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Maximum shear modulus of a Brazilian lateritic soil from in situ and laboratory tests

L. Décourt
Luciano Décourt Consulting Engineers, São Paulo, SP, Brazil

J.M. de Camargo Barros, O.C.B. Gandolfo
Institute for Technological Research of the State of São Paulo, Brazil

A.R. Q. Filho
Engesolos Soil and Foundation Engineering, São Paulo, SP, Brazil

F.D. Penna
Damasco Penna Associate Engineers, São Paulo, SP, Brazil

ABSTRACT: This paper presents the results of the in-situ tests (SPTT, crosshole, SDMT and CPTU) and laboratory tests (characterization, compaction, MCT classification, consolidation, UU triaxial and resonant column) to investigate the properties of a lateritic soil from Santo André, metropolitan region of São Paulo, Brazil, for purposes of foundations of a building. Lateritic soils are frequently found in tropical regions and present a peculiar behavior compared to temperate zone soils. It was possible to obtain the maximum shear modulus from different tests and could be verified that lateritic soils present stiffness significantly higher than that derived from empirical correlations obtained for temperate zone soils.

1 INTRODUCTION

Lateritic soils are formed by the process of laterization in hot and wet tropical regions and present high iron and aluminum oxide and hydroxide concentration. They can be either residual or transported, generally present red or yellow coloration and the clay fraction is compound mainly of kaolinite.

The oxides involve the surface of the individual clay particles producing a cementation between adjacent particles. As a result, lateritic soils exhibit a peculiar behavior, showing a more rigid behavior than non-lateritic soils.

Nogami and Villibor (1981) developed the MCT classification system of tropical soils and Ignatius (1991), based on the inclination of dry side of the Standard Proctor compaction curve, proposed a Laterization Index L to identify lateritic soils.

An opportunity for improving the knowledge of engineering properties of this type of soil occurred a few years ago when the first author was engaged in the design of footing foundations for a twenty five-story building.

For design purposes, standard penetration tests complemented by torque measurement (SPTT), crosshole (CH) and seismic dilatometer test (SDMT) have been performed. After that, during the construction of the building, it was decided to carry out others in situ tests, to better understand the behavior of this type of soil. Piezocone tests (CPTU), load test on a block foundation and many other SPTTs and laboratory tests have been performed.

2 IN SITU AND LABORATORY TESTS

The in situ investigation was conducted in three distinct areas at the site, named Site I, II and III (Figure 1). A total of eight SPTT were carried out in the three sites.

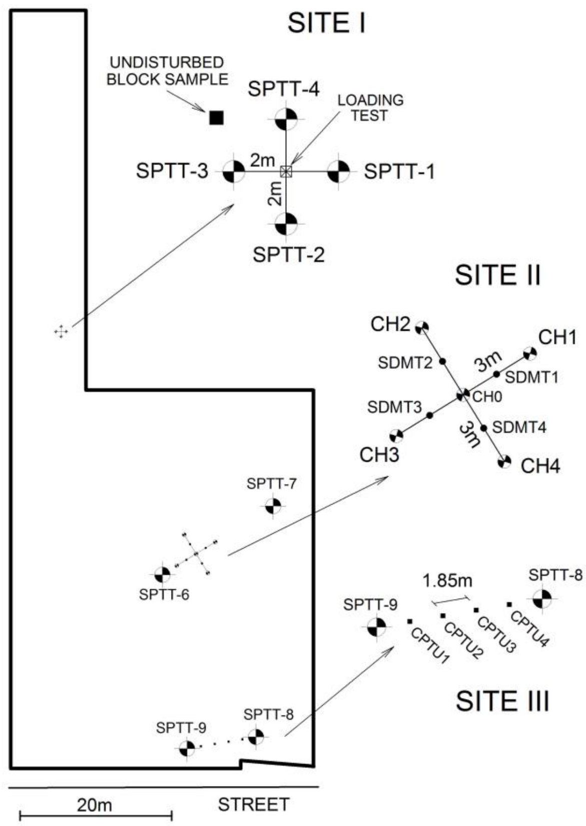

Figure 1. Site test layout.

