LONG TERM BEHAVIOUR OF MEXICO CITY CLAY

COMPORTEMENT A LONG TERME DES ARGILES DE LA VILLE DE MEXICO

ДЛИТЕЛЬНОЕ СОПРОТИВЛЕНИЕ ГЛИН МЕКСИКО СИТИ

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SYNOPSIS. The critical condition for slope stability on clays occurs after a long period of time. Accordingly,to study this problem for Mexico City clays,a research of their rheological properties was carried out.

The results of consolidated-undrained tests with pore pressure measurement in undisturbed and remoulded specimens are described. The tests were done at constant strain rate ranging from 1.5 to 0.0007% /min.

The test indicates a noteworthy strength reduction after a long period of time in terms of effective stresses; they also indicate that the generated pore pressure does not depend of the strain rate for equal confining pressure.The results are disucssed on the basis of the characteristics of the triaxial testing equipment used, in which the axial load is transmited through wires under tension.

1. INTRODUCTION

The creation of a lake in the old Texcoco basin, bordering upon the City of Mexico, has been proposed for the storage and control of the waters of the Valley of Mexico. To analyse the construction methods of the artificial lake,dredging was carried out to excavate an area 32m wide to a depth of 8m. Three months after the work was completed,various failures of the slope of the cut were observed (Proyecto Texcoco,1969).In order to determine the safety factor of the cut, the shear strength of the clays was measured by means of a field vane. Upon the basis of the data obtained and having localized the failure surface, established from readings of the inclinometers which had been installed previously,a safety factor of 1.5 was obtained. Thus the analysis $\phi=0$ is incorrect regarding its prediction of the stability of the cut shortly after the completion of construction.An effective stress analysis for drained conditions revealed that the safety factor of the slope was equal to 1,for an angle of friction of the clays equal to 28°. It should be noted here that the values of the effective angles of friction reported,for the clays of the Valley of Mexico,are 43° and 47° for consolidated undrained tests,(Marsal, R.J.,1960 and Lo,L.Y.,1962)and vary between 28° and 34° for consolidated drained triaxial and direct shear tests (Marsal,R.J.,1969,and Resendiz,D.,1964).In view of the very considerable difference between the effective angles of friction under drained and undrained conditions,a study was undertaken of the effect of time upon the mechanical behaviour of these clays. The results obtained from consolidated undrained triaxial tests with different rate of strain are presented herein.

2. CHARACTERISTICS OF THE TRIAXIAL EQUIPMENT AND OF THE SAMPLE. TYPE OF TEST

2.1 Triaxial Equipment

The axial load is applied to the specimen by means of an upper cap linked to three wires which pass to the sides of the sample and apply the load downwards (Fig.1). The upper cap drains through a thin tube tied by a chain to a counterweight which rests upon the load plate of a controlled displacement machine (Wykeham Farrance),with a minimum velocity of displacement of 0.00062mm/min.When the test is begun,there is a delay in the deformation of the sample due to the effect of the deformation of the load system. However, after 1 per cent deformation the velocities of displacement of machine and sample are equal.The triaxial cells employed are described in detail by Santoyo,E., 1971.

The confining pressure and the back pressure are transmitted along 3m long narrow connecting lines,filled with water to reduce the process of the diffusion of air.

Errors in the measurement of loads due to uncontrolled friction are of the order of 10gr/sq cm and are practically independent of the rate of axial strains and of the occasional horizontal loads (Santoyo,E. 1971). Errors in the confining pressure are $\pm$7gr/sq cm.

