An Analysis of the Volume Change, Distortional Deformation and Induced Pore Pressure of Soils under Triaxial Loading

Une analyse du changement de volume, de la déformation de cisaillement et de la pression interstitielle des sols soumis à une charge triaxiale

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Summary

Triaxial tests carried out by the author and others have shown that the volume change of soils is influenced by both normal stress and shear stress.

Under shear stress, sands generally expand in volume and this has so far been described as swelling. Conversely, silt and clay often shrink. The pore-pressure induced under rapid loading is closely related to volume change, thus considerably influencing the stress-strain relationship and the ultimate strength of a soil.

After analysis of several triaxial test results and allowing for the effect of shear stress on the change in volume of soils, the author has revised his theory of soil mechanics presented to the London Conference in 1957. This theory has been applied to and compared with some typical laboratory test results under triaxial loading, with analyses of volume change and deformation observed under slow and rapid loading, as well as the induced pore-pressure under rapid loading.

Sommaire

Les résultats des essais de compression triaxiale effectués dans le laboratoire de l'auteur et ailleurs ont montré que le changement de volume des sols est affecté non seulement par la contrainte moyenne normale mais aussi par l'effort de cisaillement. D'ordinaire les sols sablonneux sous l'application de l'effort de cisaillement font preuve d'une expansion de volume qui jusqu'ici s'appelle dilatation. D'autre part les sols limoneux et argileux montrent très souvent une contraction de volume. La pression de l'eau interstitielle induite au cours de la compression triaxiale rapide s'associe étroitement au changement de volume, ayant ainsi une influence considérable sur la relation entre contrainte et déformation ainsi que sur la résistance à la limite de rupture de sols.

Après analyse des résultats d'un nombre d'essais de compression triaxiale et après avoir considéré l'effet de l'effort de cisaillement sur le changement de volume des sols, l'auteur a revisé sa théorie de la mécanique des sols présentée au Congrès de Londres, 1957. La théorie revisée a été appliquée avec beaucoup de succès pour analyser le changement de volume et la déformation de cisaillement observés dans les compressions triaxiales lente et rapide ainsi que la pression de l'eau interstitielle induite pendant la compression triaxiale rapide.

1. Volume change of soils under shear stress

In soil mechanics, if stress changes are very small, the relationship between stress and strain according to the theory of elasticity is :

$$\Delta\varepsilon_1 = \frac{1}{E}\Delta\sigma_1 - \frac{\mu}{E}(\Delta\sigma_2 + \Delta\sigma_3),$$

$$\Delta\varepsilon_2 = \frac{1}{E}\Delta\sigma_2 - \frac{\mu}{E}(\Delta\sigma_1 + \Delta\sigma_3),$$

$$\Delta\varepsilon_3 = \frac{1}{E}\Delta\sigma_3 - \frac{\mu}{E}(\Delta\sigma_1 + \Delta\sigma_2)$$

The author's theory of soil mechanics was developed on this basis and presented to the London Conference (1957).

If we denote the normal and shear stresses on an octahedral plane respectively by σ_m and τ_m ;

$$\sigma_m = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3),$$

$$\tau_m = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2{}^2 - \sigma_3{}^2)^2 + (\sigma_1 - \sigma_3)^2},$$

it can readily be shown that the volume change is proportional to the normal stress and independent of shear stress. This may lead to the conclusion that the triaxial test result must give a continuous curve of volume change plotted against normal stress with a common tangent at the point of entry to axial compression process from pure compression. Analyses of several triaxial test results have shown that the tangent of the curve changes suddenly at this point, as illustrated in Fig. 1. The change is usually negative or expansive for sand, while very often it is positive or contractive for silt and clay. The shear stress induced in axial compression will cause volume change in a soil, thus leading to possible rejection of the theory of elasticity in soil mechanics even if the change of stress is very small. Volume change is one of the most important properties of soil, particularly in the case of rapid loading, when it is strictly correlated with the induced pore-pressure, having a considerable effect on the stress-strain relationship and ultimate strength of a soil.

After allowing for shear stress on the volume change of a soil, the author revised his theory (1957) and applied it to the analysis of triaxial test results. The pore pressure equation proposed by A.W. SKEMPTON (1954) has been generalized.

2. Revised expressions of strains, volume change and energy

Three principal stresses are the same in expression as in the former theory.