For design purposes two tests were performed before the building at the Site II: crosshole (CH) and SDMT, which are based on the propagation of elastic waves through the ground. As seismic waves impose strains smaller than $10^{-4}\%$ on the media, the maximum modulus is developed where seismic field tests are used. Equation (1) shows the relationships for the maximum shear modulus (G_0) and shear wave velocity (V_S).

$$G_0 = \rho \cdot V_S^2 \tag{1}$$

The CH is a well-established geophysical method that allows for the direct measurements of V_S using cased boreholes. The SDMT is the combination of the mechanical flat dilatometer (DMT), introduced by Marchetti, with a seismic module for measuring V_S.

Four crosshole tests were performed, in accordance with ASTM D4428 (2007), using five boreholes (20m depth, source at CH0 and receiver at CH1 to CH4). Each test was carried out using two boreholes. As shown in Figure 1, SDMT tests were performed down to 13 meters depth at the middle of the two boreholes used for each crosshole test. Figure 2 shows the typical soil profile of the investigated site, namely SPTT-6, beside the CH and SDMT tests. Six layers of soil were clearly identified.

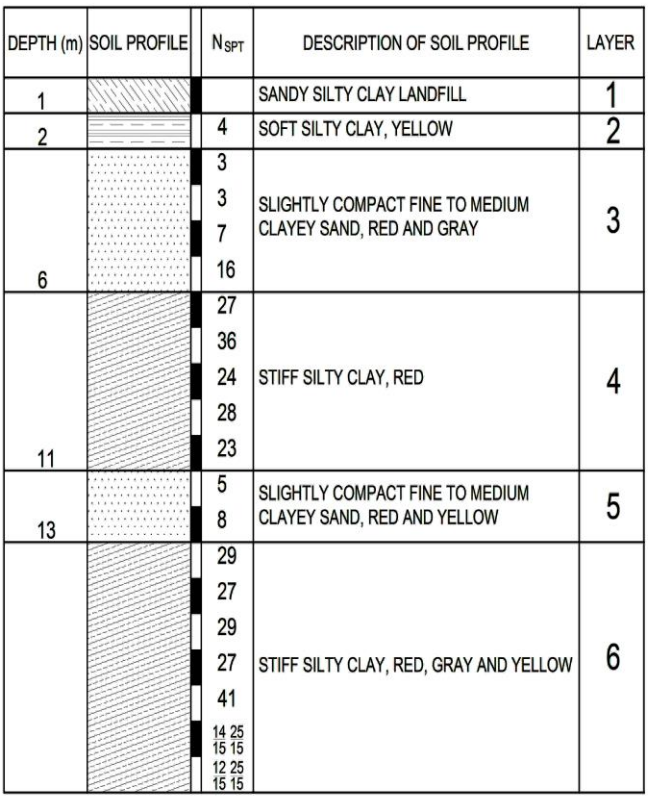

Figure 2. Soil profile (SPTT-6).

After that, during the construction of the building, it was decided to carry out other in situ and laboratory tests.

At the Site I a loading test was performed on a block foundation. The results of this test are presented by Décourt et al. in this same conference. Two blocks were extracted from upper clay layer (Layer 4 in Figure 2) and the following laboratory tests were carried out: characterization, MCT classification, determination of the Laterization Index using Proctor compaction test, one-dimensional consolidation, UU triaxial and resonant column.

Finally, at the Site III piezocone tests (CPTU) were performed adjacent to two SPT-T tests. The CPTU provides a continuous measurement of tip resistance, sleeve friction and pore pressure.

Four CPTU, going down around 19.5 meter deep, were executed in the same line of the SPT-Ts.

The site investigation presented in this paper focuses on the maximum shear modulus of Layers 2 to 5, which present lateritic characteristics.

3 RESULTS AND ANALYSIS

3.1 *In situ tests*

Figure 3 show the maximum shear modulus (G_0) determined by crosshole and SDMT tests at the Site II (average of the four tests). As can be seen, the data showed excellent agreement. The greatest values of G_0 were observed within the stiff silty clay layer (Layer 4) with an average value of 418 MPa.

