Geotechnical characterization of the estuarine deltaic deposits in the Guayaquil City through in situ and laboratory tests.

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HIGHLIGHTS

Geotechnical and geophysical characterization of a test site in Ecuador

Comparative analyses of the geotechnical and geophysical parameters for soft clays with diatoms

Shear wave velocity predictive equations using different in situ tests

Stiffness decay curves estimated using the seismic dilatometer test

ABSTRACT

Guayaquil city is located on the West margin of the Guayas River along the Pacific coast of South America. According to research and zoning from previous studies a large area of the city sits on estuarine deltaic deposits which consist of weak and highly compressible clays with diatoms. The nature of these soft clays may determine difficulties in the use of some methods or equations, and consequently in the reliability of the obtained interpretation of the results. This paper focuses on evaluating the most recommended methods and equations for this type of deposits. In this respect, a detailed geotechnical and geophysical characterization of the study area has been carried out. Borehole logs, standard penetration tests (SPT), piezocone tests (CPTu), seismic dilatometer test (SDMT), non-invasive geophysical survey and laboratory tests were performed and compared to analyze static and dynamic geotechnical parameters of these soft clays, resulted sensitive to the presence of diatoms.

Keywords: seismic dilatometer test, piezocone test, soft clays, diatoms, estuarine deltaic deposits, geotechnical characterization.

1. Introduction

In the last decades, Guayaquil soils have been widely studied because of an increasing urban process that the Ecuadorian city has experienced. Nevertheless, limited information is available in the literature about estimation of geotechnical parameters related to this area.

The estuarine zone of the Guayas River deposits is highly heterogeneous. The soil stratigraphy consists of very soft, weak, and highly compressible sediment over hard rocks of Piñon and Cayo Fm. (Vera-Grunauer, 2014). These soils, once analyzed microscopically, show in their matrix clay minerals of heterogeneous composition. One of these components are diatoms. Diatoms are single shelled plants that grow in fresh or salty water rich in dissolved silica, consuming the dissolved silica to build up their skeletons (Treguer et al., 1995; Antonides, 1998). The chemical composition of diatoms and their porous microstructure affect clay behavior, because the diatom skeletons or frustules contain a large number of voids, approximately between 60 and 70% according to Losic et al. (2007). These spaces allow great absorption of water which leads to a possible alteration of the soil properties. Díaz-Rodríguez et al. (2013) also determined that microfossils in significant quantities influence the soil behavior, especially with reference to compressibility parameters.

Caicedo et al. (2018) established that for Bogotá soils, diatoms increase the plasticity index (PI), compromising the use of the Unified Soil Classification System (USCS, ASTM, 2011). Shiwakoti et al. (2002) reached a similar consideration indicating that the Atterberg limits have a significant increase due to the presence of diatoms. Moreover, as long as the concentration of diatoms increases, the coefficients of compressibility and permeability increase. Due to the rough surface and interlocking shape of the minerals, the effective friction angle and shear strength rise too.

Vera-Grunauer (2014) developed several studies on Guayaquil clays with diatoms, confirming the above mentioned evidence. Moreover, Torres et al. (2018) studied the space-temporal variability of phytoplankton and oceanographic variables in the Gulf of Guayaquil between 2013 and 2015, finding 166 species of phytoplankton, 32 of which were diatoms and whose distribution depends on the depth, being more abundant in the first 20 m. This considerable presence of diatoms in Guayaquil deposits assumes importance considering that the majority of the methods or geotechnical correlations are calibrated on datasets that do not consider the diatom content in soft clays. A proper characterization of soil parameters requires an integrated approach whereby the geophysical method, in situ and laboratory tests are used. However, data obtained from in situ tests depend on many factors including stress history, grain size, minerals, composition and packing of the particles. Consequently, a generalized correlation, consistent for some soil types, not necessarily fit well for other geomaterials (Mayne, 2006).

In Ecuador, the standard penetration test (SPT) is overused for the geotechnical design. This practice is attributed to its widespread use worldwide during the last decades, which has led to the collection of a considerable number of data and correlations, considering the limited cost of execution during the cores, the usual availability of the SPT equipment, and the easy execution. However, its use should not be generalized in all soils, especially in soft clays. The results are difficult to interpret in cohesive deposits and, consequently, not conclusive due to the low number of blows. Besides, the samples obtained are highly altered, and therefore they are not representative of the in situ conditions (Mayne et al., 2009). In this respect, it is advisable to use other in situ tests, such as the piezocone test (CPTu) and the seismic dilatometer test (SDMT), to better capture the undrained and drained behavior of cohesive and incoherent soils, respectively. Among the main advantages of in situ tests, there are the ease of execution, the economic savings between taking the sample and its analysis in the laboratory, the reduction in the alteration of the soil by evaluating its natural state, and the possibility of investigating in greater detail the spatial variability of the subsoil according to Devincenzi et al. (2007). This paper aims to compare laboratory test data and in situ test results at a soft clay site in Guayaquil to evaluate and provide the most appropriate correlations to capture the behavior of these estuarine deposits.

2. Site investigation

2.1. Geological setting

Ecuador is considered a country with high seismic risk, being located on an active subduction tectonic margin with direction N80°E (Benítez, 1995; Egüez et al., 2003), where the Nazca plate collides and subducts with the Continental segment formed by the Northern Andean block and the Southern American plate (Chunga et al., 2019).

The study area of this research, Guayaquil city, is located in the Ecuadorian coastal region. This area presents different geological formations, where the three main representative geological units are known as the formations of Guayaquil, Cayo, and Piñon (Benítez et al., 2005). These geomorphological features of Guayaquil supports the convergence of three geological macro-domains: (1) alluvial plain of the Daule and Babahoyo rivers; (2) Chongón-Colonche Cordillera hills; (3) estuarine deltaic complex of the Guayas River.

Vera-Grunauer (2014) developed a seismic microzonation map of Guayaquil, classifying the city into seven lithological units. The study area, namely Murano, is located in Kennedy Norte sector (North-East of the city), along two estuarine branches and characterized by soft unconsolidated sediments. According to Vera-Grunauer (2014), the site corresponds to the lithological unit D3. This zone is defined as Holocene estuarine deltaic deposits, in which sub-classification D3A and D3B correspond to initial elastic periods $T_e < 1.6$ s, $T_e > 1.6$ s, respectively (Figs. 1a and 1b). Kennedy Norte zone presents different lithological features that can amplify or attenuate differently seismic waves into the ground during an earthquake (Chunga et al., 2005). For this reason, it is important to establish geotechnical parameters for a specific site.

