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Maximum shear modulus and modulus degradation curves of an unsaturated tropical soil

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Article

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Abstract

The maximum shear modulus (G_0) and the modulus degradation curve (G/G_0 *versus* γ) are important information in the evaluation of the soil mechanical behavior, both for dynamic and static loads. Dynamic tests (resonant column and cyclic triaxial tests) are not routinely performed in geotechnical practice in Brazil, and the geotechnical literature on the dynamic behavior of unsaturated tropical soils is limited. This paper presents and discusses seismic dilatometer (SDMT), resonant column, and triaxial test with bender elements and internal instrumentation to determine G_0 and the modulus degradation curve in an unsaturated tropical sandy soil profile. It was observed that G_0 tends to increase non-linearly with soil suction and net stress (σ - u_a). It was also observed that the in situ G_0 values determined with the SDMT were higher than those from laboratory tests (bender elements and resonant column). The modulus degradation curves determined with resonant column were used to define the reference curve via SDMT for the studied site. Soil suction influence in shear modulus degradation curves determined with unsaturated triaxial compression tests with local instrumentation is also presented and discussed.

1. Introduction

The maximum shear modulus (G_0) and the modulus degradation curve (G/G_0 *versus* γ) are important information to analyze the mechanical behavior of soils. It is necessary to determine these parameters and this curve due to the increase demand for nuclear facilities, offshore structures, and machine foundation design. The ground motion of the site is significantly affected by the local site condition during an earthquake, and the average shear wave velocity (*Vs*) up to 30 m is the key variable for site characterization in geotechnical earthquake engineering (Bang & Kim, 2007; ICC, 1997). Moreover, the G_0 values can be used for a static deformation analysis such as slope stability, settlement estimative, an evaluation for ground improvements, as well as assessment of collapsible soils (Burland et al., 1977; Kim & Park, 1999; Rocha et al., 2022). Tests to determine soil dynamic parameters are not currently performed in Brazil and the geotechnical literature on the dynamic behavior of tropical soils is limited.

The crosshole test is the most effective technique for determining *Vs*, and to calculate the maximum shear modulus (G_0) via Elasticity theory. Recently, the seismic dilatometer (SDMT) has being widely used since it allows the site characterization together with the determination of *Vs* profiles, consequently G_0 (Marchetti et al., 2008). Resonant column tests and the bender elements incorporated to triaxial tests can be used to determine *Vs* under controlled conditions in laboratory, such as confining stress, strain amplitude and soil suction influence.

The soil behavior is highly non-linear and has an important influence on the selection of design parameters for simple routine geotechnical projects (Atkinson, 2000). So, the direct application of G_0 to evaluated deformations problems is not applicable, and the shear modulus decay curve is necessary. The non-linear soil stress-strain behavior can be estimated with in situ and laboratory tests. In situ tests, like the crosshole and downhole can be used to determine shear modulus at small strains; dilatometer, pressuremeter, and plate load tests for medium strains; cone penetration and standard penetration tests for largely deformed soils (Amoroso, 2011; Atkinson, 2000; Ishihara, 2001). Laboratory tests, such as the bender elements or the resonant column, the cyclic triaxial or torsional shear tests, or even monotonic triaxial tests, or double specimen direct simple shear can be used to estimate the non-linear soil behavior (Amoroso, 2011).

A large portion of Brazil is covered by unsaturated tropical soils and the geotechnical literature about dynamic parameters of these soils is limited since dynamic tests are not currently carried out. The term tropical soil includes both lateritic and saprolitic soils. Saprolitic soils are residual and retain the macro fabric of the parent rock. Lateritic soils can

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be either residual or transported and are distinguished by the occurrence of the laterization process, which is an enriching of a soil with iron and aluminum and their associated oxides (cementation), caused by weathering in regions which are hot, acidic, and at least seasonally humid (Nogami & Villibor, 1981). Cementation and soil suction affects the soil behavior of unsaturated tropical soils, both in situ and in laboratory (Fernandes et al., 2022; Giacheti et al., 2019; Rocha et al., 2021). The contribution of microstructure (cementation) and soil suction to the soil stiffness depends on the strain level the soil will experience (Atkinson, 2000). These characteristics increase the overconsolidation stress and cohesion intercept (Vaughan et al., 1988) and the most existing empirical correlation should be employed with caution (Robertson, 2016).

In this paper, SDMT, triaxial tests with bender elements and internal instrumentation, as well as resonant column carried out in an unsaturated tropical soil are presented and discussed. G_0 values determined by these different techniques were compared. The modulus degradation curves (G/G_0 *versus* γ) determined via resonant column tests were used to define the reference curve for the SDMT based on the approach proposed by Amoroso et al. (2014). In addition, the effect of the unsaturated soil condition on the modulus degradation curves obtained from triaxial tests with suction control and internal instrumentation are presented and discussed.

2. Study site

SDMT, resonant column and triaxial tests with bender elements were conducted at the Experimental Research Site at the São Paulo State University (Unesp), located in the city of Bauru, State of São Paulo, Brazil. The study site includes a colluvial Neo-Cenozoic deposit up to about 13 m depth, followed by a residual soil formed during the Quaternary (De Mio, 2005). The soil profile consists in an unsaturated clayey fine sand with lateritic behavior up to about 13 m depth. The MCT Classification System (Mini, Compacted, and Tropical) proposed by (Nogami & Villibor, 1981) for tropical soils was used to define and classify the soils with regards to the lateritic behavior. These soils have undergone pedogenic and morphogenetic processes (Giacheti et al., 2019). Consequently, this soil has high porosity, high saturated hydraulic conductivity (10^{-5} to 10^{-6} m/s), and a cohesive-frictional behavior. A major geotechnical problem for this soil is collapsibility caused by soil wetting.