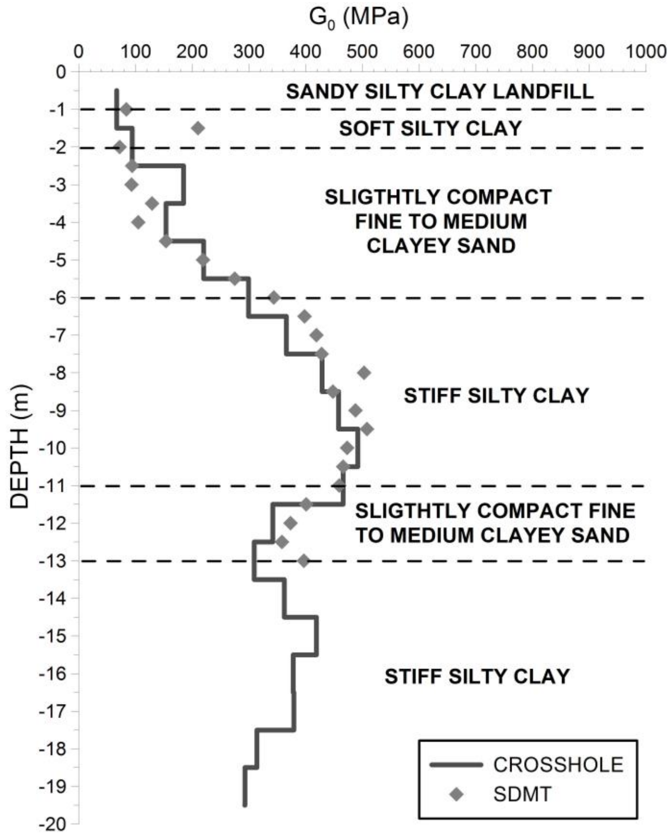

Figure 3. Crosshole and SDMT tests results.

Figure 4 shows the variation of G_0 from crosshole test with N_{SPT} for the lateritic layers (Layers 2 to 5). As the energy level in typical tests performed in Brazil is 72% (Décourt et al., 1989), the N_{SPT} values in this figure were converted to N_{60}.

The following relationship (Imai & Tonouchi, 1982) obtained from Japanese soils is considered in this paper as representative of non-lateritic soils:

$$G_0 = 11.96\ N_{60}^{0.68} \qquad (2)$$

The original relationship was modified to N_{60} considering energy level in SPT performed in Japan equal to 76% (Décourt et al., 1989).

This expression underestimates in a significant way the G_0 values of the lateritic soils at this site. The relationship between measured values and estimated by Imai and Tonouchi relationship is greater than 2.5 times.

In the same figure the two forms of regression proposed by Barros & Pinto (1997) for Brazilian lateritic soils are also presented, according to the following relationships, also converted to N_{60}:

$$G_0 = 48.9\ N_{60}^{0.665} \qquad (3)$$

$$G_0 = 56 + 16.9 N_{60} \qquad (4)$$

The measured values show a much better adjust with Barros & Pinto expressions, manly with the non-linear equation.

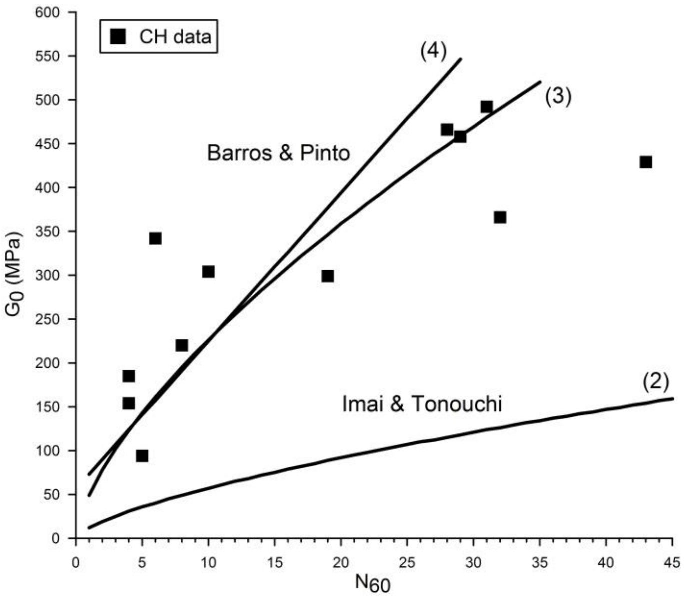

Figure 4. G_0 x N_{60} (SPTT-6).

Figure 5 shows G_0 profile obtained from CPTU data, based on Robertson & Cabal expression (2010). For the investigated soil, the moduli determined by CH and SDMT are higher than those from CPTU (average of the four CPTU tests values performed at the Site III).

