Paper No. 517

DETERMINATION OF COHESION & ANGLE OF INTERNAL FRICTION OF PARTIALLY SATURATED SOILS FROM STRESS - STRAIN MODEL

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DETERMINATION OF COHESION & ANGLE OF INTERNAL FRICTION OF PARTIALLY SATURATED SOILS FROM STRESS-STRAIN MODEL

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ABSTRACT:

The determination of cohesion (c) and angle of internal friction (ϕ) of partially saturated cohesive soils has been investigated from the unconfined compression test. It omits the usual triaxial compression test (unconsolidated undrained, quick) for which atleast three sets of experimental procedures at different lateral or minor principal stresses are adopted. It is now possible to use the unconfined compression test data alongwith stress-strain model of the soil specimen for determining the above parameters of shear strength. This method shortens the time for determination of the above characteristics of cohesive soils.

On the basis of experimental data collected at the Irrigation Research Institute, Lahore (Pakistan), some mathematical relationships have been developed to compute the values of cohesion and angle of internal friction for partially saturated cohesive soils on the basis of unconfined compression test data.

INTRODUCTION:

The civil engineer, in facing the practical problems arrising out of the use of the soil as foundation and as a construction material, has frequently to strike a balance between the need for careful experimental investigation and simplicity of the means employed. His decision will depend on his own experience and the magnitude, or uniqueness, of the particular problem. His difficulty in reaching a decision is often increased by the lack of certainty as to what testing procedure is appropriate and practicable in particular case.

The use of accurate and elaborate methods of testing requires some justification in the research laboratory. The extent to which these methods should be adopted in routine testing depends largely on whether or not they reduce the margin of uncertainty in design sufficiently enough to justify their cost. The answer is in many cases self-evident. In many less important cases, however, a wider issue is involved. The engineer's own fun of empirical knowledge is made more certain in its future application if it is based on quantitative measurements of the relevent soil properties and of the subsequent performance of the structure itself.

For the laboratory measurement of shear strength, the engineer is largely dependent on the triaxial test[1]. The test may, however, be performed in various ways; and in order to distinguish between the different types of the test and relate/apply them to the more common practical problems, it is necessary to make a brief survey of the basic factors controlling shear strength and deformation.

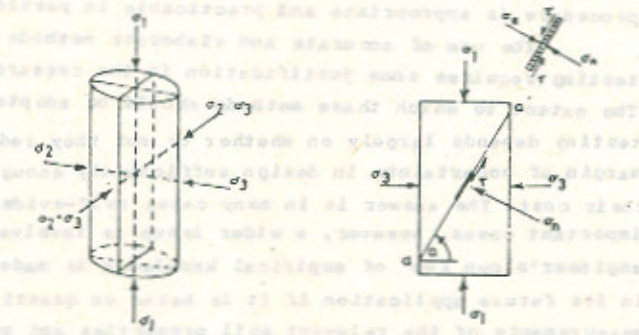
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A shear stress-strain model for cohesive soils, which includes a wide range of parmeters, is only an approximation for observing their behaviour. However, this simple model can serve important purposes. The most common application of this test is to the samples of earth fill material which are compacted in the laboratory under specified conditions of water content and density. It is also applied to undisturbed samples of strata which are not fully saturated (for example, residual soils). A rational interpretation of the parameters of a simple model is a useful approach to the understanding of shear properties of soil possessing both cohesion and internal friction (c-φ soils). The analysis of the parameters can also help in identifying the most important variables (c & φ) to develope and simplify the existing models.

The basic aim of the present paper is to critically analyse the shear stress-strain data of partially saturated cohesive soils and thus explore some empirical relationship under the prescribed testing conditions. Consequently the results of undrained unconsolidated triaxial compression and unconfined compression tests have been considered for this purpose.

TRIAXIAL SHEAR COMPRESSION OF SOIL: 234

The triaxial compression test on a soil which is actually a shear test, is the application of constantly controlled principal stresses, σ_1 , σ_2 & σ_3 along the three orthogonal axes of the soil specimen under test, (fig. 1a). Since the soil samples, subjected to a triaxial compression test, are generally cylindrical in shape, the lateral stresses about the soil sample are equal from all sides, ie. $\sigma_2 = \sigma_3$.



(a) Axonometric view of a soil sample showing principal stress and principal plane.

(b) Vertical cross section through a soil sample showing principal stresses inclined shear plane a-a, normal stress σ_n and shear stress τ.

Fig. 1 Stresses acting on a soil sample in triaxial compression test

In the triaxial compression test the cylindrical soil sample is subjected to a uniform lateral stress σ_3 and then the major principal stress, σ_1, is applied until failure in shear takes place along some shear plane, a-a', fig. 1b. Knowing the magnitude of the applied principal

stresses and the angle α of the rupture plane a-a', it is possible to calculate the normal stress, σ_n, and shear stress, τ, on the rupture plane. The value of σ_n and τ can not be determined directly but has to be computed through some empirical relationship. Triaxial compression test results permit one to determine the values of angle of internal friction φ and cohesion (c) of soils.