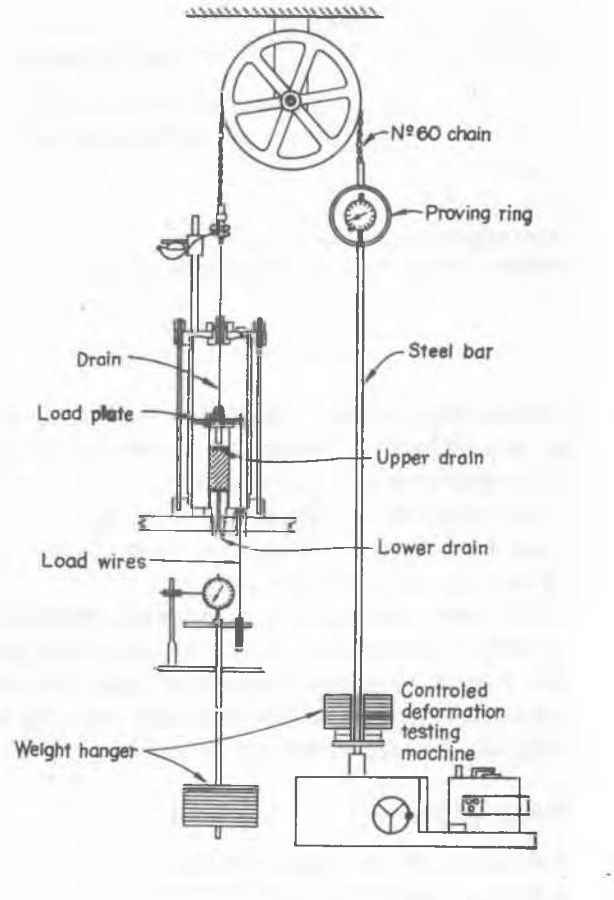

Fig.1. Triaxial equipment

2.2. Volumetric strains and pore pressure

Drainage of the sample is at the upper and lower caps, of a conventional type. The connection between the upper cap and the burette is shown in Fig.1. It consists of a stainless steel tube with an interior diameter of 0.06cm and an exterior diameter of 0.22 cm and equipped with a null displacement valve. The water drained through the lower head is collected in the burette by means of a 1/8 in-diameter copper tube,30cm long. The precision of the burette is of the order of 0.01cm^3. Its upper part is jointed to a line containing, at the burette end, silicon oil with a viscosity of 0.5cp and a surface tension of 15.9 din/cm.The other end of the line joins a tank partially filled with water. Variations in the volume of the sample are measured against the displacement of the water-oil interphase in the burette. Oil is used to avoid both the evaporation of the drained water and the dissolution of air in the interstitial liquid of the sample.

Pore pressure is measured by Statham pressure transducer of the unbounded type, compensated for temperature changes and connected to a W.T.Bean bridge equipped with a cell for calibration and zero adjustment. This system is connected up, one hour prior to beginning each test. The flexibility of this system for the measurement of pore pressure varies with the pressure measured: thus for pore pressures lower than 0.5 kg/sq cm it is equal to $5x10^{-3}$ cu cm/kg/sq cm, while for higher pressures it is $2.5x10^{-3}$ cu cm/kg/sq cm. In such conditions, for the clay under study, the maximum time needed for the measurement system to respond to pore pressure is 19 seconds.

Whitey null displacement valves are used in the system together with Swagelock connections.

The tests were carried out in a room where a temperature of $20^{\circ}\pm1^{\circ}C$ was maintained.

2.3 Characteristics of the sample and type of tests

The soil samples studied are cubic and were taken from the old Texcoco basin. They were obtained at a depth of 2.5m, the ground water-level being at a depth of 1.5m. Numerous vertical cracks were observed in the excavated cut, some filled with sand, other with cementing materials. The specimens tested in the laboratory were carefully prepared, avoiding all visible fissures and heterogeneities.

The clayey minerals of the Valley of Mexico are classified as allophanes (Girault, P., 1964). The average index properties of the tested specimens are: $w_L=343$; $I_p=279$; wnat=406, $S_y=100$ per cent and G=2.54. Sensitivity is 8. A standard consolidation test gives a consolidation coefficient, in an undisturbed state of $2x10^{-3}$sq/cm/sec for the virgin range. The preconsolidation load is equal to 0.45 kg/sq cm, implying a small degree of consolidation due to crust drying; the coefficient of volumetric conslidation m_v is 0.15 sq cm/kg.

