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The Influence of Saturation and Stratification on the Shearing Properties of Certain Soils

L'Influence de la saturation et de la stratification sur les caractéristiques de cisaillement de certains sols

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SUMMARY

Part A illustrates that the degree of saturation of soils depends not only on the pore water pressure but also on its history and on the solubility of air in soil water. The ultimate stage depends on the stability of the form of air bubbles in the soil. In Part B, the author has determined the angles of internal friction φ at different displacements Δl of soil. The shear strength of a stratified medium is defined by the maximum shear resistance at equal displacement of all layers of soils on the surface of failure. The author proposes to call the shear strength in sand and gravel at the normal stress $\sigma = 0$ the initial shear strength τ_0. In cohesive soils only, it may be called cohesion as is usual.

SOMMAIRE

La partie A démontre que le degré de saturation d'un sol dépend non seulement de la pression de l'eau interstitielle mais aussi de son histoire et de la solubilité de l'air dans l'eau du sol. Le stade dernier dépend de la stabilité de la forme des bulles d'eau dans le sol. La partie B mesure l'angle de frottement interne à différentes valeurs de déplacement Δl du sol. La résistance au cisaillement d'un milieu stratifié se définit par la résistance maximum à un déplacement égal de toutes les couches de sol sur le plan de rupture. L'auteur propose d'appeler la résistance au cisaillement initiale, τ_0, c'est-à-dire la résistance au cisaillement des sols pulvérulents à la contrainte normale $\sigma = 0$. Pour les sols cohérents, celle-ci peut s'appeler cohésion comme d'habitude.

A. RELATIONSHIP BETWEEN THE DEGREE OF SATURATION OF SOILS AND THE PORE PRESSURE, ITS HISTORY, AND THE EFFECT OF SOLUBILITY OF AIR IN WATER

THE MAIN MECHANICAL PROPERTIES of soils such as their permeability, compressibility, and shear strength (namely the pore-pressure coefficients A and B) depend on the degree of saturation of the soils, the value of which is intensively influenced by pore water pressure, its history, and by the effect of the solubility of air in soil water.

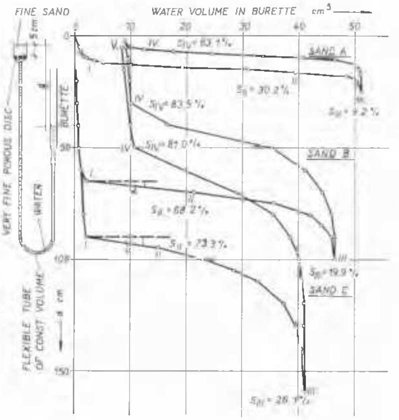

FIG. 1. Relationship between the degree of saturation of sands A, B, and C and the water pressure.

Influence of Pore Water Pressure

The influence of pore water pressure on the degree of saturation of dense sands A, B, and C (Table I) was studied by the experiments illustrated in Fig. 1. The distance d was gradually increased, while the volume of water in the burette was observed.

Until stage I was reached (see Fig. 1) the sand remained practically saturated. The small decrease in degree of saturation, S_r, was due to changes of the form of water meniscus. After surpassing stage I air began to flow into the pores of the sand, when the degree of saturation, S_r decreased to: $S_I = 1 - (\cot \alpha / F.n)$, where α is the inclination angle of the curve I–II near I, F is the area of the horizontal cross-section of the sample, and n is its porosity. The approximate values of S_I are given in Table I.

TABLE I. CHARACTERISTICS OF SAND USED IN TESTS

Sand	Range of size of grains in mm	Porosity n (per cent)			Degree of saturation	
		max	min	during the test	S_I %	S_{IV} %
A	0.5–1.0	46.1	37.0	37.1	29	83
B	0.1–0.2	50.4	36.8	36.8	68	83
C	0.05–0.1	51.7	35.0	35.1	73	81

Stage II was reached when the air that penetrated into the sample reached the base of the sample. It was visually observable, that stage II was reached when the water level in the burette was approximately 5 cm (height of the sample) lower than at the stage I.

While both free and pellicular waters were flowing out of the sample during the period I-II, only pellicular water was leaving it during II–III and the sand was therefore air permeable. During III–IV the water was coming back into the sand. The sample remained air permeable until the water began to divide the pore air into discrete bubbles. This process ended at the stage V, when S_{IV} was reached. The sand was then air impermeable, because during the period IV–V the sand sucked practically no more water. Only small compression of air bubbles due to increase of pore water pressure was observed (Havlíček, 1963, 1964).