The stress conditions on the shear plane of cylindrical soil sample can be determined grapho-analytically and analysed by Mohr's stress circles[2]. To determine the two unknowns φ and c atleast three sets of triaxial tests (at different lateral or minor principal stresses) should be run on the same soil specimen under the similar conditions of loading etc.

UNCONFINED COMPRESSION STRENGTH OF SOIL:

The unconfined compression strength of a soil sample is obtained by means of the unconfined compression test. This test is a special case of the triaxial compression test of soils where the minor principal stresses are zero ($\sigma_2 = \sigma_3 = 0$) fig. 2. The test is based on the assumption that there is no moisture loss during the test. In this test except for the

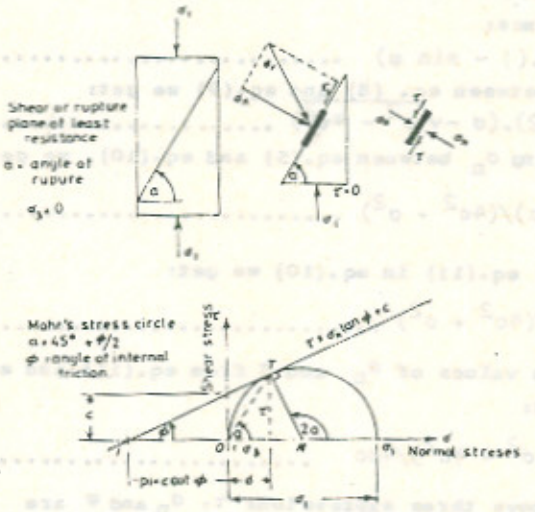


Fig. 2 Principle of unconfined compression test

granular soils, carefully prepared cylindrical soil samples are subjected to gradually increasing vertical pressure, and simultaneous measurements of the deformations of the soil sample are noted. The unconfined compression test is one of the simplest and quickest tests used for the determination of the shear strength of cohesive soils.

MATHEMATICAL ANALYSIS:

The shear strength of c-φ soils obtained in an unconfined compression test can be expressed by Coulomb's shear strength equation as[3]:

$$\tau = \sigma_n \tan \varphi + c \quad \ldots\ldots(1)$$

see fig.2 where all symbols are as before. When $\sigma_3 = 0$ as in an unconfined compression test, the co-ordinates of the point T, lying on Mohr's circle, fig.(2) are (σ_n, τ). The equation of the circle passing through the point T and the origin (0,0)

will be:

$$\sigma_n^2 + \tau^2 - \sigma.\sigma_n = 0 \quad \ldots\ldots(2)$$

where

σ_n = normal stress

τ = shear stress

σ = major principal stress

Differentiating eq.(2) for τ, w.r.t σ_n we get:

$$d\tau/d(\sigma_n) = (\sigma - 2\sigma_n)/2\tau$$

or $$\tan\varphi = (\sigma - 2\sigma_n)/2\tau \quad \ldots\ldots(3)$$

From eq.(1):

$$\tan\varphi = (\tau - c)/\sigma_n \quad \ldots\ldots(4)$$

Substituting the value of $\tan\varphi$ in eq.(3) we get:

$$(\tau - c)/\sigma_n = (\sigma - 2\sigma_n)/2\tau$$

which simplies to:

$$\sigma.\sigma_n - 2\tau.c = 0 \quad \ldots\ldots(5)$$

From geometry of the fig.(2):

$$\sigma_n = \sigma.\cos^2\alpha \quad \ldots\ldots(6)$$

$$\tau = (\sigma/2).\sin 2\alpha \quad \ldots\ldots(7)$$

Substituting $2\alpha = 90 + \varphi$ [1], eq.(7) becomes:

$$\tau = (\sigma/2).\cos\varphi \quad \ldots\ldots(8)$$

and eq.(6) becomes:

$$\sigma_n = (\sigma/2).(1 - \sin\varphi) \quad \ldots\ldots(9)$$

Eliminating φ between eq. (8) and eq.(9) we get:

$$\sigma_n = (1/2).(\sigma - \sqrt{\sigma^2 - 4\tau^2}) \quad \ldots\ldots(10)$$

Again eliminating σ_n between eq.(5) and eq.(10), we get:

$$\tau = (2\sigma^2 c)/(4c^2 + \sigma^2) \quad \ldots\ldots(11)$$

and substituting eq.(11) in eq.(10) we get:

$$\sigma_n = 4\sigma c^2/(4c^2 + \sigma^2) \quad \ldots\ldots(12)$$

Substituting the values of σ_n and τ from eq.(11) and eq.(12) in eq.(3) we get:

$$\tan\varphi = (\sigma^2 - 4c^2)/4\sigma c \quad \ldots\ldots(13)$$

In the above three expressions τ, σ_n and φ are independent of the angle of rupture which, in general, is not easily and accurately determined after the test procedure, especially in soils possessing sufficient water content. τ, σ_n and $\tan\varphi$ depend upon the values of σ and c where σ is determined experimentally as the unconfined compression strength and c, by the consideration of stress-strain curve and Mohr's circle diagram having the major principal stress equal to σ. This very consideration, when $\sigma_3 = 0$, forms the basic concept for the present study to procede further.