The triaxial tests were consolidated undrained with pore pressure measurement and at a rate of strain of between 0.045 and 94 per cent/h. Consolidation was isotropic and the same confining stress maintained at the failure stage. Since the tests lasted for periods of up to 20 days, two latex membranes, both 0.0065 cm thick, were used. The time required to achieve uniformity at 90 per cent of pore pressure in the specimens without lateral drainage (Gibson, R.E., 1963) was 8h in the case of the undisturbed and 160 h for the remoulded samples.

The interstitial water in the surface layers of the Texcoco basin has a high salt content, twice that of the sea. A qualitative analysis of the interstitial water of the samples revealed the presence of the following anions: CO_3^{--}, PO_4^{---}, and Cl^- as well as the cations Na^+, NH_4^+ and K^+.

3. TEST RESULTS

The result of consolidated undrained triaxial tests, upon undisturbed and totally remoulded samples are summarized in Tables I and II respectively.

3.1 Types of failure and deformation

In the case of undisturbed samples the failure is of the brittle or plastic type, depending upon the magnitude of the confining pressure. For null confining pressure the failure plane is marked, forming an angle of approximately 45° with the horizontal. For confining pressure of 0.25 kg/sq cm or larger the failure plane disappears. Also, strain at failure increases as the void ratio at failure decreases, in the case of both undisturbed and remoulded samples.

The fact that strain at failure, defined as the point at which ($\sigma_1 - \sigma_3$) reaches a maximum, is practically independent of the velocity of deformation, is worthy of note. For example, the strain at failure for tests 2.2[1] 5 and 8 varied between 6.9 and 5.6 per cent, while the rate of strain ranged from 1.5 to 0.0007 per cent/min.

Table I. Undisturbed samples. Consolidated undrained triaxial tests

Test No	e_f	W_i %	σ_3 kg/cm²	e_f	W_f %	$\dot{\epsilon}$ %/h	Failure for $(\sigma_1 - \sigma_3)_{max}$: u_f kg/cm²	$(\sigma_1-\sigma_3)_{max}$ kg/cm²	ϵ_f %	A_f	t_f in hours	u_f/σ_3	u^* kg/cm²	A_f^*	u_f^*/σ_3
1	10.18	406.7	0.25	9.76	390.1	94	0.16	0.41	3.2	0.39	0.034	0.64	0.16	0.39	0.64
2	10.34	406.5	0.50	8.83	347.7	94	0.33	0.64	6.6	0.50	0.070	0.66	0.33	0.50	0.66
2'	9.71	384.4	0.50	8.26	327.3	94	0.33	0.64	6.9	0.50	0.073	0.66	0.33	0.50	0.66
3	10.16	408.8	1.00	6.65	271.0	94	0.70	1.16	7.1	0.60	0.076	0.70	0.70	0.60	0.70
4	9.72	387.2	0.25	9.33	371.8	1.88	0.13	0.35	3.2	0.36	1.70	0.52	0.13	0.36	0.52
5	10.22	406.7	0.50	7.96	317.5	1.88	0.32	0.53	6.0	0.60	3.19	0.64	0.32	0.60	0.64
6	10.71	426.8	1.00	6.94	277.6	1.88	0.71	1.04	7.6	0.68	4.04	0.71	0.71	0.68	0.71
6'	9.77	388.6	1.00	6.59	263.4	1.88	0.68	0.97	7.0	0.70	3.72	0.68	0.68	0.70	0.68
7	10.18	402.6	0.25	9.76	386.1	0.045	0.16	0.26	3.4	0.61	75.6	0.64	0.16	0.61	0.64
8	10.33	409.9	0.50	8.49	337.3	0.045	0.34	0.46	5.6	0.74	124.4	0.68	0.30	0.65	0.60
9	10.06	398.4	1.00	6.80	270.7	0.045	0.82	0.88	7.9	0.93	175.6	0.82	0.70	0.80	0.70
10	11.12	437.7	0	11.18	444.1	94	–	0.24	2.1	–	0.022	–	–	–	–
11	11.08	436.2	0	11.12	438.7	1.88	–	0.16	3.1	–	0.69	–	–	–	–
12	10.68	420.7	0	10.79	424.9	0.045	–	0.17	2.5	–	55.6	–		–	–

