Creep behaviour of an undisturbed lightly overconsolidated clay
Le comportement face au fluage d'une argile légèrement sur consolidée

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ABSTRACT

The purpose of the investigation reported in this paper was to study the creep effects on stress-strain-time behaviour of Mexico City lacustrine sediments. The studied site studied is localized in the Central Park, the soil specimens were obtained from 7.3 and 22.6 m depth. The paper describes the results of 8 creep tests; isotropically consolidated specimens to consolidation pressure σ'_o= 80 kPa; the creep stage caused sustained stress were applied for over 10,000 minutes. For depth 7.3 m were applied shear stress levels D=36%, 58% and 93% and for depth 22.6 m were applied shear stress levels D=22%, 37% and 56%. The test results show that the influence of creep in the destructured range (σ'_y/σ'_o > 1) is not important at low shear stress levels, D. Creep rupture was not present in the carried out tests. The post-creep strength not clearly shows tendency to increase or decrease by comparing with reference strength before σ'_y. The parameters shown: *m*, α and A.

RÉSUMÉ

L'objectif de la recherche renseignée dans cet article, consistait à étudier l'effet du fluage sur les sédiments lacustres de la ville de Mexique. Le site étudié est localisé au Parc Alameda Central, Les spécimens de sol ont été extraits de 7,3 et 22,6 m. de profondeur. Le résultant de 8 essais de fluage, consolidés isotropiquement avec un effort de consolidation σ'_o= 80 kPa, est décrit. L'étape de fluage soumis aux efforts maintenus a duré 10000 min. Pour la profondeur de 7,3 m, des niveaux d'effort D=36%, 58% et 93%, ont été appliqués et pour la profondeur de 22,6 m. on a appliqué les efforts D=22%, 37% et 56%,. Les résultants montrent que l'influence du fluage à la branche structurée (σ_y/σ'_o > 1), est peu importante sous de hauts niveaux d'effort D. Les essais n'ont pas montré de rupture dû au fluage. La résistance post-fluage ne montre pas de tendance à augmenter ou à réduire quant à la résistance de référence avant du σ'_y. Les paramètres *m*, α et A sont montrés.

Keywords : Creep, strain rate, normal strength, creep rupture, post-creep strength, yielding, Mexico City.

1 INTRODUCTION

This paper presents experimental results for characterizing the creep effects on stress-strain-time response of the Mexico City lacustrine sediments.

Contributions for the creep characterization are reported by Murayama and Shibata (1961), Christensen and Wu (1964), Vyalov and Meschyan (1969), Borja (1992).

This phenomenon was studied by Singh and Mitchell (1968), Mitchell et al. (1968), Singh and Mitchell (1969) they developed a semi-empiric model for characterizing the interrelation between creep rate (deformation), stress level(D) and time (t).

Casagrande and Wilson (1951) reported creep studies under a sustained load and its effect on shear carried out Mexico City lacustrine sediments. The tests were performed under unconfined compression conditions and after this creep strength was observed at constant water content; the shear strength of Mexico City clay was reduced to about 80 per cent of its normal value in about 30 days.

The time-dependent responses of soils may assume a variety of forms depending on such factors as soil type, soil structure, stress history, drainage conditions, type of loading, and other factors. Fig. 1 (modificated from Singh and Mitchell 1968), ilustrates the wide variety of creep curves that may be exhibited by different soils.

Because of space limitations, only some of the more important aspects of the literature are presented. Fig. 2 (modificated from Mitchell 1976) shows basic behaviour of undrained creep tests. Fig. 2a shows a curve of strain versus time curve for three different tests with different stress level, D. The stress level, D, is a fraction of the undrained stress causing failure in standard test (Mitchell 1976). The specimen 1 exhibits only "primary" creep, as the strain decreases with time. Specimen 2 first undergoes primary creep, and then exhibits "secondary" creep, as the strain rate becomes almost constant. Specimen 3, at higher stress level, exhibits "tertiary" creep where strain rate accelerates, eventually ending in a creep rupture. Fig. 2b and 2c show the basic diagram used for presentation creep dates and evaluation parameters *m*, α and A, according to Sing and Mitchell (1968).

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2 MATERIALS PROPERTIES AND EXPERIMENTAL METHODS

2.1 *Site location*

The site from where the specimens were sampled is located in the lacustrine zone of Mexico City. The selected site (19.26°N, 99.08°W) is located in the Central Park (Alameda), one of the most damaging area during the 19 September 1985 Mexico City earthquake (Ms magnitude 8.1). The properties of Mexico City lacustrine sediments are usually variable from place to place and with depth as well (Díaz-Rodríguez et al. 1998). The tests were performed for material obtained from two different depths: 7.3 m and 22.6 m.