Let a soil sample having the required density and moisture be subjected to unconfined compression test. The soil sample fails at a major principal stress σ. With the increasing stress, deformation of the soil specimen is simultaneously noted. In fig.(3), Mohr's circle when lateral pressure is equal to zero is drawn and on the same graph per cent strain scale is set along shear stress axis in such a manner that per cent strain scale corresponds to two fifth of the normal stress scale in digital sequence as shown in the figure.

Consequently if on the normal stress scale, we read 5 pound per square inch then on the shear stress axis the strain should be two per cent and so on. This has been explored experimentally on the basis of a large number of tests. With this set of scales a stress-strain curve is drawn (shown in dotted form in fig. 3). This curve intersects the Mohr's circle at a point D. Let the co-ordinates of the point D be σ'_n, τ'. The scale for the per cent strain has been set in such a way that the point D will always lie on the circle such that $\sigma'_n > \sigma/2$.

About sixty soil samples having different classifications

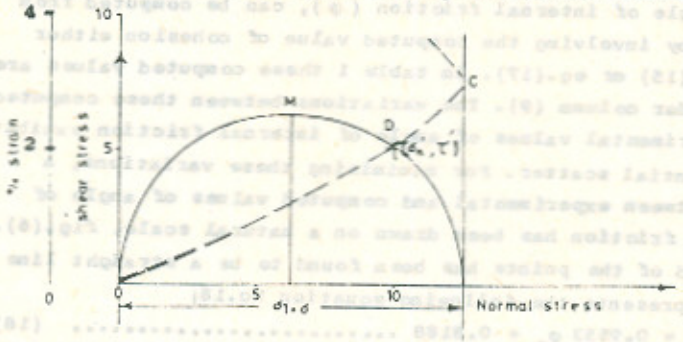


Fig. 3 Mohr's circle and stress-strain curve

were subjected to triaxial compression test and unconfined compression test under the identical conditions of density and moisture. The preference was given to Proctor's maximum density and optimum moisture content. From triaxial compression test (unconsolidated-undrained, quick) the values of cohesion (c) and angle of internal friction (φ) were determined with varying lateral pressures while from unconfined compression test, the values of σ, σ'_n and τ' were found out.

The data thus obtained was analysed for the purpose of establishing any inter-relationship between these parameters. It was observed that the difference between half of the circumferece of Mohr's circle ($\pi\sigma/2$) and the sum of the co-ordinates of the point D ($\sigma'_n + \tau'$) is very close to the experimental value of cohesion of the soil. In table 1 the computed value of this difference (M) is shown under column (7) whereas the experimental values of cohesion under column (5). This finding encouraged drawing of a graph between the experimental values of cohesion on Y-axis and the computed values of M on X-axis. The trend of the points marked on a simple scale showed a parabolic shape, the solution for which is assumed as under:

$$Y = aX^2 + bX + C \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (14)$$

Solving this equation with the help of fig.(4), we get:

$$C = -5.5228 \times 10^{-3} M^2 + 1.0096 M + 0.1937 \quad \ldots\ldots\ldots (15)$$

On further scrutiny of the data as given in table 1 it was revealed that eq.(15) shows a remarkable agreement between the experimental and computed values of cohesion upto

the level when $\sigma'_n - \tau' > 4.5$. But as soon as this difference becomes equal to or less than 4.5, these values diverge alarmingly. Therefore, a second consideration to this situation was focussed on drawing a seperate figure with such deviating points shown encircled in table 1. On the basis of this data fig.(5) has been drawn on simple scale between σ(X-axis) and the product $c \times \sigma'_n \times \tau'$ (Y-axis). We again obtain a parabolic shaped figure, the solution for which is assumed as under:

$$Y = a(X)^b \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (16)$$

Solving this equation with the help of fig.(5), we get:

$$c \times \sigma'_n \times \tau' = 0.1533(\sigma)^{2.9115} \quad \ldots\ldots\ldots\ldots\ldots \quad (17)$$

The second parameter of the triaxial shear compression test, angle of internal friction (φ), can be computed from eq.(13) by involving the computed value of cohesion either from eq.(15) or eq.(17). In table 1 these computed values are given under column (9). The variations between these computed and experimental values of angle of internal friction exhibit a substantial scatter. For minimising these variations, a graph between experimental and computed values of angle of internal friction has been drawn on a natural scale, fig.(6). The trend of the points has been found to be a straight line which represents the following equation No.18:

$$\varphi = 0.9553\,\varphi_c + 0.8188 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (18)$$

where

φ_c = computed value of angle of internal friction from eq.(13), in degrees

By using the eq.(18) the variation between φ and φ_c greatly reduces to an appreciable level as shown in table 1.

