CONSOLIDATION – AN ANALYSIS WITH PORE PRESSURE MEASUREMENTS

CONSOLIDATION – UNE ANALYSE AVEC MESURES SUR PRESSION PORE

КОНСОЛИДАЦИЯ – АНАЛИЗ ДАННЫХ ИЗМЕРЕНИЯ ПОРОВОГО ДАВЛЕНИЯ

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SYNOPSIS. An investigation is undertaken to study the role of adsorbed water in governing the mechanism of the process of consolidation of varied electro-viscous soil systems with pore pressure measurements at the bottom face and at different depths of consolidating systems. For the purpose conventional size new consolidometer and large size consolidometer were designed and fabricated.

An attempt is made to explain the variations on the basis of soil-water system by developing a physical model. The model is transformed into a rheological model and a mathematical theory is developed. The proposed rheological theory helps accounting for the observed pore water pressure and its performance in time-curves gives good agreement when compared to other consolidation theories.

INTRODUCTION

The deviations exhibited by the classical theory on consolidation has led to an extensive research work on the consolidation characteristics of soils. These researches in turn have resulted in contributing the various theories of consolidation analysis as putforth by Taylor-Merchant (1940), Gibson-Lo (1960), Wahls (1962), Barden (1965, 1969), Tsytovich-Dalmatov (1969), and Zaretsky et al (1969). It has also been accepted that pore pressure variations and consolidation characteristics as observed in testing programs are not in confirmity with the theoretical results, Christie (1965), Katti-Sonpal (1967), Tsytovich-Dalmatov (1969) and Zaretsky et al (1969).

Hence an attempt is made in this investigation to evaluate the role of adsorbed water in governing the consolidation characteristics on varied electro-viscous systems with pore pressure measurements at the bottom face and at one-fourth, one-half and three-fourth the depth of the consolidating layer with the same measuring device. For the purpose, selected soil systems with varying base exchange capacity (B. E. C.) are tested at different void ratios in the conventional size new and large size consolidometers.

LABORATORY INVESTIGATION

Material :

Black cotton soil from Baroda region and sodium bentonite were the major systems for the investigation program. Saurashtra soil and South Gujarat soil samples were also selected for a comparative study. Property data of those soils are given in Table 1. Consolidation tests are conducted on 5-micron and 2-micron clay fractions separated by sedimentation process without using any dispersing agent and having base exchange capacity variations of 24-134 m.e./100 g.

Table 1
PROPERTY DATA OF SOIL SYSTEMS

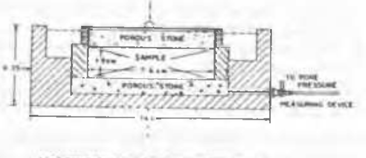
Soil Systems	L. L.	P. L.	B. E. C.
	%	%	(me./100g)
Black Cotton Soil	64.8	42.3	30
5 µ — Clay Fractions	98.6	60.4	57
2 µ — Clay Fractions	—	—	69
Sodium Bentonite			
5 µ — Clay Fractions	336	65	112
2 µ — Clay Fractions	—	—	134
Saurashtra Soil	42.50	30.80	—
2 µ — Clay Fractions	—	—	24
South Gujarat Soil	54.30	36.40	—
2 µ — Clay Fractions	—	—	50

Preparation of specimens and testing :

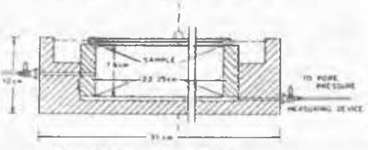
To study the different states of soil systems three void ratios 1.00, 1.25 and 1.50 were adopted for the conventional size new consolidometer and these values were 1.25, and 1.50 for large size consolidometer. The common void ratio of 1.25 in both the consolidometers testing programs corresponds to the field density of black cotton soil.

For the conventional size consolidation testing program, the samples were moulded in the rings 7.6 cm in diameter and 1.9 cm in height and were covered at the top and bottom with filter papers. Grease was applied to reduce friction. As shown in Fig. 1(a), the base was connected to Bishop's pore pressure apparatus with rigid copper tubing connections. The drainage was allowed in the upward direction only.

The effect of flexibility of pressure measuring equipment has been well analysed by Christie (1965) and he has reported that even in the absence of the flexibility of the measuring equipment, pore pressure build ups are observed. Tsytovich-Dalmatove (1969) and Zaretsky et al (1969) have putforth the reasoning of compacted nature of the soil water system for such observations. Data in this investigation have helped corroborating this analysis.