e_i, e_f = initial, final void ratio
w_i, w_f = initial, final water content
σ_3 = confining pressure
$\dot{\epsilon}$ %/h = strain rate
u_f, u_f^* = measured and corrected pore pressure at failure
$(\sigma_1 - \sigma_3)_{max}$ = maximum principal stress difference
ϵ_f % = strain at failure
A_f = Skempton's pore pressure parameter at failure
t_f = time to failure

Table II. Remoulded samples. Unconsolidated undrained triaxial tests

Test No	e_f	w_f	$\dot{\epsilon}$ %/h	$(\sigma_1-\sigma_3)_{max}$	e_f %	u_f	t_f
16	7.29	288.7	1.88	0.126	2.9	0	1.54
17	6.80	267.5	1.88	0.230	4.5	–0.003	2.40
18	8.0	321.0	1.88	0.096	2.0	0.006	1.06

3.2 Pore pressures

The magnitude of the pore pressures observed at the base of the samples during the failure stage, is related to: (1) the time of response of the measuring system (2) the time required to achieve uniform interstitial pressure throughout the sample, (3) the hydraulic permeability of the membrane and (4) the osmotic pressure generated between the confining and interstitial liquids. The time of response of the measuring system is 19 seconds under the most unfavorable conditions while the time required to achieve 90 per cent uniformity of pore pressure is 8h in the case of undisturbed samples and 160h for remoulded samples. Thus differences in the observed pore pressure at a rate of a strain of 94,1.88 and 0.045 per cent/h for undisturbed samples is to be expected, taking only the factor time required to achieve uniformity into account.The pore pressures measured during tests upon remoulded samples at a velocity of 1.88 per cent/h are practically worthless since, in this case,the time required to achieve 90 per cent uniformity is far greater than time to failure.

Besides the effect of the time required to achieve uniform pore pressure,there also exists the phenomenon of the seepage of the confining liquid through the sample's protective membrane.In the case of long term tests this is significant,especially since a high osmotic pressure is generated as a consequence of the high salt content of the interstitial liquid.To take into account the effect of seepage upon the pore pressure generated within the sample,a correction was made,based upon the following reasoning.Designating the volume of water penetrating the membrane V_w and the consequent increment in the pore pressure u,there occurs a variation in the volume of the sample equal to the sum of the following:

- Compression of the interstitial liquid. This is equal to $V_0 C \Delta u$; where V_0 is the volume of the interstitial liquid of the sample and C the coefficient of volumetric compressibility of water.

-The increase in volume due to the flexibility, F, of the system by which the pore pressure is measured, equal to $F\Delta u$.

-The increase in volume ΔV_m of the sample due to a reduction of the effective confining stress, equal to $SV_m \dfrac{\Delta u}{\sigma_3 - u}$ where V_m is the volume of the sample and S is the initial swelling ratio of the soil in discharge, equal in this case to 0.03 (Poulos, S.J., 1964).

Assuming that the variation in volume of the sample-measurement system is equal to the volume of water filtered through the membrane, then

$$V_w = \left[V_0 C + F + \frac{SV_m}{\sigma_3 - u} \right] \Delta u$$

$$\Delta u = \frac{V_w}{V_0 C + F + \dfrac{SV_m}{\sigma_3 - u}}$$

The volume of water passing through the membrane is equal to (Poulos, S.I., 1964):

$$V_w = \left[kA \frac{\sigma_3 - u}{L} + KA \frac{\Delta p_v}{L} \right] t$$

where

t = duration of test

k = Darcy's permeability coefficient of the membrane

K = membrane permeability constant

A = area of filtration

L = membrane thickness

Δp_v = difference between vapour pressures of the confining liquid and the interstitial liquid.