Dissolving of Air in Pore Water

The dissolving of air in water is a dynamic process. Therefore the dissolved air diffuses through the soil water from its contact with air of higher air pressure towards contact with the soil and air of lower pressure. Further, it is known that the soil air pressure u_a always is greater by Δu than the pressure of neighbouring water. This difference Δu is due to the surface tension of water and depends on the curvature of the air-water contact.

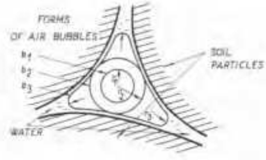

FIG. 2. Forms of air bubbles in soils.

Therefore for a bubble in a given pore the minimum of air pressure is reached, when the bubble has the spherical form touching the neighbouring soil particles (b_2 of Fig. 2). It follows, therefore, for the soil which is in contact with atmospheric air that:

(1) If for a spherical air bubble in a soil $u_a > u_{atm}$ (u_{atm} — the atmospheric pressure), the bubble will dissolve and its u_a will increase till the bubble disappears.

(2) If for a spherical bubble $u_a < u_{atm}$, the dissolved air will separate out of the water to enter the bubble which gets enlarged. Therefore, the air pressure in the bubble must decrease till it attains the form b_3 (Fig. 2).

(3) If for a bubble of the star form (b_3 in Fig. 2), $u_a > u_{atm}$, the air dissolves and its u_a decreases till u_{atm} is reached.

(4) For the same case when $u_a < u_{atm}$, u_a will increase till u_{atm} is reached.

Spherical air bubbles, therefore, are not stable in soils, but are always either decreasing or increasing. The bubble of the form b_3 in Fig. 2 is stable when its $u_a = u_{atm}$.

The above results were partly proved by experiments as shown in Fig. 3. The samples of unsaturated fine sand

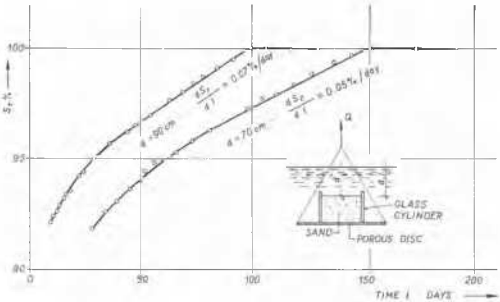

FIG. 3. Dissolving of air below the water level.

FIG. 4. Sample below the water level. Air bubbles, not yet dissolved, are visible in the middle part.

were retained and weighed below the water level. Both the weight and the average S_r of the samples increased until their S_r reached 100 per cent. The observed process was the effect of dissolving of air because: (*a*) the process proceeded faster when the sample was kept deeper below the water level; (*b*) the saturation of the sand proceeded visibly from sand surfaces (Fig. 4).

Degree of Saturation

The degree of saturation that occurs after a considerable lowering or rising of groundwater level is approximately as shown in Fig. 1, the distance d being replaced by the height h above the ground water level (or h , h_{II}, etc. for stages I, II, etc., respectively). S_r ranges from S_{min} to S_{max} according to the previous movement (raising or lowering) of ground water level. Then the process due to the solubility of air in soil water commences. Below the level which is $h_0 = \Delta u_{min}/\gamma_w$ (γ_w being the unit weight of water) above groundwater level, the air pressure is always greater than the atmospheric pressure. Therefore all bubbles that are below this level must dissolve after some time and the soil beneath become fully saturated. Immediately above the h_0 level the minimum of S_r occurs only in cases where all pores are engaged by the air bubbles of the form b_2 (Fig. 2). Above this level the stable air bubbles must have the form b_3 of Fig. 2. They are discrete and therefore the soil is air impermeable. This layer of air-impermeable soil reaches a level from h_{IV} to h_I depending on the direction of the last movement of the ground water level.

Immediately above the air-impermeable soil the degree of saturation may range between S_{IV} and S_I depending on whether the level is raised by the enclosing of the air by water or by the breakdown of the capillary water meniscus. For higher levels the soil is air permeable and its S_r is less than S_{IV}.

Long-duration tests to further support these statements are in progress. They are expected to reveal also the range of validity of the above mentioned statements for clays.

