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To cite this version:

Bernardo Caicedo, Cristhian Mendoza, Arcesio Lizcano, Fernando Lopez-caballero. Some contributions to mechanical behaviors of lacustrine deposit in Bogotá, Colombia. Journal of Rock Mechanics and Geotechnical Engineering, 2019, 11 (4), pp.837-849. 10.1016/j.jrmge.2018.12.016 . hal-02119573

HAL Id: hal-02119573

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Journal of Rock Mechanics and Geotechnical Engineering 11 (2019) 837–849

Contents lists available at ScienceDirect

Journal of Rock Mechanics and Geotechnical Engineering

journal homepage: www.rockgeotech.org

Full Length Article

Some contributions to mechanical behaviors of lacustrine deposit in Bogotá, Colombia

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ARTICLE INFO

Article history:
Received 12 May 2018
Received in revised form
18 October 2018
Accepted 28 December 2018
Available online 3 May 2019

Keywords:
Cyclic triaxial tests
Triaxial compression test
Time-dependent response
Lacustrine soils
Soil compressibility
Yield curve

ABSTRACT

This paper presents the mechanical behaviors of the lacustrine deposit, a representative soil in Bogotá, Colombia. Initially, the physical characterization of the deposit is performed via laboratory tests (grain size distribution, scanning electron microscopy, Atterberg limits and water content). This characterization intends to explain the special characteristics of the mechanical behaviors of this soil. Then, various triaxial tests are carried out with controlled loading path, strain rate change, relaxation, extensile stress, and cyclic loading. The test results reveal the shape of the yield curve for Bogotá soil (in a natural state), and also show that an increasing effect of the strain rate depends on the liquid limit. This effect is also preserved with extensile stresses (which are poorly studied in soil mechanics). Finally, other effects, such as the loss of structure in the reconstituted samples and the effect of shear modulus at low strains, are studied for Bogotá soil.

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1. Introduction

The lacustrine deposit in Bogotá (Colombia) is located on a high plateau of the Andes Mountains at 2550 m above sea level. More than 60% of the city area is located on soft soil deposits. At some sites of the plateau, the depth of the lacustrine deposit can reach 586 m (Torres et al., 2005). Shallow soil deposits at depths of 5–10 m are overconsolidated, but in deeper layers, the soil can reach extremely high values for some geotechnical properties. For example, this soil has high Atterberg limits (nearly 400% for the liquid limit), water contents at around 200%, void ratios of nearly 5, the coefficient of compressibility close to 5, and a consistency index lower than 0.5. Furthermore, this soil has a large amount of microfossils in its structure. Díaz-Rodríguez (2003, 2011) studied natural lacustrine deposits and showed that the presence of microfossils in the clay of Mexico City can modify its shear strength, suggesting its high plasticity index. Although Bogotá soil has the same lacustrine origin as the soil of Mexico (Moya and Rodríguez, 1987), the knowledge of this soil is still limited. Indeed, the mechanical behaviors of clayey soils and specific soft soils have been extensively studied by Roscoe et al. (1958), Tavenas et al. (1979), Burland (1990), Leroueil and Vaughan (1990), Beaumelle (1991), Tatsuoka et al. (2000), Diaz-Rodriguez (2003, 2011), and Horpibulsuk et al. (2011), among others. These researches focus on soils located in London, cities of Canada, Tokyo, Mexico City, Boston, and Bangkok, among others. Solutions for geotechnical works placed on these soils are extrapolated to soils in other parts of the world. Regarding the city of Bogotá, geotechnical structures need to deal with a soil having low bearing capacity and undergoing excessive settlements. Considering the previous studies, the mechanical behaviors of the soil of this type require more research.

The soils with microfossils (diatom frustules) in natural state have been presented by Tanaka and Locat (1999), Díaz-Rodríguez (2003), Kwon et al. (2011), Suganya and Sivapullaiah (2017), and Caicedo et al. (2018), among others. These soils have counter-intuitive properties for the majority of soils in the world, for example, high plasticity indices and high friction angles. For this reason, some authors have conducted controlled tests on mixtures of soil and diatoms to study more clearly the effect of diatoms on mechanical properties. Researches on mixtures of fine soil and diatoms by Shiwakoti et al. (2002), Díaz-Rodríguez (2011), Frýbová

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Peer review under responsibility of Institute of Rock and Soil Mechanics, Chinese Academy of Sciences.

https://doi.org/10.1016/j.jrmge.2018.12.016
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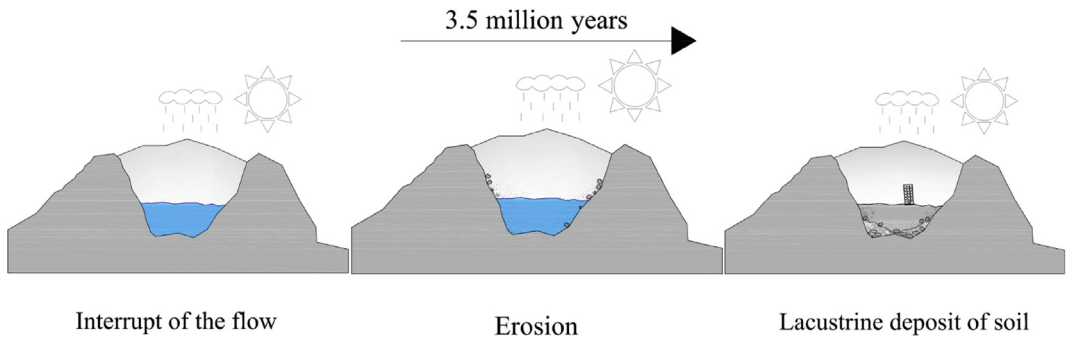

Fig. 1. Formation of lacustrine deposit.

et al. (2017), Wiemer and Kopf (2017), and Wiemer et al. (2017) show that the increment in diatom content raises the liquid limit, water content, void ratio, coefficient of compressibility, and friction angle. However, these researches do not necessarily address the problems that the geotechnical engineer has to face, as the geotechnical structures are generally resting on undisturbed soils.

The effect of stress increase due to a change in strain rate is up to 14% in relation to the reference stress. In addition, the increasing effect of strain rate on the soil is a function of the liquid limit; this effect is preserved with extensile stresses (which are poorly studied in soil mechanics). Also, the yield curve shape of the Bogotá soil (in natural state) is shown. Other effects on Bogotá soil are the structure loss of soil in the reconstituted sample and the effect of shear modulus at low strains (see Nakagawa and Soga, 1995). This work presents the behaviors of the Bogotá soft soil based on an experimental program for testing undisturbed and reconstituted samples. The experimental program includes, initially, the physical characterization of the lacustrine deposit via basic laboratory tests (grain size distribution, scanning electron microscopy (SEM), Atterberg limits and water content). This characterization intends to explain the specific mechanical behaviors of this soil. Subsequently, the experimental work continues with triaxial tests with controlled loading path, strain rate change, relaxation, extension stress and cyclic loading. In addition, several practical correlations are adapted for this soil type for shear moduli and shear strain at low strains.

2. Location and characterization of lacustrine deposit

The Bogotá City, the Colombian capital, is located at latitude 4°N and longitude 74°W with an average altitude of 2550 m in the central area of the country. The central area of the country is a high plain with a lacustrine soil deposit made of very soft soils. Its origins lie in a Pliocene and Pleistocene lake that is filled over time. The deep deposit soils are assumed by subsidences from the bottom of the basin and gradual accumulation of mainly lacustrine sediments during the last 3 million years (Hooghiemstra and Sarmiento, 1991). The chronostratigraphy of the deposit was studied by Andriessen et al. (1993) using the fission track-dating method. They estimated the soil's age of 3.2 Ma approximately at a depth of 586 m. In addition, chronostratigraphical data were provided by Torres et al. (2005) based on pollen analysis. The tectonic processes associated with the rise of the Andean Cordillera created a large lake, leading to a lacustrine deposit (Fig. 1). The whole process of soil deposition took place in a lapse covering about 3.5 million years until today (Torres et al., 2005).

Some of the deposition processes that have taken place in the basin of Bogotá include the deposition mechanism which can be explained by the water table controlled by the differential rate between the subsidence of deposit bottom and sedimentation. Possible deposition mechanisms are: (i) sedimentation of predominantly fine grained materials in a low energy regime taking place under deep water; (ii) sedimentation of predominantly silt and sand in a high energy regime occurring in low water tables near river mouths; and (iii) swamp and fluvio-lacustrine deposits with high proportions of organic matter in shallow water tables.

