in natural condition. Sp , Final shoar resistance after complete consolidation due to total load aftor addition of structure.

η Doing a coofficiont having values from 0 to 1

$\eta = 0$ for initial stage and $\eta = 1$ for complete consolidation case.

Thus eq.(3) and (4) can bo used to find shoar strength properties aftor partial consolidation in a time T. ST can bo expressed in torms of Cw & ϕ w similar to (3).

To have a closor picture of the behaviour due to difforont clay contonts soils with difforont clay

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contents have been used. The experimentation method has been described.

The experimentation is only a part of a big research programme on partially consolidated soils which can be extended for finding fundamental shear strength characteristics with respect to pore water pressures, thixotropic effects etc.

CHAPTER III

EXPERIMENTATION

3.1. General Approach :- The soil samples were obtained from the U.P. Irrigation Research Institute Roorkee. The soils were from the borrow pits for the construction of Kalin Dam in Bijnor District in Uttar Pradesh. These samples were taken from a depth of 5 to 8 ft below the ground level. The soil samples were disturbed ones.

Classification tests show that there is very little variation in soils for testing. The samples could be classified in general as

Soil A	Inorganic Silts (M.L.) or Silty clay
Soil B	Inorganic clay of (C.L.)
Soil C	medium plasticity (C.L.)

This is according to Casagrande's chart based on plasticity index & L.L. Soil C was that part of Soil B which completely passed 200 U.S.Sieve.

Soil samples were prepared from remoulded soils

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at water contents corresponding to their liquid limit.

The testing of shearing strength was done in a standard shear box apparatus.

The main idea of the experimentation is to partially consolidate the soil samples in a shear box for a certain time and then shear it quickly. Shearing resistance is noted. The water content at the plane of failure is then found out immediately. Graphs are plotted to know the characterstics of the curves shear vs. water content.

The normal pressures under which the tests are made for a soil, are 1 Kg/Cm^2, 3Kg/Cm^2 and 5 Kg/Cm^2 These loads are nearer to loads which may be uncounted in a field problem.

The time for consolidation after which the samples have been sheared are 20 secs, 5 mts, 15 mts, 25 mts, 1 hr, 5 hrs and 20 hrs. It is assumed that after 20 hrs. full consolidation takes place under all normal pressures. Sometimes a few more timing for consolidation have been taken to get a better scatter of points on curves., this is so especially for higher loads.

The method is parallel to the one suggested by N.N.Maslov [2] for partially consolidated clays.

3.2. The Soil Samples :- The soil samples were obtained from the vicinity of the proposed earthen - dam across Malin river in district Bijnor of Uttar Pradesh. The borrow pit site, from which the soil sample were taken, is in a place where water gets collected during rains and nearly dries up during summers. These samples were taken from a depth of 6 to 8 ft. Since the site of borrowpit is usually covered with water it may as well be assumed that the soil beneath would have a moisture content nearer to the liquid limit.

Soil samples A and B were taken from neighbouring borrow pits. Soil sample C is the component of soil B passing U.S. Sieve No. 200.

For the purpose of the tests, however, soil A and B were passing No. 20 U.S.Sieve. The material retained in both the cases, on No. 20 U.S. Sieve was negligible.

Properties of soil Samples- The following standard tests were performed to identify the soil type .

1. Mechanical grain size analysis.
2. Hydrometer analysis for finer contents.
3. Liquid limit.
4. Plastic Limit.
5. Specific gravity.

A visual examination for colour was also done.

The following table gives the salient properties of the soils A., B., and C.

Soil	Colour	L.L.	P.L.	P.I.	Sp. Gr.	Casagrande's Classifica-tions.
A	dull yellow	32%	23%	9%	2.7 to2.72	Silty clay (M.L.)
B	"	34%	20%	14%	-do-	Clay of medium plasticity (C.L.)
C	"	34%	18%	16%	-do-	Clay of medium plasticity (C.L.)

The grainsize distribution curves for soils A, B & C are given in figure 6.

3.3. The Apparatus :-

3.3.1. Shear testing Device : A standard shear box testing device of size 6 cms. x 6 cms. x 4.5 cms was used. The apparatus was constant strain type. The use of this apparatus is justified due to the fact that

a. it is simple to operate.

b. Its value for shear at peak point do not differ very much from that obtained from other devices.

c. The material passing No. 30 U.S. sieve could easily be tested.

d. The value of normal stress on the failure plane and shear resistance are obtained directly.

e. Water content at failure can be determined quickly.

The loads were applied with the help of the lever arrangement which is included in the apparatus. A description is not necessary it is standard device.

The porous plates were of porous copper. Only modification that was done in the apparatus was,

1. placing of a filter paper of the plate size to avoid the clogging of pores of the porous plate.

2. Closing of drainage holes to avoid escape of soil by extrusion at higher water content and havier loads.

3.3.2. <u>Device for Keeping Initial Moisture Content, of the prepared Sample constant</u>.

In order to keep the initial moisture content of the sample, which was at liquid limit prior to testing, uniform and intact, an assumed quantity of soil was mixed thoroughly with the predetermined water content. This was placed in a tray which in its turn was placed in a canister with tight lid. The cannister contained a small quantity of water which surrounded the tray containing the soil. Whenever the sample was not being taken out, the lid of the cannister was closed to avoid loss of moisture in the sample. This arrangement ensured that there would be no loss of moisture since the loss of moisture from a sample having water content at liquid limit is almost the same as that from a free water surface. It is assumed that the sample was fully saturated in such conditions.

3.4. THE TECHNIQUE

About 1000 gms of dry soil was taken. This was just sufficient to perform 5 to 6 shear experiments in a day. This soil was then mixed thoroughly with the calculated amount of water such that the whole soil sample was at the same consistency i.e. at liquid limit. The whole soil sample thus obtained was placed in a tray which was in turn lowered into a canister having water as has been described earlier. This ensured no loss of moisture from the sample which was being left for the next shear testing.

The shear box was cleaned well and dried. The lower frame of the box was then fitted with its base. Over the base were then placed the following in succession :-

a. The porous copper plate.

b. The filter paper

c. The perforated discwith corrugations visible.

The lower frame was then placed into the trolly which carries the whole device on ball-rollers.

The upper frame of the shear box was then placed keeping in mind, the proper position with respect of to the lower frame. The two bid screws were then screwed into their respective positions so as to ensure perfect tightness at the plane of separation of the two frames.

The soil sample was thoroughly pugged and sufficient amount of it was filled into the box thoroughly such that no air pockets remained in the sample. Care was taken to fill the corners of the shear box properly. The soil in the box was filled upto a mark which is generally indicated by drainage -hole position.