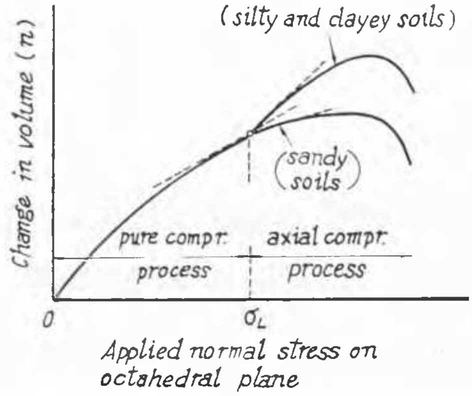

Fig. 1 Volume change in relation to the average normal stress in pure compression and axial compression during the triaxial test.
Changements de volume en fonction de la contrainte moyenne normale au cours de deux opérations de compression latérale et compression axiale dans l'essai triaxial.

$$\sigma_1 = \sigma_m + \sqrt{2}\, j_1 \tau_m$$
$$\sigma_2 = \sigma_m + \sqrt{2}\, j_2 \tau_m$$
$$\sigma_3 = \sigma_m + \sqrt{2}\, j_3 \tau_m$$

where :

$$j_1 = \cos\theta_m,\ j_2 = \cos\left(\frac{2}{3}\pi - \theta_m\right),\ j_3 = \cos\left(\frac{2}{3}\pi + \theta_m\right),$$

in which σ_m, τ_m and θ_m denote normal stress, shear stress and a directional angle of the shearing stress on an octahedral plane respectively (Fig. 2). Three principal strains may take, by adding the (Fig. 2) last term on the right hand side, the revised forms of

$$\frac{d\varepsilon_1}{1-\varepsilon_1} = \frac{1}{3V} d\sigma_m + \frac{\sqrt{2}}{3U} d(j_1\tau_m) + \frac{K}{3U} d\tau_m$$
$$\frac{d\varepsilon_2}{1-\varepsilon_2} = \frac{1}{3V} d\sigma_m + \frac{\sqrt{2}}{3U} d(j_2\tau_m) + \frac{K}{3U} d\tau_m \quad \ldots. (2)$$
$$\frac{d\varepsilon_3}{1-\varepsilon_3} = \frac{1}{3V} d\sigma_m + \frac{\sqrt{2}}{3U} d(j_3\tau_m) + \frac{K}{3U} d\tau_m$$

in which V and U denote cœfficients of volume change and distortional deformation respectively. The change in volume (n) and the total distortion (m) may be given by

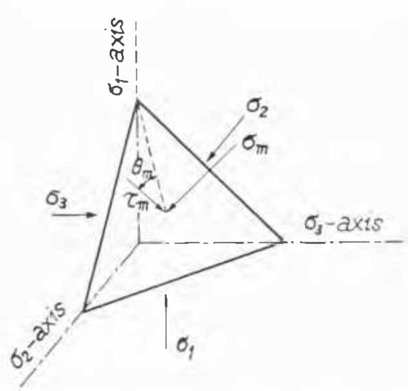

Fig. 2 Octahedral stresses.
Contraintes octaédriques.

$$\frac{dn}{1-n} = \frac{1}{V} d\sigma_m + K \frac{1}{U} d\tau_m$$
$$\frac{dm}{1-m} = \frac{1}{U} d\tau_m \qquad (3)$$

The second term on the right hand side of the first equation gives the change in volume under the action of shear stress, which is assumed to be proportional to the total distortion due to shear stress, K being a coefficient of proportionality. Increment of the energy (A) is

$$dA = \sigma_1 \frac{d\varepsilon_1}{1-\varepsilon_1} + \sigma_2 \frac{d\varepsilon_2}{1-\varepsilon_2} + \sigma_3 \frac{d\varepsilon_3}{1-\varepsilon_3}$$
$$= \frac{\sigma_m}{V} d\sigma_m + \frac{\tau_m}{U} d\tau_m + \frac{K\sigma_m}{U} d\tau_m \qquad (4)$$

which may be divided into two parts, dA_n associated with the change in volume and dA_m with distortion;

$$dA_n = \frac{\sigma_m}{V} d\sigma_m + \frac{K\sigma_m}{U} d\tau_m$$
$$dA_m = \frac{\tau_m}{U} d\tau_m \qquad (5)$$