Several site characterization programs including Standard Penetration Tests (SPT), Standard Penetration Tests with Torque (SPT-T), Seismic Cone Penetration (SCPT), Flat Dilatometer (DMT), Pressurementer (PMT), and Seismic tests (crosshole - CH and downhole - DH) were carried at the site. Sample pits were also excavated to retrieve undisturbed and disturbed soil blocks. Soil samples from these blocks were tested in laboratory for soil characterization and determination of mechanical properties and parameters. Figure 1 summarizes laboratory and in-situ tests carried out at the study site: grain size distribution (with and without dispersant), some index properties, SCPT, SPT, PMT and Seismic tests data along the soil profile.

3. In situ and laboratory tests

Four SDMTs were carried out at the study site up to 15 m depth. The SDMT testing procedures were conducted

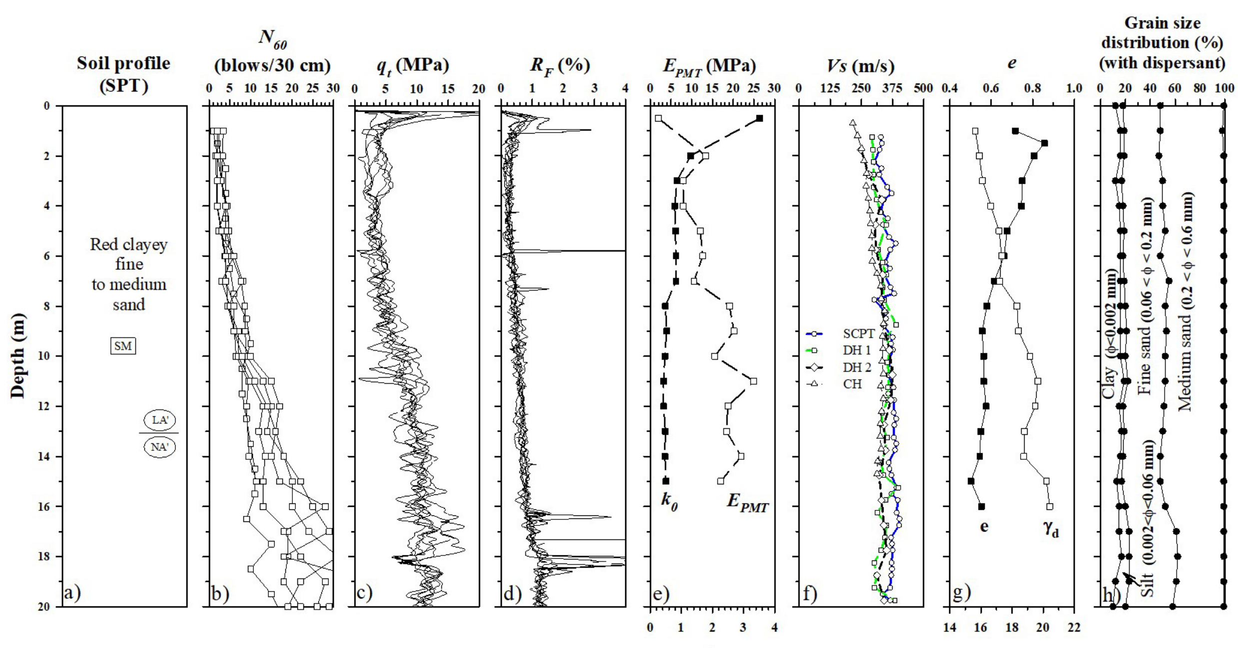

Figure 1. Summary of in situ and laboratory tests carried out at the study site [adapted from Rocha & Giacheti (2018)].

in accordance with Marchetti et al. (2006). A multi-function penetrometer with a 150 kN thrust capacity (Model Pagani TG 63 – 150 DP), which was anchored to the ground by helical augers, was used to carry this in situ test. The SDMT blade was pushed into the ground at a constant rate of 20 mm/s. The readings A-pressure and B-pressure were taken at intervals of 200 mm, and then these pressures were corrected for membrane stiffness and converted into p_0 and p_1. The three intermediate DMT parameters (I_D: material index; K_D: horizontal stress index; E_D: dilatometer modulus) were calculated from the p_0 and p_1 values. Field measurements of the shear wave velocity (Vs) were taken at 0.5 or 1 m depth interval.

The resonant column tests were presented by Giacheti (1991). The triaxial tests were performed with internal instrumentation and bender elements by Fernandes (2022). The modulus degradation curves were determined from the resonant column, triaxial and SDMT test data.

Cylindrical specimens of about 36 mm in diameter and 80 mm in height were used in the resonant column tests. They were rigidly fixed to the base of the triaxial chamber by means of a blade embedded in the porous stone (Giacheti, 1991). Table 1 shows some geotechnical indexes, the confining stresses, and the moisture content conditions for each of them. The multi-stage technique was employed in the resonant column tests, as described by Anderson & Stokoe (1978). A very low amplitude torsional excitation was applied to the top of the specimen and the shear wave velocities were determined over the logarithmic time interval up to 1,000 minutes or up to 10,000 minutes in some cases for each confining stress stage. Subsequently, the excitation force was gradually increased and the variation of the ratio of shear modulus to strain amplitude was determined.

The saturated and unsaturated triaxial tests with internal instrumentation and bender elements were performed using 50 mm diameter specimens with the height ranging from 100 to 120 mm. The determination of Vs (and hence G_0) via bender elements was performed for the samples collected at 1.5, 5, 7, 11, and 13 m depth. The phase angle between the waves and the frequency domain method (Ferreira, 2002) were used to determine the wave propagation time. Suction values of 0 (saturated), 50, 200 and 400 kPa were imposed for samples collected at 1.5 and 5 m depth and suction values of 0 (saturated), 50, 100 and 200 kPa for samples collected at 7, 11 and 13 m depth. The applied confining stresses were 25, 50, 100, and 200 kPa for all samples tested. The axial (ε_a) and radial (ε_r) strains were measured by internal instrumentation (LVDTs with axial and radial displacement measurement) for the sample collected at 2 m depth, with a confining stress of 50 kPa and suction values equal to 0, 50, 200, and 400 kPa. The shear strain for individual soils elements (ε_s) can be calculated from Equation 1 based on ε_a and ε_r, and it was transformed in shear strain (γ) with Equation 2. The modulus of elasticity (E) was obtained from the triaxial test data and a Poisson ratio (μ) equal to 0.2 was assumed to determine the modulus degradation curve (Equation 3).