Replacing the literals by their numerical values, we obtain :

$$1)\ \Delta u_{kg/cm^2} = \frac{0{,}35(\sigma_3 - u)_{kg/cm^2} + 9{,}5 \Delta p_v\, gr/cm^2}{3 + 4{,}5 + \dfrac{2400}{(\sigma_3 - u)_{kg/cm^2}}} t_{dias}$$

The vapour pressures of the confining and interstitial liquids were determined with an isoteniscope, in function of the temperature in both a deaired state and without deairing.

The results of these measurements are given in Fig.2

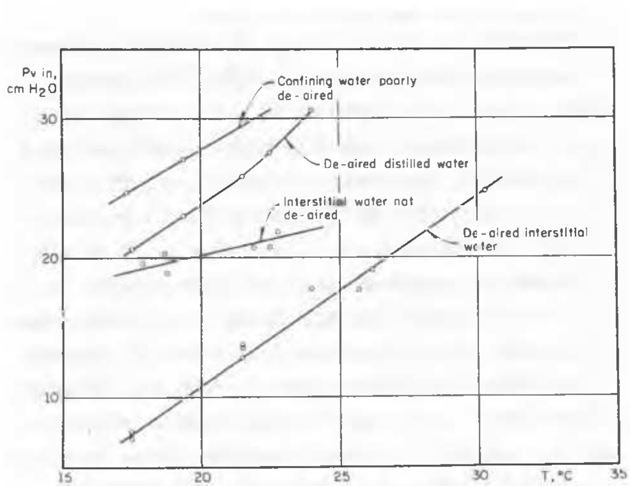

Fig.2. Vapour pressures

At 20°C the difference in vapour pressure Δp_v of the confining and interstitial liquids is 8 gr/sq cm. Upon this basis the observed pore pressures, u_f, were corrected by means of Eq 1. The corrected pore pressures are given in Table I, in column denoted u_f^*. The variation in the pore pressure, u_f^* is shown in Fig 3 as a function of axial strain for the set of tests on undisturbed samples. It may be seen that for confining pressures of 0.25 and 0.5 kg/sq cm, pore pressure is strictly independent of the rate of strain.

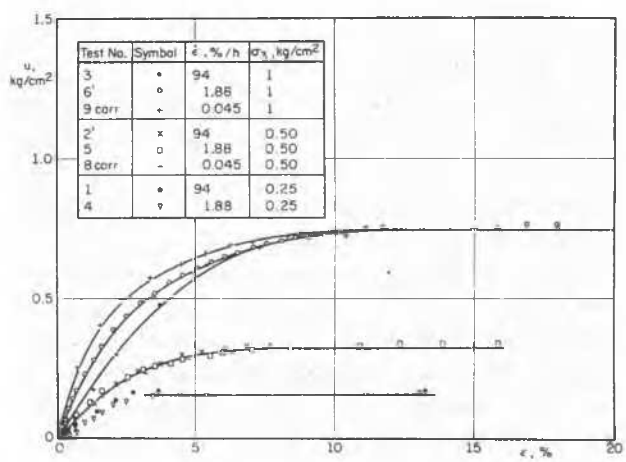

Fig.3 Pore pressure versus axial deformation

In those tests carried out with a confining pressure of 1 kg/sq cm, the pore pressure observed for the same value of strain increases with a decrease in the rate of strain. However, the pore pressures determined in this latter case and for high strain values coincide. It may be assumed, therefore, that in this case the discrepancies observed in the $u_f^* - \varepsilon$ relationship are due to the time required to achieve uniform pore pressure.

To conclude, in the case of samples consolidated under the same pressure σ_3 there is a unique relationship between axial strain and pore pressure corrected for effects due to seepage through the membrane. This relationship is independent of the rate of strain, particularly over the interval of failure strains. This result is of the greatest importance, since it allows the construction of Mohr's envelopes in function of effective stresses for any rate of strain provided only that the corresponding value of $(\sigma_1 - \sigma_3)$ max is known.