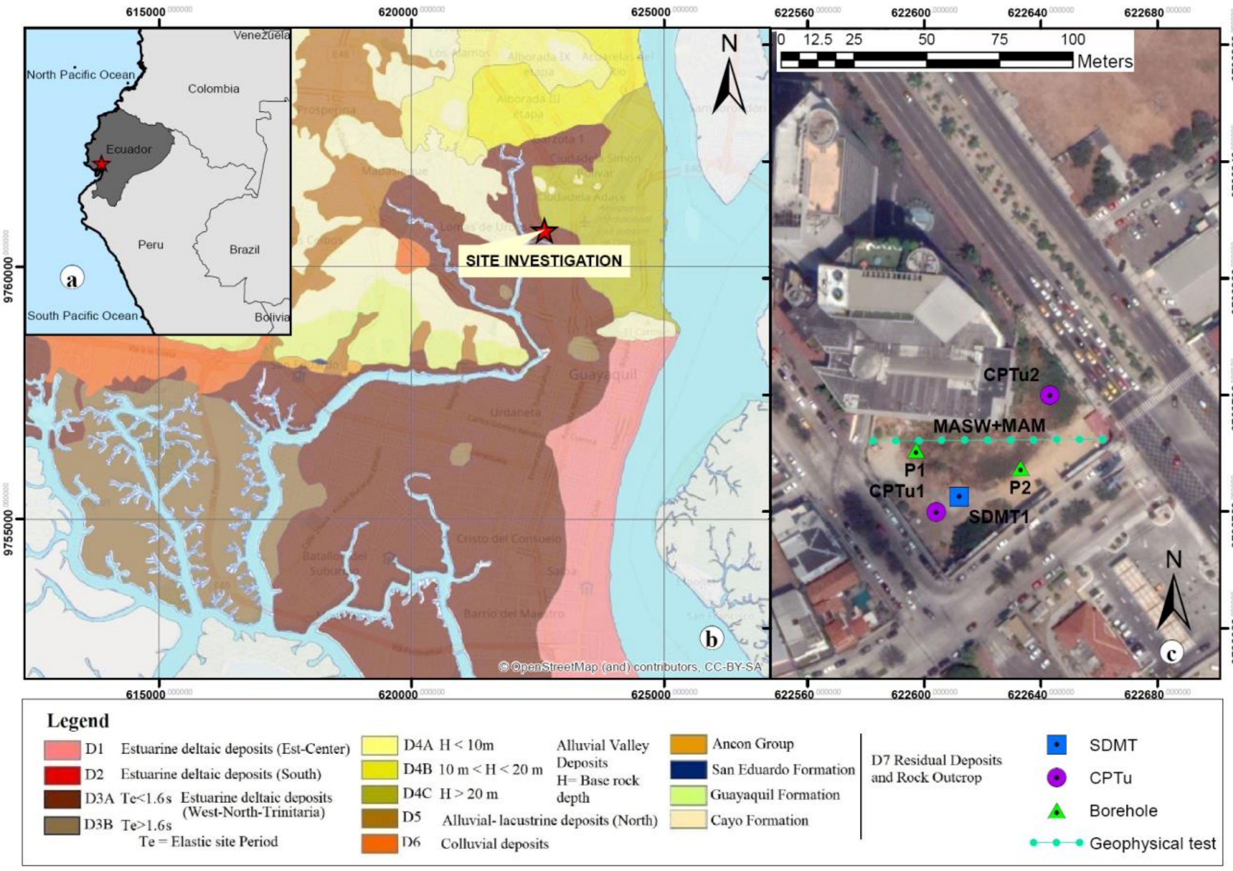

Fig. 1. Location of the test site (a) and geological map of the study area (b). Location of the site investigations at the Murano site (c).

2.2. Description of the Site Campaign

Multiple geotechnical and geophysical surveys were carried out at the site to reconstruct a more accurate subsoil characterization at Murano site. The location of the boreholes and the other investigations is reported in Fig. 1c. The investigations included n. 2 boreholes (P-1 and P-2) at depths of 46 and 45 m respectively, with the execution of SPTs and the retrieval of disturbed samples for soil classification (i.e. sieve analyses and Atterberg limits) approximately each meter of depth. Moreover, n. 3 undisturbed samples were collected at 3.70, 6.60 and 7.50 m of depth to perform oedometric and unconfined compression tests. In situ tests included n. 2 piezocone tests, namely CPTu1 and CPTu2, at 41 and 30 m of depth respectively, and n. 1 seismic dilatometer (SDMT1) test at 31 m of depth. Due to the irregular topography of the area for the presence of a non-penetrable fill, in situ tests needed a predrilled hole up to 2 m depth. Dissipation tests were performed at specific depths for both CPTu and SDMT tests (see Table 1 for details). With reference to geophysical measurements a multichannel analysis of surface waves (MASW) survey was performed with a microtremor array measurement (MAM) test for a total length of 80.5 m.

Table 1. Summary of the field investigations performed at Murano site.

Field test	Depth* (m)	Dissipation test depth (m)	Disturbed samples	Undisturbed samples	SPT per borehole	GWT depth (m)	Test date
P1	46.00	-	45	1	45	1.80	14/11/2018
P2	45.00	-	45	2	45	2.00	08/11/2018
CPTu1	41.00	10.60; 13.45	-	-	-	2.05	12/11/2018
CPTu2	30.00	8.00; 12.00	-	-	-	1.82	09/11/2018
MASW+MAM	80.50	-	-	-	-	-	-
SDMT1	31.40	8.00	-	-	-	3.00	05/08/2018

*For MASW+MAM this refers to length.

2.3 Direct Measurements

Fig. 2 summarizes the results of the direct measurements obtained from the in situ geotechnical and geophysical investigations. In particular for SPT the blow counts (N_{SPT}), necessary to penetrate the sampler 300 mm into the ground, after advanced first penetration of 150 mm, are reported. For CPTu, the readings of the corrected cone resistance (q_t), sleeve friction (f_s), and pore water pressure (u_2) are plotted. For the flat dilatometer (DMT) test, introduced by Marchetti (1980), the profiles of the two corrected pressure readings, namely p_0 (1st reading) and p_1 (2nd reading), are illustrated in Fig. 2. The low N_{SPT} and q_t measurements and the high f_s and u_2 values in the upper 30 m of depth, together with the proximity of p_0 and p_1 pressures depth by depths, agree to preliminarily identify the profile of a soft and clayey soil. Moreover, for DMT one measurement for the 3rd corrected pressure reading (p_2) is available and equal to 135 kPa in a thin sandy layer located at 17 m of depth. According to Marchetti et al. (2001), p_2 values are generally used to estimate the hydrostatic water level in incoherent deposits. Therefore, the ground water level (GWT) can be estimated at 3 m of depth at the DMT test site. CPTu test can be also used to estimate the GWT through u_2; in this case GWT is at about 2 m for CPTu1 and 1.82 m for CPTu2. For boreholes P1 and P2, GWT was measured at 1.8 and 2 meters of depth respectively. CPTu tests and boreholes were performed roughly in the same wet period (see Table 1), which justifies the good agreement between GWT results. On the contrary, SDMT1 was performed in the Ecuadorian dry season, explaining the GWT variation due to seasonal fluctuations.

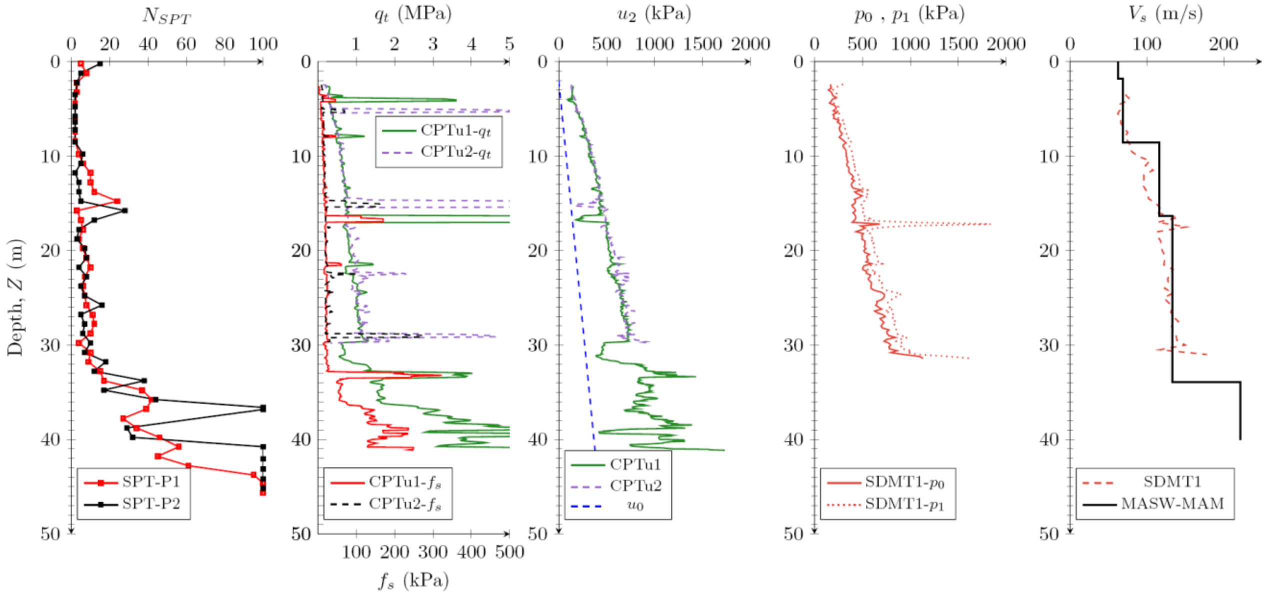

Fig. 2. Measured parameters for geotechnical and geophysical tests at Murano site: SPT blow counts (N_{SPT}), corrected cone resistance (q_t), sleeve friction (f_s), pore water pressure (u_2), corrected DMT readings (p_0, p_1), shear wave velocity (V_s).