$$G = \frac{E}{2 \cdot (1 + \mu)} \tag{1}$$

$$\varepsilon_s = (2/3 \cdot (\varepsilon_a - \varepsilon_r)) \tag{2}$$

$$\gamma = 3/2 \cdot \varepsilon_s \tag{3}$$

3.1 Modulus degradation curve via SDMT

The modulus degradation curve can be estimated from a reference degradation curve determined in the laboratory by cyclic testing (Marchetti et al., 2008). This curve can be defined from two points obtained by means of SDMT: (1) maximum soil shear modulus (G_0), and (2) shear modulus at the working condition (G_{DMT}). Amoroso et al. (2014) presents a procedure to estimate the modulus degradation curve via SDMT based on the findings of Marchetti et al. (2008). This procedure is schematically represented by Figure 2, and it consists of the following steps:

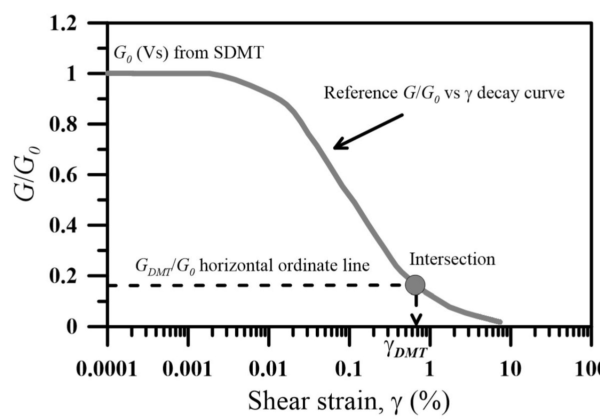

Figure 2. Amoroso et al. (2014) approach to derive modulus degradation curves from SDMT [adapted from Amoroso et al. (2014)].

Table 1. Some information of the previously performed resonant column tests [adapted from Giacheti (1991)].

Depth (m)	Confinant stresses (kPa)	Liquid limit W_L (%)	Plastic index P_I (%)	Unit weight γ_n (kN/m^3)	Water content condition
0.95	25, 50 and 100	19	4	16.45	Natural
4.8	50, 100 and 200	22	5	17.1	Natural
8.85	50, 100 and 200	23	7	17.9	Natural

- Determine G_0 based on *Vs* from SDMT, at the same depth of the available reference modulus degradation curve;
- Calculate G_{DMT} based on the constrained modulus obtained from SDMT data (M_{DMT}) (Equation 4) and normalized by its maximum shear modulus (G_0).

$$G_{DMT} = \frac{1-2\mu}{2\cdot(1-\mu)} \cdot M_{DMT} \tag{4}$$

Where μ is the Poisson ratio.

- Assume the shear strain associated with working strain DMT moduli (γ_{DMT}) based on the available information [e.g., Amoroso (2011)];
- Then, use Equation 5, proposed by Amoroso (2011), to assess the shear modulus reduction curve by SDMT.

$$\frac{G}{G_0} = \frac{1}{1+\left(\frac{G_0}{G_{DMT}}-1\right)\left(\frac{\gamma}{\gamma_{DMT}}\right)} \tag{5}$$

Therefore, the ratio G_{DMT}/G_0 obtained from SDMT and the estimated shear strain γ_{DMT} were used to plot the corresponding hyperbolic curve at each investigated test site.

According to Amoroso et al. (2014), the "typical range" of shear strain (γ_{DMT}) associated to the working strain moduli G_{DMT} can be approximately assumed as 0.01 to 0.45% for sand, 0.1 to 1.9% for silt and clay, and higher than 2% for soft clay. The authors considers that this approach can provide a first estimate of the modulus degradation curve (G/G_0 *versus* γ) of the soil.

3.2 SDMT

Figure 3 shows the intermediate DMT parameters (I_D, K_D, and E_D) and the shear wave velocity, and consequently maximum shear modulus profiles for the study site. I_D, K_D and E_D were calculated by Marchetti's equations (Marchetti, 1980). Shear wave velocity determined with SDMT, and total mass density (ρ) determined using undisturbed soil samples collected in a sample pit excavated at the study site were used to calculate G_0 values based on Elastic Theory (Equation 6).

$$G_0 = \rho \cdot V_s^2 \tag{6}$$

Where *Vs* is shear wave velocity, and ρ is the total mass density.

Figure 3 shows a good agreement between the *Vs* and G_0 profiles determined by all the performed tests. The values of *Vs* and G_0 increase with depth up to 10 m and this trend becomes almost constant after that depth.

3.3 Triaxial tests with bender elements (BE) and internal instrumentation

The shear wave velocity (*Vs*) and consequently G_0 were determined using bender elements. G_0 shows a tendency to increase non-linearly with the suction and with the net stress (σ - u_a) for sandy soils, and tends asymptotically to a limit (Nyunt et al., 2011). A hyperbolic function was used to evaluate the influence of the suction and the net stress variables on the maximum shear modulus (Equation 7).

$$G_0 = G_{0,sat} + a(\sigma - u_a) + \frac{s}{b+c(s)} \tag{7}$$

Where $G_{0,sat}$ is the maximum saturated shear modulus, s is the soil suction, and a, b, and c are empirical parameters of the fit. G_0 and $G_{0,sat}$ are expressed in MPa and the net stress (σ - u_a) and suction in kPa.