Geotechnical study of the basin presented in this research concerns the upper 50 m of the deposit corresponding chronologically to the last 250,000 years. There are some sporadic episodes of swamp deposits with high proportions of organic matter. This corresponds to alternating rises and falls of the water table. At the same time, the mean value of the liquid limit increases from 50% to 200%. In this deposit, the mean percentage of particles with sizes less than 2 μm fluctuates from 0% to 20%. From 50 m deep to the surface, the chronological measurement carried out by Torres et al. (2005) found that the deposition rate is 3440 yr/m. Nowadays, the deposit soil has an overconsolidated surface in the first 5–10 m, and then presents a very soft consistency ($I_c < 0.5$). Also, the soil presents an unusual structure compared with classical clayey soils due to the presence of microfossils (diatomaceous) in its structure. Diatomaceous microfossil is one example of natural micro- and nano-particles that create a soil with structure modifying the mechanical behaviors of the soil. Indeed, soils with diatomaceous particles do not follow most of the useful geotechnical

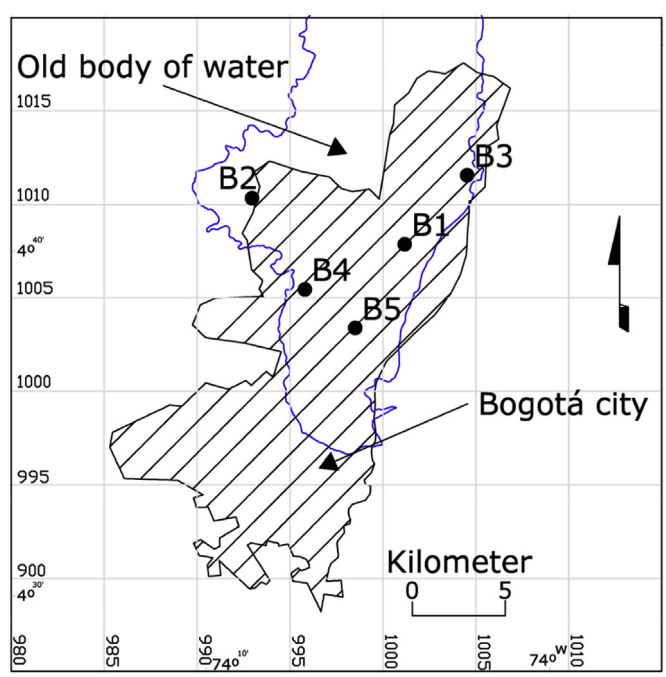

Fig. 2. Old body of water, Bogotá City, and location of undisturbed samples.

correlations linking index properties and stress–strain parameters (Diaz-Rodriguez, 2011).

Geotechnical exploration was conducted in field by drilling, and samples were derived respectively from the downtown zone, north zone, northeast zone, and northwest zone (Fig. 2). The drillings were made to the depths more than 40 m and the soil blocks up to 3 m. Note that all samples obtained were from the lacustrine deposit. Drilling samples were obtained in undisturbed state by using the stationary piston technique. The main reason for the depth of the drillings was that most geotechnical structures based on this type of soil were found in the first 50 m depth.

The mineralogical composition of the lacustrine soil includes 50% quartz, 15%–42% feldspars, and 15%–30% clay which was predominantly smectite. Other minerals such as mica and vermiculite appear in less quantity. In addition, the organic matter content is in the range of 6%–10% in the first 5 m and then increases to around 25% at 30 m depth. For deeper layers, the organic matter content decreases gradually. This mineralogical composition was obtained for the B1 zone. Table 1 shows the content of elements from the B3 zone. Fig. 3 shows some particle size distribution by laser granulometry tests from the B1 zone.

Images in Fig. 4 were obtained using SEM on an undisturbed sample of B1 and B3 drillings. Fig. 4a and c shows a large number of diatoms. Also, it is rather difficult to characterize a typical soil structure with high diatomaceous concentration based on classical soil mechanics (Fig. 4b and d). Fig. 4 shows an isotropic structure (non-oriented structure), which is formed by an aggregate of flocculated particles and a large inter-aggregate porosity. This structure is similar to that found by Sides and Barden (1971), but without diatoms, which is typically a structure that characterizes a slow deposition, as is the lacustrine deposit of Bogotá. Fig. 4 shows that diatoms are intact (Fig. 4c) or broken (Fig. 4e and f), and the highly porous materials account for the high water content of these soils.

Fig. 5 presents the average values of the drillings at 50 m, i.e. liquid limit, water content, plastic limit, consistency index, and undrained cohesion. The standard deviation ranges of the above features were as high as 12%–99% for the liquid limit, 2%–33% for the plastic limit, 14%–97% for the water content, and 4–46 kPa for the undrained cohesion. Also, it is noted that the high variation of the indices mentioned above was due to the geological processes of filling the lake which formed the lacustrine soil of Bogotá. It also describes the general division for the layers (Fig. 5d) at the site (based on the classification of Lobo-Guerrero (1996)), in accordance to the following depths:

(1) 0–1 m: Anthropic fills, brown color, high in organic matter;
(2) 1–5 m: Oxidized silty clay, soft consistency ($0.5<I_c<0.75$), gray and brown colors, plasticity index of 80%, liquid limit around 120%, water content of 80%, and plastic limit around 40%, with water level around 1.5 m;
(3) 5–23.5 m: Silty clay, orange color, very soft consistency ($I_c<0.5$), plasticity index around 120%, liquid limit of 140%, water content between 80% and 140%, and plastic limit around 30%;
(4) 23.5–40 m: Silty clay, yellow color with peat, very soft consistency ($I_c<0.5$), plasticity index of 90%, liquid limit of 140%, water content between 120% and 140%, and plastic limit between 40% and 50%;
(5) Deeper than 40 m: Light brown silt, plasticity index of 90% (equal to the previous layer), liquid limit of 140%, water content of 120%, plastic limit of 42%, and soft consistency ($0.5<I_c<0.75$).

Table 1
Contents of elements from a sample of B3 drilling.

Element	Symbol	Content (%)
Carbon	C	14.34
Oxygen	O	58.05
Magnesium	Mg	0.39
Aluminum	Al	7.02
Silicon	Si	16.9
Sulfur	S	0.59
Potassium	K	0.58
Calcium	Ca	0.51
Iron	Fe	1.62

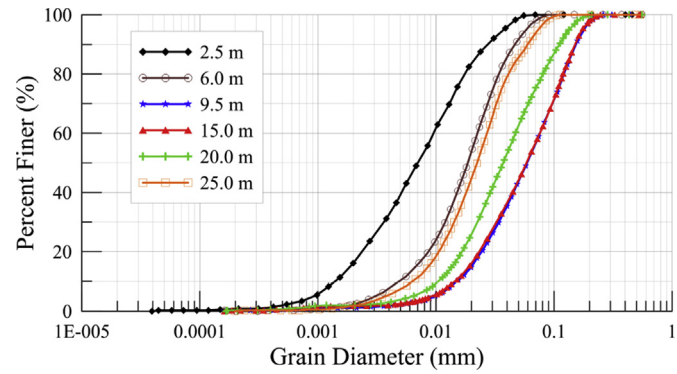

Fig. 3. Distribution of particle size by laser granulometry tests.

3. Mechanical behaviors of lacustrine deposit

The mechanical characterization of the soil is divided into three parts: yield curve, time-dependent response, and behavior of small strains. This characterization is based on a triaxial test with controlled loading path, strain rate change, relaxation, extension stress, and cyclic loading. The experimental program includes four undrained triaxial tests and five triaxial compression tests with anisotropic stress paths in the yield curve; four triaxial compression tests with controlled loading path, strain rate change, and relaxation in the time-dependent response to drained compressive behavior; four undrained triaxial tests with strain rate changes in the time-dependent response to undrained behavior; and 23 cyclic triaxial tests and bender element tests at small strains.

3.1. Yield curve

The yield curve of Bogotá soil was explored following the same methodology used by Roscoe et al. (1958), Tavenas and Leroueil (1977), Graham et al. (1983), and Díaz-Rodrìguez et al. (1992). This methodology consists of undrained triaxial tests and consolidation tests following anisotropic paths.

As characterizing the yield curve requires a completely undisturbed sample, this study is done on a block of soil obtained at 3 m depth in the new "El Dorado" airport (B2). The characterization of the block is shown in Table 2. An explanation for the low water content and low limit liquid compared with the high values presented in this work is because the sediments are close to the surface and may be mixed with alluvial sediments having low liquid limit and low water content. Another point is that these sediments have suffered from some cycles of flooding and drying in their geological history in some parts of the Bogotá deposit (Lobo-Guerrero, 1996).