Finally, Fig. 2 plots the shear wave velocity (V_s) data carried out from the SDMT test (Marchetti et al., 2008) and from the combined interpretation of the dispersion curves related to the active MASW survey and the passive MAM measurements (Park et al., 2007). The two independent V_s profiles highlight a good agreement between the geophysical and geotechnical methods, thus strengthening the reliability of the acquisitions at the test site.

The passive measurements also provide the ambient noise vibrations, in terms of Horizontal to Vertical, namely H/V, curves, as shown in Fig. 3. The graph detects a peak frequency (f_0) at 0.796 Hz, that corresponds to an elastic period (T_e) of 1.256 s. This value correctly matches with the seismic microzonation study (Vera-Grunauer, 2014) that identifies the Murano area as D3A zone, namely estuarine deltaic deposits with T_e < 1.6 s.

Other direct measurements obtained at the site are related to dissipation tests from CPTu and DMT tests, as for the coefficient of consolidation in horizontal direction (c_h) (Robertson et al., 1992; Marchetti and Totani, 1989). Fig. 4a shows the results of the CPTu pore water pressure (u_2) with the time (t) together with the points corresponding to the measured time for the 50% of the dissipation (t_{50}), while Fig. 4b illustrates the profile of the non-corrected 1st DMT reading (A) with the time (t) in combination with the contraflexure point of the curve (t_{flex}). c_h values obtained in the layer at 8.00 m depth by CPTu and DMT is in good agreement and is equal to an average of $3 \cdot 10^{-5}$ m²/sec, while c_h provides values between $2 \cdot 10^{-5}$ and $5 \cdot 10^{-5}$ m²/sec in the bottom layer between 10.60 and 13.45 m of depth.

All together the SPT, CPTu, SDMT and MASW+MAM direct measurements preliminarily agree to identify a homogeneous site. However, the data interpretation that will follow in the next paragraphs will provide further details.

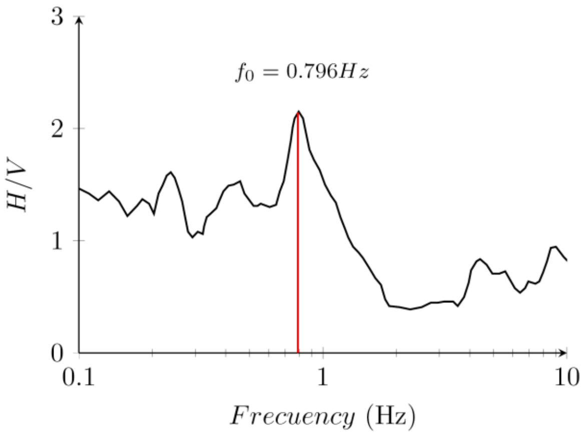

Fig. 3. H/V curves from passive measurements.

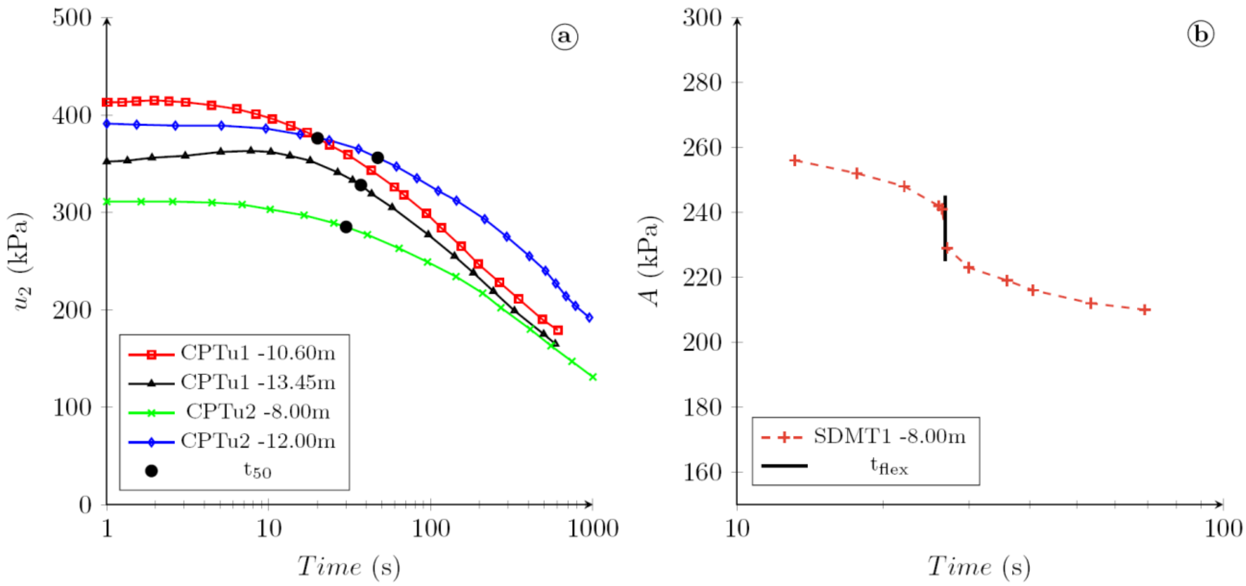

Fig. 4. Dissipation tests from CPTu (a) and DMT (b) tests.

3. Geotechnical characterization of the test site using in situ and laboratory tests

3.1. Soil classification

Laboratory and in situ testing were analyzed to obtain a detailed soil classification. Fig. 5 shows the borehole log using USCS soil classification, the soil composition, the Atterberg limits (liquid limit LL, plastic limit LP), the plasticity index (PI) and the water content (w), the CPT soil behavior type index (I_c) and the DMT material index (I_D).