B. SHEAR STRENGTH OF STRATIFIED SOIL

In the solution of stability of earth masses consisting of layers of various kinds of soils, the shear strength determined by the laboratory tests is usually considered without taking into account the value of displacement Δl at which the shear failure of soil occurs. The value of displacement Δl on surface of failure needed for mobilizing the full shear strength varies with different kinds of soils.

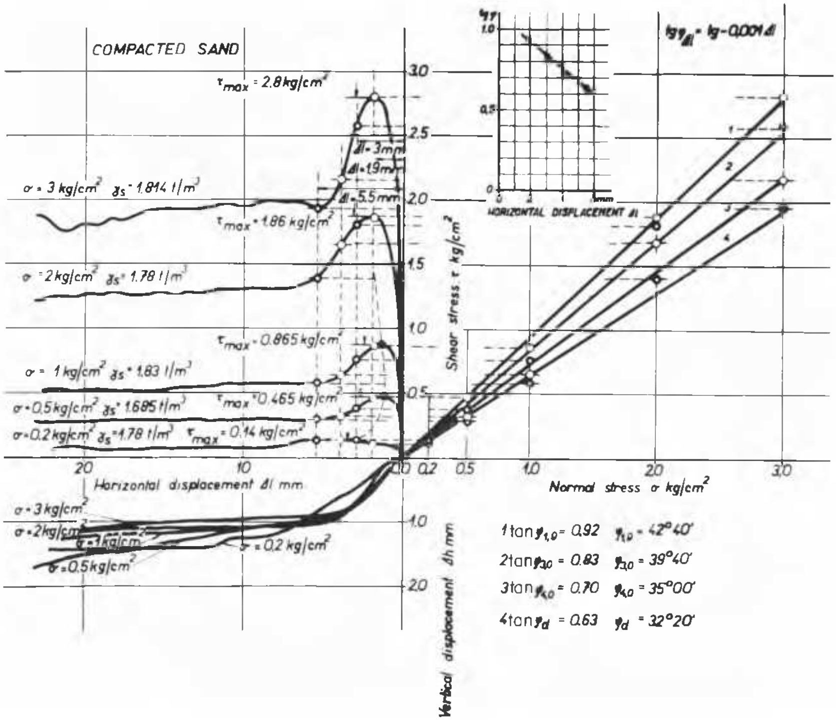

FIG. 5. Shear resistance of compacted sand.

In the case of the displacement of certain magnitude, one layer fails by shear whereas the other layer, consisting of a different kind of soil, fails at a greater displacement Δl. After slip of the second layer, the resistance of the first layer goes down because its shear resistance has been surpassed.

Tests with Sand

Tests with sand were carried out in the direct shear box at the maximum density (Fig. 5) and at the minimum density (Fig. 6). The composition of the sand is given by the following: $d_{10} = 0.22$ mm, $d_{30} = 0.28$ mm, $d_{50} = 0.35$ mm, $d_{70} = 0.45$ mm, $d_{90} = 0.8$ mm, $d_{100} = 4.0$ mm. The sand was sheared at a constant rate of 0.2 mm/min. and the shear resistance was measured by means of a dynamometer. The value of shear strength of the sand at maximum density was reached at the displacement of $\Delta l = 1.9$ mm. In the case of the loose sand, the shear strength was attained at the displacement $\Delta l = 3.5$ mm. The tops of stress-strain curves are always approximately situated at the same vertical line.

Angle of Statical and Dynamic Friction

Considering the tops of curves, we may determine the angle of internal friction φ which may be called the angle of static friction in terms of physics.

After the shear strength of the soil has been attained, its shear resistance reduces and settles at a certain minimum value. Considering this minimum shear resistance of the soil, we can again determine the relationship between the stresses σ and $\tau_{f\min}$. We obtain again a straight line forming with the horizontal axis an angle which may be called the angle of dynamic friction of soil φ_d.

Degree of Loosening

The ratio between the coefficients of static and dynamic friction may be called the degree of loosening $\lambda = \tan \varphi / \tan \varphi_d$. In the case of loose sand, we have usually $\tan \varphi = \tan \varphi_d$ and the degree of loosening $\lambda = 1$ as measured in the loose sand at the normal stress $\sigma = 0.2$ kg/sq.cm.