SOLVED EXAMPLE

a) when $\sigma'_n - \tau' > 4.5$:

With the help of the following data as obtained from unconfined compression test, find out cohesion and angle of internal friction of a given partially saturated cohesive soil:

Maximum normal stress (σ) = 23.10 psi

σ'_n = 20.00 psi

τ' = 8.25 psi

Solution:

$$M = \pi\sigma/2 - (\sigma'_n + \tau')$$
$$= (22 \times 23.10)/7 - (20 + 8.25)$$
$$= 8.04$$

Substituting the value of M = 8.04 in eq.(15) we get:

$$c = -5.6228 \times 10^{-3}(8.04)^2 + 1.0096 \times 8.04 + 0.1937$$
$$= 7.95 \text{ psi}$$

For determining angle of internal friction, substituting the values of c and σ in eq.(13), we get:

$$\tan\varphi_c = \frac{(23.10)^2 - 4(7.95)^2}{4 \times 23.10 \times 7.95} = 0.3823$$

$$\varphi_c = 20.9 \text{ degrees}$$

Again substituting the value of $\varphi_c = 20.9^\circ$ in eq.(18) we get the corrected value of φ = 20.8 degrees

For confirmation, the same soil sample was subjected to triaxial compression test with varying lateral pressures. The values of cohesion and angle of internal friction come out to be as under:

c = 8.0 psi

φ = 20.7 degrees

b) when $\sigma'_n - \tau' < 4.5$

With the help of the following data as obtained from unconfined compression test, find out cohesion and angle of internal friction of a given partially saturated cohesive soil:

Maximum normal stress (σ) = 18.5 psi

σ'_n = 10.5 psi

τ' = 8.5 psi

Solution:

Substituting the above values in eq.(17), we get:

$c \times 10.5 \times 8.5 = 0.1533\ (18.5)^{2.9115}$

= 8.40 psi

For determining the angle of internal friction, substituting the values of c and σ in eq.(13), we get:

$$\tan\varphi_c = \frac{(18.5)^2 - 4(8.4)^2}{4 \times 18.5 \times 8.4} = 0.0965$$

φ_c = 5.51 degrees

Again substituting the value of $\varphi_c = 5.51^\circ$ in eq.(18) we get the corrected value of φ = 6.08 degrees

For confirmation, the same soil sample was subjected to triaxial compression test with varying lateral pressures. The values of cohesion and angle of internal friction come out to be as under:

c = 8.35 psi

φ = 6.0 degrees

Both the solved examples and the attached data given in table 1 show a marvellous agreement with the values of cohesion and angle of internal friction computed with the present method to confirm its authenticity.

CONCLUSIONS:

From the data observed during unconfined compression tests on various soils and its analysis, we can conclude:-

1. Triaxial shear tests on partially saturated soils are not mandatory for the determination of cohesion and angle of internal friction
2. The above two parameters can also be determined through unconfined compression tests of partially saturated soils
3. The data and analysis confirm that the equation Nos. 13, 15, 17 and 18 can safely be used to determine the above referred parameters thus avoiding the necessity of carrying out the more time consuming and cumbersome triaxial shear tests on partially saturated soils

RECOMMENDATIONS:

Based on the analysis of test data and conclusions drawn therefrom it is recommended that:-

1. Unconfined compression tests should replace the triaxial shear tests for the determination of cohesion and angle of internal friction of partially saturated cohesive soils
2. There is a definite need of conducting further research in this field opening on undisturbed soils and also when $\sigma n' < \tau'$. The efforts to develope some relationship, if any, are made on the basis of experimentations made under the above referred conditions.

ACKNOWLEDGEMENT:

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REFERENCES:

1. A.W.Bishop & D.J. henkel — The measurement of soil properties in the triaxial test. The English Language Book Society & Edward Arnold (Publishers) Ltd., England BLBS edition reprint-1972
2. A.R. Jumikis — Soil Mechanics. D. Van Nostrand Company, Inc. Princeton, New Jersey-1968
3. Karl Terzaghi — Theoratical Soil Mechanics. John Willey & sons Inc., New York-1942

Tab. 1.1

UNCONFINED & TRIAXIAL SHEAR COMPRESSION TEST DATA ALONGWITH COMPUTED VALUES OF THE SHEAR PARAMETERS

Sample No.	Unconfined compression test			Triaxial compression test		M = $\pi\sigma/2 - (\sigma'_n + \tau')$	Computed values of		Corrected value of (φ_c) degrees	Variations	
	unconfined comp. strength (σ) psi	abscisa of pt. of intersection (σ'_n)	ordinate of pt. of intersection (τ')	cohesion (c) psi	Angle of int. friction (φ) degrees		Cohesion (c) psi	Angle of internal friction (φ_c) degrees		Cohesion psi	Angle of int. friction degrees
1	2	3	4	5	6	7	8	9	10	8-5	10-6
1	17.2	13.5	7.0	6.5	15.5	6.52	6.53	15.6	15.7	+0.03	+0.2
2	32.0	20.9	15.7	13.0	12.5	13.67	12.94	12.1	12.4	-0.06	-0.1
3	44.0	37.2	15.6	15.5	21.0	16.32	15.17	20.8	20.7	-0.33	-0.3
4	39.4	35.0	11.7	14.0	18.2	15.19	14.23	18.3	18.3	+0.23	+0.1
5	45.8	42.0	13.2	15.5	21.0	16.74	15.52	21.7	21.5	+0.02	+0.5
6	26.0	17.9	12.0	11.0	12.5	10.94	10.57	11.8	12.1	-0.43	-0.4
7	47.0	45.8	8.8	18.0	17.0	19.43	17.53	16.6	16.7	-0.47	-0.3
8	40.0	29.9	17.0	15.0	16.5	15.93	14.85	16.8	16.9	-0.15	+0.4
9	56.0	53.3	13.1	20.0	20.5	21.56	19.35	20.7	20.6	-0.65	+0.1
10	40.0	31.2	16.2	15.0	18.0	15.43	14.43	18.4	18.4	-0.57	+0.4
11	38.0	31.6	13.9	13.5	19.5	14.19	13.39	19.7	19.6	-0.11	+0.1
12	28.0	20.8	12.5	10.0	17.6	10.68	10.30	17.3	17.3	+0.30	-0.3
13	34.0	24.6	15.6	13.2	17.6	13.21	12.55	17.1	17.2	-0.65	-0.4
14	39.0	25.3	18.4	16.3	11.2	17.56	16.19	10.6	10.9	-0.11	-0.3
15	27.0	20.6	11.5	10.0	16.5	10.31	10.0	16.9	17.0	± 0	+0.5