The magnitude of the pore pressure at failure varies linearly with the confining stress and is independent of the rate of strain. Fig 4 shows the line representative of the variation of u_f^* against σ_3, with a slope of 0.65. According to the data given in Table I, the coefficient of pore pressure A_f^* increases in all cases with a decrease in the rate of strain.

3.3 Strength

All of the stress-strain curves for undisturbed samples during the failure stage, reveal an increase in shear stress with an increase in strain, followed by a slow decrease. In no case were axial strains above 20 per cent reached, so the residual resistances could not be established. For remoulded materials, a decrease in resistance was observed after the maximum value for $(\sigma_1 - \sigma_3)$ had been reached, which was in turn followed by an increase due to the restriction imposed by the membranes.

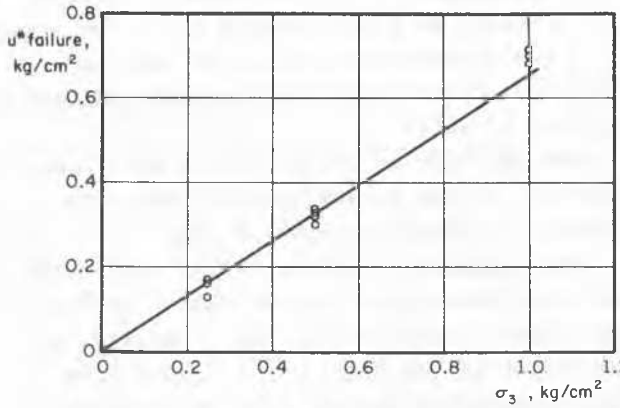

Fig.4 Pore pressure at failure versus isotropic consolidation pressure

In Fig. 5 the variation in maximum deviator stress is shown in function of void ratio at failure for both undisturbed and remoulded samples, tested at different rates of strain.

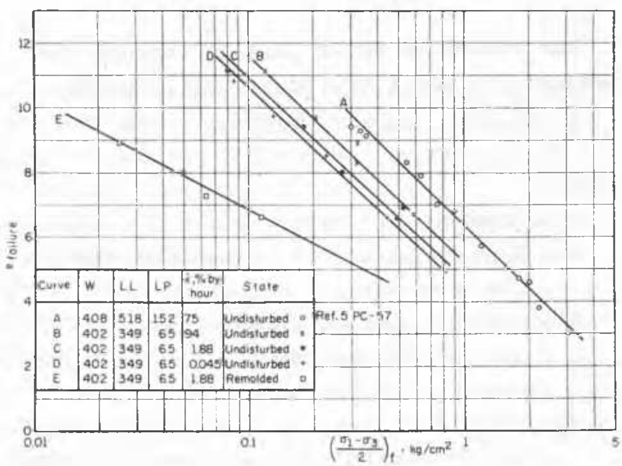

Fig.5 Void ratio at failure versus maximum deviatior stress at failure

The straight lines, e_f versus long $\frac{(\sigma_1 - \sigma_3)}{2}$ move to the left as the rate of strain diminishes and remain parallel to each other.

It is worth emphasizing that, although the preconsolidation load determined by means of a standard consolidation test, is equal to 0.45 kg/sq cm for the material represented by the straight lines B,C and D,and to 1 kg/sq cm for the material represented by the straight line A,the e_f versus log $(\frac{\sigma_1-\sigma_3}{2})$ curves show no break whatsoever for the void ratios corresponding to these preconsolidation loads.Thus,this clay behaves as an essentially cohesive material, in Hvorslev's sense,since the relationship between strength and void ratio is univocal and independent of the loading history.Consequently,the true angle of friction, Φ_e must be closed to zero.This point is in agreement with the empirical relationship between the plasticity index and true angle of friction (Lo,K.Y.,1962). It further coincides with the fact that the frictional resistance of open structured materials such as the Mexico City clays,and for small deformations,is very low (Schmertmann,J.H. 1963).

Thus,it is not surprising that this essentially cohesive resistance should vary significantly as the rate of strain is reduced,due to viscosity effects. Mohr envelopes obtained in function of the effective stresses,varying the ratio of strain,are given in Fig.6.