At greater values of normal stress σ the sand has been compacted and therefore, the degree of loosening was $\lambda = 0.69/0.6 = 1.15$. In the compacted sand, the degree of loosening $\lambda = 0.92/0.63 = 1.46$. The angle of dynamic friction decreases with increasing loosening. The angle of friction φ and the coefficient of friction $\tan \varphi$ may be determined for various values of displacement Δl (Fig. 5). The value of displacement in mm is given by the index of the angle φ. For example, the value of $\tan \varphi_4 = 0.7$ indicates that this coefficient of friction has been measured at the displacement of $\Delta l = 4$ mm. The angle of dynamic friction has been measured at the displacement $\Delta l = 5.5$ mm.

The author has determined the relationship between the coefficient of friction $\tan \varphi$ and the displacement Δl after surpassing the shear strength τ_f. This relation is given by the following equation: $\tan \varphi_{\Delta l} = \tan \varphi - \psi . \Delta l$. For compacted sand, the coefficient $\psi = 0.081$ mm^{-1} and for loose sand $\psi \doteq 0$.

For the exact solution of a stability problem we need the value of displacement, at which the angle of internal friction

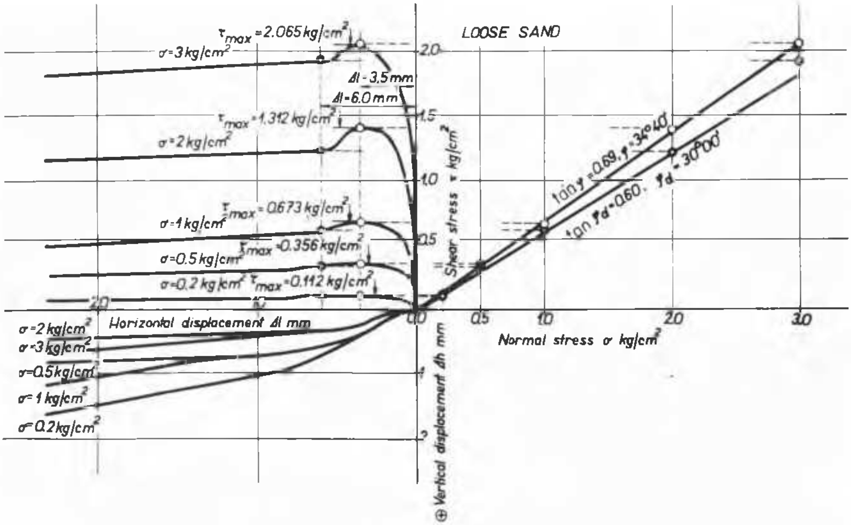

FIG. 6. Shear resistance of loose sand.

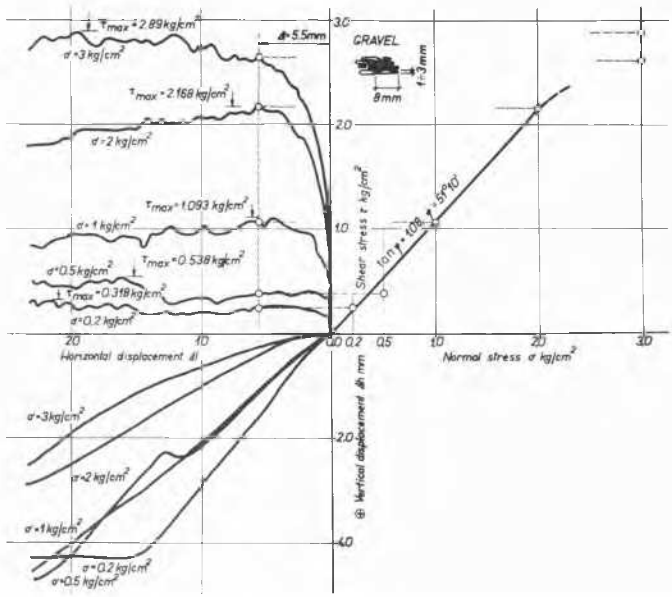

FIG. 7. Shear resistance of flat grained gravel with orientation in the horizontal direction.

has been measured. In the case of various values of displacement Δl, the coefficient of friction $\tan \varphi$ will be determined for different displacements.

Shear Resistance of Stratified Soils

The shear strength of a stratified medium is solved in such a way, that we consider the displacement Δl of the layer, which exhibits the greatest resistance, and for this displacement the values of shear resistance of the other soils should be determined. A little greater displacement Δl is then considered and the respective shear resistance is again determined until the maximum shear resistance at the same displacement Δl of all layers is obtained.