The yield curve expresses two kinds of behaviors: dilation and contraction. The difference between dilation and contraction can be observed from the excess angle, as shown in Bolton (1986) and Xiao et al. (2014). The excess angle is defined as the difference between

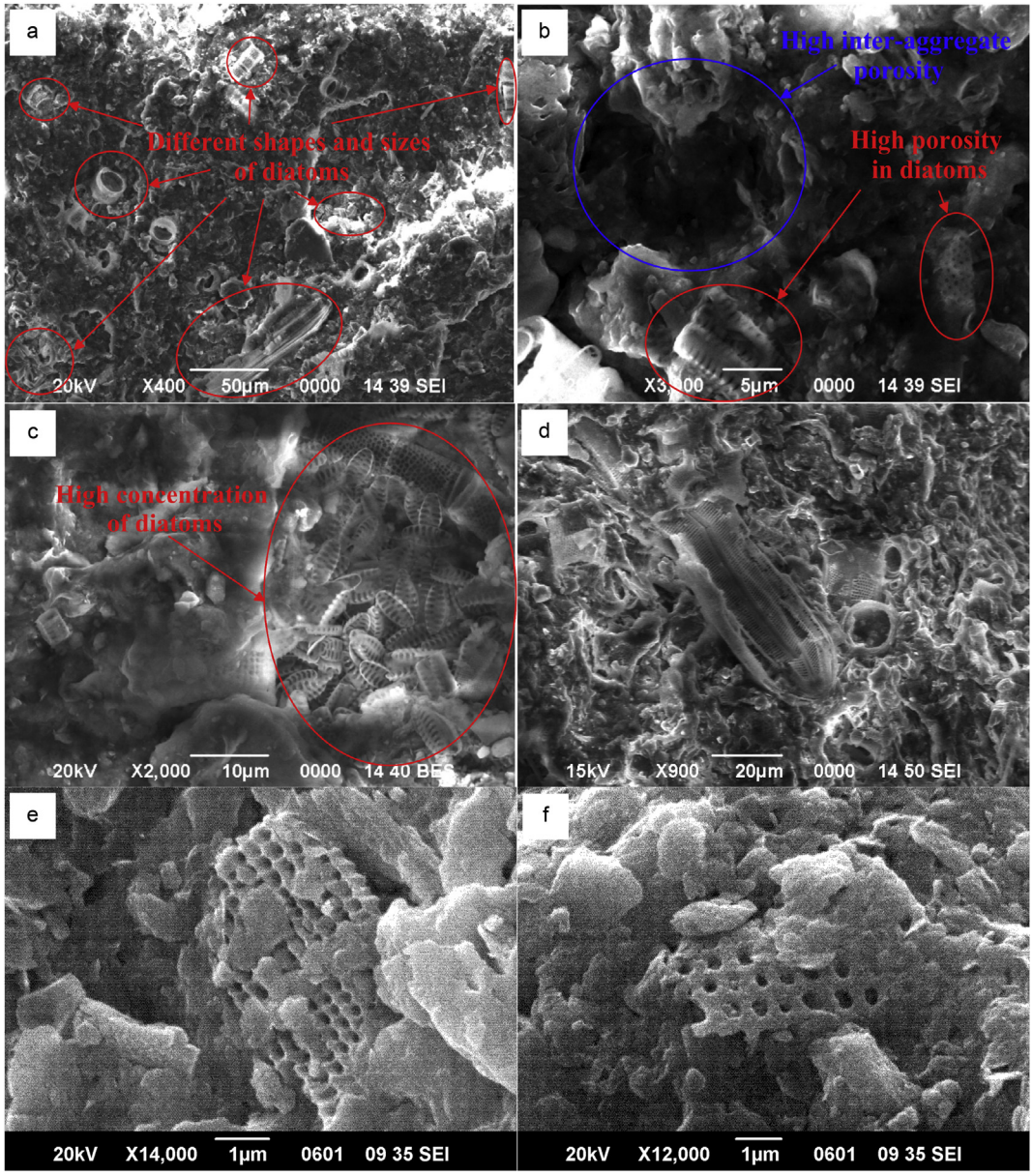

Fig. 4. SEM photos of an undisturbed sample from B1 and B3 zones.

the peak friction angle and the critical state friction angle. When the excess angle is equal to zero, it is demonstrating contraction; when the excess angle is greater than zero, the dilation appears. The dilation of the material was assessed using four triaxial tests. These tests were isotropically consolidated to a mean effective stress $p' = 100$ kPa and then overconsolidation ratio (OCR) values of 1.3, 1.5, 2 and 8 were considered. Fig. 6 shows the shear response measured in these tests, where q denotes the deviatoric stress and ε_1 the axial strain. The contraction behavior was studied using five anisotropic consolidation tests (CA) (see Fig. 7a). Prior to this, all tests were isotropically consolidated to $p' = 100$ kPa and then continued using anisotropic stress paths. The anisotropic consolidation was carried out using following equation:

$$\sigma_3 = \frac{\sigma_1(3-\eta)+3U\eta}{3+2\eta} \tag{1}$$

where σ_3 is the confining stress, σ_1 is the vertical stress, η is the stress ratio of q/p', and U is the excess pore pressure.

Eq. (1) was incorporated in a conventional triaxial test. However, as the yield stress is not well defined in the consolidation curves (Fig. 7c) in the plane of void ratio e and $\log_{10}p'$, the methodology proposed by Becker et al. (1987) and then updated by Beaumelle (1991) was used. This procedure is based on the calculation of the strain energy (work per unit volume), W:

$$W = \int \sigma'_v \mathrm{d}\varepsilon_v = \sum \sigma'_v \mathrm{d}\varepsilon_v \tag{2}$$

where σ'_v is the vertical effective stress and ε_v is the volumetric strain.

Next, the yield stress is determined using a bilinear method that is adjusted according to the intersection of two lines. The first line is the extension of the mean effective stress and the second line is the extension of the work per unit volume.

The yield stresses are obtained for each curve in Fig. 7b. Another important aspect is the variation of the normal consolidation slope C_C (Fig. 7c), which presents an unusual behavior considering the

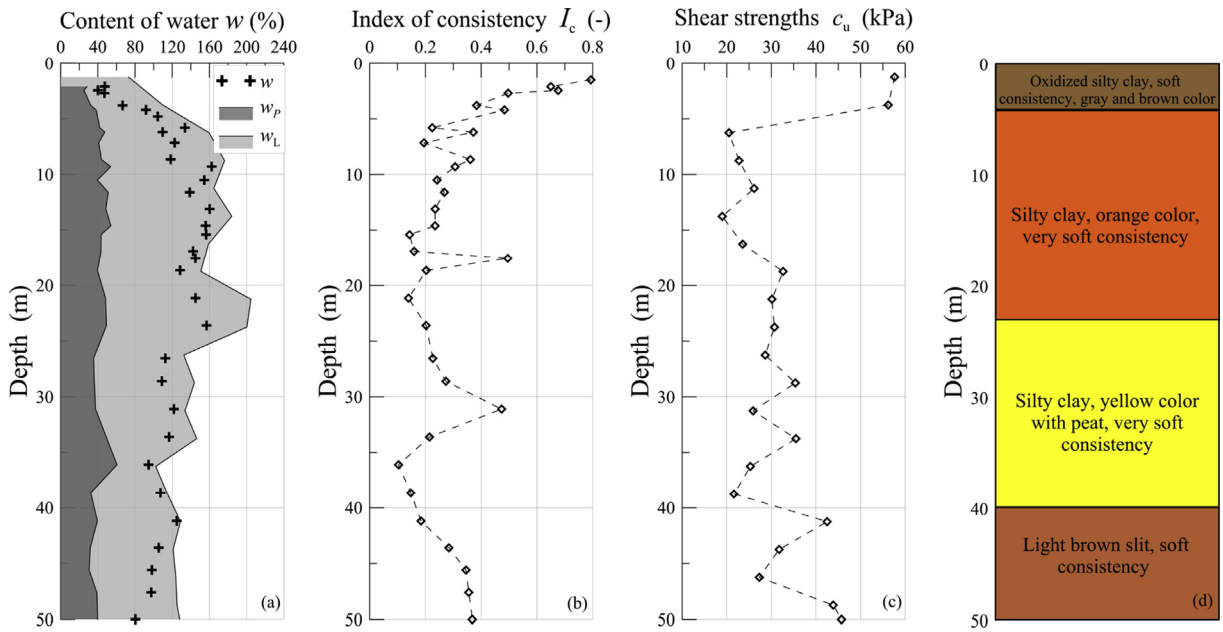

Fig. 5. Comparisons between (a) Plastic limit, liquid limit, and water content; (b) Consistency index; (c) Undrained shear strengths; and (d) Stratigraphy.