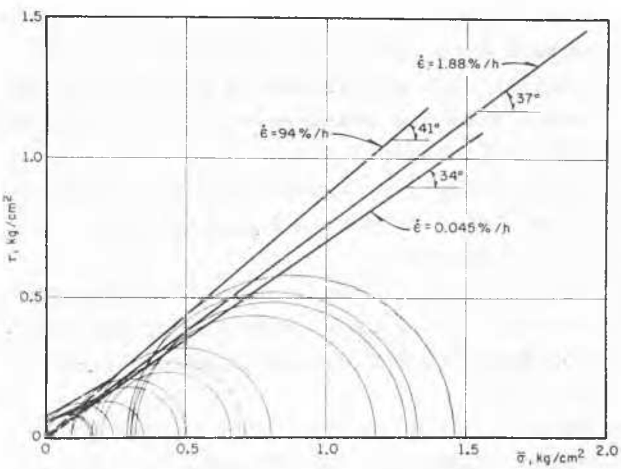

Fig.6 Mohr envelopes.Effective stresses

Here it may be seen that the apparent angle of friction obtained with the maximum principal stress difference criteria of failure falls from 41° to 34°, as the rate of strain changes from 94 per cent/h to 0.045 per cent/h. Both maximum shear strength and apparent angle of friction, in function of effective stresses,vary proportionately with the logarithm of time to failure (Figs 7 and 8). In Fig 8, there is also shown the variation in function of time to failure of the apparent angle of friction Φ',obtained with the maximum effective principal stress ratio criteria of failure. The difference between Φ and Φ' is small. To reach values of Φ or Φ' equal to 30° a time to failure of the order of 4 months would be necessary,according to the correalation given in Fig 8.This agrees with the value of 30° reported for drained tests which lasted from 3 to 5 months (Marsal,R.J.1969).

Using the conclusions related to the study of the pore pressures,the flow limits of these clays may be determined upon the hypothesis that the maximum value of the coefficient of pore pressure A_f^* is equal to 1 for confining pressures greater than preconsolidation stress. If A_f^* =1, then $\sigma_1 - \sigma_3 = u$, that is $\bar{\sigma}_1 = \sigma_3$. But $\frac{u^*f}{\sigma_3}$ =0.65 and $\bar{\sigma}_3 = \sigma_3 - u^*$ =0.356 , from which $\frac{\sigma_1}{\sigma_3} = tg^2\left(\frac{\pi}{4}+\frac{\Phi'}{2}\right) = \frac{1}{0.35} = 2857$ Thus Φ' =29°

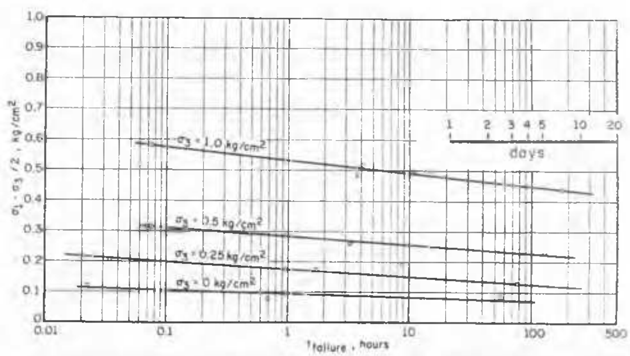

Fig.7 Maximum deviator stress versus time to failure

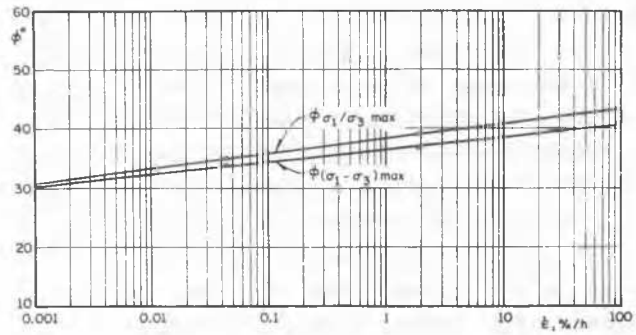

Fig.8 Effective friction angles versus axial strain rate

In conclusions, it would seem reasonable to assume that the clay behaves like a Bingham body (Fig. 9) with an apparent minimum angle of friction Φ' of 29°.