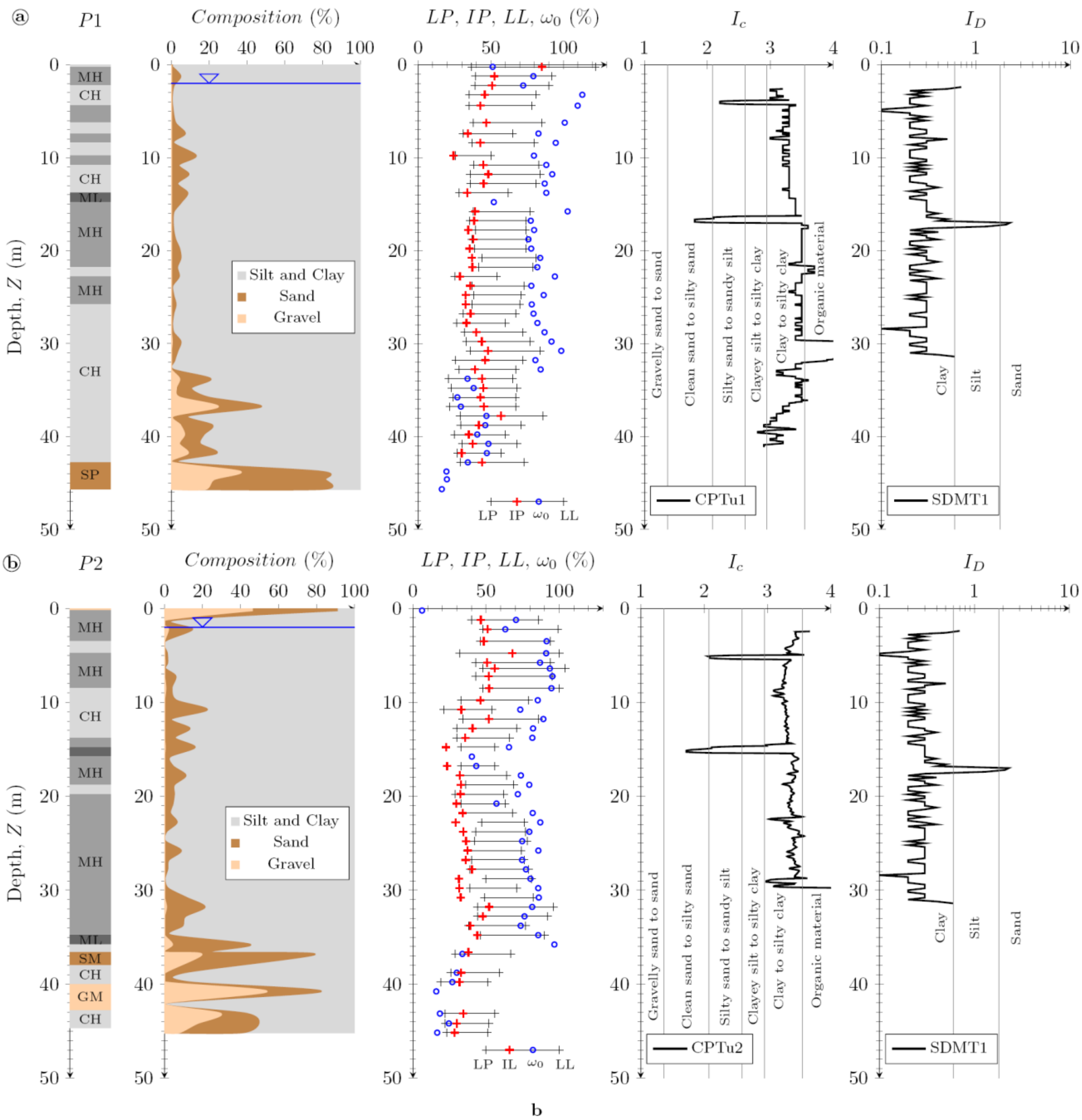

Fig. 5. Soil classification using USCS method, CPTu and DMT interpretations, soil composition and basic properties for boreholes P1 (a) and P2 (b).

The soil stratigraphy is apparently quite uniform up to approximately 33-37 m, showing mainly clays with high plasticity (mean plasticity index PI > 40%) and liquid limit (mean liquid limit LL >

70%). In particular, from 0 to 15 m the predominance of silts and clays soil is observed, characterized by an average PI of 46% and w of about 86%. The fines continue to predominate from 15 to 30-37 m, but the percent of sand starts to increase and the IP and w values decrease staying in a range of 30-50% and 70-90%, respectively. Below 30-37 m of depth, the percentage of sand continues to increase up to 60% and also a relevant presence of gravel (37-54%) is encountered. Consequently, Atterberg limits and water content values decrease.

I_c and I_D profiles, estimated using Robertson and Cabal (2015) and Marchetti et al. (2001) respectively, are in broad agreement with the soil stratigraphy obtained from the boreholes and the lab testing, since in situ tests detect on average a clay layer up to 40 m depth with a thin sandy sand lens between 15 and 17 m depth. However, there is not a perfect correspondence between the CPT-DMT geotechnical description and the grading curves (i.e. soil composition percent), since both I_c and I_D are parameters related to the mechanical soil response and not strictly to the grain size of the soil deposits (e.g. Boncio et al., 2020). The integrated information of gradations and index properties may find better agreement with I_c and I_D values. For example, correspondence to low-plastic deposits by P2 (Fig. 5b) is noticed for the silty sands detected by CPTu2 at about 15 m.

Finally, it can be observed that for most of the soil samples within the upper 30-37 m depth w is generally higher than 70%, recording also values bigger than 100%. Particularly, highest PI, LL, LP and w values are concentrated in the upper 15 m depth. This information is in relevant agreement with Vera-Grunauer (2014) who performed scanning electron micrographs for soil samples taken at sites close to the studied area. Vera-Grunauer (2014) observed that the microporous structure of diatoms has a diameter less than 0.5 μm which generates a large specific surface area and allows the absorption of a large amount of water. Moreover, Vera-Grunauer (2014) developed a graph in which it is evidenced that for the lithological zone D3, where the study area is located, the relationship between w and LL varies mainly between 0.8 to 1.2, presenting a greater tendency to values close to 1. This information is consistent with the present research where it can be verified that the water content is greater or very close to the liquid limit, as shown in Fig. 5. Therefore, the greater water content is because of water absorption caused by the diatom pores as mentioned above.

3.2. Strength and compressibility

The soil total unit weight (γ) is an important parameter because it indirectly shows an idea of the field state of stress of a soil at a desired depth (Rodríguez et al., 2015). The total unit weight value depends on the water content located in the voids as well as the density or weight of the mineral grains. Recommended values of total unit weight were proposed by Look (2007) for cohesive soils, from soft organic with $\gamma \approx 14$ kN/m^3, to soft non organic with $\gamma \approx 16$ kN/m^3 and to stiff to hard with γ between 18 to 20 kN/m^3. Laboratory tests on undisturbed samples (ED-P1, ED-P2, Fig. 6) carried out at the Murano site show $\gamma \approx$ 14-16kN/m^3. These low values are in agreement with typical behavior of soft soils and are probably related also the presence of siliceous diatoms randomly distributed in the soil mass (Vera-Grunauer, 2014).

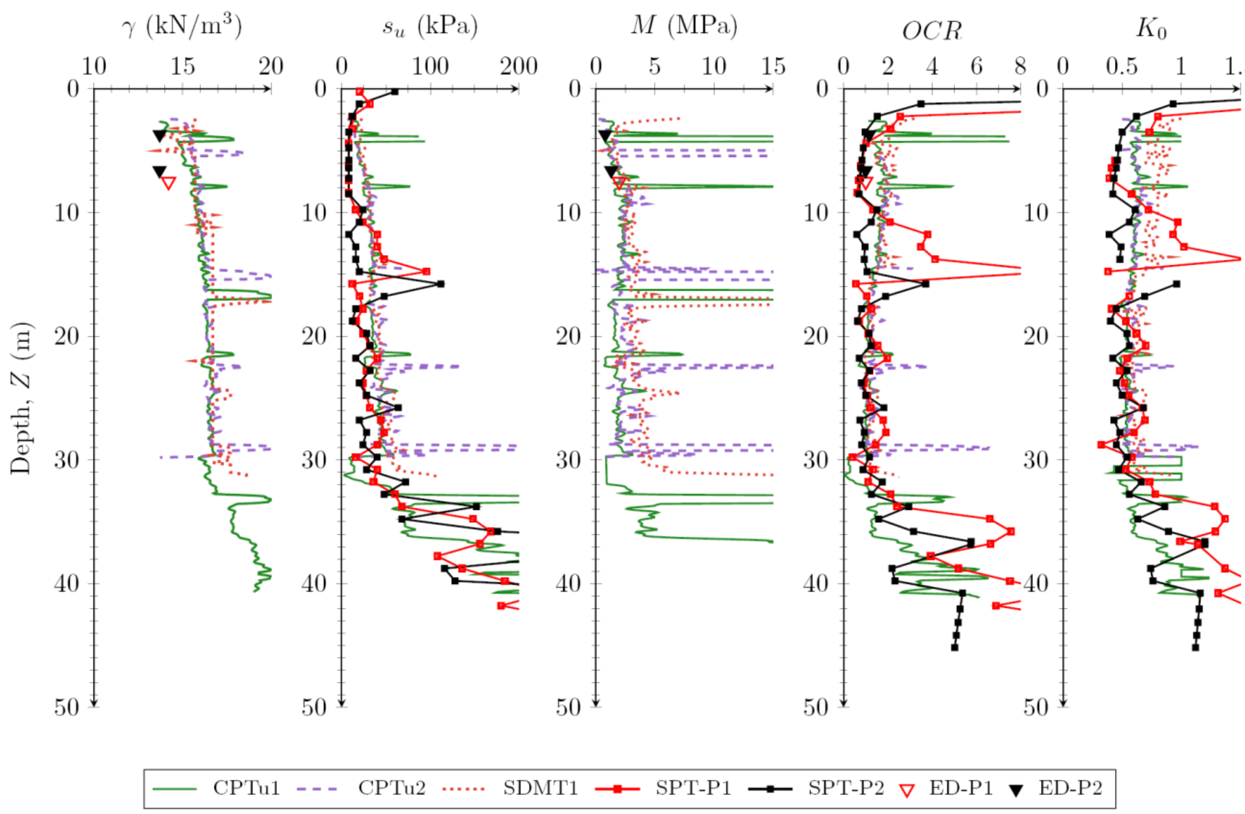

Fig. 6. Geotechnical parameters estimated from laboratory and in situ tests.