The top of the soil " sample into the box was levelled and over it were placed the perforated disc digging into soil, filter paper and at the top, the porous copper plate in succession.

Over the porous plate was placed the block of metal that distributed the normal load uniformly over the sample, in the shear box. To avoid eccentricity while placing load a ball is used over this block on which the normal load is suspended.

Necessary weights, which are required to produce the necessary normal stress over the sample in the shear box, were placed gently to avoid any jerk.

Stop watch was started simultaneously with the loading. Partial consolidation was allowed to take place for the required period of time already mentioned.

The sample was then sheared fast by hand, the rate of loading being 25 m.m./ minute approximately. Care was taken to remove the screws fastening the box frames and also to lift the frames slightly with the help of another set of screws to avoid any friction between the metal surfaces of the two frame. The fast rate of shearing is necessary in order to get the moisture content at failure after a definite period of partial consolidation.

The shear resistance was noted, which was indicated by the proving ring at failure.

The weights were removed immediately to avoid any consolidation after failure has taken place. The box was then quickly removed from the totally and the two frames were pulled so as to part at the plane of failure

A small sample was then taken from the middle of the plane of failure of soil sample. The sample was collected neither too deep from the surface from the plane of failure of soil sample nor too much from near the edges of the sample of soil.

This sample thus collected was immediately transferred to a crucible and weighed accurately before and after drying the sample to get the moisture content at failure.

Similar experiments were conducted with different normal pressure and with different periods of partial consolidation. For each soil and each normal load for a particular time of consolidation at least three tests were performed.

3.6. PRECAUTIONS:

It was necessary to take the following precautions for better results :-

1. The soil sample was thoroughly mixed with the required water content to bring the whole sample to a consistency of liquid limit. The initial moisture

content of the dry soil was taken into account. This was 2% to 3% of the dry weight of soil. Some allowance was kept for the water which evaporates, or sticks to hand and container while mixing. This was about 1% extra water.

2. The sample was mixed one hour prior to the testing. This ensured thorough wetting and saturation of soil.

3. The initial moisture content was always calculated for each sample which put into the shear box for testing.

4. The apparatus was always cleaned throughly before filling it with soil sample. Any dust between the contact plane was avoided.

5. The soil sample was packed throughly, and upto the drainage holes in upper frame so that no air pocket remained and the thickness of sample was about 2.5 cms. The corners of the box were filled thoroughly.

6. While loading the arm with weights jerks due to putting of weights in the pan, were avoided.

7. The screws from the upper frame were removed.

20 sec. before the time of shearing.

8. Other set of screws was used to lift the upper frame slightly from the lower frame without disturbing the state of soil.

9. No sooner the time for consolidation was over the sample was sheared quickly. Removal of load and taking sample for water content did not take more than 15 to 20 secs. in all cases.

10. The sample for finding water content was taken from the middle of the failure plane such that neither it was from deeper side (this will give less water content) nor too much towards the edge of the box (this gives more water content).

11. For readings of more than 25 mts. duration of loading water was filled arround the shear box to avoid drying of the sample in the shear box. This water was removed 10 minutes before shearing the sample.

12. The filter paper and the porous plates used were of the same type throughout the test to provide a fixed condition for drainage.

13. Porous plates were cleaned with pressed air from time to time to remove any fine soil which may have clogged the pores and have reduced the permeability.

14. The filter paper was fixed with water over the porous plate in such a manner that no air bubble remained between the two surfaces.

3.6. Difficulties During Experimentation and Their Remedy :-

The following difficulties were encountered during experimenting with the soil samples. The remedial measures taken have been also described.

1. Difficulty and remedy for keeping the initial moisture content of the prepared sample has already been described.

2. Since the sample was at a consistency of liquid limit the sample followed out through the gaps and spaces described below, when heavier loads were placed :-

The flow took place through

a. holes for drainage in the upper and lower the frame . This was remedied by plugging them with plaster of paris. All the experiments were conducted with the holes plugged, for uniformity.

b. <u>Gap between the two frames of the shear box.</u> This could be remedied by screwing tightly. But even after this, if the same state prevails the shear box may be replaced by a better one., with better finish.

c. <u>Gap between the sides of the perforated plate and the upper frame of the shear box.</u> This gap could be avoided by having the plates which just fit into the frame.

3. Tilting of upper plates after loading was a common and serious difficulty this could be avoided by filling and levelling the soft sample properly, so that all plates are placed horizontally and the load is not applied eccentrically.

4. At times when the copper porous plates were slightly bent, they got struck up with the sides of the upper frame. This might stop consolidation. Some load will be transferred to frame also which would lead to wrong shearing strengths. This also create a difficulty when the upper frame is slightly lifted with the help of screws to avoid friction between

metallic surface.

Remedy for this is to make the plates flat, again, by pressing, such that they just fit into the frame without touching the sides of the frame. Another remedy for this may be to use porous stone instead of porous copper plates.

3.7. Discussions on the Experimenting Technique.

1. The rate of shear strain was about 25 m.m./minute which is very high as compared to that specified for quick tests which is nearly 0.54 m.m./minute. This may affect the results of shear strength which would be higher in the fast rate. However, there was no other possible way toshear the sample without change of water. It was later observed from the graph however that the consolidated quick values of the soil types used were giving the values of internal friction within the limits which have been generally observed by others. More over this fast rate of strain is a common factor for all samples and thus is eliminated automatically.

2. The zero minute reading although important for the curves is difficult to get, especially so when

higher normal loads are acting on the shear plane. Thus, instead of zero minute reading a 30 secs reading were taken.

3. Use of filter paper between the plates does not affect very much the rate of drainage. Moreover, it is used for all testing it can be considered a constant factor for all the sample testings.

4. Plugging of drainage holes reduced the drainage rate thus consolidating time of the sample increased. But the drainage from the top and bottom i.e. through the porous plates, was sufficient for experimenting. This is also a constant factor for all the sample tested and hence does not affect the value.

3.8. Limitations of the Experimental problem.

Only the shearing strength and water contents have been measured and thus relation is only with respect to these two factors all other factors such as thixotropy, the volume change etc have been ignored.

CHAPTER IV

PRESENTATION AND DISCUSSIONS

The curves of figures 1,2 and 3 have been plotted from the readings given in table 1,2 and 3, which were obtained as a result of the average value taken from at least three runs of experiments on similar samples for a definite period of partial consolidation. As has already been mentioned, three soils A, B and C have been taken for experiments to note the characterestics on different soils. These soils are respectively classified as M.L. , C.L. and C.L. according to A Casagrande's chart. For figures 4 & 5 values have been obtained from figures 1,2 and 3.

The following is the presentation and discussion of the curves obtained in Figures 1,2 and 3.