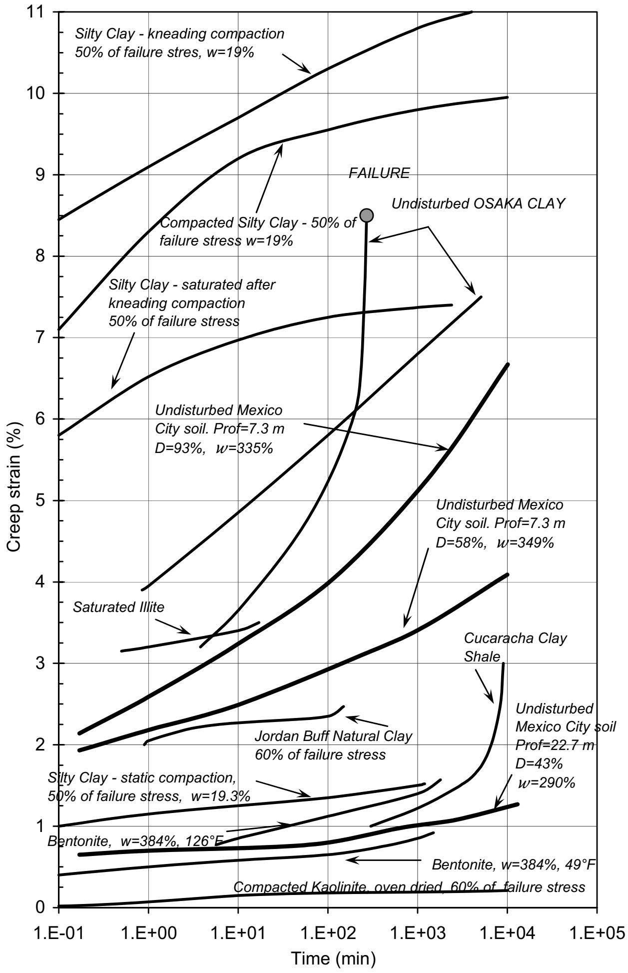

Fig. 1. Creep curves exhibited for differents soils (Modificated from Singh and Mitchell 1968).

Some physical properties of the soil specimens are indicated in Table 1. In the following, the term Mexico City lacustrine sediments, refers only to the soil that was tested in this investigation.

Table 1. Physical properties of the testing soil.

Depth	w_{nat}	w_L	w_P	I_P	SUCS	e_o	G_s
m	%	%	%	%			
7.3	442	473	79	394	CH	10.4	2.5
22.6	329	346	117	230	MH	7.8	2.4

2.2 *Laboratory testing*

The creep tests were conducted on specimens isotropically consolidated by increments using triaxial-cell method. The cylindrical specimens were carefully trimmed resulting in 36 mm diameter and 80 mm height.

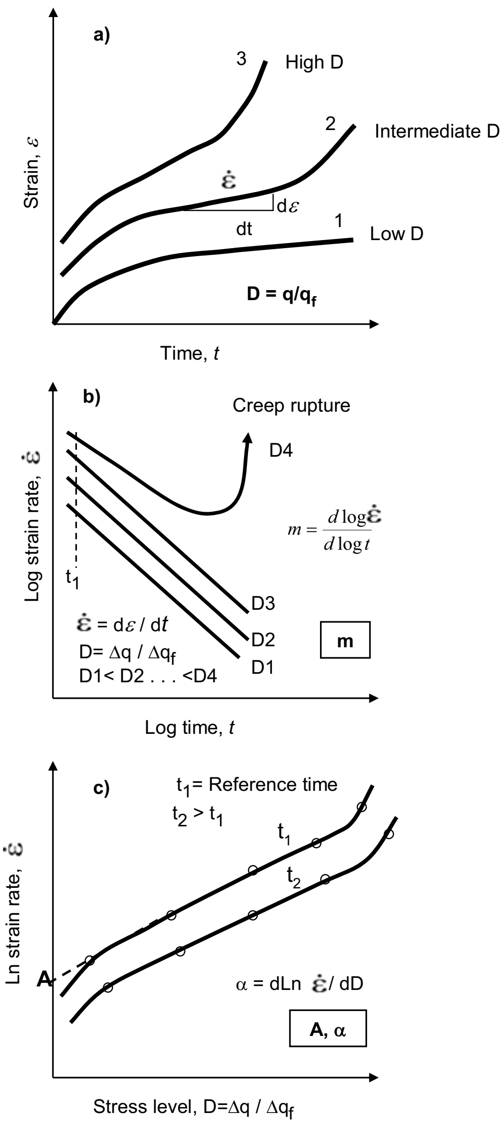

Fig. 2. Tests undrained creep basic behaviour plots (Modificated from Mitchell 1976).