UNCONFINED & TRIAXIAL SHEAR COMPRESSION TEST DATA ALONGWITH COMPUTED VALUES OF THE SHEAR PARAMETERS

Sample No.	Unconfined compression test			Triaxial compression test		$M=\pi\sigma/2-(\sigma'_n+\tau')$	Computed values of		Corrected value of (φ_c) degrees	Variations	
	unconfined comp. strength (σ) psi	abscisa of pt. of intersection (σ'_n)	ordinate of pt. of intersection (τ')	cohesion (c) psi	Angle of int. friction (φ) degrees		Cohesion (c) psi	Angle of internal friction (φ_c) degrees		Cohesion psi	Angle of int. friction degrees
1	2	3	4	5	6	7	8	9	10	8-5	10-6
16	29.0	17.2	14.5	11.0	15.0	-	11.13	15.0	15.1	+0.13	+0.1
17	38.5	28.8	16.9	13.5	18.4	14.77	13.88	18.4	18.4	+0.38	±0
18	40.5	34.2	14.4	14.0	20.0	15.02	14.09	20.3	20.2	± 0	+0.2
19	29.2	22.9	12.0	11.0	18.0	10.97	10.59	18.1	18.1	-0.41	+0.1
(20)	18.4	10.2	8.9	8.0	8.0	-	8.13	7.1	7.6	+0.13	-0.4
(21)	13.0	7.3	6.4	5.8	7.2	-	5.74	7.1	7.6	-0.06	+0.4
22	24.0	20.0	9.8	7.9	24.0	7.9	7.82	23.8	23.6	-0.08	-0.4
23	51.0	45.2	16.2	17.6	22.0	18.71	17.11	22.3	22.1	-0.49	+0.1
24	60.0	56.8	14.4	21.0	21.5	23.05	20.48	21.4	21.3	-0.52	-0.2
25	30.5	24.5	12.5	11.0	21.0	10.91	10.54	20.7	20.6	-0.46	-0.4
26	27.6	22.8	9.6	10.5	15.0	10.64	10.57	15.1	15.2	+0.07	+0.2
(27)	31.2	19.5	15.0	12.0	16.5	-	11.74	16-1	16.2	-0.26	-0.3
28	31.2	24.8	12.5	11.0	18.5	11.71	11.25	18.4	18.4	+0.25	-0.1
29	40.0	37.3	9.3	15.0	16.0	16.23	15.10	15.9	16.0	+0.10	± 0
30	33.0	25.0	13.8	13.0	15.8	13.04	12.40	16.1	16.2	-0.50	+0.4

Tab. 1.3

UNCONFINED & TRIAXIAL SHEAR COMPRESSION TEST DATA ALONGWITH COMPUTED VALUES OF THE SHEAR PARAMETERS

Sample No.	Unconfined compression test			Triaxial compression test		M= $\pi\sigma/2 - (\sigma'_n+\tau')$	Computed values of		Corrected value of (φ_c) degrees	Variations	
	unconfined comp. strength (σ) psi	abscisa of pt. of intersection (σ'_n)	ordinate of pt. of intersection (τ')	cohesion (c) psi	Angle of int. friction (φ) degrees		Cohesion (c) psi	Angle of internal friction (φ_c) degrees		Cohesion psi	Angle of int. friction degrees
1	2	3	4	5	6	7	8	9	10	8-5	10-6
31	14.2	12.6	4.2	5.5	13.8	5.51	5.59	13.6	13.8	+0.09	± 0
32	18.4	17.8	4.0	7.5	15.0	7.10	7.08	14.8	15.0	-0.42	± 0
33	30.5	20.9	14.3	12.7	13.0	12.71	12.12	13.0	13.2	-0.58	+0.2
34	24.8	17.0	12.0	10.0	14.5	9.96	9.69	14.0	14.2	-0.31	-0.3
35	25.0	17.0	11.5	11.0	10.5	10.77	10.41	10.4	10.8	-0.59	+0.3
36	41.8	37.1	12.6	15.0	19.0	15.96	14.87	19.1	19.1	-0.13	+0.1
37	32.4	31.2	7.0	12.0	17.0	12.69	12.10	16.5	16.6	+0.10	-0.4
38	31.3	21.0	14.8	13.0	12.0	13.37	12.69	11.9	12.2	-0.31	+0.2
(39)	19.6	10.0	10.0	9.0	6.0	-	8.87	5.7	6.3	-0.13	+0.3
(40)	22.0	13.7	11.0	8.5	17.0	-	8.20	16.6	16.7	-0.30	-0.3
41	31.6	25.8	12.1	11.5	19.5	11.74	11.27	19.0	19.0	-0.23	-0.5
42	45.0	43.6	6.9	18.0	12.0	20.19	18.29	11.8	12.1	-0.20	+0.1
43	54.0	51.7	11.4	19.0	18.5	21.72	19.47	18.4	18.4	+0.47	-0.1
44	34.2	28.6	12.6	12.0	20.0	12.52	11.95	20.1	20.0	-0.05	± 0
45	28.0	20.2	12.0	12.0	12.0	11.78	11.31	12.1	12.4	-0.69	+0.4