1(a) CONVENTIONAL SIZE NEW CONSOLIDOMETER UNIT



1(b) LARGE SIZE CONSOLIDOMETER UNIT

FIG. 1

Load increments adopted were 0.1 to 0.2, 0.2 to 0.4, 0.4 to 0.8, 0.8 to 1.6, 1.6 to 3.2 and 3.2 to 6.4 kg/cm.^{2} Readings were taken for a period of 24 hours after the application of each load increment.

Similar procedure was also adopted for large size consolidometer testing program except that in this case pore pressures are measured at four points-at bottom, three-fourth, one-half and one-fourth the height of the sample by repeating the same test three times. Details of the large size consolidometer are shown in Fig. 1(b). Further the pore pressures were measured by inserting surgical tubes called cannula having 2 mm diameter bore and length of 16 cm. The cannula when inserted in the consolidometer will have its tip exactly at the centre of the sample placed in the consolidometer ring.

Results :

The consolidation characteristics as evaluated from the testing programs are presented and analysed in sequence first of conventional size consolidometer unit and subsequently of large size consolidometer unit.

Pore pressure characteristics :

For the conventional size consolidometer unit, pore pressures were measured at the bottom and the pore pressure curves obtained for the various systems are shown in Fig. 2.

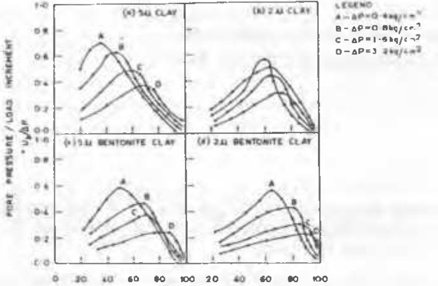


FIG. 2

AVERAGE DEGREE OF CONSOLIDATION U %

Complete examination of Figures 2(a)—(d) reveal two important characteristics :

(i) as the load increment value is doubled, the pore pressure peaks in a system as well as in the series, achieve lower values;

(ii) as the load increament value is doubled, the pore pressure peaks start shifting in a system, as well as in the series, to higher degree of consolidation achieved in the systems.

Physical model analysis :

Researches on clay mineralogy have established that soil particles-clay particles possess unbalanced electrical charges at surface and water molecules are held by these particles due to unbalanced electrical charges on the particles and dipolar nature of water molecules. The effective thickness of adsorbed water layer depends upon electro-chemistry of the soil-water system and on the property of water under consideration. As the distance increases from the clay particles, the strength with which the adsorbed water layer is held on the clay particle decreases at a faster rate and beyond a certain distance, there may not be any effect on the water present in soil-system.

If it is visualised that free-water existing in a soil water system is not running in continuous phase and has locked-up position having the surrounding of the adsorbed water layers of the adjoining clay particles, it would follow that the increment of load when applied, will be taken up by the adsorbed water layer forming the viscous medium which soon would start yielding and transfer the stress to free water existing in the locked-up position. This in turn would give rise in the pore pressure values as time elaspes and a built-up pore pressure peak, commensurate with the viscous state prevelent, would be observed.

To analyse the role of adsorbed water in governing the mechanism involved in the process of consolidation, it will be necessary to study the variation of pore pressure peaks in the soil systems having not only the varied B.E.C. values and the same state of void ratio but also the same equivalent particle size in the systems. Pore pressure data obtained in this regard for the four soil systems with 2-micron size fractions of Saurashtra Soil Sample, South Gujarat Soil Sample, Black Cotton Soil Sample and Bentonite Clay Sample have been presented in Figures 3(a)—(d).

Inspection of the characteristics in Fig. 3(a) reveal that the soil systems with (or without) the same equivalent size of the particles and the same initial void ratio, at a given increment of load, higher the B.E.C. value of a system, lesser would be the pore pressure peak value. Further as the void ratio decreases, interparticle spacing prevalent in a system also decreases leading to the increase in the resistance of adsorbed water around clay particles which in turn would permit lesser magnitudes for the pore pressure peaks at a given load increment. This point is corroborated from the examination of curves reported in Figures 3(b)—(d).