Figure 4 shows the variation of G_0 with suction and with net stress, as well as the fitting for the samples collected at

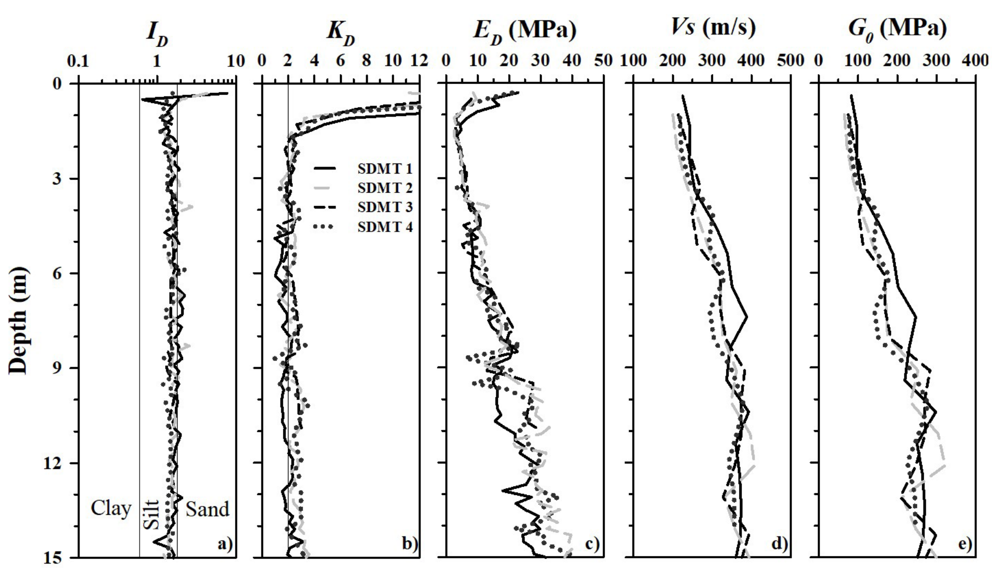

Figure 3. SDMT data at the study site.

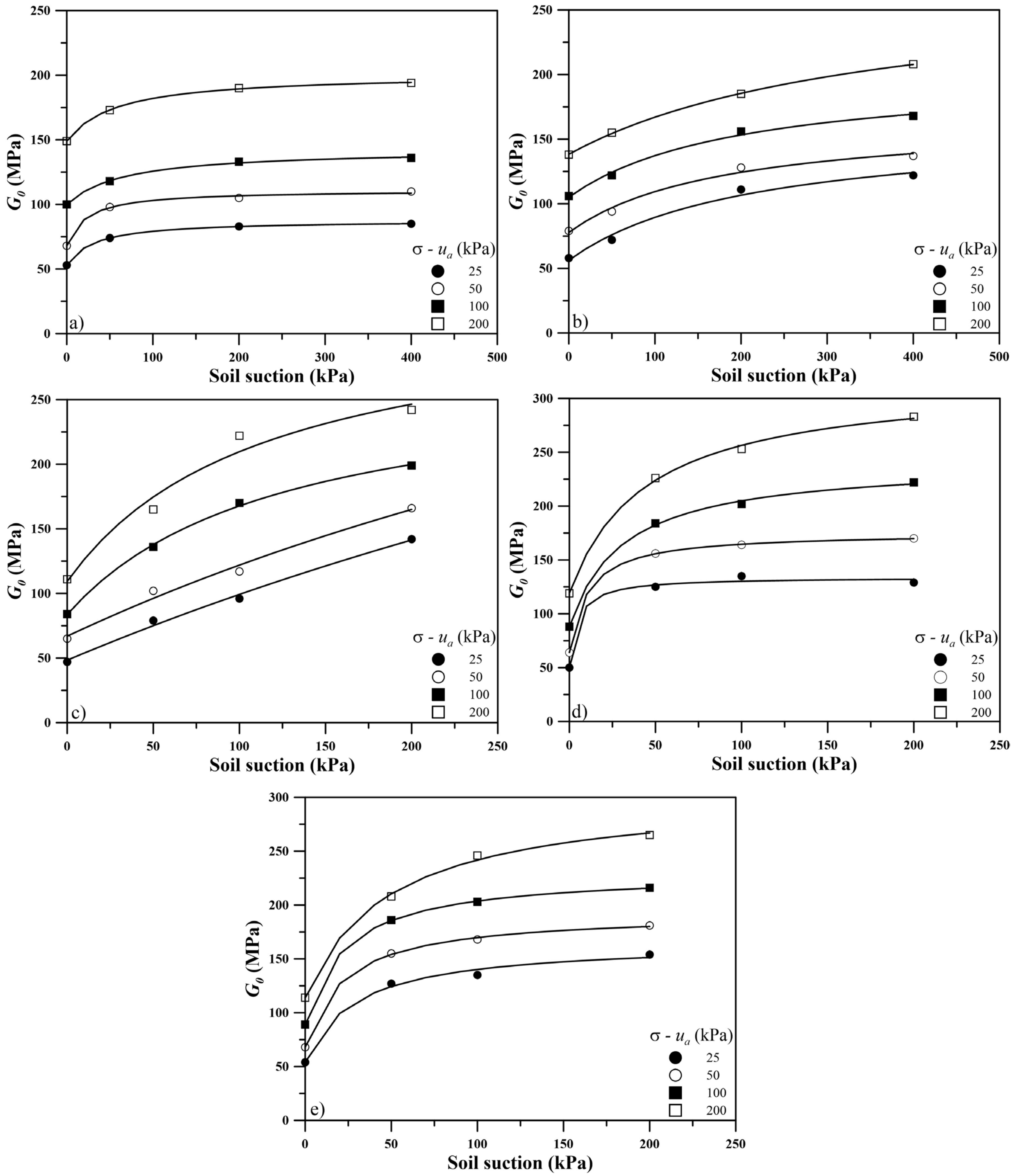

Figure 4. Variation of G_0 with net stress (σ - u_a) and with suction for the undeformed soil samples collected at (a) 1.5 m, (b) 5 m, (c) 7 m, (d) 11 m, and (e) 13 m depth.