Tab. 1.4

UNCONFINED & TRIAXIAL SHEAR COMPRESSION TEST DATA ALONGWITH COMPUTED VALUES OF THE SHEAR PARAMETERS

Sample No.	Unconfined compression test			Triaxial compression test		$M=\pi\sigma/2-(\sigma'_n+\tau')$	Computed values of		Corrected value of (φ_c) degrees	Variations	
	unconfined comp. strength (σ) psi	abscisa of pt. of intersection (σ'_n)	ordinate of pt. of intersection (τ')	cohesion (c) psi	Angle of int. friction (φ) degrees		Cohesion (c) psi	Angle of internal friction (φ_c) degrees		Cohesion psi	Angle of int. friction degrees
1	2	3	4	5	6	7	8	9	10	8-5	10-6
46	28.5	18.3	13.2	12.5	6.5	13.27	12.80	6.1	6.6	+0.30	+0.1
47	22.7	12.0	11.3	10.2	7.0	-	10.21	6.1	6.6	+0.21	-0.4
48	17.9	10.4	8.9	8.0	12.0	-	7.30	11.6	11.9	-0.70	-0.1
49	22.0	15.3	9.8	9.0	10.0	9.46	9.24	9.9	10.3	-0.24	+0.3
50	25.0	17.9	11.4	10.5	14.5	9.97	9.70	14.3	14.5	-0.80	± 0
51	25.4	14.3	12.7	10.0	12.0	-	10.4	11.4	11.7	+0.40	-0.5
52	16.6	11.6	7.3	7.0	15.0	-	6.46	14.2	14.4	-0.54	-0.6
53	23.1	19.8	8.2	8.5	19.0	8.29	8.18	19.4	19.4	-0.32	+0.4
54	10.0	7.7	4.3	4.0	16.0	-	3.78	15.8	15.9	-0.22	-0.1
55	19.6	10.8	9.8	8.0	9.0	-	8.38	8.9	9.3	+0.38	+0.3
56	11.9	8.5	5.5	4.5	16.5	-	4.44	16.5	16.6	-0.06	+0.10
57	22.2	13.5	11.1	8.5	15.5	-	8.51	15.0	15.2	+0.01	-0.30
58	25.3	22.3	8.3	9.0	18.5	9.19	9.0	19.1	19.1	±0	+0.60