4.1. Shearing Strength Vs. Water Content at Failure (Figures 1,2 and 3).

The three different curves for the different consolidating loads are presented in Figs. 1,2 and 3. The initial condition in all the cases is kept more or less the same, i.e. the tests are carried over on a sample prepared at or nearly equal to the liquid limit. As per usual conventions, here also, they show the small amount of shearing strength at liquid limit. However this initial shearing strength has an increasing trend with increase in consolidating load. This can be attributed to the fact that immediately after the application of load a small time is taken before it is sheared and hence under higher loads comparatively higher degree of consolidation has taken place and therefore it shows the higher initial strength. However this trend is maintained throughout the test procedure viz. The shearing strength is higher for the same moisture content at higher consolidating loads. The final moisture content represents the degree of consolidation and hence lesser the moisture content higher is the degree of consolidation and therefore it shows higher shearing strength.

It is found that at higher loads due to prolonged consolidation the moisture content is more even though the shearing strength is more. This increased moisture may be attributed to the effect of dilatancy. Dilatancy is the expansion of a soil due to shear at a constant value of pressure. Here the soils are having coarse and fine silts which would have greater void ratio when consolidated under smaller loads and thus would contract during shear under smaller loads. Whereas under higher loads the void ratio will be less and so when sheared under higher loads they will show dilatancy and hence increased moisture content due to absorption of water. In this case dilatancy may be there during shear under 3 Kg/Cm^2 but since it is smaller load than 6 Kg/Cm^2 less void ratio will result in case of 6 Kg/Cm^2 pressure and therefore greater dilatancy can be observed and as a result greater water content at failure might be obtained at higher consolidation periods.

In case of soil A, the gradient of the curves are steeper as compared to that of soils B and C. Also the shearing strength after 20 hrs consolidation in corresponding curves for all soils are remaining almost same. However the moisture content corresponding to this shearing strength is more in case of soil A, this may be due to the lesser compressibility of soil A as compared to soils B and C.

In all the three soils it is noticed that the same shearing strength can be achieved by smaller loads .

if applied for longer duration. However if the time for application of load is kept same in all cases higher loads give higher shearing strength. This may be due to the fact that at higher loads rate of drainage is faster and so a greater effective stress will result in greater shearing strength as compared to smaller loads where the drainage will be slower and hence effective stress will be lesser for the same time.

The points which donot fall on the curve can be attributed to the errors and mistakes which are unavoidable when experimenting with a shear box device.

4.2. Mohr's Envelopes for Failure at Particular Water Contents (Figure 4.)

Mohr's envelopes of, figure 4 have been drawn by plotting the values obtained from the figure 1,2 and 3 for water contents which are common to all the three curves in each figure. These values have been tabulated in table 4. The three value of water contents for which the mohr's envelopes have been plotted are two extreme values the minimum and the maximum, which are common to all the curves and one intermediate value. For these three water contents shear strength values have been noted from the figures 1,2 and 3 corres-ponding to all the three curves in each figure.

The dashed line show the Mohr's envelope which is obtained by plotting the values for shearing str-ength after 20 hrs. in all the three cases of nor-mal load in other words it is approximately the enve-lope for consolidated quick test . The shearing strength values are the extreme values of the curves in Figure 1, 2, and 3 at the least water contents. These envelopes provide a comparison for those obtain-ed from partial consolidation case. The values are given in table 8.

It can be seen from the graphs that for lesser degree of consolidation the soils signify content Cw and ϕ_w values, under any loading. But as the conso-lidation progresses the value of cohesion Cw becomes less prominent as the load is increased although the actual shearing strength increases. This indicates that the ϕ_w plays the significant role in shearing strength at this moment of increasing load.

As the consolidation approaches the final stage two distinct portions of Mohr's envelope can be seen, one portion of Mohr's envelope indicating constant

C_u & ϕ_u which is obtained at lower loads under all water contents and the other with increased ϕ_u and smaller cohesion at higher loads with water content decreasing. However for complete consolidation the whole soil behaves as if C' tends to zero and gives constant maximum value for $\emptyset$.

In case of soil A the values of C_u & u remain constant under any load, at lesser degree of consolidation. As the degree of consolidation increases respective values of C_u and ϕ_u increases this may be due to the higher density achieved by consolidation. Howeve in case of soils B and C the constant value of C_u and ϕ_u are obtained only at the earlier stages of partial consolidation. At a later stage after a certain water content the Mohr's envelope is distinctly divided into two parts. One is that which is for loads smaller than 3 Kg/Cm^2 And the other for normal load greater than 5 Kg/Cm^2. C_u and ϕ_u at a water content for both the loads are different. This can again be attributed to the increase in density and decrease in water content of the soil in the first part of the Mohr's envelope in the second case of Mohr's envelope the cause is same but the part played by the high normal stress is much more prominent and thus gives a steeper slope and less value of Cu. This tendency of envelopes is more pronounced in soil B and C which may be due to the greater contact because of more number of particles since these are clays of medium plasticity and contain more of silt.

The Mohr's envelope plotted as dotted lines are for consolidated quick test and are not for the same water content for different normal loads. Since the soils are completely disturbed soils these envelope should pass through origin only. But they are not doing so. This may be due to the errors and mistakes in experimenting which are beyond control or they may be due to incomplete consolidation.

4.3. Cohesion and Angle of Internal Friction Vs. Water Content (Figure 5)

Figure 5 gives these curves. The values for these have been obtained from figure 4 and have been tabulated in table 6. The values of Cu and ϕ_u are for two distinct portions of Mohr's envelopes at different water contents, one for the part with loads lesser than 3 Kg/Cm^2, the other greater than 3 Kg/Cm^2. For an comparison with the state of consolidated quick test the values for these curves are also shown as reading No. 4 in table 6.

A general survey of the curves show that in initial stages of consolidation and for all loads, all values of Cv and Φu are same. As the stage of consolidation advances the values of Cv and Φu at higher loads start changing. It is seen from graph that the rate of change of Φu is much more than that of Cu. Also the rate of change of Φ_w at smaller loads with progressing consolidation is less than that for higher loads. Where as for Cu, rate of change is more at lower loads and smaller at higher loads for ~~progressing consolidation. However Cu~~ and Φ_w in both cases are increasing. However it is evident from the trend of the envelopes for higher loads (Fig. 4.) that the cohesion ultimately must reduce to an insignificant quantity after a certain water content.

This phenomena of reduction of cohesion at higher loads for greater consolidation may not be explained in a way other than that the cohesion at a load is intercepted of the tangent to the Mohr's envelope at the ordinate where p = 0 and since the curve goes steeper at higher loads cohesion must decrease.