1.5, 5, 7, 11, and 13 m depth. As the soil suction increases from 0 to 400 kPa, the shear moduli at small strain (assumed equal to 0.001%) also increase. Table 2 shows the fitting parameters for the Equation 7 at the 100 kPa net stress for all the investigated depths. It can be seen from this figure that the experimental data are well represented by a non-linear relationship (Equation 7) between G_0 and the variables suction and net stress, except for the net stresses of 25 and 50 kPa for the depth of 7 m, which showed a quasi-linear behavior.

Figure 5 shows the absolute values of the shear modulus at a small shear strain of 0.001% and at a finite strain of 1% for the sample collected at 2 m depth. It also can be seen from this figure that the soil shows a relatively high and fast variation of the shear modulus as the shear strain increases from 0.001 to 1%.

3.4 Resonant column (RC) tests

Giacheti (1991) presented the maximum shear modulus (G_0) as a function of time of confinement by performing resonant column tests and observed that the samples tested (Table 1) showed an almost linear increase of G_0 with logarithmic time, practically from the beginning of drainage, which is a typical behavior for sands. Figure 6 shows the value of G_0 at 1,000 minutes of confinement at different confinement stresses for the samples collected at 0.95, 4.8, and 8.85 m depth (Giacheti, 1991). Figure 7 shows the modulus degradation curves (G/G_0 *versus* γ) determined for different confining stresses for the samples collected at 0.95, 4.8, and 8.85 m depth (Giacheti, 1991).

Figure 7 shows the modulus degradation curves for the samples collected at 0.95, 4.8, and 8.85 m depth at different confinement stress (σ_3). It can be seen from Figure 7 that the modulus G presents a small reduction for shear strains higher than 10^{-4}%, which is accentuated from γ greater than 10^{-3}%. Furthermore, a lower influence of confining stresses and depth on G values is observed for the range of strains investigated, with a tendency for a lower degradation of the modulus with increasing confining stress (σ_3).

4. Discussion

4.1 G_0 from in situ and laboratory tests

In order to compare G_0 values determined by SDMT, resonant column (RC) and bender elements (BE), in situ confining stresses were defined based on at-rest earth pressure coefficient (K_0) estimated from the Jaky (1948) equation.

All values were considered for the bender element tests with suction equal to 50, 100, and 200 kPa for the investigated depths (1.5, 5, 7, 11, and 13 m depth) since the SDMT and resonant column tests were performed in the natural soil condition. These suction values were defined from the suction monitoring by tensiometers, and watermark sensors presented by Giacheti et al. (2019). In addition, an average G_0 profile from four SDMTs was adopted.

Figure 8 shows the differences between the G_0 values determined by the average SDMT, average SDMT plus and

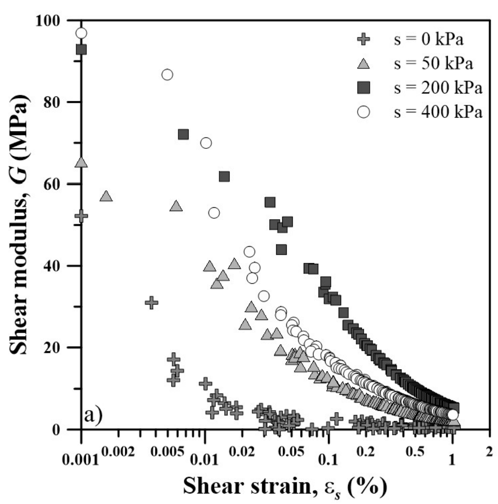

Figure 5. Modulus degradation curves for the tested samples from 2 m depth with suction values equal to 0, 50, 200, and 400 kPa.

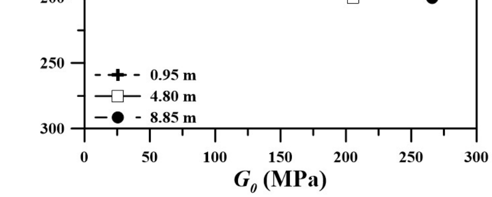

Figure 6. G_0 at 1,000 minutes of confinement for the samples collected at 0.95, 4.8, and 8.85 m depth at different confinement stresses [adapted from Giacheti (1991)].

Table 2. Fitting parameter for the Equation 7 at 100 kPa net stress.

Fitting paramenters	Depth (m)				
	1.5	5	7	11	13
$G_{0,sat}$ (MPa)	52.1	53.3	40.9	47.9	53.1
a	0.478	0.517	0.427	0.401	0.334
b	1.534	2.059	0.656	0.202	0.703
c	0.023	0.010	0.005	0.006	0.006