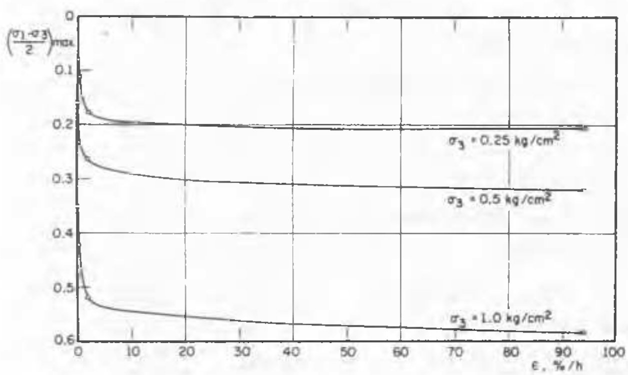

Fig.9 Maximum deviator stress at failure versus axial strain rate

4.CONCLUSIONS

Consolidated undrained triaxial tests carried out upon Mexico City clays at different rates of strain allow it to be stated that:

1.The magnitude of axial failure strain is independent of the rate of strain applied .

2. The relationship between pore pressure, corrected for the effects of filtration through the membrane, and axial strain is independent of the rate of strain, particularly over the interval of failure strains.

3. The magnitude of the pore pressure at failure varies linearly with consolidation stress and is independent of the strain rate.

4. The coefficient of pore pressure A_f increases with a decrease in the strain rate.

5. This clay behaves like an essentially cohesive material, in Hvorslev's sense, with a true angle of friction close to zero.

6. The apparent angle of friction, Φ, in function of the effective stresses, falls from 41° to 34° when the rate of strain varies from 94 to 0.0045 per cent/h, in consolidated undrained triaxial tests.

7. Assuming that in the long term the maximum value of the pore pressure coefficient A_f^* is equal to 1, then the minimum apparent angle of friction Φ', in function of the effective stresses, is equal to 29°. This finding agrees with the results of long term drained tests.

5. ACKNOWLEDGEMENTS

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6. REFERENCES

1. Gibson, R.E., 1963. "An analysis of system flexibility and its effect on time-lag on pore water pressure measurements", Geotechnique, London, Vol. 13, pp 1-11.

2. Girault, P., 1964, "Mineralogia de las arcillas del Valle de Mexico, Ingenieria, Mex. Vol. 34, p.511-519.

3. Lo, K.Y., 1962, "Shear strength properties of a sample of volcanic material of the valley of Mexico", Geotechnique, London, Vol.12 p.303-319.

4. Marsal, R.J., & Salazar, J.1960, "Pore pressure and volumetric measurements in triaxial compression tests", Research Conference on Shear Stength of Cohesive Soils, ASCE, Boulder, Colorado, pp. 965-983.

5. Marsal, R.J., & Mazari, M. 1969. "The subsoil of the City of Mexico", Universidad Nal. Autonoma, Mexico.

6. PoulosS.J.,1964,"Control of leakage in the triaxial test" Harvard Soil Mechanics Series, N° 71, Cambridge,Mass.
7. Proyecto Texcoco,1969. "Memorandum of works undertaken and conclusions",Secretaria de Hacienda,Mexico.
8. Resendiz,D., 1964 "Propiedades mecanicas de los suelos de cimentacion de la laguna Zumpango",Instituto de Ingenieria,UNAM,Mex.
9. Santoyo,E., & Resendiz,D., 1971 "A precision triaxial cell", Journal of Materials ASTM, Vol 6, pp 788-799
10. Schmertmann,J.H.,1963, "Generalizing and measuring the Hvorslev effective components of shear resistance" ASTM, Special Technical publication No 361, pp 147-157.