Monitoring ground improvement using in situ tests in Guayaquil, Ecuador

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Abstract

This paper describes the use of the seismic dilatometer test (SDMT) and the piezocone test (CPTu), to assess the effects of ground improvement in preventing liquefaction damage at a wastewater treatment plant in Guayaquil, Ecuador. The ground improvement consisted of 15 m-long, 0.55 m-diameter and 2 m-spacing stone columns built with vibro-replacement technique. The tests were carried out both in natural and in treated soils, in order to compare the variation of the geotechnical parameters in the analyzed deposits, also combining DMT and CPTu results in sandy deposits to estimate the overconsolidation ratio (OCR), the at-rest lateral earth pressure coefficient (K_0) and the ratio between the constrained modulus and the corrected cone resistance (M/q_t). Due to the presence of liquefiable soils at the trial site, the test results were then used to evaluate the pre and post-treatment liquefaction severity indexes, using different methods based on CPT, DMT, combined CPT-DMT, and shear wave velocity (V_S) approaches for a design ground motion. The results show a certain

sensitivity of the DMT over the CPTu tests to the ground improvement into the layer composed by sands and sandy silts, while V_S values show a limited increase in the treated area.

Keywords

Ground improvement, liquefaction, in situ testing

List of notations

FC	is the fines content of the soil
PI	is the plasticity index of the soil
SC	is stone column
I_i	is the improvement index
NS	is natural soil (before the installation of the SC)
TS	is the treated soil (after the installation of the SC)
GWT	is the ground water table
σ'_{v0}	is the vertical effective stress
u_0	is the equilibrium pore water pressure
N_{SPT}	is the SPT blow count
$(N_1)_{60}$	is the SPT corrected penetration resistance
$(N_1)_{60,cs}$	is the equivalent clean-sand corrected standard penetration resistance
q_c	is the cone resistance
q_t	is the corrected cone resistance for pore water pressure in cohesive soil
q_{c1}	is the corrected cone resistance for overburden stress
q_{c1N}	is the normalized corrected cone resistance for overburden stress
$(q_{c1N})_{cs}$	is the equivalent clean-sand normalized cone resistance (CPTu)
I_c	is the soil behavior index
D_R	is the relative density
φ'	is the effective friction angle
I_D	is the material index
K_D	is the horizontal stress index
K_0	at-rest lateral earth pressure coefficient
OCR	is the overconsolidation ratio
M	is the constrained modulus
V_S	is the shear wave velocity
V_{S1}	is the corrected shear wave velocity (V_S)
M_w	is the moment magnitude
MSF	is the magnitude scaling factor
PGA	is the peak ground acceleration
r_d	is the shear stress reduction coefficient
$CSR_{M=7.5}$	is the cyclic stress ratio at 7.5 moment magnitude

$CRR_{M=7.5}$ is the cyclic resistance ratio at 7.5 magnitude

FS_{LIQ} is the factor of safety to liquefaction

LPI is the liquefaction potential index

H_1 is the non-liquefiable capping layer

LPI_{ISH} is the Ishihara-inspired liquefaction potential index

ε_v is the post-liquefaction volumetric strain

LSN is the liquefaction severity number

S is the liquefaction-induced vertical settlement

1. Introduction

Ground improvement is a field that involves different techniques to modify the soil response under different conditions. The decision regarding ground modification performance is based on~~:~~ the assessment of difficult soils, liquefaction potential, soil instability, insufficient bearing capacity and/or excessive settlement, seepage (U.S. Army Corps of Engineers, 1999).