Table 2
Characterization of block of soil from "El Dorado" airport.

Water content, w (%)	Liquid limit, w_L (%)	Plastic limit, w_P (%)	Plasticity index, PI (%)	Unit weight (kN/m^3)	Content (%)		
					$d > 74$ mm	74 mm $> d >$ 2 mm	$d < 2$ mm
65.71	76	42	34	25.72	7.4	68.5	24.1

Note: d denotes the particle size.

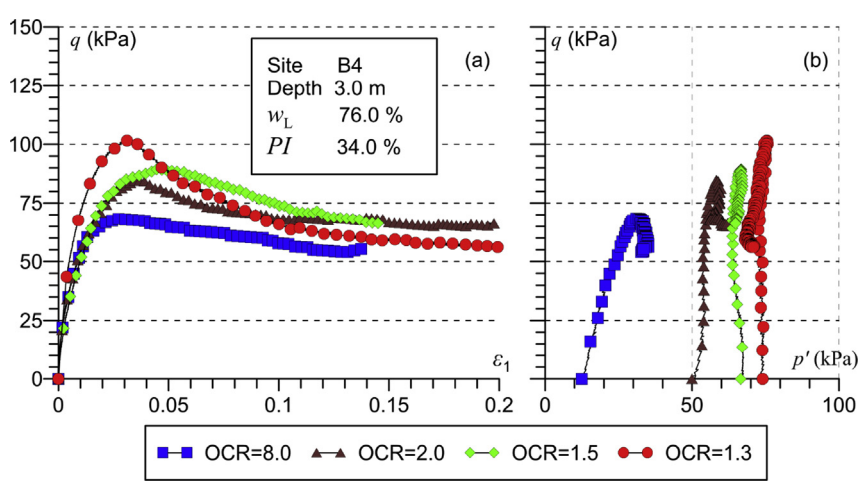

Fig. 6. Triaxial compression test (CIU) results of a sample obtained from 3 m depth.

fundamentals of classical soil mechanics (Roscoe et al., 1958; Tavenas and Leroueil, 1977; Graham et al., 1983). This behavior suggests that for these kinds of lacustrine soils, the normal consolidation slope C_C is not constant and can be considered as a state variable, as illustrated in Fig. 7d.

Finally, the yield curve of Bogotá soil in a natural state is plotted in Fig. 8. This curve was obtained by the combination of the surfaces of Roscoe and Hvorslev, as shown in Roscoe et al. (1958, 1963). The meaning of the curve is that all stress paths within the curve are in an overconsolidated state (elastic behavior), and all paths that touch the yield curve are in a normally consolidated state (elastoplastic behavior). This methodology has a shortcoming that not all points are of the same void ratio e. For this reason, the curve can be passed in different yield surfaces. However, the yield curve can have a similar shape to the yield surface (Díaz-Rodríguez et al., 1992). On the other hand, this methodology has been widely used to obtain the yield surface. The first surface proposals were made for reconstituted soils, as shown in Roscoe et al. (1958) and Cotecchia and Chandler (2000). However, the work presented by Cotecchia and Chandler (2000) showed that, when using natural or reconstituted samples, the size of the surface changes, but not its shape. Another point of the surface is the rotation about the

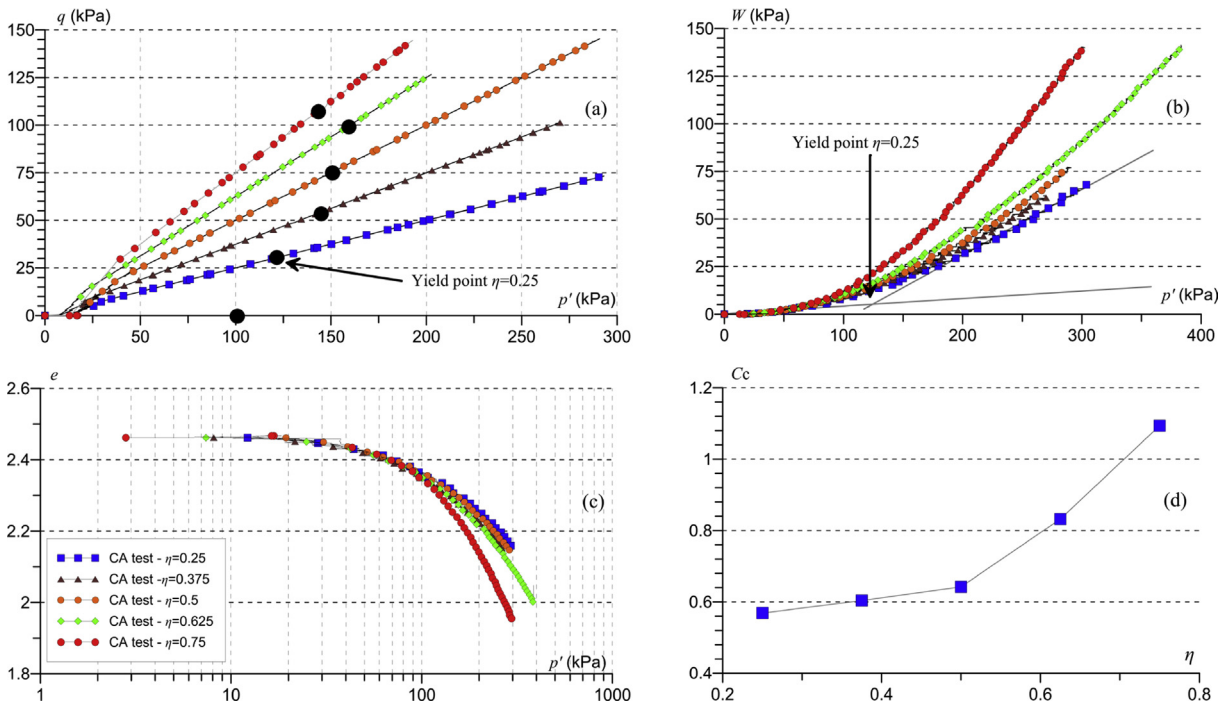

Fig. 7. Triaxial consolidation test results of a sample obtained from 3 m depth.

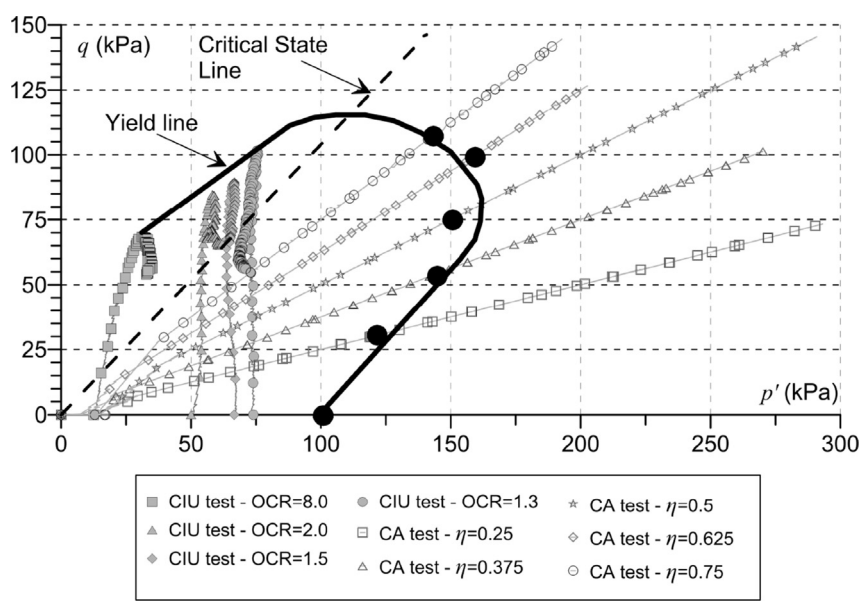

Fig. 8. Yield curve of a sample obtained from 3 m depth.

isotropic axis, which has also been found by Roscoe et al. (1958), Tavenas and Leroueil (1977), Tavenas et al. (1979), Graham et al. (1983), and Díaz-Rodríguez et al. (1992), among others. This shape appears due to the process of one-dimensional (1D) compression during the deposition of the soils, where the vertical stress is greater than the lateral stress. Therefore, the yield surface rotates at a stress ratio K_0 (coefficient of earth pressure at rest), as shown in Díaz-Rodríguez (2003).