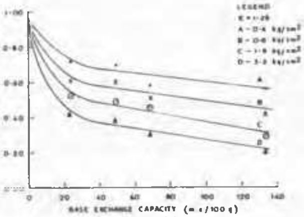


FIG. 3 (a)

PORE PRESSURE PEAK AND BEC RELATIONSHIP

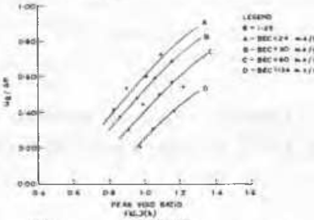


FIG. 3(b)

PORE PRESSURE PEAK AND VOID RATIO RELATIONSHIP

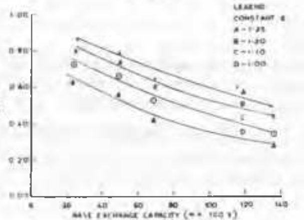


FIG. 3 (c)

PORE PRESSURE PEAK BEC RELATIONSHIP (CONSTANT VOID RATIO)

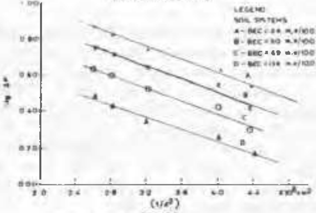


FIG. 3(d)

PORE PRESSURE PEAK AND VISCOUS RESISTANCE RELATIONSHIP

PORE PRESSURE DATA FROM LARGE SIZE CONSOLIDOMETER :

The validity of the physical model will now be examined for the isochrones of the soil systems. For the purpose, a typical isochrone for 2-micron (Black Cotton Soil) clay system is fully analysed at different stages of consolidotion and is shown in Figures 4(a)—(f).

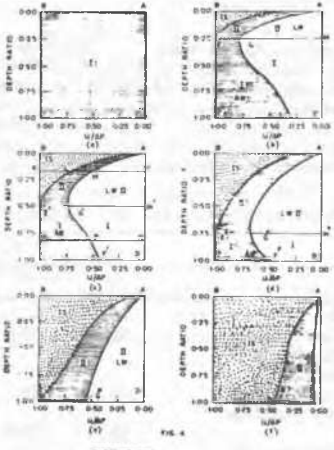


FIG. 4

ISOCHRONE ANALYSIS FOR 2μ CLAY SYSTEM

For an electroviscous soil system, when an increment of load is applied, the full load will initially be taken up by the adsorbed water layers forming the viscous medium and pore water pressure will have zero initial value throughout the depth of the consolidating system. Hence in Fig. 4(a), line AD represents the intial line of pore water pressure for the proposed physical model.

When pore pressures reach a peak value at one-fourth the depth of the sample, nature of the isochrone is represented by line AGF in Fig. 4(b). At this stage consolidation as indicated by pore pressure peak, has advanced just to one-fourth the depth of the sample. Hence the degree of consolidation would be zero at this level which in turn would give the general nature of consolidation line as AE.

Inspection of Fig. 4(b) reveals that the consolidating soil system has two distinct states, along its depth, as separated by line EH. Soil system in its part, above this line is undergoing consolidation while the one below this line has still to initiate the consolidation. The granular region IS of the figure shows skeleton part of the system under stress, shaded region AW indicates adsorbed water stress while the plain region LW depicts liquid water pressure in the soil system.

In Fig. 4(c), as the pore pressure peak is observed at one-half the depth, consolidation degree at this depth would be zero. The degree of consolidation at one-fourth the depth would be equal to the pore pressure peak value at that depth less the actual value of pore pressure in the present stage. This in turn would give ALE as the consolidation line for the stage of the soil system.

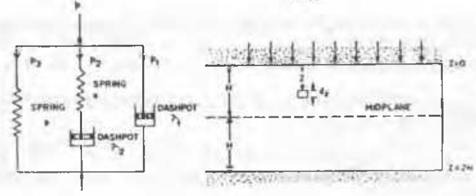
Further, from Fig. 4(c), it can be seen that any line such as XY in the consolidating depth of the system would have its magnitude equal to the load increment value and would be the summation of pressures taken up by soil grains (effective stress) as represented by XL, viscous resistance as represented by LM and pore pressure magnitude equal to YM. Beyond the depth, where consolidation is still to be initiated and where build-up of pore water pressure continues, any line such as MN, having its magnitude equal to the load increment, would consist of the summation of transient effective pressure represented by MP and pore water pressure represented by PN.

As the consolidation progresses, pore pressure peaks are achieved successively at remaining three-fourth and full (bottom) depth of the consolidating system as shown in Figures 4(d)—(e). Fig. 4(f) shows the degree of consolidation, isochrone at 90% dissipation of pore pressure at the bottom and the corresponding magnitude of viscous resistance accompanying consolidation to continue secondary compressions in the soil system.

Load transfers in the consolidating system, as revealed from the above analysis, in the varied phases of the soil water system, is in complete accordance with the proposed physical model.