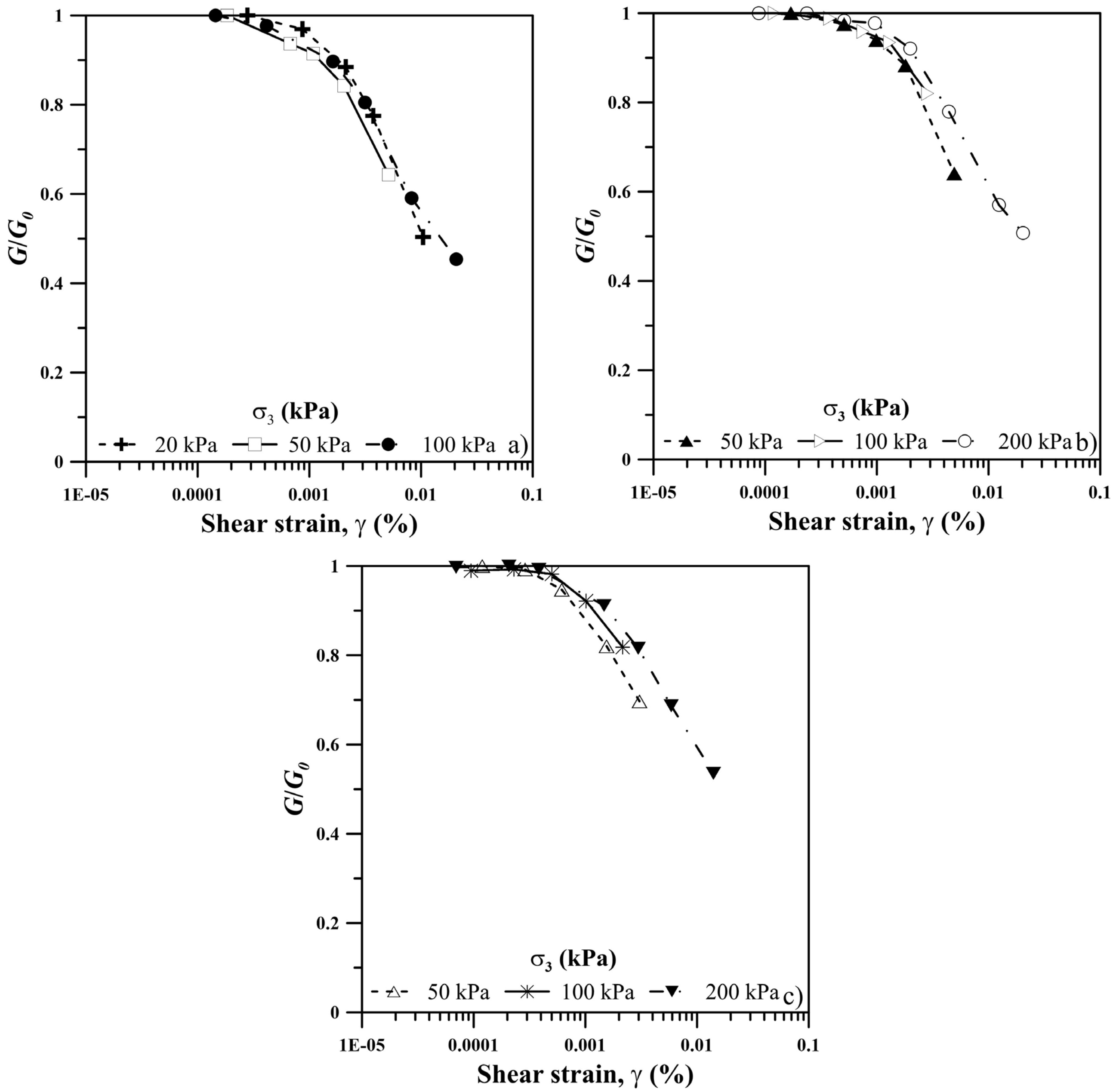

Figure 7. Modulus degradation curves (G/G_0 *versus* γ) for samples collected at 0.95 (a), 4.8 (b) and 8.85 (c) m depth [adapted from Giacheti (1991)].

minus one standard deviation (SD), RC and BE tests. The average SDMT values were higher than those determined by RC and BE. These values were 8% and 35% higher than those determined via BE, and 6% and 28% higher than those determined via RC. It is important to mention that the G_0 values determined by RC and BE are positioned at the lower limit or slightly outside the range. These differences may be related to possible disturbances during the sampling process and specimens preparation, errors in the estimation of the in situ confining stresses (Ferreira et al., 2011) as well as the influence of soil suction in G_0 (Nyunt et al., 2011).

4.2 Modulus degradation curve

4.2.1 SDMT and resonant column

Amoroso et al. (2014) suggest a method to estimate the modulus degradation curve (G/G_0 *versus* γ) by using SDMT, based on the parameters G_0, G_{DMT} and γ_{DMT}, as previously discussed in item 3.1. This approach allows a preliminary definition of the modulus degradation curve, which needs to be interpreted in conjunction with reference G/G_0 *versus* γ determined in the laboratory via cyclic triaxial or resonant

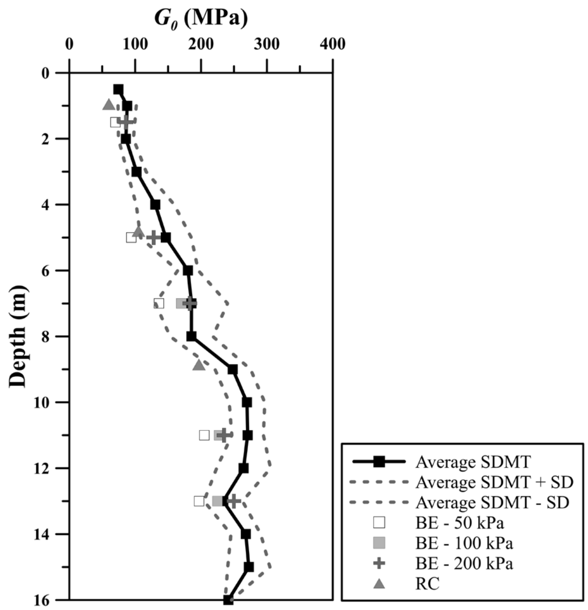

Figure 8. G_0 values determined by SDMT, bender elements, and resonant column for the study site.

column tests. So, the degradation curves presented in Figure 7 were considered representative and an average degradation curve was assumed. Table 3 presents the average values of G_0, M_{DMT}, G_{DMT}/G_0 determined by means of the four SDMTs performed, as well as the shear strain imposed with the expansion of the DMT blade (γ_{DMT}) determined from the average degradation curve assumed by the resonant column tests.

Figure 9 shows the value of the G_{DMT}/G_0 ratio determined via SDMT, the shear strain fitted from the resonant column data (γ_{DMT} - gray symbol), and the average stiffness degradation curve obtained by Equation 3 (in gray dashed line). The G_{DMT}/G_0 value is equal to 0.051 for the studied site, which is slightly lower than the typical values reported in the literature (shaded areas), which can be associated to the natural cementation of the particles and the unsaturated condition, typical of tropical soil sites. At the working condition (G_{DMT}), the stiffness due to cementation and soil suction is lost. On the other hand, the value of the shear strain imposed by SDMT blade pushing into the soil (γ_{DMT}) for the studied soil is in the range of values commonly reported in the literature for sands and silty sands to sandy silts (0.1 to 0.5%) (Amoroso et al., 2014). It is important to mention that the multistage technique (Anderson & Stokoe, 1978) used to perform the resonant column tests can generate accumulated deformations and disturbances in the structure of the specimen, resulting in cementation bond breakage, realignment of grains, and changes in void ratios (Barros, 1997); therefore, the modulus degradation for the soil of the study site can be lower than those presented in Figures 7 and 9.