FIG 4

COHESION (C) Vs

$\frac{\pi \sigma}{2} - (\sigma_n' + \tau') = M$ CURVE

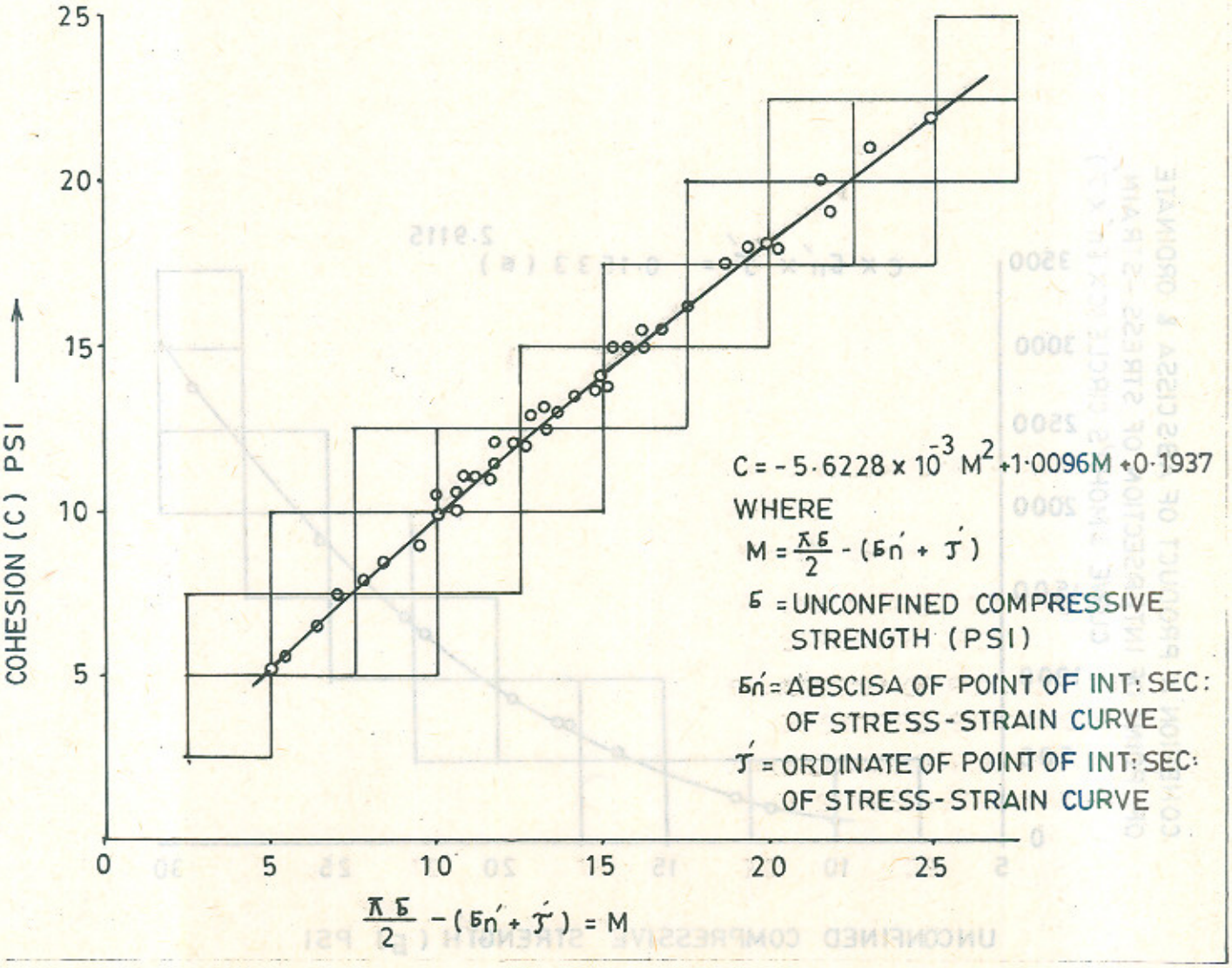


Fig 5

UNCONFINED COMPRESSIVE STRENGTH Vs COHESION x PRODUCT OF ABSCISSA & ORDINATE OF POINT OF INTERSECTION OF STRESS-STRAIN CURVE & MOHR'S CIRCLE WHEN $\sigma_3 = 0$