When the clay contained a layer of compacted sand, in some cases the coefficient of dynamic friction of sand had to be taken into account.

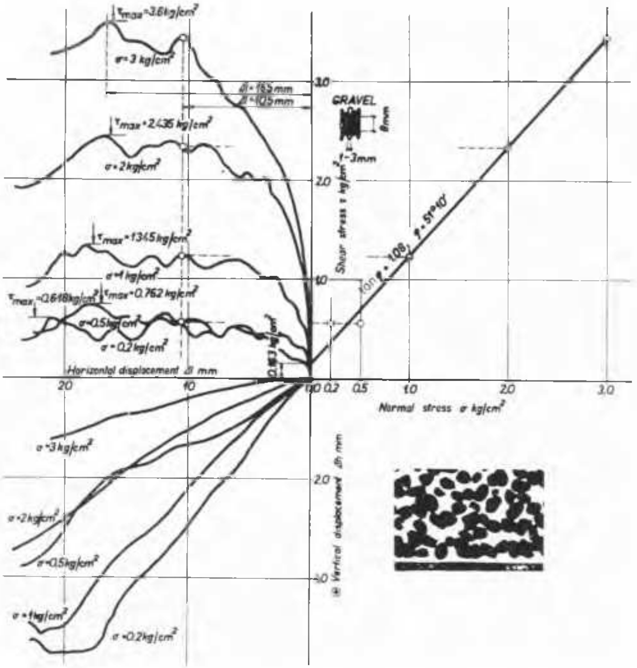

FIG. 8. Shear resistance of flat grained gravel with orientation in the vertical direction.

Tests with Gravel

In the case of compacted gravel, the straight line indicating the relationship between the normal stress σ and the shear strength τ_f does not always pass through the origin of coordinates (point 0—Fig. 8). In the direct shear box, gravel with flat grains was examined. The grains had a thickness of 1 to 3.0 mm and a length of 4 to 8 mm. The grains of gravel were situated in the horizontal direction (Fig. 7) and the measured value of the coefficient of internal friction was $\tan \varphi = \tan \varphi_d = 1.08$. The degree of loosening was $\lambda = 1$ and the horizontal displacement $\Delta l = 5.5$ mm.

Initial Shear Strength of Gravel

The same gravel was examined, with the grains situated vertically in the direct shear box as shown on Fig. 8. The measured coefficient of friction was $\tan \varphi = 1.08 = \tan \varphi_d$, the initial shear strength τ_0 0.163 kg/sq.cm. and the horizontal displacement $\Delta l = 10.5$ mm. Also in this case, we may conclude that the degree of loosening $\lambda = 1$. In both cases, the gravel was freely piled up and not compacted. During the shearing, a slip zone developed, the grains moved and rotated, and the surface of the sample rose by 4.6 mm at $\sigma = 1.0$ kg/sq.cm. and by only 0.6 mm at $\sigma = 3$ kg/sq.cm.

The Coulomb equation may be written in the following general form: $\tau_f = \tau_0 + \sigma.\tan \varphi$, where τ_0 denotes the initial shear strength which, in the case of clay only, is called the cohesion c. In sand, the initial shear strength is caused by resistances arising when the grains move up and rotate at the rise of slip zone. It is interesting that in the case of vertical arrangement of grains after overcoming the initial shear strength, the value of the angle of friction was the same as with the horizontal orientation of the flat grains. The initial shear strength has been measured by many authors and depends on the degree of density of sand and of the form of grains.

When bricks are dumped in a pile, the pile attains an angle of repose φ. When, however, they are arranged one over the other, $\varphi = 90°$ and the height depends on the mode of arrangement, i.e., on whether they are interlocked or not. This is because of the arrangement of bricks and is not due to a change in frictional property of bricks themselves.

Conclusion

In the case of a stratified medium consisting of layers of different kinds of soils in which the shear strength is mobilized at different values of displacement on a slip surface, the maximum shear resistance at equal displacement of all layers should be determined. Therefore, we have to define the degree of loosening which indicates the decrease of shear resistance in relation to the shear strength τ_f. Further, the angle of internal friction for different values of displacement of soil is needed. In the case of the compacted sand and gravel the straight line representing the limiting shear stress does not pass through the origin and gives on the shear stress axis an abscissa which may be called the initial shear strength τ_0. For cohesive soils, the abscissa represents the cohesion c.

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