On the contrary, the γ profiles obtained from CPTu and DMT interpretations, according to Robertson and Cabal (2015) and Marchetti and Crapps (1981) charts respectively, provide higher unit weight values in correspondence of coarser layers. Simultaneously, a general increase of γ is detectable with the depth into the homogeneous clay, due to the increase of effective vertical stress (σ'_{v0}). In general, soil unit weights are best obtained by relatively undisturbed samples, while the main scope of CPTu and DMT charts is not an accurate estimation of γ, but the possibility of constructing an approximate σ'_{v0} profile, needed in the elaboration of the in situ tests (Robertson and Cabal, 2015; Marchetti et al., 2001).

Undrained shear strength (s_u) coupled with total stress analysis is often used to examine the failure state of geotechnical structures under undrained conditions in Guayaquil City (Vera-Grunauer, 2014). For SPT, Brown and Hettiarachchi (2008) recommend a correlation based on the energy corrected SPT blow count (N_{60}) values:

$$s_u=4.1\cdot N_{60} \tag{1}$$

with

$$N_{60} = C_E \cdot N_{SPT} \tag{2}$$

C_E is the energy correction factor, equal to 1.02-1.04, obtained from the measurements of the measured hammer energy at the Murano site.

A theoretical solution for CPTu test interpretation is in the form:

$$s_u=(q_t-\sigma_{v0})/N_{kt} \tag{3}$$

where σ_{v0} is the total vertical stress and N_{kt} is a factor that varies from 10 to 18, with 14 as an average (Robertson, 2010).

Finally, for DMT test s_u is obtained by the following equation (Marchetti, 1980):

$$s_u= 0.22\cdot\sigma'_{v0}(0.5\cdot K_D)^{1.25} \tag{4}$$

with

$$K_D=(p_0-u_0)/\sigma'_{v0} \tag{5}$$

where p_0 is the corrected 1st reading and u_0 is the hydrostatic pore water pressure. K_D is the horizontal stress index that provides the basis for several soil parameter correlations, coming out as the key result of the dilatometer test. In this respect, K_D can be regarded as an amplified in situ earth pressure coefficient (K_0) because (p_0 - u_0) is an "amplified" horizontal effective stress (σ'_{h0}), due to penetration. In genuinely NC clays (no aging, structure, cementation) the value of K_D is approximately equal to 2, and this justifies that the K_D profile is similar in shape to the overconsolidation ratio (OCR) profile, hence generally helpful for "understanding" the soil deposit and its stress history (Marchetti, 1980; Jamiolkowski et al., 1988).

For s_u estimations CPTu and DMT present a satisfactory agreement, while SPT provides lower values within the upper 20 m depth, moving closer to DMT and CPTu prediction at greater depth. This is related to the fact that SPT may not capture the effect produced by diatoms and cementation in clays due to the natural confinement and microstructure. SPT evaluation is susceptible to distortion produced by the incorrect data processing and the calibration for the test execution.

Penetration test results are most commonly used to estimate the settlement behavior of the soils, using the constrained modulus (M), that depend on the stress state, soil type, and degree of preconsolidation. These dependencies are incorporated into CPT and DMT empirical correlations since M from CPT (Robertson, 2009) is related to the soil behavior type index (I_c) and to the in situ vertical stress, and M from DMT (Marchetti, 1980) is a function of the material index (I_D), of the horizontal stress index (K_D) and of the dilatometer modulus (E_D, i.e. the elastic modulus of the horizontal load test performed by the DMT membrane with 60 mm diameter and the 1.1 mm displacement). The results evidence a correspondence between the samples evaluated by oedometer test and in situ CPT and DMT tests within the upper 7 m depth, providing $M \approx 0.5$-2.03 MPa. CPT and DMT predictions are still in reasonable agreement between 7 and 15 m depth ($M \approx 2.5$ MPa) while at greater depths, DMT always provides higher values compared to CPT. According to the numerous case histories available in the literature (e.g. Monaco et al. 2006, 2014; Schmertmann, 1986, 1988; Mayne, 2005; Berisavljevic 2017) DMT usually provides a good agreement between measured and DMT-predicted settlements thanks to the high reliability of the DMT constrained modulus M that is a working strain modulus. M by DMT is therefore associated with an intermediate strain level, more appropriate for the settlement calculations. In contrast, penetration tests, like CPT, working at higher strains due to the considerable distortion induced by the CPT conical tip, produce a less reliable M estimation (Baligh and Scott, 1975; Mayne, 2001).

For evaluating the overconsolidation, the abovementioned strong dependence between K_D and stress history in uncemented NC clay allowed the development of the following equation (Marchetti, 1980):

$$OCR= (0.5 \cdot K_D)^{1.56} \tag{8}$$

Later, Kulhawy and Mayne (1990), Mayne and Liao (2004) and Mayne (2016) noticed that the OCR in CPT tests significantly influences the normalized values of the q_t, suggesting to use the following formula in the analysis of fine-grained soils:

$$OCR=0.3 \cdot (q_t - \sigma_{v0})/\sigma'_{v0} \tag{9}$$

For SPT analysis, OCR was estimated using SHANSEP approach and site parameters S and m, proposed by Vera-Grunauer (2014):

$$s_u/\sigma'_{v0}=S \cdot (OCR)^m \tag{10}$$

The values of S and m can be assumed equal to 0.22 and 0.75, respectively, for D3 estuarine deltaic zone of Guayaquil, while s_u was obtained using Eq. (1). Fig. 6 shows OCR values from in situ and oedometer tests. CPT and DMT are in good agreement for the entire profile, estimating OCR ≈ 2 within the upper 15 m and a more defined NC behavior (OCR ≈ 1) approximately between 15 and 30 m. On the contrary, oedometric tests underestimate OCR up to 15 m depth. This may be due to the difficulties to retrieve high quality samples on soft clay soils which prevents to preserve soil structure (e.g. Berisavljević et al., 2014).

CPT and SPT tests also provide a rough estimate of the in situ earth pressure coefficient (K_0) for low plastic fine-grained soils, using the OCR values estimated by each own test (Kullhawy and Mayne, 1990):

$$K_0=0.5\cdot OCR^{0.5} \tag{11}$$

On the contrary DMT provide a reliable K_0 correlation in clay obtained experimentally by Marchetti (1980) and theoretically by Yu (2004):

$$K_0=(K_D/\ 1.5\ )^{0.47}-0.6 \tag{12}$$

SPT and CPT underestimate K_0 in the upper 15 m depth where DMT evaluates $K_0 \approx 0.8$. This inaccuracy is due to the considerable scatter that exists in the CPT and SPT database used to determine these correlations (Robertson and Cabal, 2015). On the contrary K_0 estimations by all the in situ tests are in reasonable agreement at greater depth providing an average NC value of 0.6 between 15 and 30 m depth. This difference between the geotechnical behavior of the upper 15 m-thick layer and the lower 15 m-thick layer, as detected by OCR, K_0 and index parameters (LL, LP, IP, w), may be interpreted as a different concentration of diatoms, higher in the top layer rather than in the bottom one. This assumption is consistent with the analyses of some authors such as Vera-Grunauer (2014), who determined for clays in areas very close to the Murano site that up to 14.5 m depth diatoms are abundant with a density between 5 to 6 million per gram and very well preserved. Moreover, Torres et al. (2018) after studying the temporal variability of diatoms determined that the highest abundance occurs in the first 20 meters.

3.3. Permeability (k)

The vertical coefficient of permeability (k_v) was calculated from the oedometric test, for which the compressibility curve was constructed with the Casagrande methodology and its respective Schmertmann correction (Schmertmann, 1955). In situ tests were also used to determine permeability. Robertson (2010) developed a correlation between the soil behavior type index (I_c) and the coefficient of permeability (k) to obtain an entire but approximate permeability profile that is not sensitive to the anisotropy of the soil. However, better estimation of the horizontal permeability (k_h) can be provided by dissipation tests from both CPTu and DMT. Teh and Houlsby (1991), Parez and Furiel (1988) and Robertson (2010) relationships were used for CPTu tests, once t_{50}, and consequently c_h, were estimated from dissipation curves (Fig. 4). These three correlations provide similar values, and therefore for clarity in Fig. 6 only Robertson (2010) estimation is shown. Similarly, for DMT test t_{flex} and c_h were used to estimate k_h according to Marchetti and Totani (1989).