Comparing (4) with the total differential of the energy:

$$dA = \frac{\partial A}{\partial \sigma_m} d\sigma_m + \frac{\partial A}{\partial \tau_m} d\tau_m$$

we obtain

$$\frac{\sigma_m}{V} = \frac{\partial A}{\partial \sigma_m}, \qquad \frac{\tau_m + K\sigma_m}{U} = \frac{\partial A}{\partial \tau_m} \qquad (6)$$

3. Coefficients of volume change and distortion expressed as functions of stresses

Assuming that (a) the initial conditions before stressing are $\sigma_m = \sigma_0$, $\tau_m = 0$, $V = V_0$, $U = U_0 = \nu^2 V_0$; (b) under pure compression, V is in proportion to A_n; (c) under pure shear, U is in proportion to $A_n - A_m/\lambda^2$, where λ is a constant, and (d) under the combined stress of compression and shear, the amount of energy is an exclusive function of the existing stress state irrespective of the stress path in the past, we can derive the following equation of the energy (A).

$$A = \frac{\sigma_0}{V_0}\sigma_m \left\{ (1+\lambda^2) - \lambda^2 \frac{\cos\theta}{\cos\theta_0} - (1+\lambda^2)\tan\theta_0\,(\theta - \theta_0) \right\} \qquad (7)$$

in which, in order to simplify the expression, we put

$$R_0 = \sqrt{(\lambda\nu)^2 + (k\lambda^2)^2},\ \sin\theta_0 = -\frac{K\lambda^2}{R_0}$$
$$\cos\theta_0 = \frac{\lambda\nu}{R_0},\ \tan\theta_0 = -\frac{K\lambda}{\nu} \quad \ldots. (8)$$
$$\sin\theta = \frac{1}{R_0}\frac{\tau_m}{\sigma_m} + \sin\theta_0$$

By putting partial differentials of (7) by σ_m and τ_m into (6), we arrive at the following expressions for the two coefficients as functions of stresses.

$$\frac{1}{V} = \frac{1}{V_0} \frac{\sigma_0}{\sigma_m \cos\theta} \left\{ G(\theta) + \tan\theta_0(\sin\theta - \sin\theta_0) \right\}$$

$$\frac{1}{U} = \frac{1}{U_0} \frac{\sigma_0 \cos\theta_0}{\sigma_m \cos\theta} \quad \cdots\cdots \quad (9)$$

in which

$$G(\theta) = \cos\theta_0 + (1+\lambda^2)(\cos\theta - \cos\theta_0) + 2\lambda^2 \tan\theta_0 (\sin\theta - \sin\theta_0) - (1+\lambda^2)\tan\theta_0(\theta-\theta_0)\cos\theta.$$

In the case of pure compression when $\tau_m = 0$ or $\theta = \theta_0$, the first equation becomes

$$\frac{1}{V} = \frac{1}{V_0} \frac{\sigma_0}{\sigma_m} \quad (10)$$

The change in volume and the distortion are, from (3),

$$\frac{dn}{1-n} = -d\log_e(1-n) = \frac{\sigma_0}{V_0}\left\{\frac{G(\theta)}{H(\theta)} - \tan\theta_0\right\} d\theta$$

$$\frac{dm}{1-m} = -d\log_e(1-m) = \frac{\sigma_0}{U_0}\frac{d\tau_m}{d\sigma_m}\frac{\cos\theta_0}{H(\theta)} d\theta \quad \cdots\cdots \quad (11)$$

in which

$$H(\theta) = \frac{1}{R_0}\frac{d\tau_m}{d\sigma_m} - \sin\theta + \sin\theta_0$$

The three principal strains in (2) transform into

$$\frac{d\varepsilon_1}{1-\varepsilon_1} = \frac{1}{3}\frac{dn}{1-n} + \frac{\sqrt{2}\,j_1}{3}\frac{dm}{1-m} + \frac{\sqrt{2}\,\tau_m}{3U} dj_1$$

$$\frac{d\varepsilon_2}{1-\varepsilon_2} = \frac{1}{3}\frac{dn}{1-n} + \frac{\sqrt{2}\,j_2}{3}\frac{dm}{1-m} + \frac{\sqrt{2}\,\tau_m}{3U} dj_2$$

$$\frac{d\varepsilon_3}{1-\varepsilon_3} = \frac{1}{3}\frac{dn}{1-n} + \frac{\sqrt{2}\,j_3}{3}\frac{dm}{1-m} + \frac{\sqrt{2}\,\tau_m}{3U} dj_3$$