In particular, liquefaction is the soil response to the loss of stiffness and strength due to pore pressure increment, reducing the effective stress. This increment is caused by an elevation of the hydraulic gradient or through dynamic loading of the soil (Knappett and Craig, 2012). A form to mitigate the liquefaction potential is through densification of the soil by the following treatments: deep dynamic compaction, vibro-compaction, blasting and vibro-replacement (Mitchell, 1981; U.S. Army Corps of Engineers, 1999; Shenthan *et al.*, 2004; Mackiewicz and Camp, 2007). The characteristics of the soil, such as gradation and density, project requirements, limitation of space, adjacent structures and ground water table, are aspects to consider for selecting the appropriate treatment. Mitchell (2008) discussed the applications and limitations of these densification methods, noting that the degree of improvement given by the deep dynamic compaction, vibro-compaction and blasting is greater in clean sands and decreases as the fines content (FC) increases. Nevertheless, several studies document mitigation works using a variety of FC values (including rather elevated percentages) highlighting an increase of improvement given by the vibro-replacement stone columns: Mackiewicz and Camp (2007) used an improvement index (I_i), given by the ratio between the cone resistance (q_c) after and before the treatment minus one, to provide an improvement of $0.3 < I_i < 2.8$ for FC < 5%, and of $0 < I_i < 1.6$ for 15% < FC < 40%; Luehring *et al.* (2001) showed an increase of 95% for the corrected SPT blow count $(N_1)_{60}$, and of 180% for the normalized corrected cone resistance q_{c1N}, using vibro-replacement stone columns in combination with vertical drains in deposits with FC < 65%. Mitchell and Wentz (1991) showed an average 100% increase for the corrected cone resistance for overburden stress (q_{c1}) and an average 45% increase for the SPT corrected penetration resistance, $(N_1)_{60}$, when comparing pre and post treatment results in soil layers with FC < 55%. Vibro-replacement stone columns installation may have the double beneficial effect to cause densification of the surrounding soil during installation and to facilitate the dissipation of the excess of pore water pressure developed during an earthquake, by providing a shorter path of drainage (Adalier and Elgamal, 2004).

Therefore, it results of interest to verify the effectiveness of the improvement using in situ tests. These tests allow for developing a quick assessment, which consists in the comparison of selected geotechnical parameters obtained before and after the treatment. Currently, there is a wide selection of in situ tests.

Depending on the location, the availability of specific in-situ testing equipment is also a factor to consider in evaluating the ground improvement. Numerous authors (Schmertmann, 1986; Mackiewicz and Camp, 2007; Mitchell, 2008; Monaco *et al*., 2014; Bałachowski and Kurek, 2015; Amoroso *et al*., 2018, 2020; Massarsch and Fellenius, 2019; Massarsch *et al*., 2020) evaluate the change of the soil characteristics achieved, using at least one of the following tests and their parameters: SPT blow count N_{SPT} in the standard penetration test (SPT), horizontal stress index K_D and constrained modulus M in the flat dilatometer test (DMT), corrected cone resistance q_t in the piezocone penetrometer test (CPTu), and shear wave velocity V_S in the geophysical measurements provided by invasive or non-invasive tests (e.g. seismic piezocone SCPTu, seismic dilatometer SDMT, down-hole DH, cross-hole CH, multichannel analysis of surface waves MASW). Moreover, several studies discuss the change in the at-rest lateral earth pressure coefficient K_0, the overconsolidation ratio OCR and the ratio M/q_t when monitoring the densification effectiveness and the lateral stress increase. To estimate the parameters mentioned above, a combination of CPT and DMT tests is performed, as suggested in previous studies (e.g. Baldi *et al*., 1986; Marchetti *et al*., 2001; Massarsch *et al*., 2020).

Therefore, the present study describes the effects of ground improvement using the seismic dilatometer test (SDMT) and the piezocone test (CPTu). The location of the trial site is a project site for a wastewater treatment plant located in a sector known as “Las Esclusas” in Guayaquil, Ecuador. Ground improvement, realized in different zones of the facility, consisted of 15 m-long, 0.55 m-diameter and 2 m-spacing stone columns built with the vibro-replacement technique to mitigate the liquefaction potential (Luque, 2018). In this respect, CPTu, DMT, combined CPTu-DMT parameters and V_S measurements were executed in natural and treated soils and the results were compared. Moreover, liquefaction analyses were carried out to verify the column effectiveness in this key aspect.

2. Combination of SDMT and CPTu for monitoring ground improvement

Single-parameters derived separately from SDMT and CPT tests can be used to detect the modification in soil characteristics due to improvement works. As stated by various authors (e.g. Schmertmann, 1986; Bałachowski and Kurek, 2015; Amoroso *et al*., 2018, 2020; Massarsch and Fellenius, 2019; Massarsch *et al*., 2020), these parameters can be identified in the horizontal stress index K_D and the constrained modulus M from DMT, the corrected cone resistance q_t (or the cone resistance q_c) and the relative density D_R from CPT. K_D is directly derived from the corrected DMT membrane lift-off pressure reading and contains information about the stress history of the soil, while M is a function of the three DMT intermediate parameters (horizontal stress index K_D, dilatometer modulus E_D and material index I_D), representing a working strain modulus, i.e. the modulus that, when introduced into the linear elasticity formulae, provides realistic estimates of the settlement of a shallow foundation under working loads (Marchetti, 1980, 2008; Marchetti *et al.*, 2001). Parameter q_t (or q_c) is a direct measurement from CPT while D_R is usually based on correlations as function of the cone resistance and effective stress (Juang *et al.*, 1996). According to previous ground improvement studies related to densification techniques (e.g. Massarsch and Fellenius, 2002, 2019; Massarsch *et al.*, 2019), the horizontal stress tends to increase after compaction making K_D (and therefore M) more sensitive than q_t (and consequently D_R) to detect the modifications induced in by the treatment.