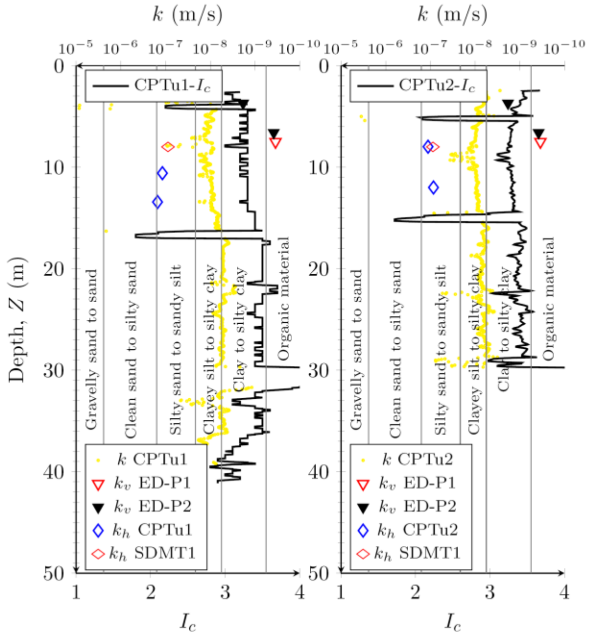

Fig. 7. Permeability estimates together with soil behaviour index profiles for CPTu1 and CPTu2.

The results obtained at Murano site show that all the correlations used to estimate k_h from CPTu and DMT dissipation tests give very similar interpretations in the soft clays between 8.00 and 13.45 m depth ($k_h \approx 10^{-7}$ m/s), while the continuous k profile derived from I_c only agrees with the laboratory data in the upper and most permeable clayey layer ($\leq$ 4 m depth). Higher permeability is encountered in sandy soils ($k \approx 10^{-6}$ m/s) while lower values are confined to soft clay ($k \leq 10^{-9}$ m/s) in reasonable agreement with permeability ranges obtained by Holtz and Kovacs (1981). In summary, it results indistinct the selection of one permeability correlation in place of another one as long as a dissipation test is performed, while the “approximation” of k to I_c should only be used as a guide and option in the absence of dissipation tests, since this methodology can provide results different for one order of magnitude (or even more), as for the case study of Murano site. Finally, the results of the dissipation tests are not consistent with the results of the oedometric test even correcting laboratory-k_v in k_h. According to Tavenas et al. (1983) the permeability measured by oedometric test regularly gives very low values as a consequence of the hypotheses of Terzaghi's 1-

D theory, which considers that the material is isotropic and homogeneous, and therefore it implies the assumption of constant k, M, and c_v during the consolidation.

4. Dynamic soil properties at the test site using geotechnical and geophysical measurements

4.1. Shear wave velocity

The estimation of the shear wave velocity (V_s) is fundamental in geotechnical engineering design, not only for site classification and soil-structure interaction, but also for earthquake analysis and site response. Penetration tests can be used for predicting V_s through some measured parameters. In particular, DMT allows to estimate the small strain shear modulus (G_0), based on the intermediate parameters I_D, K_D, M (Marchetti et al., 2008) and hence V_s, though the theory of elasticity, as follow:

$$V_s = \sqrt{G_0/\rho} \tag{13}$$

Where ρ is soil density. The equations to predict G_0 are listed in Table 2:

Table 2. Equations to estimate V_s from DMT according to Marchetti et al. (2008).

Soil type	G_0 correlation
Silts: $0.6<I_D<1.8$	$G_0=M\cdot 15.686\cdot K_D^{-0.921}$
Clays: $I_D<0.6$	$G_0=M\cdot 26.177\cdot K_D^{-1.0066}$
Sands: $I_D>1.8$	$G_0=M\cdot 4.5613\cdot K_D^{-0.7967}$

Several authors have developed and recommended correlations for SPT, expressed as a function of N_{SPT}, N_{60}, depth (Z), soil type and geological age (Table 3). Finally, for CPT several correlations are available to predict V_s, that are related to numerous parameters like tip resistance (cone tip resistance q_c or corrected cone tip resistance q_t), sleeve friction (f_s),confining stress, depth (Z), soil type, and geologic age (Table 4).

Table 3. Main available equations to estimate V_s from SPT.

Author	Soil Type	V_S correlation	Geological description
Wair et al. (2012)	All soils	$V_s = 26 \cdot N_{60}^{0.215} \cdot \sigma'_{v0}{}^{0.275}$	Holocene
	All soils	$V_s = 34 \cdot N_{60}^{0.215} \cdot \sigma'_{v0}{}^{0.275}$	Pleistocene
	Clays and silts	$V_s = 23 \cdot N_{60}^{0.17} \cdot \sigma'_{v0}{}^{0.32}$	Holocene
	Clays and silts	$V_s = 29 \cdot N_{60}^{0.17} \cdot \sigma'_{v0}{}^{0.32}$	Pleistocene
	Sands	$V_s = 27 \cdot N_{60}^{0.23} \cdot \sigma'_{v0}{}^{0.23}$	Holocene
	Sands	$V_s = 35 \cdot N_{60}^{0.23} \cdot \sigma'_{v0}{}^{0.25}$	Pleistocene
Imai and Yoshimura (1970)	All soils	$V_s = 76 \cdot N_{SPT}^{0.33}$	-
Kalteziotis et al. (1992)	All soils	$V_s = 76.2 \cdot N_{SPT}^{0.24}$	-
	Sands and silts	$V_s = 49.1 \cdot N_{SPT}^{0.502}$	
	Clays	$V_s = 76.55 \cdot N_{SPT}^{0.445}$	
Ohsaki and Iwasaki (1973)	All soils	$V_s = 81.4 \cdot N_{SPT}^{0.39}$	-
	Sands	$V_s = 59.4 \cdot N_{SPT}^{0.47}$	
Iyisan (1996)	All soils	$V_s = 51.5 \cdot N_{SPT}^{0.516}$	Deep alluvial deposits
Jinan (1987)	All soils	$V_s = 116.10 \cdot (N_{SPT}+0.32)^{0.202}$	Soft Holocene deposits
Dikmen (2009)	All soils	$V_s = 58 \cdot N_{SPT}^{0.39}$	Quaternary alluvium
	Sands	$V_s = 73 \cdot N_{SPT}^{0.33}$	Quaternary alluvium
	Clays	$V_s = 44 \cdot N_{SPT}^{0.48}$	Quaternary alluvium
	Silt	$V_s = 60 \cdot N_{SPT}^{0.36}$	Quaternary alluvium

Table 4. Main available equations to estimate V_s from CPT.

Author	V_s (or G_0) correlation	Geological description
Robertson (2012)	$V_s = \alpha_{vs} \cdot (q_t - \sigma'_{v0})^{0.5} / p_a$; $\alpha_{vs} = 10^{0.55 \cdot I_c + 1.68}$	Holocene and Pleistocene soils, mostly uncemented
Hegazy and Mayne (1995)	$V_s = [10.1 \log(q_t) - 11.4]^{1.67} \cdot f_s / q_t \cdot 100$	All types of soils
Simonini and Cola (2000)	$G_0 = 49.2 \cdot q_c^{0.51}$	Sand, silt and silty clay of Venice Lagoon
Andrus et al. (2007)	$V_s = 2.27 \cdot q_t^{0.412} \cdot I_c^{0.989} Z^{0.033} \cdot ASF$; ASF = 1.00	Holocene soils
	$V_s = 2.62 \cdot q_t^{0.395} \cdot I_c^{0.912} Z^{0.124} \cdot SF$; SF = 1.12	Pleistocene soils
Madiai and Simoni (2004)	$V_s = 140 \cdot q_c^{0.30} \cdot f_s^{-0.13}$	Holocene cohesive soils
	$V_s = 268 \cdot q_c^{0.21} \cdot f_s^{0.02}$	Holocene incoherent soils
	$V_s = 182 \cdot q_c^{0.33} \cdot f_s^{-0.02}$	Pleistocene cohesive soils
	$V_s = 172 \cdot q_c^{0.35} \cdot f_s^{-0.05}$	Pleistocene incoherent soils
Bouchovalas et al. (1989)	$G_0 = 28.0 \cdot q_c^{1.40}$	Very soft clays
Vera-Grunauer (2014)	$V_s = \sqrt{\eta \cdot q_c e^{\alpha}}$; $\alpha = [(3 N_{kc} - 4) / 4] - [1 / (2\beta)]$; $\eta = 3g / [2 N_{kc} \cdot \gamma_s \cdot (1+\nu)]$	Clays with diatoms

p_a = atmospheric pressure; ASF = Age scaling factor; SF = Scaling factor; γ_s = volumetric weight; g = gravity; N_{kc}=correlation factor; β = ratio between undrained shear strength and effective vertical stress; ν = Poisson's constant.