3.2. Time-dependent response

As discussed previously, the strain rate study includes drained compressive behavior and the behavior of undrained shear stresses.

3.2.1. Drained compressive behavior

Four tests (CA) were performed to study strain rates in soils. The first test was performed with a stress ratio $\eta = q/p' = 0.5$ at a constant strain rate $\dot{s} = 0.02$ mm/min (Fig. 9a). During the test, the strain rate changes 10 times lower than the reference strain rate to $\dot{s} = 0.002$ mm/min. Furthermore, the test was relaxed (the state of stress changes without changing the volume) for 3 d until the derivative of the deviatoric stress with time approached zero, as shown in Fig. 9b and c. Fig. 9c shows the relaxation effect (red line) by letting the volume change equal zero ($\Delta Vol. = 0$) over time t; in other words, the deformation is constant over time ($\Delta e = 0$). Fig. 9b shows the stress gradually decreasing (red line). The red line in Fig. 9 represents the relaxation process in the test. Therefore, the

relaxation part in the test was characterized by constant total strain and decreased stress (Augustesen et al., 2004). Finally, Fig. 9d shows that consolidation at a strain rate 10 times lower than the reference rate leads to a normal consolidation line for that rate. This change in the normal consolidation line with strain rate was observed by Šuklje (1957) and Tatsuoka et al. (2000) and denoted by the term "isotachs".

The second test was an isotropic compression test ($\eta = 0$) with a mean effective stress up to $p' = 530$ kPa, followed by unloading to $p' = 0$ kPa (Fig. 10a and b). Next, an anisotropic compression test was performed at $\eta = 0.5$ (Fig. 10a). Also, strain rate changes were made during this test. Fig. 10b shows the influence of the strain rate on the soil. The shape of the compression curve is modified by changes in both the preconsolidated domain and the normal consolidation line following preconsolidation stress. Therefore, the preconsolidation stress decreases with the decrease in strain rate. From Fig. 10b, it can be observed that the faster the loading rate, the higher the effective stresses for a certain strain (Leroueil et al., 1985; Augustesen et al., 2004). In addition, Fig. 10b shows that the rate-dependent behavior in the overconsolidated range is small. Fig. 10c shows the volume of water drained from the soil over time. This figure shows clearly the effects of the strain rate and stress anisotropy ($\Delta Vol. = f(\check{s}, \eta)$) on the rate of drainage. Fig. 10d is plotted using the slope of volume variation against time in percent from the data obtained in Fig. 10c, and it is valid for the initial stage of the consolidation process.

The third and fourth tests were isotropic. They were performed on a sample with a liquid limit of 168%. The third test was conducted on an undisturbed sample at several strain rates. The fourth consolidation test was performed on a sample of the same soil that had been reconstituted at the same void ratio. Fig. 11a shows the loss of cementation and structure of the reconstituted sample which corresponds to a strain energy loss (Fig. 11b). Fig. 11c and d shows the variations in volume and mean effective stress against time. It is shown that the relaxation effect is presented in natural and reconstituted samples. However, the relaxation effect in reconstituted samples can be slightly less. Despite this, the viscosity effect of the reconstituted sample is preserved regardless of the loss of structure during deposition.

3.2.2. Undrained shear behavior

The effect of the strain rate on undrained shear behavior was studied with three triaxial compression tests (CIU) and one triaxial extension test with anisotropic consolidation (CAUE).

The first sample was consolidated to higher stresses than the preconsolidation stress; then the shear stress was applied at three strain rates: $\check{s}/10$, $\check{s}$ and $10\check{s}$, which were interchanged at every 2% of strain, as shown in Fig. 12. From this figure, it can be observed that changes in the shear stress depend on the strain rate. These results are similar to those obtained by Leinenkugel (1976) and Tatsuoka et al. (2000). The results show that the larger the strain rate, the stiffer the soil. Fig. 12b shows the strain rate effect on the yield surface. The stress path to normally consolidated clay tends to rise to the surface of Roscoe. The change of yield surface with strain rate was reported in Leroueil and Marques (1996).

The second test was performed on a preconsolidated soil with $OCR = 3$. The same strain path was applied up to 17% of the vertical strain; then the effect of the strain rate was studied during unloading, as shown in Fig. 13. This figure shows an isotach during unloading as proposed in Sorensen and Simpson (2007). This is an important issue because it was assumed that the effect of strain rate is only evident after the yield point (or in the elastoplastic domain); however, Sorensen and Simpson (2007) showed that the effect of the strain rate appears also in a small proportion in the elastic domain. For this, several constitutive models use the strain rate effect only in the elastoplastic part and ignore it in the elastic part, as shown in Hinchberger et al. (2010), Nakai et al. (2011), and Yang et al. (2016), among others.

The third one was conducted on an overconsolidated sample following the procedure of the above tests. Fig. 14 shows that the influence of the strain rate is small. Note that this sample was obtained from the naturally overconsolidated layer with a low liquid limit. This can be due to the low effect of strain rate. The triaxial tests were performed by Vaid et al. (1979) for overconsolidated

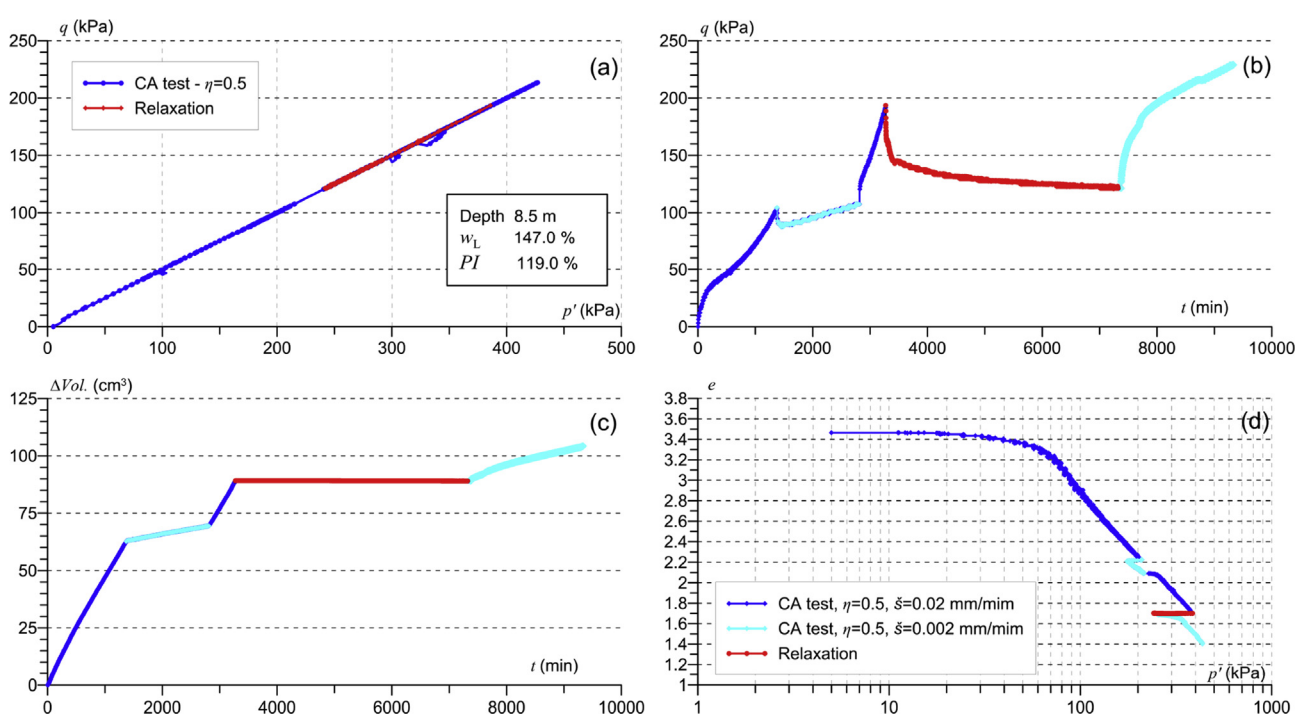

Fig. 9. Triaxial consolidation test results of a sample obtained from 8.5 m depth with stress ratio $\eta = 0.5$.