The ratio of τ_m to σ_m becomes a maximum when $\theta = \frac{\pi}{2}$, which may be considered a condition of failure. If the maximum value be $\tan\psi$, we have

$$\tan\psi = R_0(1-\sin\theta_0) = \sqrt{(\lambda\nu)^2 + (K\lambda^2)^2} + K\lambda^2.$$

Therefore

$$\lambda^2 = \frac{\tan^2\psi}{\nu^2 + 2K\tan\psi}.$$

4. An application of the theory to the triaxial test under slow loading

Each triaxial test consists of pure compression combined with axial compression. The effect of pore-pressure will become negligible or will even completely disappear under a condition of slow loading.

1. *Pure compression*

During pure compression an applied pressure (σ) is increased slowly to a certain value (σ_L). The octahedral stresses are

$$\sigma_m = \sigma_0 + \sigma, \quad \tau_m = 0.$$

Integration of the first equation of (3) by introducing (10), gives

$$-\log_e(1-n) = \frac{\sigma_0}{V_0}\log_e\left(\frac{\sigma_0+\sigma}{\sigma}\right) \doteqdot n. \quad (12)$$

The final volume change (n_L) under the pressure (σ_L) is

$$-\log_e(1-n_L) = \frac{\sigma_0}{V_0}\log_e\left(\frac{\sigma_0+\sigma_L}{\sigma_L}\right) \doteqdot n_L.$$

2. *Axial compression*

When the axially applied (deviator) stress is expressed by σ_v, the octahedral stresses are

$$\sigma_m = \sigma_0 + \sigma_L + \frac{1}{3}\sigma_v = (\sigma_0+\sigma_L)\frac{H_0}{H(\theta)}.$$

$$\tau_m = \frac{\sqrt{2}}{3}\sigma_v = \sqrt{2}\,(\sigma_0+\sigma_L)\frac{\sin\theta - \sin\theta_0}{H(\theta)}$$

in which

$$H_0 = \frac{\sqrt{2}}{R_0} \quad \text{because} \quad \frac{d\tau_m}{d\sigma_m} = \sqrt{2}.$$

Integrations of (11) give

$$-\log_e(1-n) = \frac{\sigma_0}{V_0}\Big[(1+\lambda^2)F_1(\theta) - \lambda^2(\cos\theta_0) - 2H_0\tan\theta_0)F_2(\theta) - (1+2\lambda^2)\tan\theta_0(\theta-\theta_0) - (1+\lambda^2)\tan\theta_0 F_3(\theta)\Big], \quad (13)$$

$$-\log_e(1-m) = \frac{\sigma_0}{U_0}\sqrt{2}\cos\theta_0 F_2(\theta)$$

in which

$$F_1(\theta) = \int\frac{\cos\theta}{H(\theta)}d\theta = \log_e\left\{\frac{H_0}{H(\theta)}\right\}$$

$$F_2(\theta) = \int\frac{1}{H(\theta)}d\theta = \frac{1}{\sqrt{(H_0+\sin\theta_0)^2-1}}\left[\sin^{-1}\frac{1-(H_0+\sin\theta_0)\sin\theta_0}{H_0} - \sin^{-1}\frac{1-(H_0+\sin\theta_0)\sin\theta}{H(\theta)}\right],$$

$$F_3(\theta) = \int\frac{(\theta-\theta_0)\cos\theta}{H(\theta)}d\theta = -(\theta-\theta_0)\log_e H(\theta) + \int\log_e H(\theta)d\theta.$$

5. Analyses of some triaxial test results under slow loading

1. *Penman's test on a saturated silt.*

After analysing the results of the triaxial test made by A.D.M. Penman (1953) under slow loading, the constants in each test have been assumed, as tabulated in Table-1, and plotted against the initial void ratio before testing (e_i) in Figs. 3 and 4.