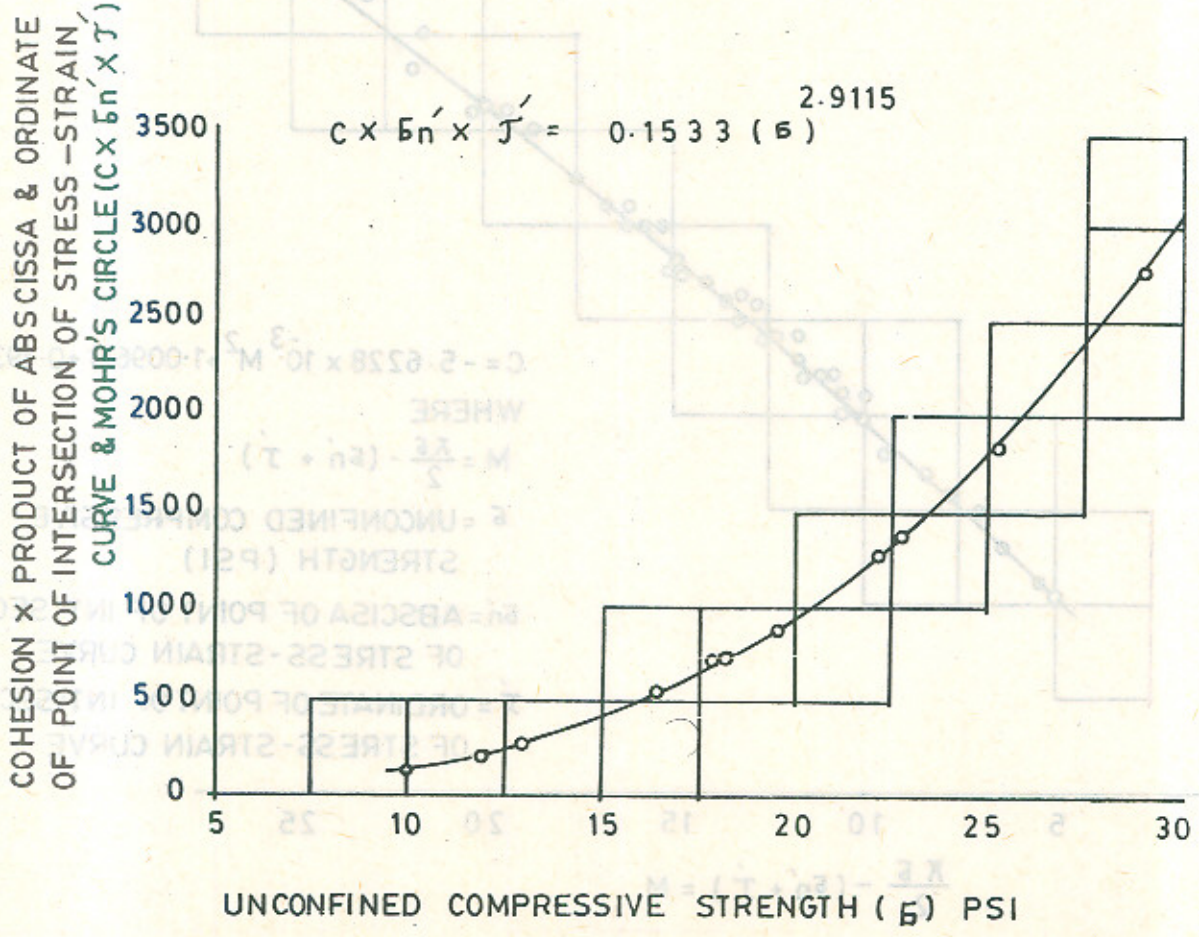
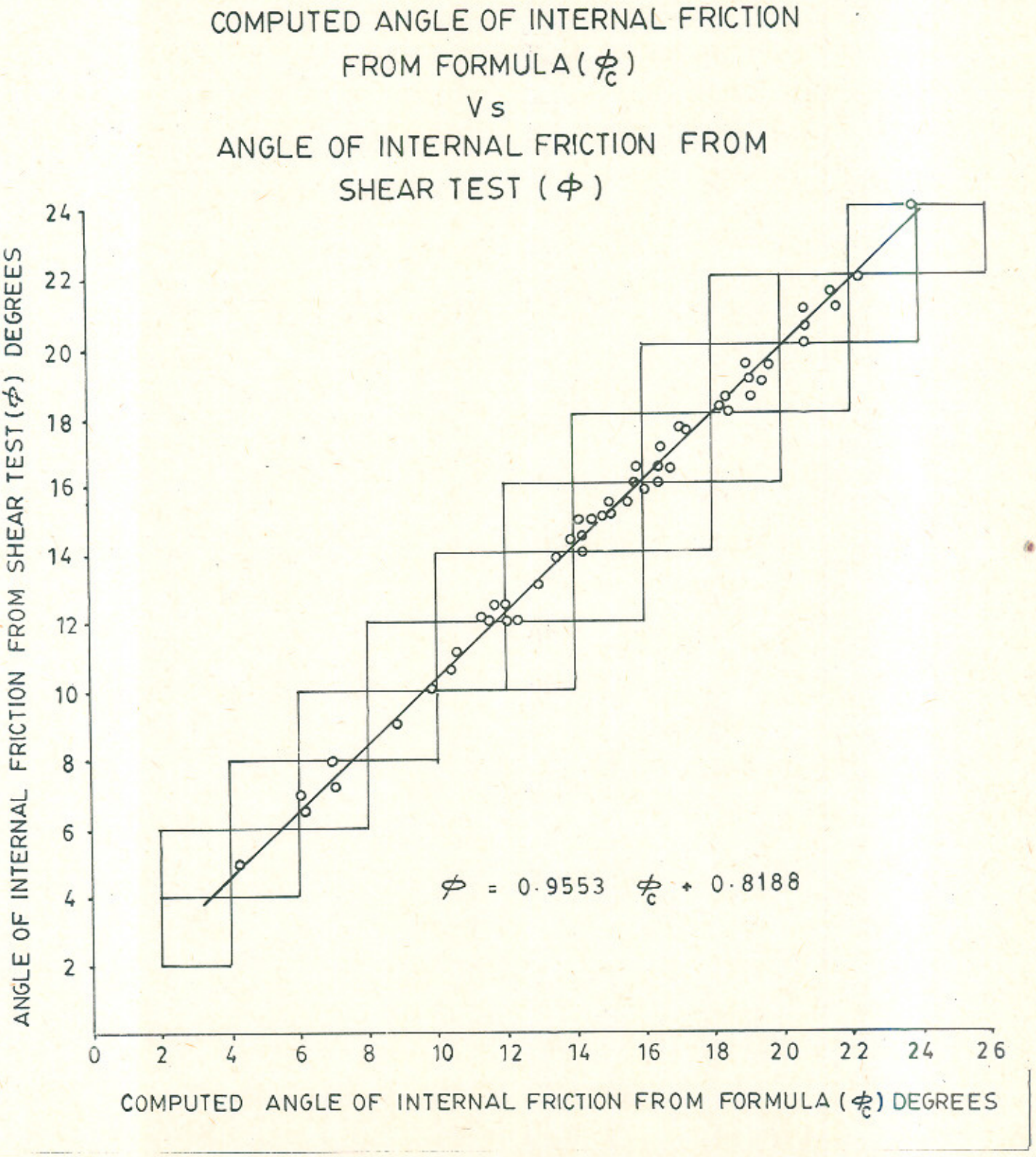


FIG 6


COMPUTED ANGLE OF INTERNAL FRICTION FROM FORMULA (ϕ_c) Vs ANGLE OF INTERNAL FRICTION FROM SHEAR TEST (ϕ)
ANGLE OF INTERNAL FRICTION FROM SHEAR TEST (ϕ) DEGREES
2
4
6
8
10
12
14
16
18
20
22
24
$\phi = 0.9553\ \phi_c + 0.8188$
0
2
4
6
8
10
12
14
16
18
20
22
24
26
COMPUTED ANGLE OF INTERNAL FRICTION FROM FORMULA (ϕ_c) DEGREES