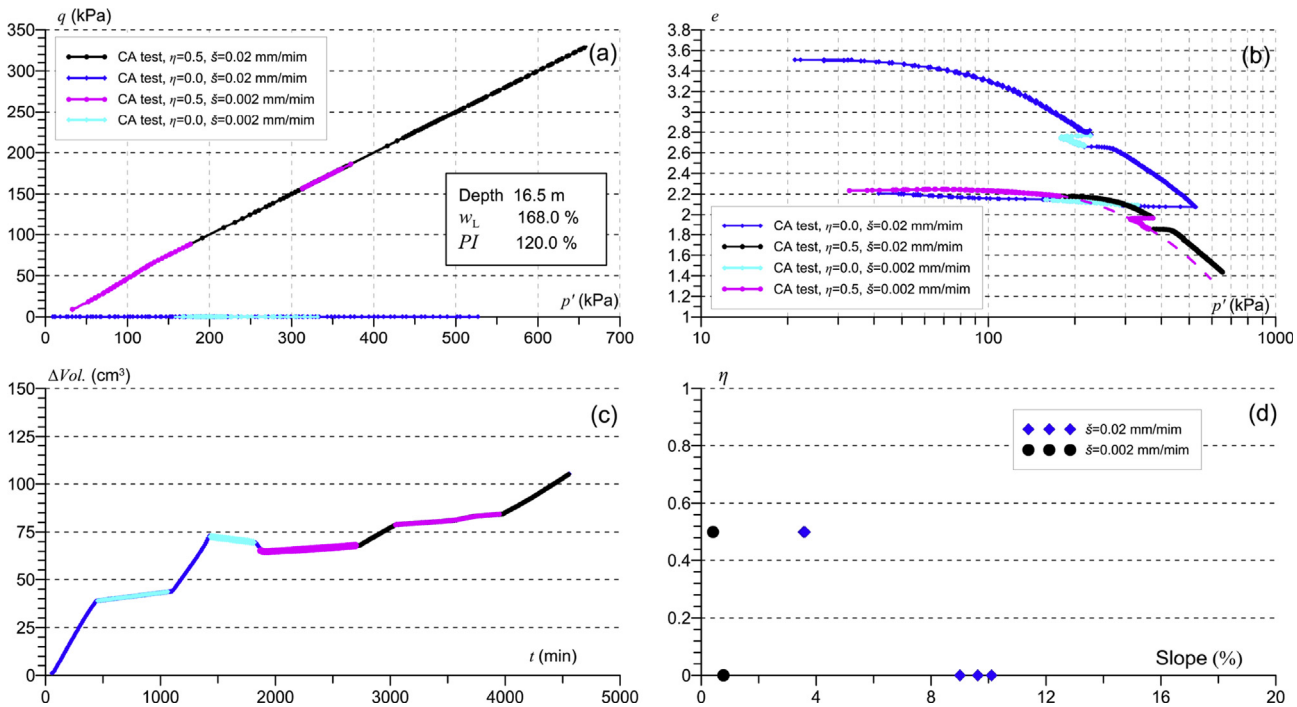

Fig. 10. Triaxial consolidation test results of a sample obtained from 16.5 m depth with stress ratios $\eta = 0$ and 0.5.

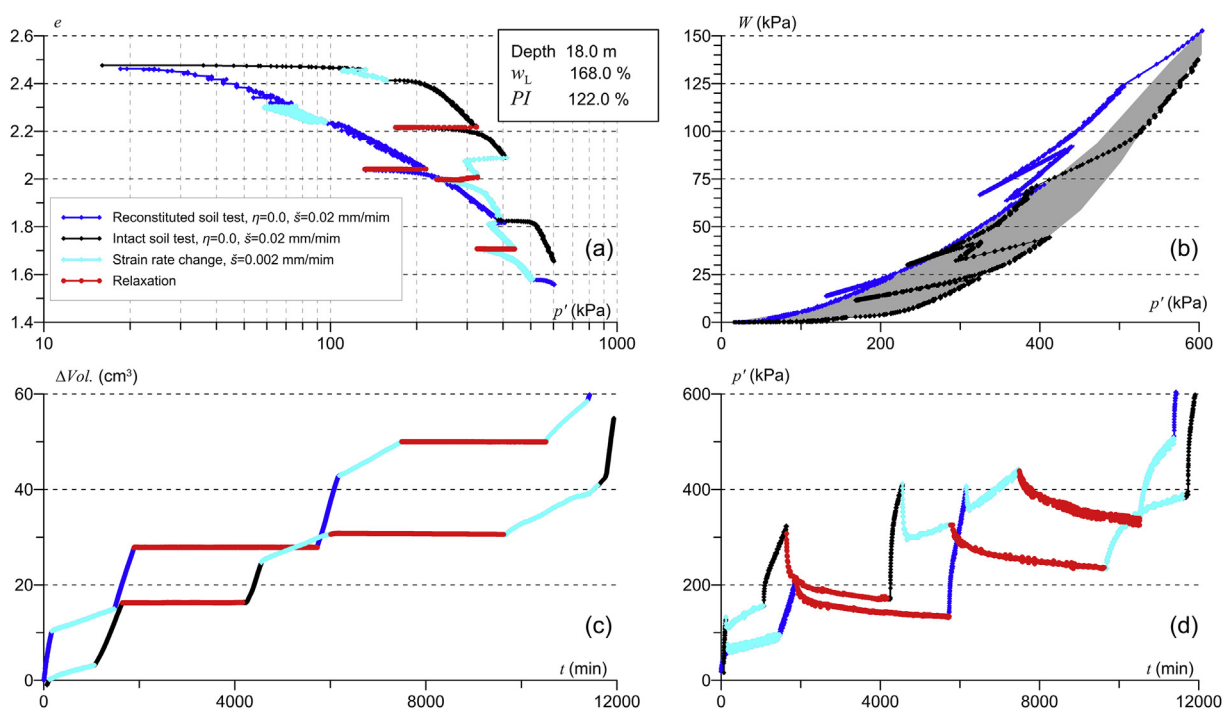

Fig. 11. Consolidation test results of a natural sample and a reconstituted sample obtained from 18 m depth.

Leda Saint-Jean-Vianney clay. These tests showed the small effect of strain rate in the elastic range. However, the larger the strain rate, the greater the peak strength.

The last test was conducted on a sample upon which anisotropic consolidation with $\eta = 0.375$ and $p' = 147$ kPa was applied, followed by deviatoric stress in extension up to a strain of 6%, as shown in Fig. 15. From this test, it can be observed that the viscosity effect is preserved in the extensile stress. The undrained extension test was performed under constant total radial stress and decreased total axial stress. Also, the test was carried out at a controlled rate of axial strain.

3.2.3. Viscosity effect on the soil

Triaxial tests presented above show that the Bogotá lacustrine clay is influenced by viscosity (Figs. 9–15). In addition, the viscosity effect also appears in the reconstituted sample (Fig. 11). These

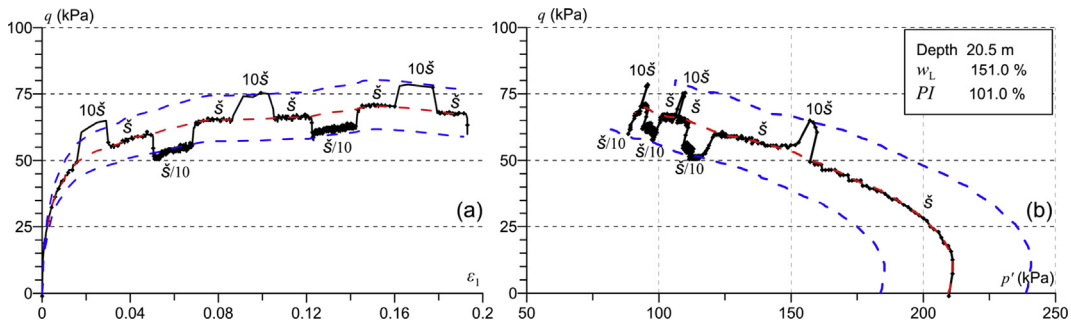

Fig. 12. Triaxial test (CIU) results of a sample obtained from 20.5 m depth.

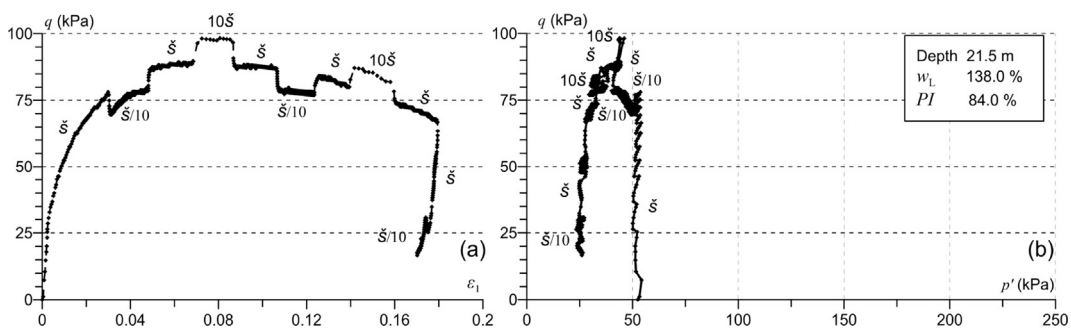

Fig. 13. Triaxial test (CIU) results of a sample obtained from 21.5 m depth with $OCR = 3$.