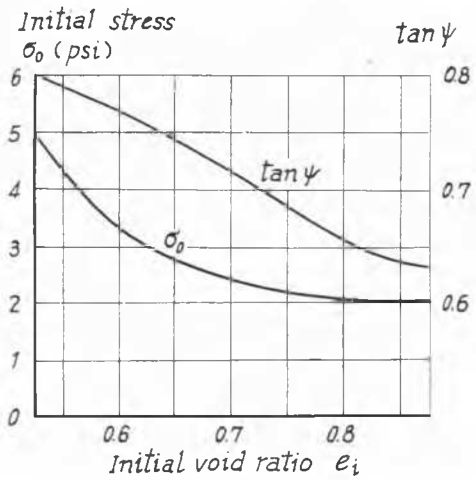

Fig. 3 Relationship between soil constants (σ_0 and $\tan\psi$) and initial void ratio (e_i) in the slow tests by Penman.

Relation entre les constantes des sols (σ_0 et $\tan\psi$) et l'indice de vide initial (e_i) dans l'essai de compression triaxial lent (Penman).

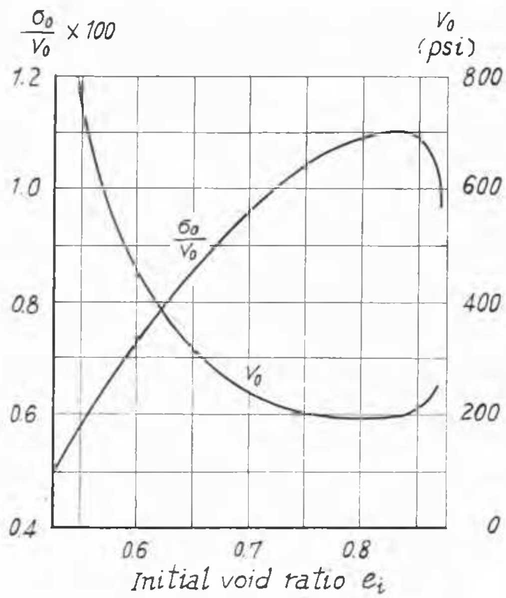

Fig. 4 Relation between the constants of soil $\left(\frac{\sigma_0}{V_0}\right.$ and $\left.V_0\right)$ and initial void ratio (e_i) in the slow tests by Penman.

Relation entre les constantes des sols $\left(\frac{\sigma_0}{V_0}\right.$ et $\left.V_0\right)$ et l'indice de vide initial (e_i) dans l'essai de compression triaxiale lent (Penman).

Test No. 10-29 has been analysed as illustrated in Fig. 5, which shows that the theoretical curve may represent the experimental curve up to about half of the maximum axial stress. Gradual increase in deviation between the two curves on the latter half may probably be due to a slipping and overriding motion on the contact surface of soil particles, which has not yet been taken into account in the present theory.

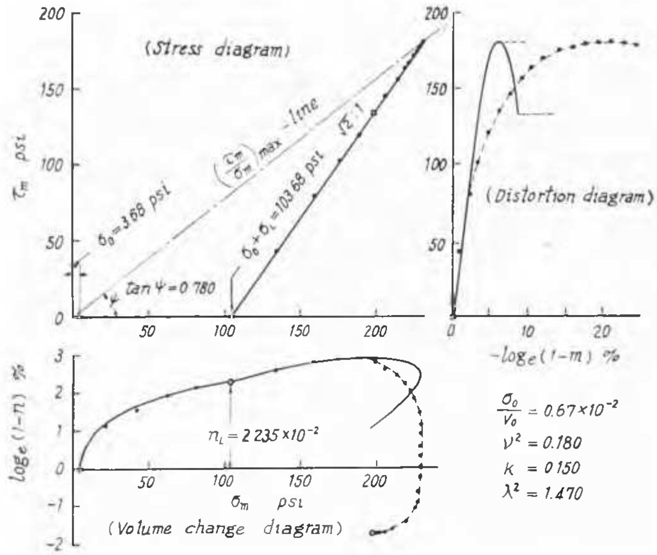

Fig. 5 Analysis of test No. 10-29 (Penman).

Les diagrammes de l'essai de compression triaxiale lent sur un limon (Penman) : test No. 10-29).

2. *The author's test on a sandy soil with gravel*

A typical example of analysis on the slow test results obtained in the author's laboratory is illustrated in Fig. 6.