Fig. 8a provides the comparison between V_s measured and V_s predicted by DMT, that shows a reasonable agreement. There is a slight overestimation of DMT values, more pronounced in the upper 15 m that could be related to the higher concentration of the diatoms as previously detected by K_D (through K_0 and OCR) that is noticeable more reactive to stress history, structure and prestraining/aging, scarcely felt by q_c (or q_t) from CPT (Amoroso, 2014). A large number of correlations have been developed for SPT, involving the soil type, the geological description, and sometimes the in situ stress. This results in a wide variability (Fig. 8b) within the V_s profiles, as previously noted also by other authors in other sites (e.g. Fabbrocino et al. 2015, Akin et al., 2011), and confirmed for the soft clay deposits of Murano test site (Fig. 8c) (e.g. Jinan, 1987 estimates values up to two times the measured ones).

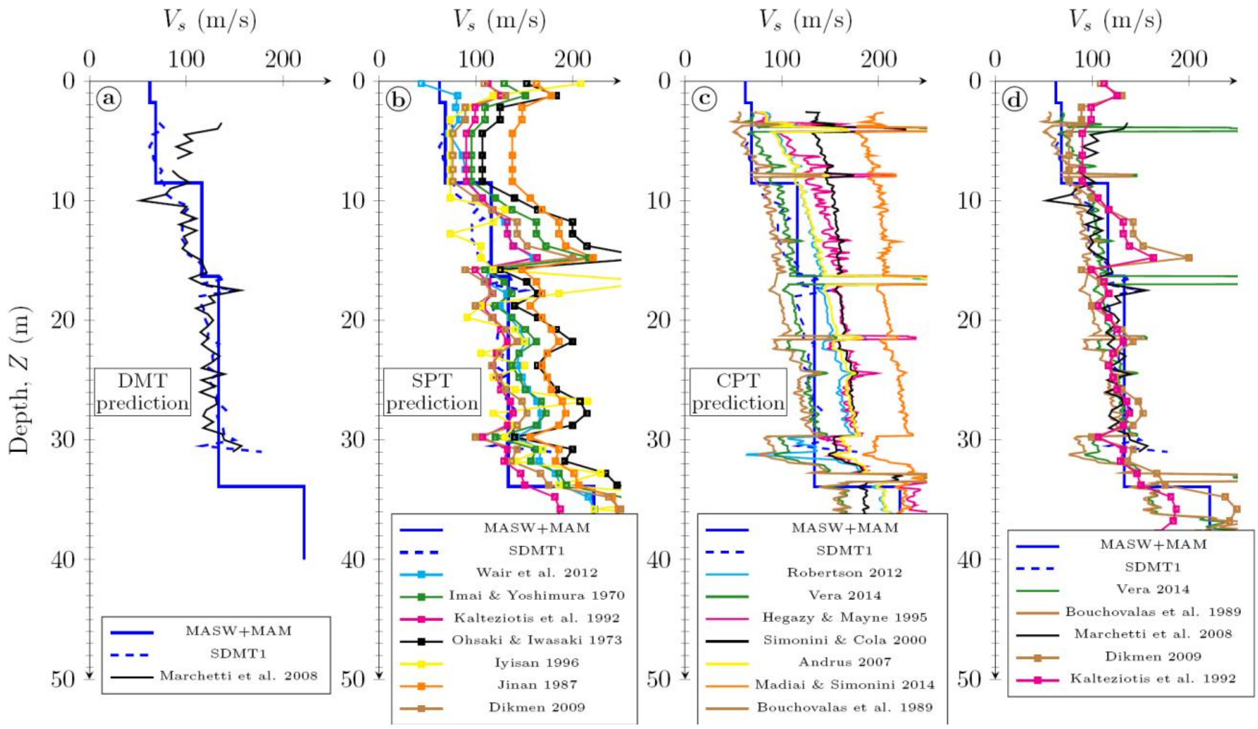

Fig. 8. Comparison of V_s measured and V_s predicted by DMT (a), SPT (b), and CPTu (c); comparison of V_s measured and V_s predicted using the best correlations. The plots correspond to borehole P1, CPTu1 and SDMT1 tests.

A similar behaviour (Fig. 8c) is observed with the V_s correlations developed for CPT test (e.g. Robertson, 2012 estimates values up to four times the measured ones). The arisen uncertainty could be due to the dependency to numerous and different parameters mentioned above that CPT and SPT parameters may not capture correctly. However, it is possible to select the best SPT-V_s and CPT-V_s predictions for soft clay deposits using the formulas proposed by Wair et al. (2012), Dikmen (2009) and Kalteziotis et al. (1992) for SPT test. Interestingly, the last two equations developed for all types of soils are in better agreement with the measured V_s profile than those made exclusively for clays. The selected Wair et al. (2012) equation is valid for Holocene clays and silts. For CPT test, Bouchovalas et al. (1989) and Vera-Grunauer (2014) resulted to fit better with Vs measurements, and they are valid for very soft clays and for clays with diatoms (Fig. 8c). In particular, Vera-Grunauer (2014) proposed a site-specific correlation calibrated using Guayaquil dataset, that for D3 zone it established the following input parameters: β=0.22; N_{kc}=12; γ_s=15 KN/m^3; ν=0.3. All together the measured (SDMT, MASW+MAM) and selected-predicted (Marchetti et al., 2008; Wair et al., 2012; Dikmen, 2009; Kalteziotis et al., 1992; Bouchovalas et al., 1989; Vera-Grunauer, 2014) V_s data presented reasonable agreement identifying V_s values increasing in the 30 m depth in range of 50-180 m/s.

4.2. Stiffness decay curves

Finally, in situ tests were used to evaluate stiffness decay curves (G-γ curves). In particular, this opportunity is offered by the seismic dilatometer that allows to estimate the in situ variation of soil stiffness with the level of deformation, as preliminarily suggested by Marchetti et al. (2008) and then tested by Amoroso et al. (2014) and Di Mariano et al. (2019). The method proposes firstly to assess the small strain modulus G_0 though the theory of elasticity using V_s (Eq. 13).

Then it is necessary to evaluate a working strain shear modulus G_{DMT} starting from the constrained modulus (M also named M_{DMT}) obtained from the usual DMT test though the theory of elasticity:

$$G_{DMT} = M_{DMT} \cdot (1-2\upsilon)/[2 \cdot (1-\upsilon)] \tag{14}$$

where υ = Poisson's ratio, assumed equal to 0.3 for all layers.

Amoroso et al. (2014) proposed an equation to determine a hyperbolic stress-strain equation to represent the non-linear soil behavior through a normalized decay curve (G/G_0-γ curve) by SDMT data:

$$G/G_0=1/\left[1+(G/G_{DMT}-1) \cdot \left(\gamma/\gamma_{DMT}\right)\right] \tag{15}$$

where G = shear modulus; γ = shear strain; γ_{DMT} = shear strain associated with the working strain DMT modulus for which Amoroso et al. (2014) suggested range of values based on the soil type.

In this particular case, being Murano site composed by soft clays, it is recommendable to use a value of γ_{DMT} = 2%. Moreover, to consider the effect of the confining stress and the different geotechnical properties of the entire soil profile into the assessment of the G/G_0-γ curves, seven homogeneous strata were identified from 3.50 to 31 m depth. The average values used to construct the non linear curves plotted in Fig. 9 are reported in Table 5. The G/G_0-γ curves estimated in the upper 15.5 m have a similar behavior, while the deeper G/G_0-γ curves decay much faster. This aspect is related to the higher values of K_D, and hence of OCR and K_0, detected for the upper layer, confirming a possible relationship with the different concentration of the diatoms.