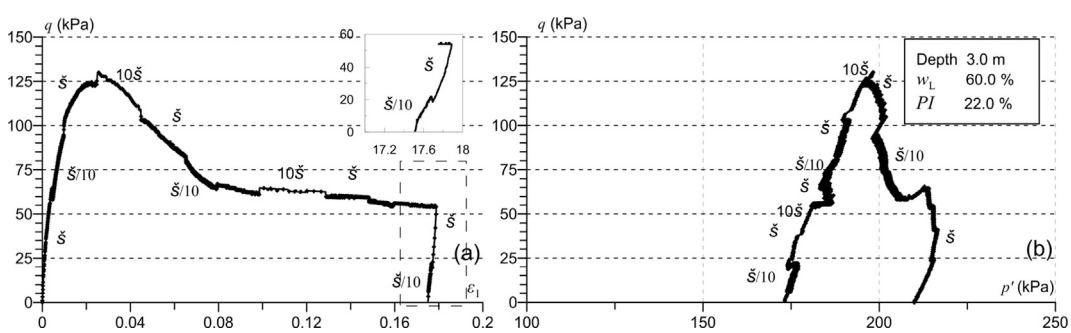

Fig. 14. Triaxial test (CIU) results of a sample obtained from 3 m depth.

results suggest that the soil composition affects its viscosity. The degree of viscosity can be assessed by calculating any increase or decrease in the soil responses dependent on the strain rate. The method proposed by Leinenkugel (1976) is useful for this purpose. This proposal used the viscosity index $I_{v\alpha}$ to relate changes in soil responses that depend on the strain rate (Eq. (3)):

$$R_{ev} = R_{ev\alpha}\left[1 + I_{v\alpha}\ln\left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_\alpha}\right)\right] \tag{3}$$

where R_{ev} is the current resistance, $R_{ev\alpha}$ is the reference resistance, $\dot{\varepsilon}_\alpha$ is the reference strain rate, and $\dot{\varepsilon}$ is the current strain rate.

In the test shown in Fig. 12, the measured variation in the soil stress of 14% leads to a value of $I_{v\alpha} = 0.06$. On the other hand, for the overconsolidated test with $OCR = 3$ (Fig. 13), the increase (or decrease) in stress by 11% leads to $I_{v\alpha} = 0.047$. In the test in Fig. 14, a 6.5% change in stress (increase or decrease) was observed with $I_{v\alpha} = 0.028$. This value could be related to the lower liquid limit of 60%, as shown in Krieg (2000) (Fig. 16). Finally, the viscosity effect on extension is 9% ($I_{v\alpha} = 0.034$) (Fig. 15). This is an important point to be considered for practical purposes, since the effect of the strain rate on extension is usually neglected.

In conclusion, the liquid limit is influenced by a high content of diatoms, as reported by Shiwakoti et al. (2002), Díaz-Rodríguez (2011), Frýbová et al. (2017), Wiemer and Kopf (2017), and Wiemer et al. (2017). Additionally, a high content of diatoms influences the viscosity effect, as shown in Fig. 16.

3.3. Mechanical behaviors at small strains

The mechanical behaviors of Bogotá soil at small strains were studied with 23 cyclic triaxial tests and bender element tests. These tests were performed on samples having different Atterberg limits to assess the effect of these intrinsic properties on soil mechanical behaviors at small strains.

3.3.1. Maximum shear modulus, G_0, under isotropic compression

The maximum shear modulus, G_0, was measured using the bender element test. In the bender device, a couple of piezoceramics transducers called bender elements were used to

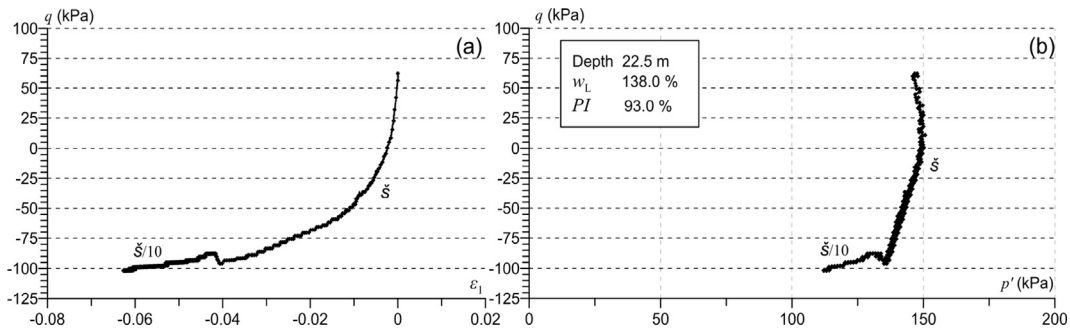

Fig. 15. Triaxial extension test (CAUE) results of a sample obtained from 22.5 m depth.

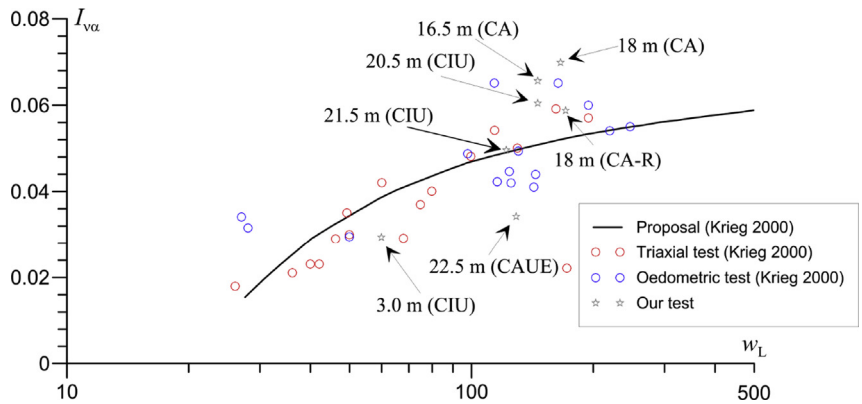

Fig. 16. Experimental observation of viscosity index (Krieg, 2000).

measure shear wave (S-wave). The piezo-ceramics transducers were embedded into the top cap and the base of a triaxial compression test apparatus (Murillo et al., 2011). An electrical pulse, which initiated a shear wave through the sample, excited the upper element, and the lower element received the shear wave signals. This technique is based on applying a high-frequency shear pulse of around 10 kHz to a sample and then deriving the shear modulus from the shear wave velocity measured (Murillo et al., 2011). Although it is recognized that bender elements apply a pulse with a very low strain level, the accurate measurement of the strain is not possible using this technique. Also, assessing the effect of strain rate is difficult due to the impossibility of applying pulses of low frequencies.

A literature review shows that the maximum shear modulus, G_0, depends mainly on the void ratio e and the effective principal stresses σ'_1 and σ'_3. For isotropic stress paths, Hardin and Richart (1963), Iwasaki and Tatsuoka (1977), Biarez and Hicher (1994), and Lo Presti et al. (1997) have shown that the shear modulus is expressed as a function of the void ratio and the mean effective stress p' or the confining effective stress σ'_3, as follows:

$$G_0 = K_G P_a f(e) \left(\frac{\sigma'_3}{P_a}\right)^n \tag{4}$$

$$f(e) = \frac{(b-e)^2}{1+e} \tag{5}$$

where K_G and n are the material constants, $f(e)$ is a function of the void ratio, b is a constant, and P_a is the atmospheric pressure. The power n characterizes the nature of grain-to-grain contacts. When it is close to 0.5 (see Fig. 17), as it is in this study, it implies some deviation from the predictions of Hertz's theory ($n = 1/3$). This can be explained by the not-necessarily-punctual nature of the contacts between grains (Dano and Hicher, 2002).

Hardin and Richart (1963) proposed the relationship for the function $f(e)$. They proposed a value of 2.17 for the parameter b, but in the case of Bogotá soil, this value results in a very high void ratio. Better agreement is obtained using $b = 10$, as shown in Fig. 17.