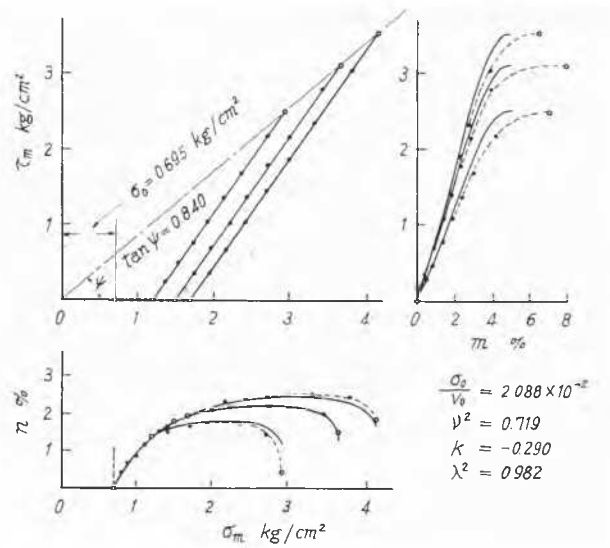

Fig. 6 A typical analysis of slow test results on a sandy soil with gravel (Hoshino).

Diagrammes types de l'essai de compression triaxiale lent sur un sable avec gravier (Hoshino).

6. Induced pore-pressure under rapid loading

If the compressibility of soil particles immersed in water may be disregarded, the solubility of void air in water and the temperature change affecting test results, the change in volume of void air must be identical with the change in volume of soils under rapid loading. The volume change of a soil or the

Table 1

Constants in the slow test by PENMAN
(Constantes de l'essai de compression triaxiale lent PENMAN)

Test No.	*Initial void ratio* e_i	*Initial stress* σ_0 (psi)	$\frac{\sigma_0}{V_0}$	tan ψ
12- 8	0·867	2·00	0·0098	0·630
10-27	0·831	2·04	0·0110	0·640
10- 1	0·807	2·07	0·0109	0·651
10-22	0·786	2·10	0·0108	0·662
10-20	0·762	2·13	0·0106	0·677
10-21	0·747	2·18	0·0104	0·686
10- 6	0·741	2·20	0·0103	0·689
10-19	0·707	2·30	0·0097	0·710
10-23	0·674	2·58	0·0091	0 729
10-17	0·653	2·68	0·0087	0·740
10-29	0·577	3·68	0·0067	0·780
12- 2	0·554	4·22	0·0059	0·788

void air (n) may be correlated with the pore-pressure (u) in accordance with Boyle's Law.

$$(p_0 + u)\ (N_a - n) = p_0 N_a \text{ or } n = \frac{u}{p_0 + u} N_a,$$

in which p_0 = initial value of the pore pressure, generally the same as the atmospheric pressure before testing, and N_a = initial volume of the void air per unit volume of the soil. As differentiation gives

$$dn = \frac{N_a - n}{p_0 + u} du,$$

we have, after some transformation,

$$\frac{dn}{1 - n} = \left\{ \frac{1}{p_0 + u} - \frac{1 - Na}{p_0 + (1 - N_a) u} \right\} du.$$

Equating this with the first equation of (3) and considering that the volume change of soils is brought about by the effective normal stress, that is, the applied normal stress less the pore pressure (σ_m — u), we obtain

$$\left\{ \frac{1}{p_0 + u} - \frac{1 - Na}{p_0 + (1 - N_a) u} \right\} du = \frac{1}{V} (d\sigma_m - du) + \frac{K}{U} d\,\tau_m. \tag{14}$$

Introducing the expressions in (8) and (9) after putting (σ_m — u) in place of σ_m, we derive the following differential equation correlating the induced pore pressure to the applied stress.

$$\left\{ \frac{1}{p_0 + u} - \frac{1 - Na}{p_0 + (1 - N_a) u} \right\} du = \frac{\sigma_0}{V_0} \frac{G(\theta)}{\cos\theta} \frac{d(\sigma_m - u)}{\sigma_m - u} - \frac{\sigma_0}{V_0} \tan\theta_0\, d\,\theta. \tag{15}$$

Integration of this equation is possible if a stress path is specified.