FORMULATION OF RHEOLOGICAL THEORY AND ITS CORRELATION :

The physical model analysed in the preceding section is transformed into a rheological model in its generalised form by introducing a Maxwell element in the Kelvin body as shown in Fig. 5(a). From the inspection of the model, it can be seen that transmission of load sequence will be governed by parameters λ_1, λ_2, a and b of the model.



5(a) PROPOSED RHEOLOGICAL MODEL

5(b) CONSOLIDATING STRATUM

FIG. 5

At any time t, after the application of a stress p, the corresponding stresses in the dashpot arm, Maxwell arm and spring arm are p_1, p_2 and p_3 respectively. The compressive strains in the arms should be equal. Hence $p = p_1 + p_2 + p_3$ .. 1

$$\text{and} \quad \frac{ds}{dt} = p_1\lambda_1 = a.\frac{dp_2}{dt} + p_2\lambda_2 = b.\frac{dp_3}{dt} \qquad ..\ 2$$

For one-dimensional consolidation of a finite layer of a soil system Fig. 5(b), at small strains,

$$\frac{k}{\gamma_w}\frac{\delta^2 u}{\delta Z^2} = \frac{\delta u}{\delta t} \qquad ...\ 3$$

Combining equation 2 & 3,

$$\frac{k}{\gamma_w}\frac{\delta^2 p_2}{\delta Z^2} = a.\frac{\delta p_2}{\delta t} + p_2\lambda_2 \qquad ..\ 4$$

$$\text{or} \quad \frac{\delta^2 p_2}{\delta Z^2} - \frac{a\gamma_w}{k}.\frac{\delta p_2}{\delta t} - p_2\lambda_2 = 0 \qquad ..\ 5$$

Boundary conditions for the consolidating system shown in Fig. 5 (b) can be written as

(i) at $Z = 0$, $p_2 = 0$ at $t = 0$
(ii) at $Z = 2H$, $p_2 = 0$ at $t = 0$
(iii) at $t = 0$, $p = p_1$
and (iv) $\frac{dp}{dt} = \frac{dp_1}{dt} + \frac{dp_2}{dt} + \frac{dp_3}{dt}$

Equation 5 is solved for dimensionless solution by applying the boundary conditions, assuming the two characteristic times (λ_1 b and λ_2/a) as equal and for the systems analysed neglecting the increase of viscosity of medium from initial to final state during an increment ($\lambda_1 = \lambda_2$),

$$\text{putting } \frac{\lambda_2}{a} = Tp = \text{Viscous Time Factor.} \qquad ..\ 6$$

$$\frac{kt}{a\gamma_w H^2} = T = \text{Time Factor} \qquad ..\ 7$$

and $\frac{Tp}{T} = R^2 = \frac{\lambda_2 \gamma_w H^2}{k}$ one gets the performance equations as :

$$p_1 = \sum_{m=0}^{m=\infty} \frac{(2m+1)\pi}{\left(R^2 + \frac{(2m+1)^2\pi^2}{4}\right)} . \sin\left(\frac{(2m+1)}{2} . \frac{Z}{H}\right) x$$

$$x \ \exp\left\{-\left(R^2 + \frac{(2m+1)^2\pi^2}{4}\right)T\right\} \qquad ..\ 8$$

$$p_2 = \sum_{m=0}^{m=\infty} \frac{4}{(2m+1)\pi} . \frac{R^2}{\left(R^2 + \frac{(2m+1)^2\pi^2}{4}\right)}$$

$$\text{Sin}\left(\frac{(2m+1)}{2} . \frac{Z}{H}\right) x$$

$$x \ \exp\left\{-\left(R^2 + \frac{(2m+1)^2\pi^2}{4}\right)T\right\} \qquad ..\ 9$$

$$U_{D_1} = 1 - \sum_{m=0}^{m=\infty} \frac{2}{\left(R^2 + \frac{(2m+1)^2\pi^2}{4}\right)} x$$

$$x \ \exp\left\{-\left(R^2 + \frac{(2m+1)^2\pi^2}{4}\right)T\right\} \qquad ...\ 10$$

$$U_{D_2} = 1 - \frac{8}{\pi^2}\sum_{m=0}^{m=\infty} \frac{R^2}{\left(R^2 + \frac{(2m+1)^2\pi^2}{4}\right)} x$$

$$x \ \exp\left\{-\left(R^2 + \frac{(2m+1)^2\pi^2}{4}\right)T\right\} \qquad ..\ 11$$

and

$$U_c = 1 - \sum_{m=0}^{m=\infty} \frac{2R^2}{\left(R^2 + \frac{(2m+1)^2\pi^2}{4}\right)^2} x$$

$$x \ \exp\left\{-\left(R^2 + \frac{(2m+1)^2\pi^2}{4}\right)T\right\} \qquad ..\ 12$$