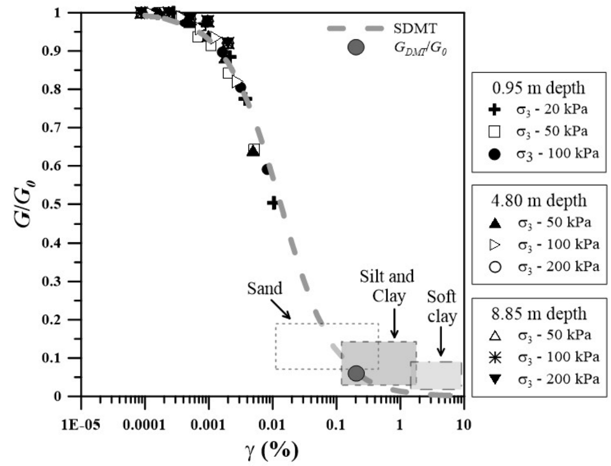

Figure 9. Modulus degradation curve via SDMT from Amoroso et al. (2014) method and the resonant column test data for the study site.

Table 3. Best-fit parameters for Amoroso et al. (2014) method for the study site.

G_0 (MPa)	M_{DMT} (MPa)	G_{DMT}/G_0	γ_{DMT} (%)
140.5	12.5	0.051	0.391

4.3 G/G_0 *versus* γ determined via unsaturated triaxial tests with internal instrumentation

The modulus degradation curves were also determined via unsaturated triaxial tests with internal instrumentation (LVDTs) for the sample collected at 2 m depth for soil suctions equal to 0, 50, 200, and 400 kPa (Figure 10). The objective is to evaluate the soil suction influence in the pattern and shape of the shear modulus degradation curve. Figure 10a shows that the shear modulus reduction curves are not monotonically related to the change in soil suction and reaches a maximum for a suction value equal to 200 kPa. The modulus degradation curve for the suction equal to 400 kPa is equivalent or slightly lower than those of suction equal to 50 kPa. It shows that the G/G_0 *versus* γ curves firstly rises and then falls in a certain range with the increase in soil suction. This trend is different from that found by Ng et al. (2021) for compacted unsaturated lateritic sandy clays and for eight different soil types as reported by Dong et al. (2018). However, Ng & Xu (2012) observed that the G/G_0 curves shift towards higher shear strain values with increasing soil suction for yellowish-brown completely decomposed tuff (CDT).

In order to describe the non-linear soil behavior, several researchers have proposed a mathematical model to capture the features of modulus reduction curve (Darendeli, 2001; Iwasaki et al., 1978; Kokusho, 1980;

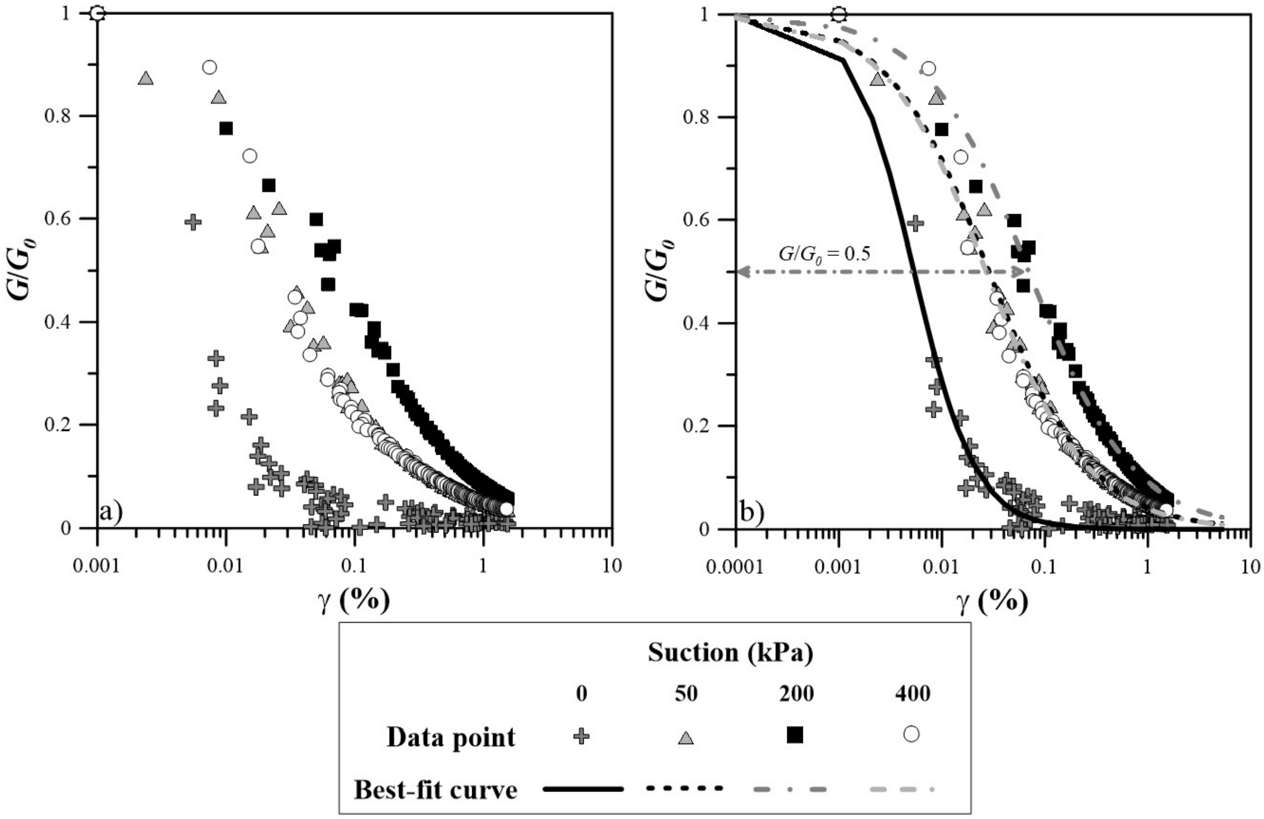

Figure 10. Modulus degradation curves: (a) determined from unsaturated triaxial tests with internal instrumentation; (b) fitted with Darendeli (2001) model.