The rate of change of ϕ_w in soil A is much more than that of to soils B & C as degree of consolidation progresses. But the difference of rate of change for higher and smaller loads is much less than in case of soils B & C.

Rate of change of cohesion Cv is less in soil A than the corresponding curves for soils B & C. But again the difference in rate of change of cohesion at lower and higher loads in case of soil A is less than those in soils B & C. This may be due to the fact that soil A is silty clay and the other two are inorganic clays of medium plasticity.

It is seen clearly that the part of the Mohr's envelope which is for greater loads than 3 Kg/Cm^2 approach more near to the envelope given by consolidated quick condition where as at lower loads it is not so.

Thus it can be seen that Coulombs law is approximately applicable for finding the shearing [illegible] strength of partially consolidated soil at different water content at failure. And is different from that of consolidated quick tests values of C and Ø and hence values of cohesion and angle of internal friction can be named as C_w and ϕ_w i.e. depending upon water content. Also it is observed that some type of adjustment takes place between the points C_w and frictional resistance as water content changes and hence the shearing strength is dependent upon load and water content both in such a case and which can be nomenclatured as S_{pw}.

CHAPTER V

CONCLUSION

5.1 Conclusion.

The following conclusions may be drawn regarding the shearing characteristics of partially consolidated cohesive soil.

1. With lesser moisture content the shearing strength is greater.
2. At greater consolidating loads the rate of change of shear strength is much more than the smaller consolidating with decrease of moisture content.
3. Shearing strength under higher loads with partial consolidation may be less than at lower loads but higher consolidation.
4. For the same duration of loading the higher loads give higher strength.
5. For higher loads the consolidation process is quicker as compared to smaller loads.
6. For partially consolidated cases at lower loads, the effect of cohesion is predominent. However for cons

the effect of cohesion at higher loads is small.

7. The effect of higher loads as concluded in 6 seems to be more prominent with increasing consolidation and decreasing water content.

8. The rate of increase of cohesion under smaller loads with decreasing water content is more as the type of soil changes from silty to clayey.

9. Rate of increase of angle of internal friction at higher loads and at lesser water contents also increases from silty to clayey soil.

10. There is a definite relation between the envelopes of partial consolidation case and the values of water content for consolidated quick case.

11. The shearing strength of partially consolidated soil also can be approximately represented by Coulomb's straight line equation.

12. All conclusions drawn are in accordance with the modern concept of shearing strength of soils.

5.2. Scope for Further Research :-

This experimentation was but a part of work on partially consolidated soils. Shearing strength is affected by several factors yet unknown to us. An accurate determination of shearing strength is a problem for us which must be investigated thoroughly keeping in view these facts the following can be suggested as the problem for further research in case of partially consolidated soils.

1. The determination of the components which contribute to cohesion and their amounts under different conditions of loading and water content.

2. The effect of thixotropy on shearing strength of partially consolidated soils. Since it must affect the shearing strength.

3. The part played by pore water pressures and its relation with shearing strength is an important part for investigation. Since pore pressures must develope when the soil is consolidated partially and sheared.

APPENDIX A

TABLE 1. (Soil A.)

Initial Water Content at Liquid Limit = 32 percent.

No.	Time "T" for partial consolidation	Shearing Strength (Spr) Kg/Cm² after T under normal stress of			WATER CONTENT (%) at failure after "T" under normal stress of		
		1Kg/sq.cm	3Kg/sq.cm	5Kg/sq.cm	1Kg/sq.cm	3Kg/sq.cm	5Kg/sq.cm
1.	30 sec.	0.0848	0.1305	0.180	32.0	31.8	31.0
2.	5 min.	0.0750	0.2530	0.693	31.7	23.1	24.0
3.	10 min.	0.1030	0.5320	1.230	30.1	23.0	24.0
4.	15 min.	0.2370	0.8330	1.350	25.7	23.6	23.5
5.	25 min.	0.2310	0.8970	1.330	24.8	23.6	23.4
6.	1 hr.	0.3010	0.9400	1.450	24.0	23.1	23.2
7.	5 hrs.	0.4000	0.9300	1.490	24.4	22.7	23.0
8.	20 hrs.	0.4050	0.9900	1.490	24.4	22.4	22.5

TABLE 2. (Soil B.)

Initial Water Content at Liquid Limit = 34%

No.	Time "T" for partial consolidation	SHEARING STRENGTH (Spr) Kg/SqCm after T under normal stress of			WATER CONTENT (%) at failure after "T" under normal stress of		
		1Kg/sqCm	3 Kg/sqCm	5 Kg/sqCm	1Kg/sqCm	3Kg/sqCm	5Kg/sqCm
1.	20 sec.	0.101	0.159	0.237	31.0	31.70	31.25
2.	5 min.	0.127	[illegible]	0.510	29.0	27.32	25.00
3.	15 min.	0.201	0.027	1.500	27.0	23.00	20.50
4.	35 min.	0.470	1.092	1.380	24.8	21.00	20.30
5.	60 hrs	0.505	1.100	1.020	22.2	19.00	21.20

TABLE 8 (Soil C.)

Initial Water Content of Liquid Limit = 34 percent.

No.	TIME "T" for partial consolidation.	SHEARING STRENGTH (Spr) Kg/Cm² after T under normal stress			WATER CONTENT (W%) at failure after T under normal stress of		
		1Kg/sq.cm	3Kg/sq.Cm	5Kg/sq.Cm	1Kg/Sq.Cm	3Kg/Sq.Cm	5Kg/Sq.Cm
1.	30 sec	0.0436	0.036		32.6	31.0	
2.	3 min			0.232			29.00
3.	5 min	0.0335	0.319	0.570	31.7	27.0	23.45
4.	8 min			1.045			24.20
5.	10 min	0.139	0.530	1.275	27.5	29.7	24.00
6.	15 min	0.140	0.700	1.330	27.2	21.6	22.00
7.	25 min	0.143	1.030	1.480	27.0	19.6	22.30
8.	5 hrs.	0.342	1.035	1.540	23.1	19.4	21.00
9.	20 hrs.	0.420	1.050	1.550	22.8	19.4	21.00

TABLE 4 (Values from Fig. 1,2,3 respectively)

No.	SOIL	Water Content W %	SHEARING STRENGTH (Srw) Kg/Cm² under Normal Stress of 1 Kg/sq.cm	3 Kg/SqCm	5 Kg/Sq.Cm.
1.	A	31.0	0.05	0.130	0.19
2.	A	28.0	0.13	0.280	0.40
3.	A	24.5	0.39	0.700	1.07
1.	B	31.0	0.10	0.165	0.24
2.	B	28.0	0.21	0.310	0.41
3.	B	23.5	0.50	0.740	1.11
1.	C	31.0	0.07	0.140	0.19
2.	C	28.0	0.11	0.220	0.37
3.	C	23.0	0.39	0.67	1.10

TABLE 5.