The base pedestal, upon which the specimens were placed, is connected to a drainage line. The specimens were encased in two latex membrane separated by a film of silicon oil. Filter paper strips were used along the length of the specimen to accelerate drainage. The cell was equipped with a ball-bearing air bushing to reduce friction along the piston. In all tests the back-pressure was maintained at 230 kPa, the measured Skempton B-value exceeded 0.97. The tests were consolidated to confining pressure of 80 kPa, the which were is close to the yielding stress, σ'_y, previously determinated (Díaz-Rodríguez et al. 1992). Drainage was permitted during all phases of consolidation at each stage.

The standard strength or reference test was carried out by incremental load, obtaining a reference strength value. Other way to obtained reference strength is developing strain rate tests at 0.1%/h to 10.0%/h, under undrained triaxial conditions. Both of this procedures cause failure of the specimen within a standard time interval of 3 minutes to 50 hours.

The creep stage was conducted at the end of consolidated stage with duration of 10,000 minutes, setting masses of 4, 6.5 and 9 kg. A post-creep test was carried out to determine strength for each specimen.

3 RESULTS AND INTERPRETATION

A summary of tests results is illustrated in Table 2. The influence of variation in the stress level, D, for each depth is shown in Fig. 3a. For 7.3 m of depth the stress levels applied were D=36%, 58% and 93%; while for 22.6 m of depth D=22%, 37%, 56% were applied. For the applied stress level, D, and for both depths studied the log strain rate decreases linearly with the log time without creep rupture. This behaviour is different compared to the generalized behaviour of other soils where the strain-rate effects or load-strain are increased with plasticity.

Figs. 3a and 3b present tests results of lightly overconsolidated specimens (σ'_y/σ'_o =1.06 up to 1.25); they also show creep parameters: *m*, A and α obtained from triaxial tests used in the rate process theory equation of Mitchell (1976), [strain rate=$Ae^{\alpha D}(t_1/t)^m$].

The creep effect on post-creep strength is indicated in Fig. 4. A post-creep strength do not shows any tendency at loss of strength after creep comparing with the reference strength obtained of incremental load testing, (D=100%), and suggests that reference strength may be obtained with strain rate tests at interval of 0.1%/h up to 10.0 %/h.

4 DISCUSSION

The parameter m=1.0 indicates low potential of loss of strength in the present creep rupture case (Mitchell 1976). For the tested soil the parameter m=0.97, and all specimens neither present creep rupture nor any tendency to increasing or decreasing strength in the post-creep tests, this material response is strongly influenced by the natural soil structure where physical-chemical changes take place in the fabric open flocculated-random of soil studied (Díaz Rodríguez 2003). Under experimental conditions of soil tested the m =0.97 value indicates that strength do not change respect reference strength; these results are different regarding to the results reported by Casagrande and Wilson (1951). The parameters m, α and A agree with value range reported from others types of soils independent of the type of test and soil properties.

The post-creep tests show that for the deviatoric stress failure $(\sigma'_1-\sigma'_3)_{máx}$ the axial strain of failure ε_f, depends on the stress level, D. This behaviour differs from controlled strain rate tests where the axial strain of failure ε_f, is essentially independent of the rate of strain (Vaid and Campanella 1977).

Table 2. Results of the triaxial undrained creep tests.

Test No.	D	m	α	A	$(\sigma_1-\sigma_3)_{ref}$	$(\sigma_1-\sigma_3)$ post-creep	ε_f
	%			min^{-1}	kPa	kPa	%
Depth=7.3 m, σ'_o=80 kPa							
1	100	- -			105	- -	5.2
2	36	0.9			- -	112	5.0
3	58	0.9	6.2	$7x10^{-2}$	- -	118	6.9
4	93	0.9			- -	115	8.2
Depth=22.6 m, σ'_o=80 kPa							
5	100	- -			133	- -	3.5
6	22	0.9			- -	113	2.9
7	37	0.9	6.2	$7x10^{-2}$	- -	130	3.0
8	56	0.9			- -	108	5.2