3.3.2. Relationships between shear modulus and shear strain at low strains

Curves of shear modulus degradation were assessed performing cyclic triaxial tests in undrained conditions with a frequency of 1 Hz. Due to the viscosity effect explained above, a strain rate effect is expected for the cyclic behavior; however, this point is not discussed herein. A complete study will be conducted to understand which strain ratio and/or strain is necessary to generate excess pore pressure, as it can influence the shear modulus.

The results from the shear modulus, G, and shear modulus degradation, G/G_0, depending on the shear strain γ, were adjusted using the hyperbolic model proposed by Nakagawa and Soga (1995), which can be presented by

$$\frac{G}{G_0} = \frac{1}{1+\alpha|\gamma|^\lambda} \tag{6}$$

This model requires two parameters, α and λ, which can be fitted from a linear regression of test data in $\log_{10}\gamma$ vs. $\log_{10}(G_0/G-1)$ plot. The parameter α is equal to $G_0/G-1$ at $\gamma = 1$, and the parameter λ is the slope of the line.

Test results indicate that the parameter λ could be related to the liquid limit, as shown in Fig. 18a. Also, Fig. 18b shows a relationship

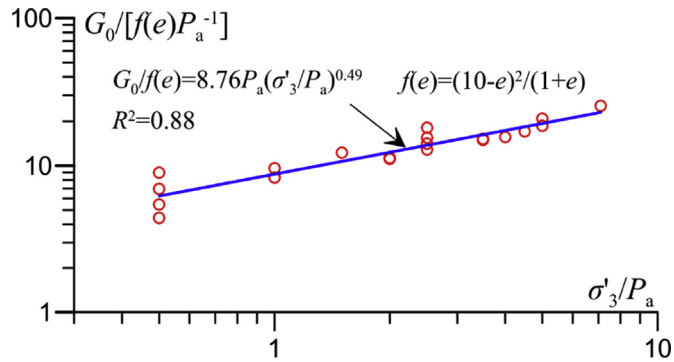

Fig. 17. Results of small-strain shear modulus under isotropic compression.

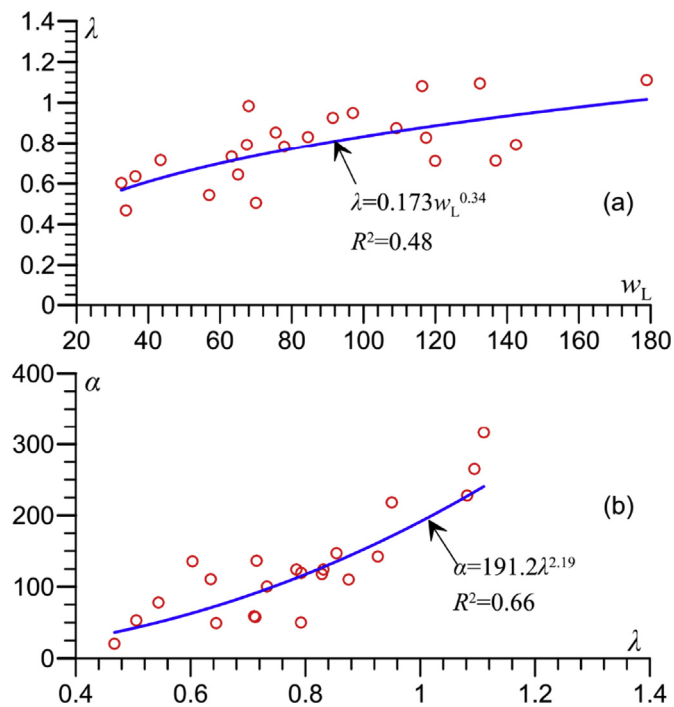

Fig. 18. (a) Parameter λ depending on liquid limit, and (b) Parameter α depending on λ as a result of fitting process.

between parameters α and λ. The following equations allow α and λ to be obtained from the liquid limit:

$$\lambda = 0.173w_L^{0.34} \tag{7}$$

$$\alpha = 191.2\lambda^{2.19} \tag{8}$$

Fig. 19a shows the results for shear modulus degradation obtained with the hyperbolic model described above. Shear modulus degradation depends on the shear strain in soils at different liquid limits. As the liquid limit of the soil increases, its elastic domain, given by the range of strains in which degradation is very small, also increases. Similar results have been presented by Vucetic and Dobry (1991) for soils obtained from Mexico City.

Fig. 19b shows the relationship between shear modulus degradations measured experimentally and calculated using Eqs. (6)–(8). This figure indicates that most of the results are close to the line at which model and test results are equal; however, better agreement is observed for soils having high liquid limits.

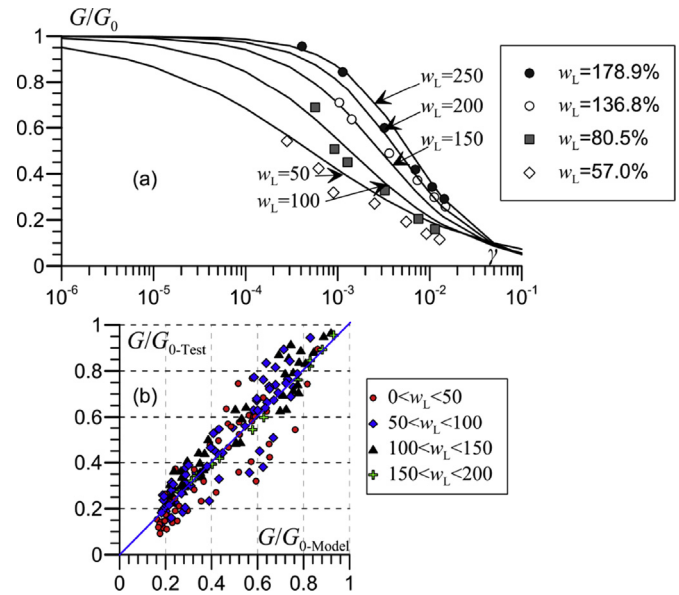

Fig. 19. (a) Shear modulus degradation obtained with the hyperbolic model at different liquid limits; and (b) Comparison of experimental and calculated values of shear modulus degradation.

4. Conclusions

This paper focuses on the experimental study of the Bogotá lacustrine soil, Colombia. The results confirm several special features that have been found in soft lacustrine soils elsewhere in the world. The conclusions are given as follows:

(1) It is found that the shape of the yield curve for Bogotá soil is similar to those of other curves proposed for normally consolidated natural soils. However, the contraction part of the yield curve shows that the normal consolidation slope is not constant, but a state variable that is a function of stress path. This is an unusual behavior considering the fundamentals of classical soil mechanics. Nowadays, the normal consolidation slope is a soil parameter, which is independent of stress state (stress path). This effect results in the strains in the soil being influenced by stress paths. This variation seems to be a characteristic of lacustrine soils with diatoms.

(2) Regarding the time-dependent response, Bogotá soil presents stress changes by approximately 14% of the shear strength under compression and undrained shear behavior. However, this value is not constant, because the stress change depends on the liquid limit and strain rate. Using the proposal by Krieg (2000), it is possible to find the correlation of the stress changes, due to viscosity of the soil, which are well related to liquid limit. On the other hand, high content of diatoms increases the liquid limit. In addition, high content of diatoms influences the viscosity effect and increases the effect of strain rate on shear stresses. Also, this effect can be important to the loading tests in piles with a constant rate of penetration (CRP).

(3) The other important characteristics are the influence of the anisotropic stress (stress path) and the strain rate on the volume change of the soil. These characteristics were found in the compression procedure in drained condition. Therefore, the settlement of Bogotá soil is influenced by stress path and strain rate.

(4) In the domain of small strains, a relationship depending on the liquid limit is proposed. It is observed that the liquid limit

increases the domain of strains when shear modulus degradation is very small, as reported by Vucetic and Dobry (1991) for Mexico City soils. These changes in stress can be induced by viscosity, which are well related to liquid limit, as proposed by Krieg (2000). In addition, the present work shows that high content of diatoms changes liquid limit. As a result, the high content of diatoms changes the seismic behavior of soil in Bogotá.

Conflicts of interest

We wish to confirm that there are no known conflicts of interest associated with this publication and there has been no significant financial support for this work that could have influenced its outcome.

Acknowledgments

The authors express their gratitude to Universidad Nacional de Colombia and Los Andes University in Colombia for their technical and financial support of this study on the deep foundations of residual soils. The authors would also like to thank Assistant Professor Olivia Flores of Fulbright Colombia for her valuable revisions and edits regarding the language of this research.

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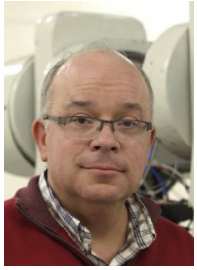

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