CONSOLIDATED QUICK TEST VALUES FROM Fig. 1,2,3

No.	Soil	SHEARING STRENGTH (Spu) in Kg/Sq.Cm after Total consolidation under stress of			Water content (W) at failure after total consolidation under stress of		
		1Kg/SqCm	3Kg/SqCm	5Kg/Sq.Cm	1Kg/SqCm	3Kg/SqCm	5Kg/SqCm
1.	A	0.40	0.95	1.49	21.4	22.0	20.0
2.	B	0.51	1.10	1.32	23.4	19.5	20.1
3.	C	0.42	1.04	1.53	22.6	19.4	21.3

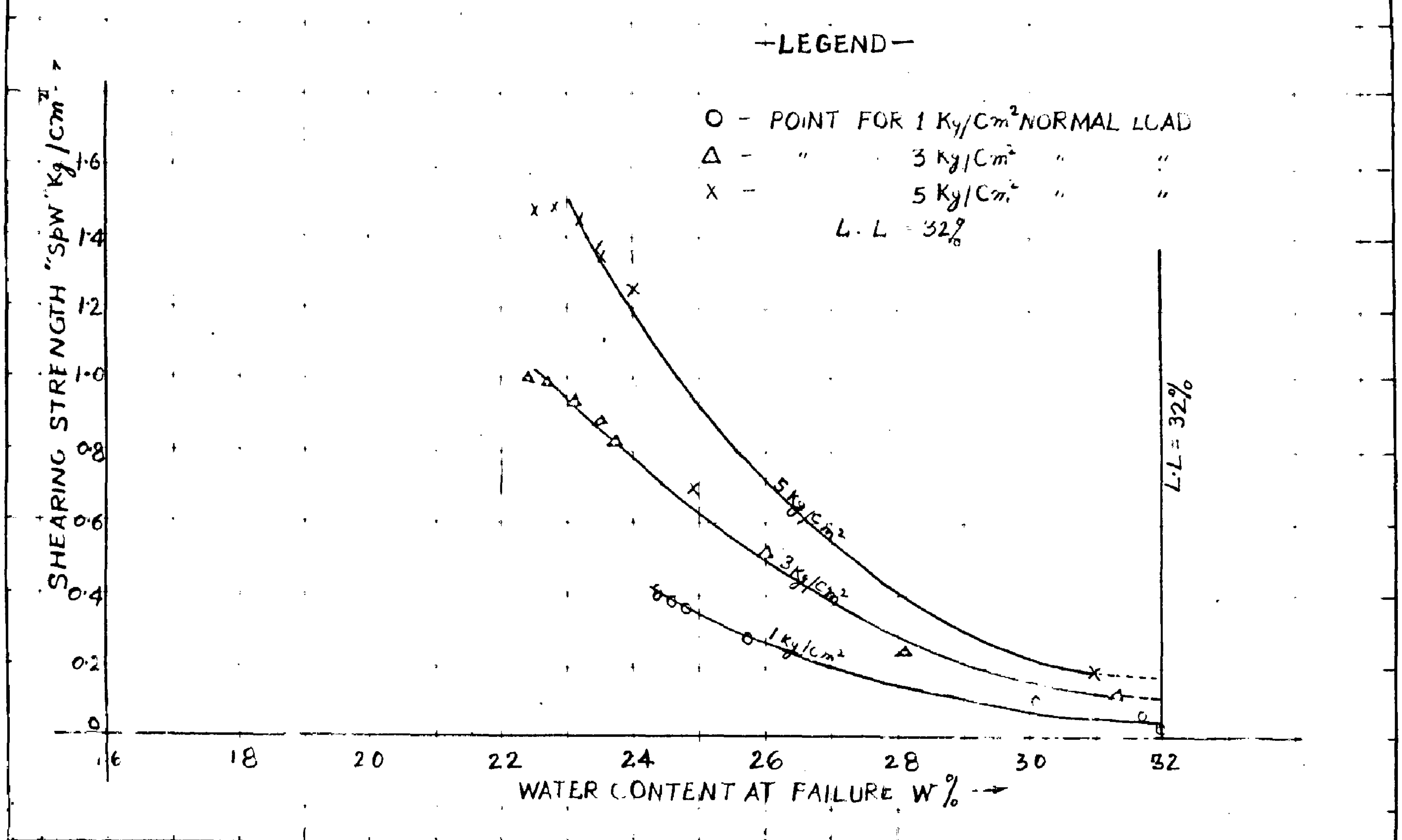
TABLE 6

Cohesion and Friction for Different Water Content Values
From Fig. 4.

No.	Water Content W%	Soil	Values for stresses lesser than 3 Kg/Cm^2		Values for stresses greater than 3 Kg/Cm^2		Remarks
			C_U Kg/Cm^2	ϕ_U degrees	C_U Kg/Cm^2	ϕ_U degrees	
1.	31.0	A	0.025	3.0	0.025	3.0	
2.	27.0	A	0.110	4.5	0.110	4.5	
3.	24.5	A	0.225	9.0	0.120	9.5	
4.		A	0.150	15.0	0.150	15.0	Complete Consolidation
1.	31.0	B	0.030	2.0	0.03	2.0	
2.	27.0	B	0.150	3.5	0.03	5.0	
3.	23.5	B	0.525	5.0	0.30	10.0	
4.		B	0.275	14.5	0.00	20.0	Complete Consolidation
1.	31.0	C	0.05	2.0	0.05	2.0	
2.	27.0	C	0.07	3.5	0.03	5.0	
3.	23.0	C	0.275	6.0	0.03	13.0	
4.		C	0.100	16.0	0.10	16.0	Complete Consolidation