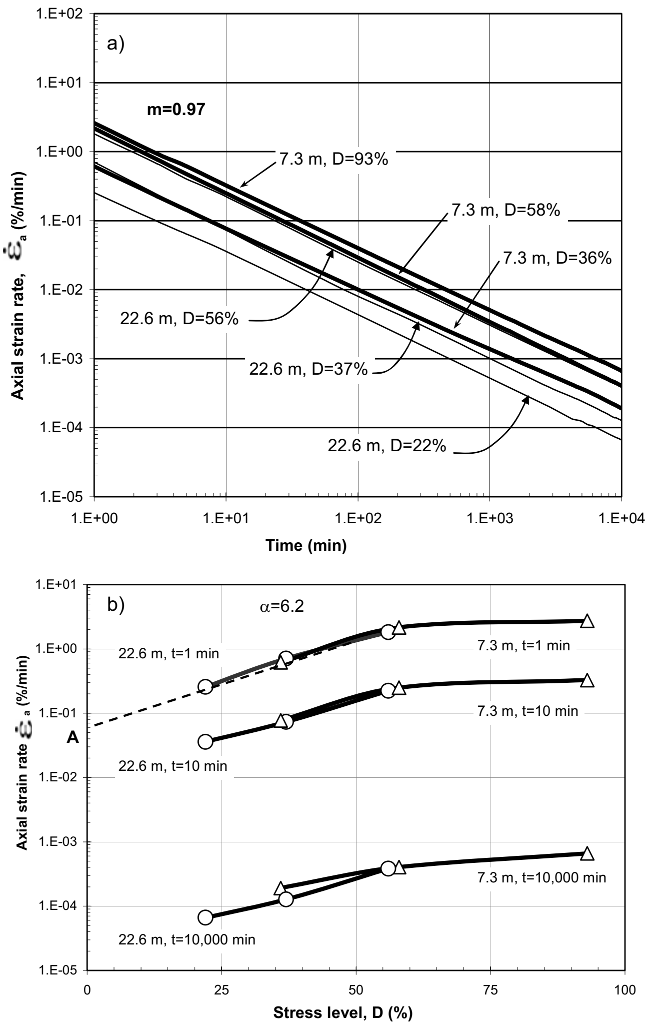

Fig. 3. Results of triaxial undrained creep on Mexico City soils.

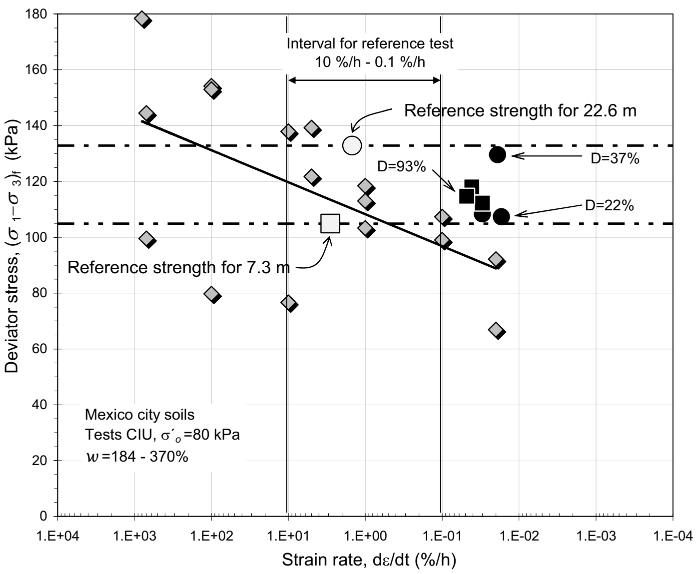

Fig. 4 Effect of the undrained creep on post-creep strength.

5 CONCLUSIONS

This paper describes results from 8 consolidated-undrained triaxial creep tests, developed on Mexico City lacustrine sediments. Specimens were consolidated at confinement pressure, σ'_o=80 kPa and subjected to different stress levels, D; Due the specimens do not present creep rupture at the end of the creep stage, they were conducted to post-creep condition for determining post-creep strength.

The following conclusions were obtained from this study:

- Mexico City lacustrine sediments, which have high plasticity, not experienced creep rupture at high stress levels, D=93%
- A creep typical behaviour has been observed in all specimens; strain rate decreases linearly with log scale of time without show creep rupture .
- The parameter m= 0.97, is similar to others values obtained for clays.
- The parameters A and α were evaluated in 7x10^{-2} min^{-1} and 6.2, respectively.
- Soils with values of m=1 seem to exhibit the same strength before and after creep (Singh and Mitchell 1969), the results of post-creep strength do not exhibit tendency either decrease or increase post-creep strength regarding to a reference strength.
- The scatter in results is attributed to heterogeneity of the specimens and testing in the structured domain of soil ($\sigma'_o < \sigma'_y$).
- The axial strain failure, ε_f for post-creep strength is dependent of the stress level, D.
- For Mexico City lacustrine sediments, the soil structure and their implications play a very important role before creep behaviour.

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