Equation (14) may be transformed into

$$du = \frac{d\,\sigma_m + K \dfrac{V}{U} d\,\tau_m}{1 + V\left\{ \dfrac{1}{p_0 + u} - \dfrac{1 - Na}{p_0 + (1 - N_a) u} \right\}} \quad .... \quad (16)$$

This is a generalized expression of the pore pressure equation proposed by Skempton. Introducing into the equation the conditions before testing;

$$V = V_0, \qquad U = U_0 = \nu^2 V_0, \qquad u = 0,$$

and the stresses in the triaxial test ($\Delta\sigma_2 = \Delta\sigma_3$);

$$\Delta\sigma_m = \frac{1}{3}(\Delta\sigma_1 + 2\Delta\sigma_3), \quad \Delta\tau_m = \frac{\sqrt{2}}{3}(\Delta\sigma_1 - \Delta\sigma_3),$$

the initial value of the induced pore pressure may be given by

$$\Delta u = \frac{1}{1 + \dfrac{Na\,Vo}{Po}} \left\{ \frac{1}{3}(\Delta\sigma_1 + 2\Delta\sigma_3) + \frac{\sqrt{2}\,K}{3\nu^2}(\Delta\sigma_1 - \Delta\sigma_3) \right\}$$

Comparing this with the last equation in SKEMPTON's paper (1954) :

$$\Delta u = B \left\{ \frac{1}{3}(\Delta\sigma_1 + 2\Delta\sigma_3) + \frac{3A - 1}{3}(\Delta\sigma_1 - \Delta\sigma_3) \right\},$$

we obtain :

$$B = \frac{1}{1 + \dfrac{Na\,Vo}{P_0}}, \quad A = \frac{1}{3} + \frac{\sqrt{2}\,K}{3\nu^2}.$$

7. An application of the theory to the triaxial test under quick loading condition

1. *Pure compression*

Integration of (15) under the stress condition of $\theta = \theta_0$ gives

$$\frac{p_0 + u}{p_0 + (1 - N_a) u} = \left(\frac{\sigma_0 + \sigma_1 - u}{\sigma_0} \right)^{\frac{\sigma_0}{V_0}}$$

$$\text{or } \sigma = u + \sigma_0 \left\{ \left(1 - \frac{N_a u}{po + u} \right)^{-\frac{V_0}{\sigma_0}} - 1 \right\}. \quad .... \quad (17)$$

which makes it possible to calculate the pore pressure (u) induced under the applied pressure (σ).

2. *Axial compression*

Introducing the stress conditions :

$$\sigma_m = \sigma_0 + \sigma_L + \frac{1}{3}\sigma_v$$

$$\tau_m = \frac{\sqrt{2}}{3}\sigma_v = \sqrt{2}(\sigma_m - u) - \sqrt{2}(\sigma_0 + \sigma_L - u)$$
$$= R_0 (\sin\theta - \sin\theta_0)(\sigma_m - u).$$

into (15), we obtain the following differential equation,

Table 2. Constants in the rapid test by Wagner

Constantes de l'essai de compression triaxiale rapide (Wagner)

Test No.	Volume of void air Na	Initial stress σ_0 (psi)	Constants				Soil type
			$\frac{V_0}{\sigma_0}$	ν^2	k	λ^2	
1-T	0·0260	5·0	48	0·875	— 0·37	1·775	Sand, well graded.
2-T	0·0623	12·0	110	0·255	0·04	1·707	
3-T	0·0733	3·0	135	0·244	0·09	1·418	
4-T	0·0813	4·0	125	0·248	0·09	1·369	
5-T	0·0927	5·0	120	0·275	0·10	1·212	
6-T	0·1149	6·0	144	0·313	0·11	1·620	
7-T	0·0530	9·0	50	0·220	0·12	1·374	Sand, with clay binder (I)
8-T	0·0380	11·7	12,5	0·200	— 0·17	2·459	
9-T	0·0480	2·8	64	0·098	0·05	3·898	
10-T	0·0680	3·5	110	0·125	0·20	1·547	
11-T	0·1068	5·0	138	0·113	0·20	1·291	
12-T	0·0500	6·7	55	0·346	— 0·07	2·458	Sand, with clay binder (II)
13-T	0·0580	9·0	43	0·167	0·12	1·205	Clay, lean
14-T	0·0880	4·0	47	0·192	0·14	0·997	
15-T	0·0680	11·0	35	0·143	0·10	1·257	
16-T	0·0766	12·0	52	0·146	0·20	0·978	
17-T	0·1393	12·0	86	0·163	0·23	0·939	
18-T	0·1750	16·0	64	0·195	0·28	0·720	
19-T	0·0450	14·0	74	0·086	0·05	2·158	Clay, moderate plasticity
20-T	0·0611	22·7	47	0·130	0·05	1·052	Clay, very plastic