APPENDIX B

FIGURE I SOIL A (SILTY CLAY)
SHEARING STRENGTH Vs WATER CONTENT



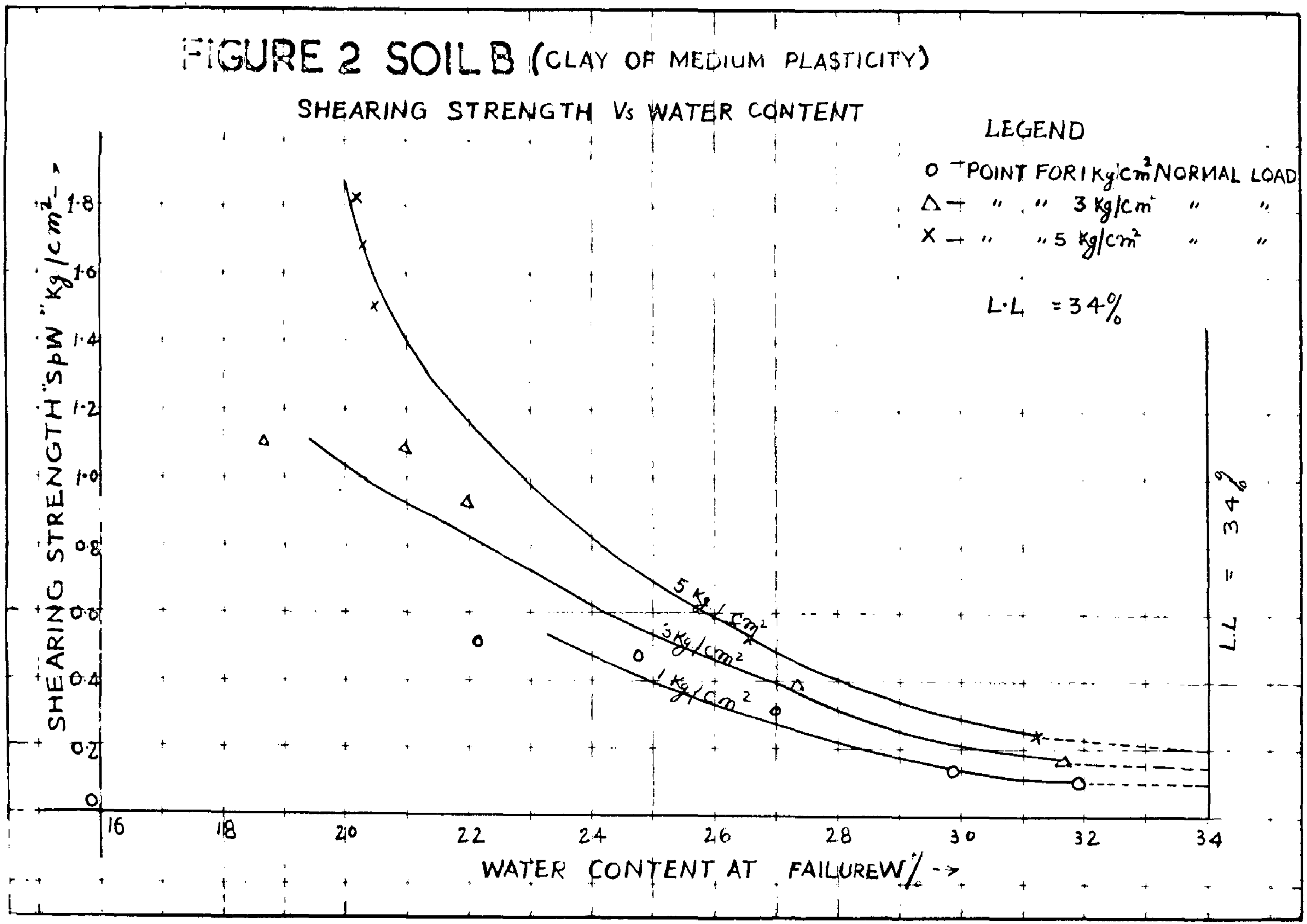


FIGURE 2 SOIL B (CLAY OF MEDIUM PLASTICITY)
SHEARING STRENGTH Vs WATER CONTENT
LEGEND
o — POINT FOR 1 Kg/cm² NORMAL LOAD
Δ — " " 3 Kg/cm² " "
X — " " 5 Kg/cm² " "
L·L = 34%
L.L = 34%
5 Kg/cm²
3 Kg/cm²
1 Kg/cm²
SHEARING STRENGTH "SpW" Kg/cm² →
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
16
18
20
22
24
26
28
30
32
34
WATER CONTENT AT FAILURE W% →

FIGURE 3 SOIL C (CLAY OF MEDIAM PLASTICITY)