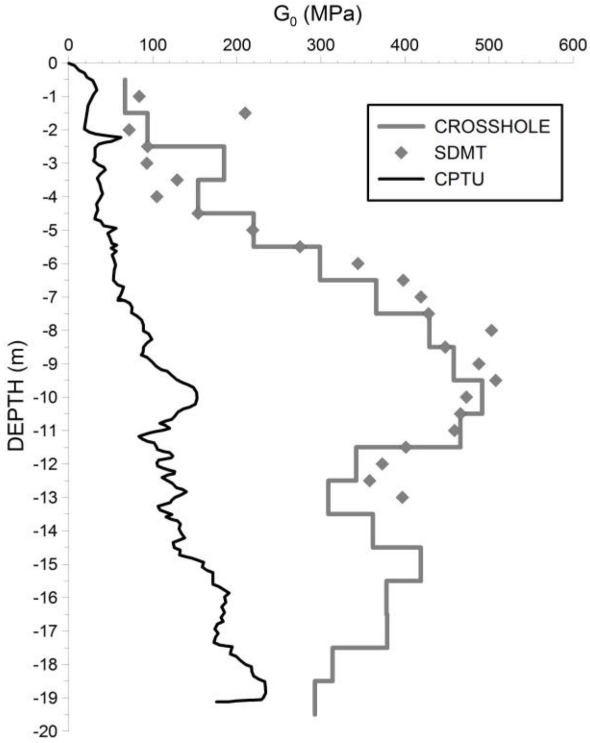

Figure 5. G_0 obtained from CPTU compared with CH and SDMT results.

3.2 *Laboratory tests*

The laboratory tests were carried out on the blocks extracted from upper clay layer (Layer 4) at Site I. The particle size distribution curve of the soil is shown in Figure 6. The Atterberg Limits and the average mass-volume relationships are presented in Table 1.

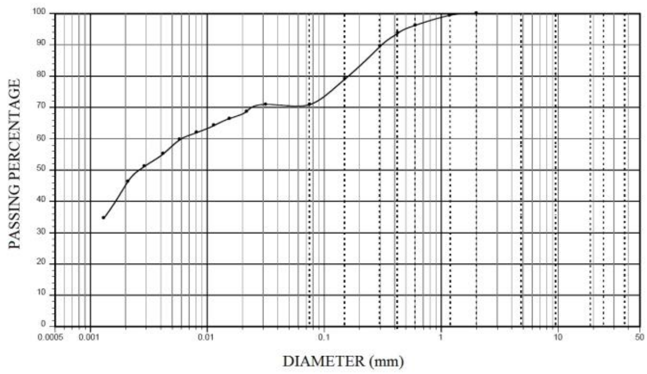

Figure 6. Particle size distribution curve.

Table 1. Atterberg limits and average mass-volume relationships

LL (%)	PI (%)	w (%)	ρ (kg/m³)	e	Sr (%)
57	24	16,4	2081	0,589	79,4

In the MCT Classification the soil was classified in the LG´ group (lateritic clay) and the Laterization Index obtained was 0.43, corresponding to lateritic soils.

Two consolidation tests were performed. As can be seen in Figure 7, although the maximum vertical stress applied has reached 2500 kPa, the virgin curve could not be well defined. However, it is possible to conclude that the maximum past vertical consolidation stress is higher than 300 kPa and probably about 500 kPa.

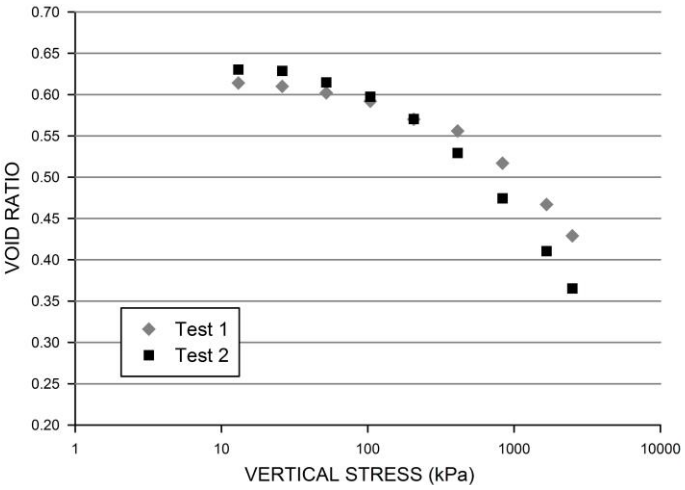

Figure 7. Consolidation test results.