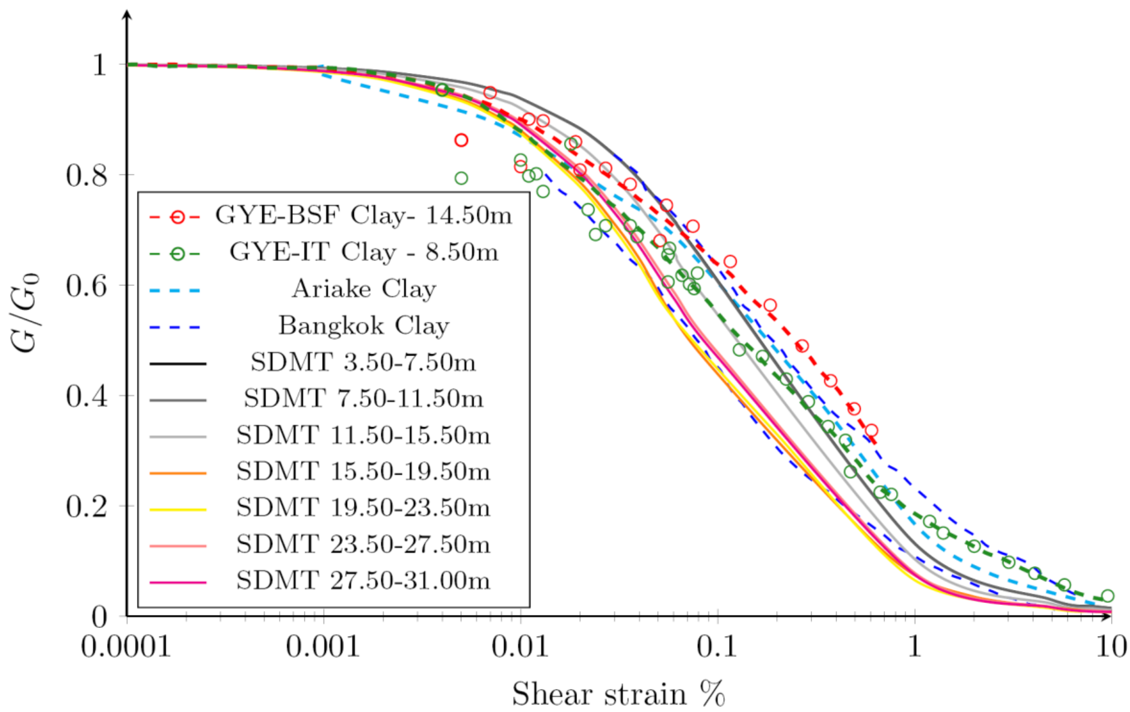

Fig. 9. G-γ decay curves for Guayaquil clays obtained by SDMT tests and comparison with results of laboratory curves.

Table 5. Average values used to construct G/G_0-γ curves.

Depth	**Soil type**	**G_0 (MPa)**	**M_{DMT} (MPa)**	υ	**G_{DMT} (MPa)**	**G_{DMT}/ G_0**	**γ_{DMT} (%)**
3.50-7.50	Soft clay	7.22	1.79	0.30	0.51	0.07	2
7.50-11.50	Soft clay	11.45	2.82	0.30	0.81	0.07	2
11.50-15.50	Soft clay	17.21	3.30	0.30	0.94	0.05	2
15.50-19.50	Soft clay	26.40	3.29	0.30	0.94	0.04	2
19.50-23.50	Soft clay	26.22	3.12	0.30	0.89	0.03	2
23.50-27.50	Soft clay	28.85	4.14	0.30	1.18	0.04	2
27.50-31.00	Soft clay	32.41	4.44	0.30	1.27	0.04	2

Fig. 9 also plots two G/G_0-γ curves developed for Guayaquil clays in geological zone D3A whose samples were retrieved at Baseball Stadium Field (BSF-dashed red line) on OC clays and at

Trinitaria Island (TI-dashed green line) on NC clays according to Vera-Grunauer (2014). The cyclic response of TI samples was evaluated by means of cyclic triaxial and simple shear tests while, for BSF clay, the decay curve was estimated from cyclic triaxial data. The conditions of the clay structure were modelled in the following way: to reproduce the conditions of the OC clay, the recompression method was used during the consolidation stage and the SHANSEP procedure was applied to model the normally consolidated soil. As reported by Vera-Grunauer (2014), the lower decay of BSF clay is due to the influence of pyrite cementation in its soil fabric. Other laboratory curves are included in Fig. 9 for naturally cemented alluvial clays with diatoms: Bangkok clay (Teachavoransinskun et al., 2002) and Ariake clay (Nagase et al., 2006).

A reasonable agreement is possible to detect by comparing the entire group of literature curves with G/G_0-γ curves by SDMT. However, the best fitting can be found between Guayaquil and Bangkok (upper limit) laboratory tests and SDMT prediction within the upper 15.5 m, probably due to the higher content of diatoms. Below that depth, SDMT assessment fits well with the lower limit of Bangkok clays.

5. Conclusions

The deep site campaign performed in Guayaquil (Ecuador) at the Murano site allowed to provide a better soil characterization for soft clays in presence of diatoms. In particular, the soil deposits were classified both using physical characteristics (i.e. USCS) and in situ tests (i.e. CPTu and DMT). In particular, index properties looked to be influenced by the diatom content. In this respect, the microstructure and porous shape of diatoms increased the average PI and w values in the upper 15 m depth, influencing the interpretation provided by USCS classification. This aspect is less visible from I_c and I_D, while it resulted well detected by in situ soil stiffness and strength.

In general terms, the parameters of resistance, compressibility and stress history provided reliable values using both CPT and DMT, while SPT and laboratory tests usually detected lower values. SPT test is not particularly effective for soft soil, while characterization of soil behavior by laboratory tests is directly dependent on the sampling process. Soft soil sampling procedure can modify the soil structure, and therefore soil behavior from laboratory differs considerably from the corresponding in situ behavior. The analysis especially of the OCR profiles by CPTu and DMT confirmed the presence of diatoms in the upper 15 m (OCR ≈ 2) and their lower concentration below this depth (OCR ≈ 1). Similar observations emerged from K_0 values obtained only by DMT: K_0 decreases from 0.8 to 0.6 moving from the upper 15 m to the lower layer. The better K_0 prediction by DMT is related to the intermediate range of strain to which the test is associated (Baligh and Scott, 1975; Mayne, 2001), and consequently to the direct correlation between K_0 and K_D, considering K_D a stress history indicator and an amplified K_0. The above findings are in agreement with Vera-Grunauer (2014) and Torres et al. (2018) who identified soft clays with diatoms in the upper 15-20 m in Guayaquil bay.

Due to the porous shape of diatoms, a considerable increase in permeability would have been expected in the upper layer. However, in situ and laboratory measurements are not available along the entire profile, but only in the upper 15 m depth. According to the results obtained, it is evident that CPTu and DMT dissipation tests gave very similar results for this type of clays.

On the contrary, parameters obtained from the oedometric tests lead to inconsistent results, probably because of sample disturbance and due to the assumptions made to interpret the permeability through the 1D Terzaghi's theory, which do not properly fit the behavior of natural clays (Tavenas et al. 1983). The comparison between predicted and measured V_s values suggested that DMT prediction is more reliable than CPT and SPT predictions. The high number of V_s correlations developed for CPT and SPT test detected a wide variability within the V_s profile of the soft clays, resulting in contrast with the single equation available for DMT (Marchetti et al., 2008). The arisen uncertainty could be due to the dependency to numerous and different parameters related to the geological age, soil type and in situ stress state that CPT and SPT parameters may not capture correctly. At the same time, DMT (through K_D) is well correlated to stress history, prestraining/aging and structure scarcely felt by q_c and N_{SPT} (Amoroso, 2014).

Finally, the nonlinear soil behavior of the soft clays at Murano site was presented by means of literature data and direct SDMT data interpretation. The G/G_0-γ decay curves in the estuarine deltaic clays (zone D3) resulted in good agreement using SDMT and cyclic triaxial tests, identifying a similar behavior in the curves of upper 15.5 m, while the deeper G/G_0-γ curves decay much faster. This aspect resulted related to the higher values of K_D, and hence of OCR and K_0, detected for the upper layer, confirming a possible relationship with the different concentration of the diatoms. The use of SDMT in estimating stiffness decay curves could be therefore advantageous for the geotechnical design, although further investigation is needed to better understand the influence of diatoms content on decay curves.

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Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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