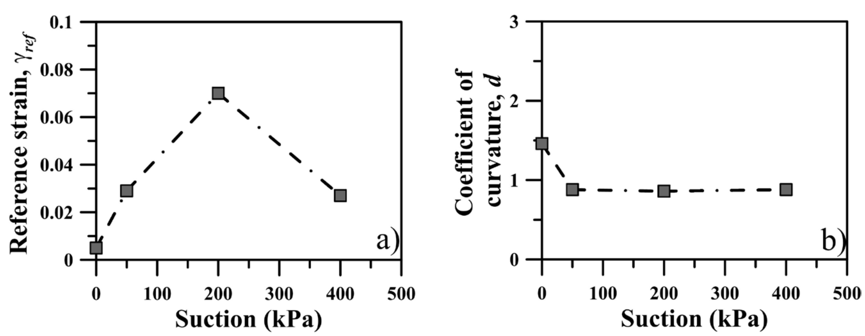

Figure 11. Soil suction influence on: (a) reference strain (γ_{ref}); (b) curvature coefficient *a* for the soil from the study site.

Seed et al., 1986; Vucetic & Dobry, 1991). According to Amoroso et al. (2014), the G/G_0 *versus* γ curves proposed by Darendeli (2001) include all other reference curves. Darendeli (2001) equation was used to represent the modulus reduction curves, as follows:

$$\frac{G}{G_0} = \frac{1}{1+\left(\frac{\gamma}{\gamma_{ref}}\right)^d} \tag{8}$$

Where *d* is the constant that represents the curvature of the modulus reduction curve, and the γ_{ref} is the reference strain controls the location where *G* decreases to half of its maximum value as the shear strain increases.

Figure 10b shows the modulus reduction curves for the tested samples. The dashed line at $G/G_0 = 0.5$ reflects the positions of the reference shear strain for each modulus reduction curve. Figure 10b also allows to examine the dependencies of the reference strain (γ_{ref}), and coefficient of curvature (*d*) on soil suction. The relationships between reference strain and suction are shown in Figure 11a, whereas the relationships between coefficient de curvature and suction are shown in Figure 11b for the soil from the study site. The reference strain increases with soil suction up to 200 kPa and decreases for soil suction equal to 400 kPa. The coefficient of curvature presents a slight reduction between saturated and soil suction equal to 50 kPa and remains approximately constant as the soil suction increases from 50 kPa to 400 kPa, indicating no suction dependence on this parameter.

5. Conclusion

The main conclusions drawn from the study are as follows:

- The maximum shear modulus of the studied soil increases nonlinearly with suction and net confining stress based on the bender elements test data;
- The maximum shear modulus values from the SDMT were higher than those determined in the laboratory via resonant column and bender elements tests. This behavior can be related to possible soil disturbances during the sampling and preparation of the specimens, errors in estimating the in situ confining stresses, as well as the influence of soil suction on G_0;
- The average modulus degradation curve defined from resonant column was used to obtain the modulus degradation curve from SDMT. The approach proposed by Amoroso et al. (2014) for SDMT is interesting and can be used as a first tentative to represent the modulus degradation curve for the soil from the study site;
- The modulus degradation curves from the suction-controlled, internally instrumented triaxial tests are not monotonically related to the change in soil suction and it was maximum for a suction value equal to 200 kPa, and this behavior is different from which was observed by other researchers. Additional tests on samples collected at other depths as well as for other suction values should be done to explore and confirm the relation between soil suction and modulus degradation curves for the studied soil.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Jeferson Brito Fernandes: conceptualization, data curation, visualization. Breno Padovezi Rocha: conceptualization, data curation, formal analysis, investigation, methodology, writing – original draft, writing – review & editing. Heraldo Luiz Giacheti: conceptualization, methodology, supervision, funding acquisition, project administration, writing – review & editing.

Data availability

The datasets generated during and/or analyzed during the current study are available from the corresponding author on reasonable request.

List of symbols

a	fitting parameter
b	fitting parameter
c	fitting parameter
d	constant that represents the curvature of the modulus reduction curve
e	void ratio
p_0	corrected first reading
p_1	corrected second reading
q_c	cone resistance
s	soil suction value
w_L	liquid limit
BE	bender elements
CH	crosshole tests
CPT	cone penetration test
$CPTu$	piezocone penetration test
DH	downhole test
DMT	flat dilatometer
E	elasticity modulus
E_D	dilatometer modulus
E_{PMT}	Menard PMT modulus
G	shear modulus
G_{DMT}	working strain modulus
G_0	maximum shear modulus
$G_{0,sat}$	maximum shear modulus at the saturated condition
I_D	material index
I_P	plastic index
K_D	horizontal stress index
K_0	in situ coefficient of lateral earth pressure
M_{DMT}	constrained modulus obtained by DMT
N_{60}	SPT N values for an efficiency of 60%
PMT	Menard pressuremeter test
RC	resonant column
R_f	friction ratio
$SCPT$	seismic cone
SD	standard deviation

SDMT	seismic dilatometer
SPT	standard penetration test
SPT-T	standard penetration test with torque measurement
Vs	shear wave velocity
εa	axial strain
εr	radial strain
γ_d	dry unit weight
γ	shear strain
γ_{ref}	reference strain
γ_{DMT}	shear strain associated to the working strain modulus (G_{DMT})
ρ	soil bulk density
σ_3	confining stress
σ - u_a	net confining stress
μ	Poisson' ratio

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