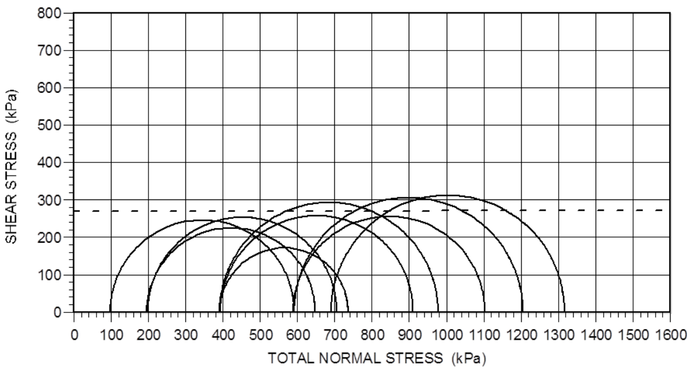

Figure 8 shows the results of the UU triaxial tests. As can be seen, the average value of the undrained cohesion was 270 kPa.

The obtained G_0/S_u ratio was 1550, with G_0 and S_u determined from CH and UU triaxial tests, respectively. Several linear correlations between G_0 and S_u are presented in the technical literature. However, it is difficult to compare the results since both S_u and G_0 values are obtained from different laboratory and field tests. Values from 488 (Hara et al., 1974) to 1800 (Bouckovalas et al., 1989) are encountered in the literature.

Resonant column test was performed in four different levels of confining pressure. Figure 9 shows the G_0 values as a function of the confining pressure. In the same figure is presented the well-known equation proposed by Hardin (1978):

$$G_0 = 625\ OCR^K\ (p_a \sigma_0)^{0.5}/(0{,}3+0{,}7e^2) \qquad (5)$$

where: OCR = overconsolidation ratio; p_a = atmospheric pressure; σ_0 = confining pressure; e = void ratio and K = parameter that depends on Plasticity Index of the soil. The OCR value was computed adopting the maximum past vertical consolidation stress equal to 500 kPa.

It can be seen that Hardin's equation, obtained from data of temperate zone soils, underestimates significantly the G_0 value of the soil investigated in this study.

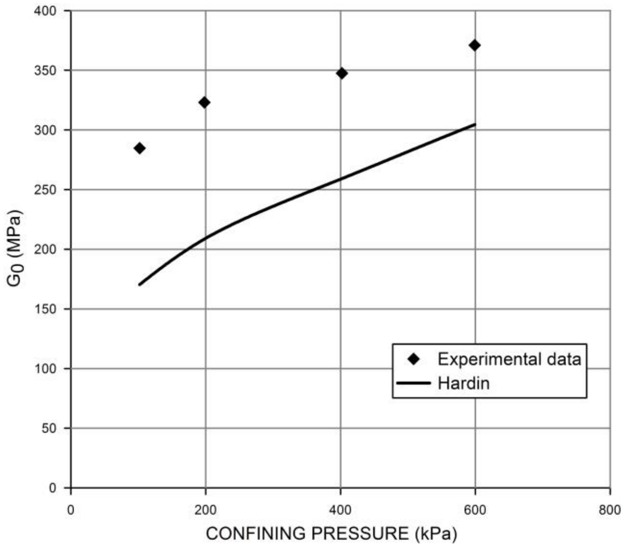

Figure 9. Variation of G_0 with confining pressure obtained in resonant column tests.

4 CONCLUSIONS

G_0 values of the lateritic soil investigated in this study were obtained from different in situ and laboratory tests.

The results of both seismic techniques performed (crosshole and SDMT) showed a very good agreement.

The well-known relationships between G_0 and N_{SPT} (Imai & Tonouchi) and G_0 and q_c (Robertson & Cabal), both representative of non-lateritic soils, underestimated in a significant way the G_0 values of the lateritic soils at the site investigated. By using the relationships proposed by Barros & Pinto for lateritic soils, a much better match was observed.

MCT Classification and the Laterization Index determined in laboratory confirmed lateritic behavior of the soil block extracted from the stiff silty clay (Layer 4). The Hardin expression also underestimated the G_0 values obtained by resonant column tests in laboratory.

Different tests performed in this study showed that lateritic soils present stiffness significantly higher than temperate zone soils. This high rigidity is resultant of the cementation that occurs between adjacent particles.

Empirical correlations commonly used for G_0 estimation for temperate zone soils are not adequate and should not be used for lateritic soils.

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