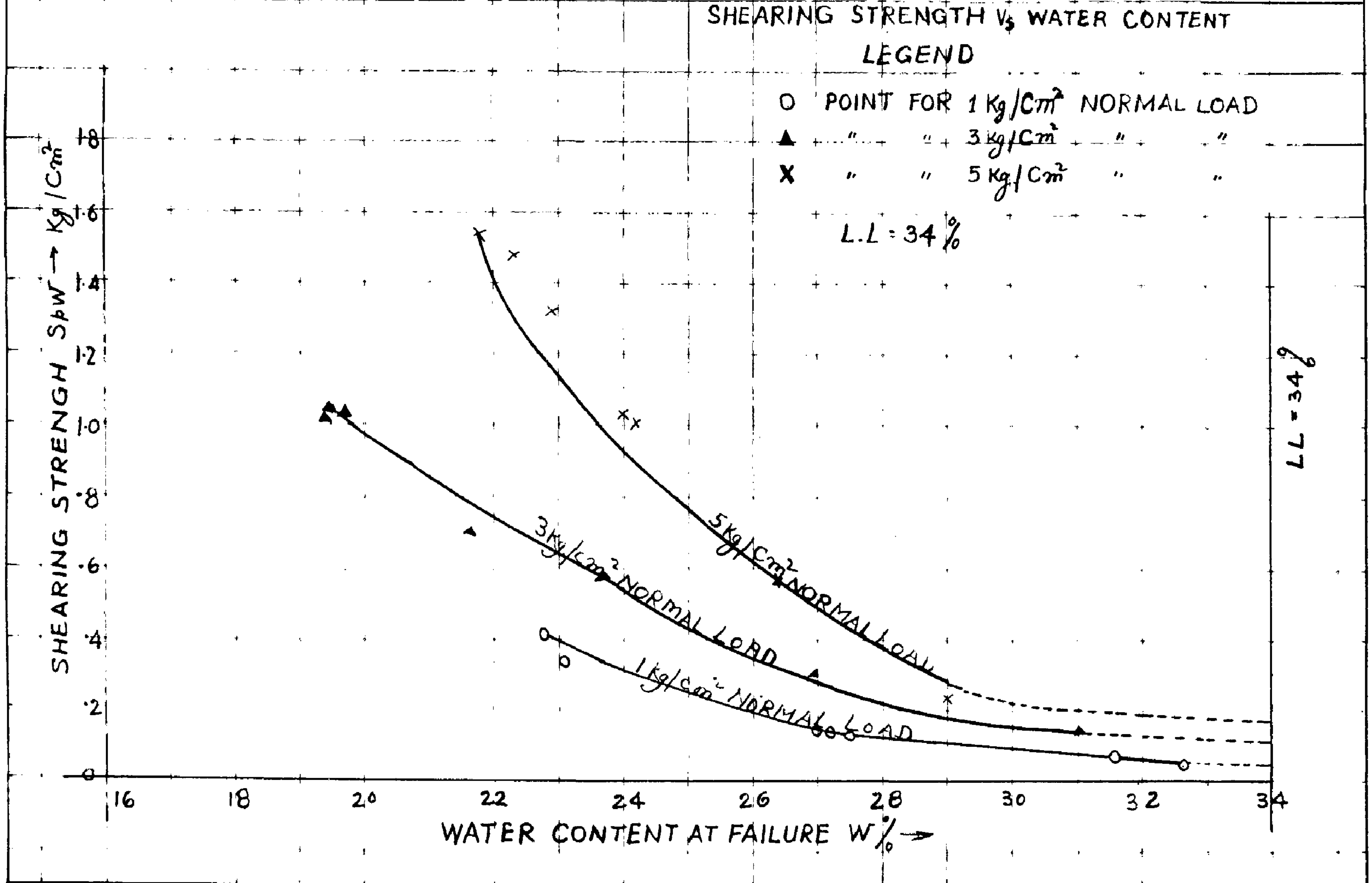


FIGURE 4 (MOHR'S ENVELOPES)

SHEARING STRENGTH "S_{pw}" Vs NORMAL LOAD "p" Kg/Cm^2

FOR DIFFERENT WATER CONTENTS & DIFFERENT SOILS

(POINTS FROM TABLE 4)

DOTTED ENVELOPE FOR CONSOLIDATED QUICK VALUES

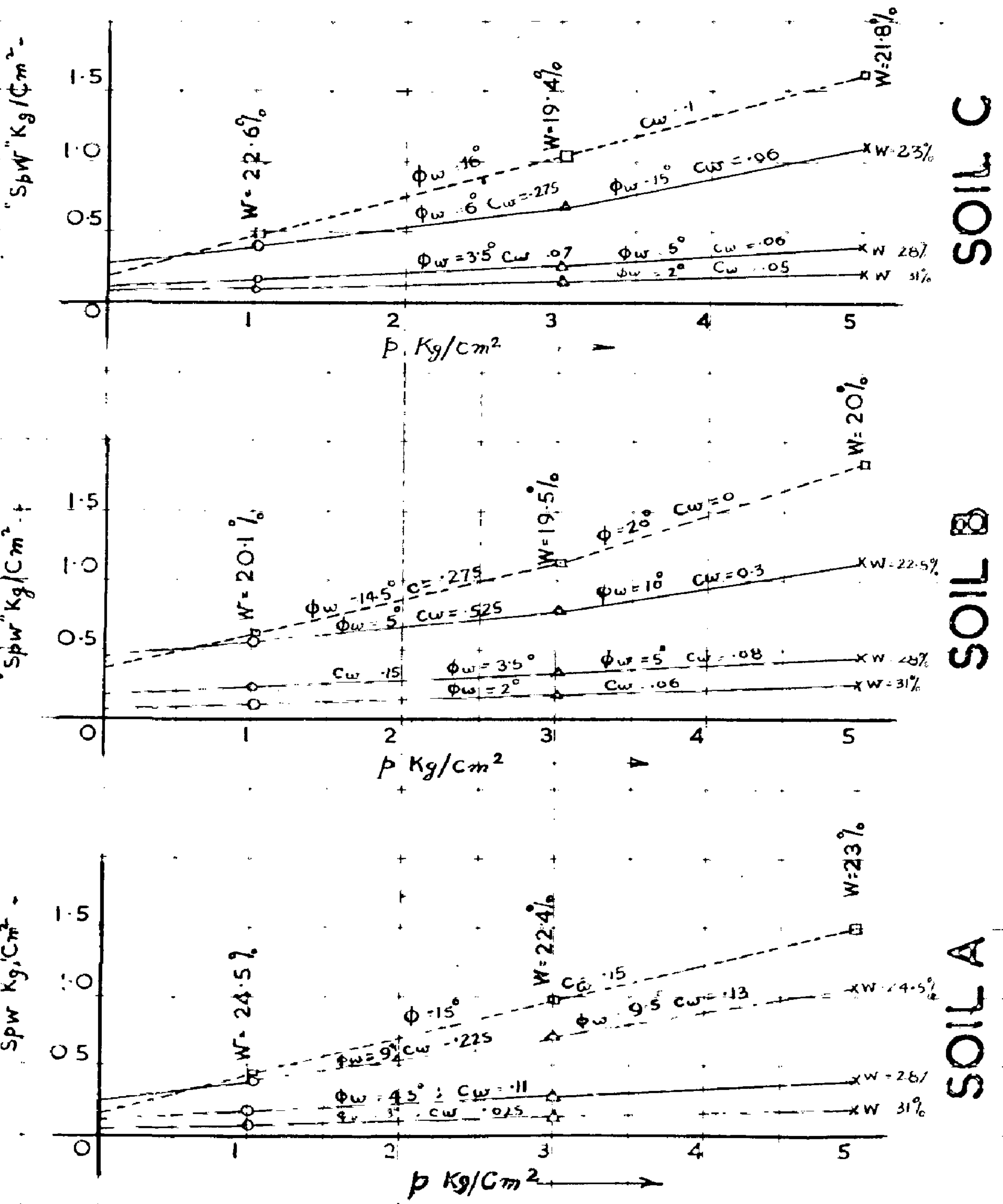
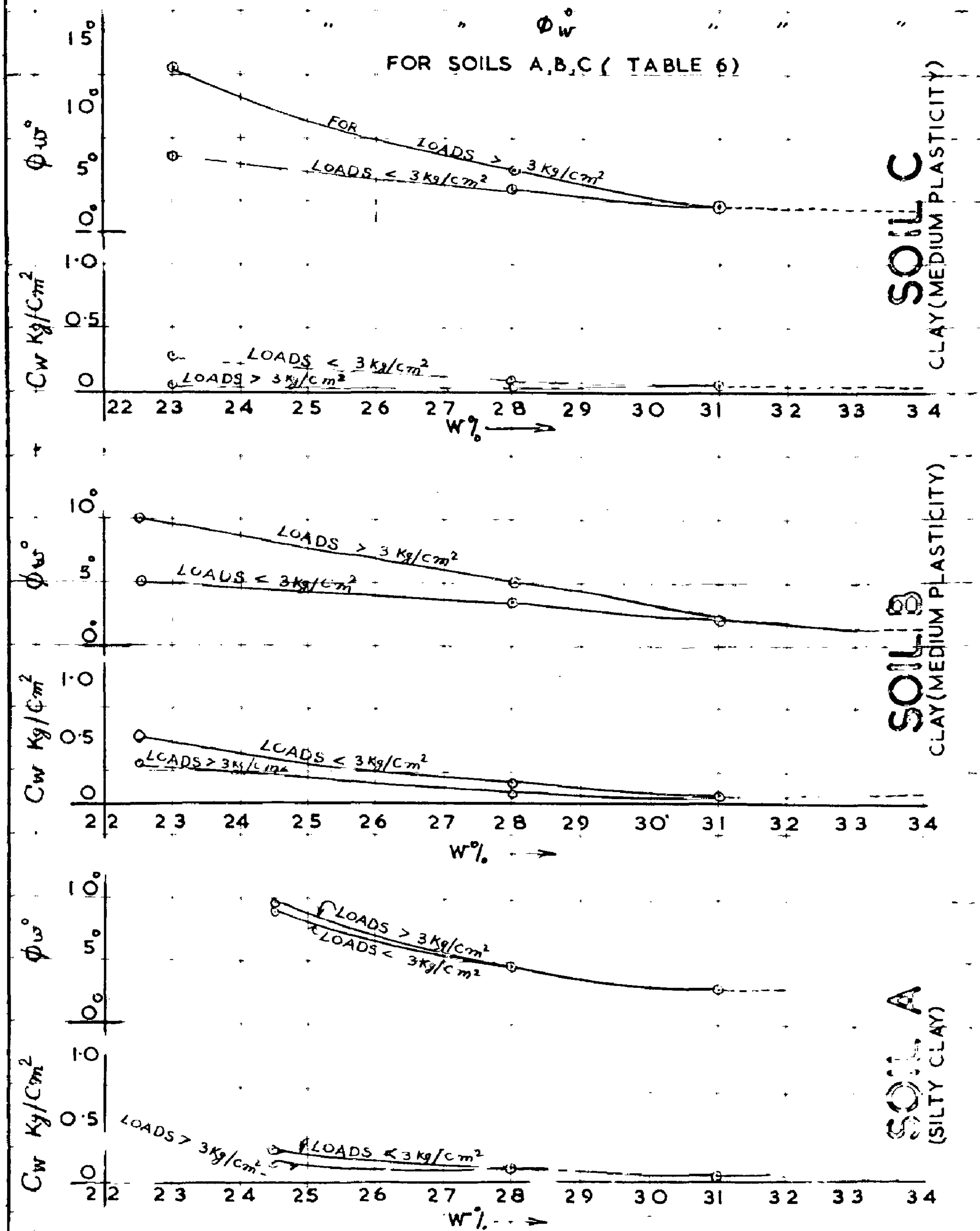