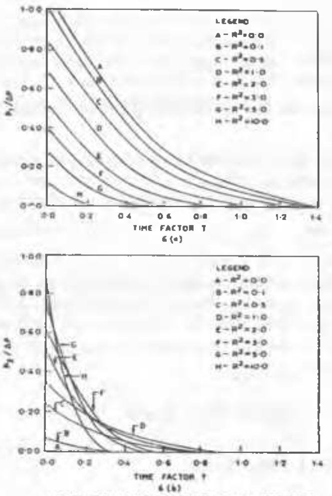
Where U_{D_1}, U_{D_2} and U_c are dissipation ratios for the first and second arms and consolidation degree respectively of the model.

On inspection of these equations, it becomes evident that when time factor ratio becomes zero, equation 8 reduces to Terzaghi equation for pore pressure and equation 10 of dissipation ratio reduces to Terzaghi equation for degree of consolidation. Theoretical solution for p_1, p_2 and U_{D_1} are shown in Figures 6(a)—(c) .

Fig. 6(a) & (b) help evaluating the pore pressure peak value and corresponding peak time factor. Pore pressure peak will be evaluated by the equation

$$u_f = (p_1)_0 + (p_2)_0 - p_{2f} \qquad ..\ 13$$

For dissipation characteristics, the curves of Fig. 6(a) are traced again and dissipation time for 90% (T_{90}) is evaluated. Correlation of the soil system as a typical case, for pore pressure peak and dissipation characteristics has been shown in Fig. 7(a).



THEORETICAL RATES OF MIDPLANE P₁ & P₂ DISSIPATION

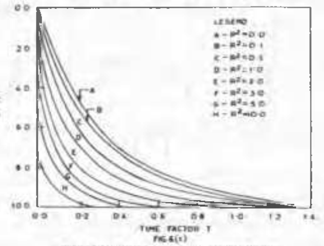
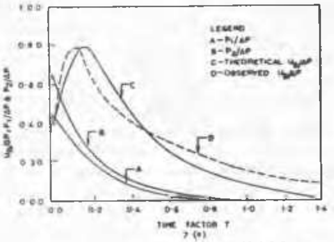
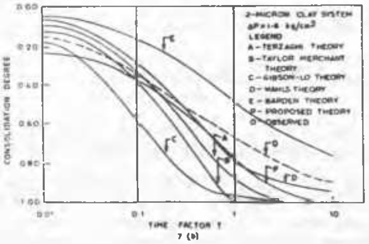


FIG.6(c)
THEORETICAL RATES OF U_{D1} CONSOLIDATION

The theoretical plots of p_1 and p_2 on logarithmic time scale indicate the process about ten times faster than the normal one. Hence the correlation of a system for consolidation characteristics requires modification to $10\,T_1$ (peak time factor) value of the corresponding time factors. Performance time curves have been correlated herein for the representative two micron (Black Cotton Soil) clay system in Fig. 7(b).



7 (a)
CORRELATION OF POREPRESSURE CURVE



7 (b)
PERFORMANCE OF TIME CURVE OF THE THEORY

The proposed theory gave good agreements with the soil systems specially when R value is less than 5. At higher values of R^2 as the skeleton takes major portion of the stress transfer leaving very little increase of stress to pore pressure, the estimated pore pressure peak showed poor correlation. However for such soil systems, the performance of the theory in U—T curves was reasonably well in agreements with the observations.

The theory further helps in corroborating the data reported by Tsytovich-Dalmatov (1969) wherein the pore pressure factors are continuously decreasing as the load increments are advanced.

In the field conditions where the consolidating stratum has thickness much higher than laboratory thin samples, the value of R^2 which otherwise amount to be exceptionally high will have a value of R^2 depending upon its internal electro-viscous state which in its performance in consolidation will trace out an appropriate solution for both pore pressure and time curves characteristics.

CONCLUSION :

The present study reveals that unbalanced electrical charges influence the consolidation characteristics of the soil system considerably. Unlike prevailing two phase analysis of a soil water system, stress transfer takes place in three phases, namely adsorbed water phase, liquid water phase and the skeleton phase. On the basis of analysis of results, a physical model with these three constituent phases is developed.

The physical model is transformed into a rheological model and a theory is formulated. The proposed rheological theory, in its application, helps accounting for the unique pore pressure characteristics of the soil systems. A comparative study of time curves obtained from the proposed rheological theory its correlation with the experimental results and the performances revealed by other theories, has shown reasonable agreements for the soil systems analysed in this investigation.

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