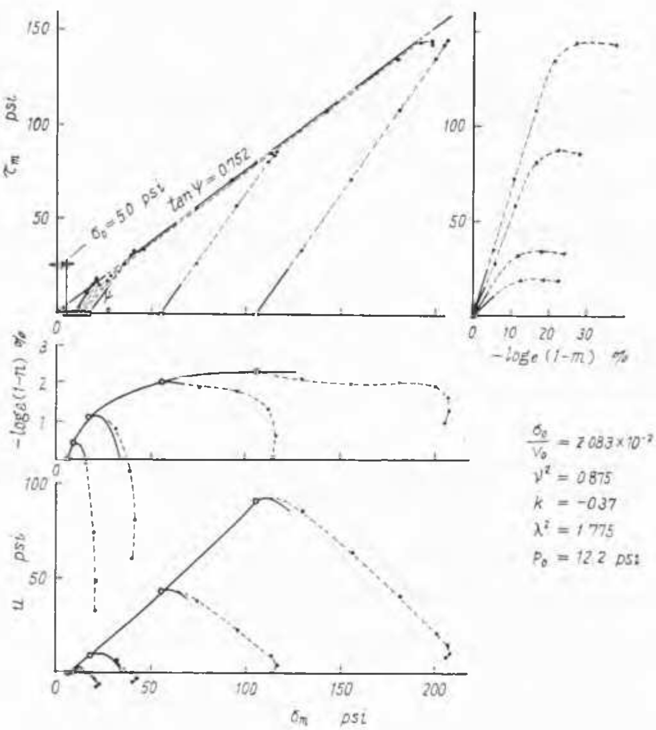

Fig. 7 Analysis of test No. 1-*T* (Wagner).

Diagrammes de l'essai de compression triaxiale rapide (Wagner) : test No. 1-*T*).

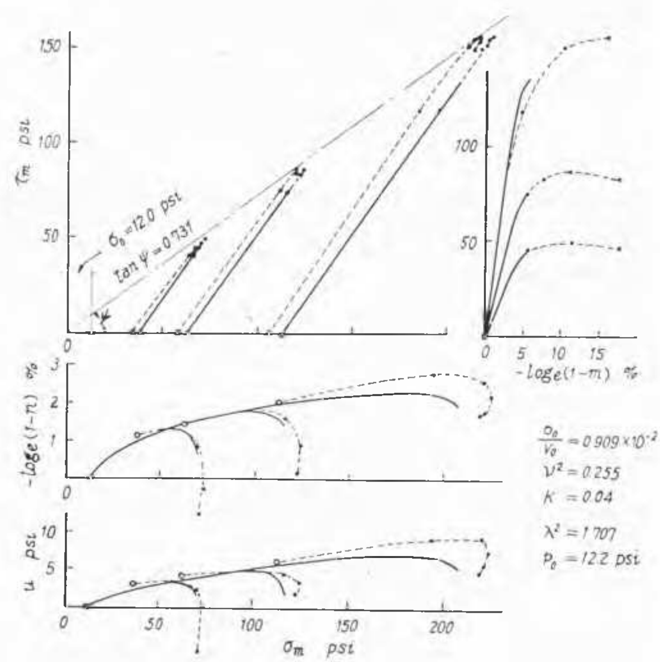

Fig. 8 Analysis of test No. 2-*T* (Wagner).

Diagrammes de l'essai de compression triaxiale rapide (Wagner : test No. 2-*T*).

integration of which can establish the relationship between applied axial stress and induced pore pressure.

$$\left\{ \frac{1}{p_0 + u} - \frac{1 - Na}{p_0 + (1 - N_a)u} + \frac{\sigma_0}{V_0} \frac{G(\theta)}{\cos\theta} \frac{1}{\sigma_0 + \sigma_L - u} \right\} \frac{du}{d\theta}$$

$$= \frac{\sigma_0}{V_0} \frac{G(\theta)}{H(\theta)} - \frac{\sigma_0}{V_0} \tan\theta_0 \qquad (18)$$

in which

$$H(\theta) = \frac{\sqrt{2}}{R_0} - \sin\theta + \sin\theta_0$$

8. Analyses of triaxial test results under rapid loading

Analyses have been made on the results of the triaxial test undertaken by A.A. WAGNER (1950) under rapid loading. Constants assumed in each test are given in Table-2.

Two examples of the analysis of test results are illustrated in Figs. 7 and 8.

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