FIGURE 5

RELATION BETWEEN COHESION C_w AND WATER CONTENT W%
" " ϕ_w° " " "
FOR SOILS A, B, C (TABLE 6)



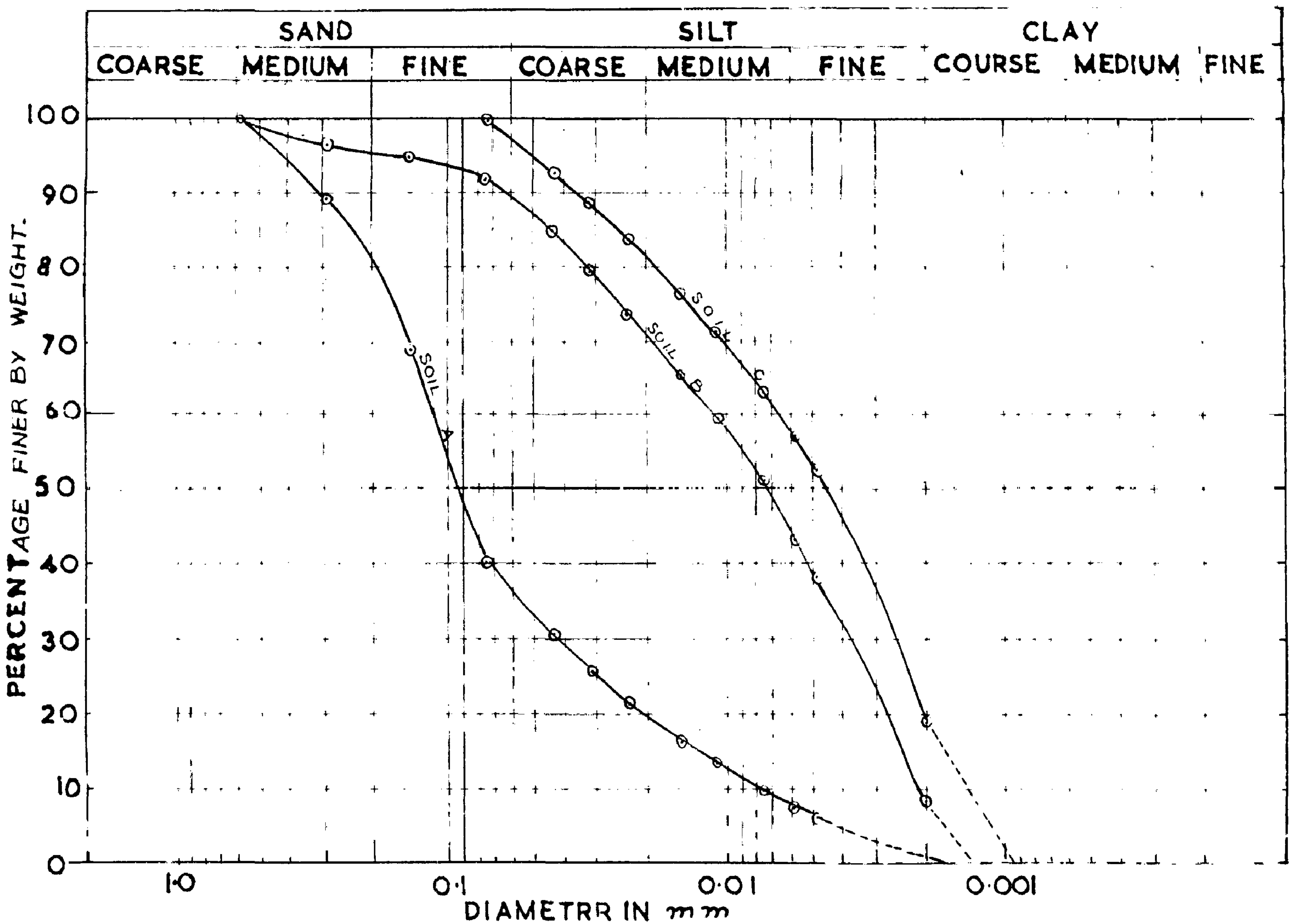


FIGURE 6. GRAIN SIZE DISTRIBUTION OF SOILS A B C (M I T CLASSIFICATION)

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