Moreover, CPT-DMT parameters can help to identify the effectiveness of the treatment, such as at-rest earth pressure coefficient K_0, overconsolidation ratio OCR, and ratio M/q_t (or M/q_c). Ground improvement techniques by compaction are usually installed in sandy soils with low FC, in order to monitor the effectiveness of the treatment through K_0 and OCR, the combination of DMT and CPT parameters is used. With reference to K_0 in sands Baldi *et al.* (1986) and later Hossain and Andrus (2016) developed correlations, mostly derived from calibration chamber (CC) tests. The present research estimated K_0 using the relationship proposed by Hossain and Andrus (2016), based on OCR, K_D and q_c/σ'_{v0}:

1. $$K_0 = 0.72 + 0.456 \log OCR + 0.035 K_D - 0.194 \log {}^{q_c}/_{\sigma'_{V0}}$$

To estimate OCR in sands Marchetti *et al*. (2001) proposed to use the ratio M/q_c ($M/q_t \approx$ 5-10 in NC sands, $M/q_t \approx$ 12-24 in OC sands) considering the results of DMT and CPT tests performed in compaction works of a sand fill (Jendeby, 1992), CC tests (Baldi *et al.*, 1988), quality control of ground improvement (Schmertmann *et al.*, 1986). Later, Monaco *et al*. (2014) carried out a field experiment constructing a temporary embankment that allowed to directly measure the OCR and to correlate it with the ratio M/q_t:

2. $$OCR = 0.0344\left({}^{M}/_{q_t}\right)^2 - 0.4174\left({}^{M}/_{q_t}\right) + 2.2914$$

Schmertmann (1985) highlighted the importance of measuring the soil lateral stress in different scenarios, remarking that the possible K_0 increment, due to compaction induced effects, depends on the initial K_0 soil condition. Lateral stress estimations obtained in a test area with an initial high K_0 ($\approx$ 1.3) increased merely 3% after the dynamic compaction, while a much greater increase occurred when natural conditions were associated to relatively low K_0 ($\approx$ 0.6), reaching an increase of 77% after the treatment.

The increment of the lateral stress can be also observed by vibratory compaction, as provided by Massarsch and Fellenius (2002) and Rayamajhi *et al.* (2016). In addition to CPT and SPT tests for the verification of the performance of the soil improvement, Baez (1995) suggested DMT tests as an important tool to validate the increase of the lateral stress. K_0 variation is used as an indicator of the treatment effectiveness in sandy deposits (Amoroso *et al*., 2018; Massarsch *et al*., 2020), combining DMT and CPT data as detailed above. Mayne and Kulhawy (1982) and Schmertmann (1985) agreed that OCR depends on K_0, and therefore OCR is also a good indicator of the modified effectiveness. Moreover, Massarsch and Fellenius (2019) evaluated the effects of vibratory compaction concluding that densification produces pre-consolidation effect of the soil causing an increment in OCR. OCR variation has been observed in several research works: Bałachowski and Kurek (2015) reported an OCR increase of 100-125% into a sandy deposits due to vibro-flotation while the ratio M/q_c increased from 120 to 160%; Kurek and Bałachowski (2015) detected an improvement between 50 and 100% in OCR, while the ratio M/q_c showed a slight increase due to heavy compaction works in sands with FC < 6%; Amoroso *et al*. (2018) monitored different techniques of improvement and the change in OCR and K_0 profiles was not clearly noticeable possibly due to the compaction technique used and/or the soil variability. Extensive literature reports that M increases at a faster rate than q_t. Therefore, the ratio M/q_t is often used as a treatment quality control criterion as it contains also information about the stress history of the soil. (Lee *et al.*, 2011) performed several CC tests finding a higher sensitivity of DMT to the changes of stress history over CPT, due to the lower soil disturbance of the wedge during its insertion in comparison with the cone. Consequently, this produces a higher increase of K_D and subsequently of M values with respect to q_t measurements. Therefore, the M/q_t ratio is also a good indicator of the ground improvement work.

3. Trial site

The trial site was located in Guayaquil (Ecuador) within a wastewater treatment plant (WTP) construction project close to the banks of the Guayas river in a sector known as "Las Esclusas" as can be observed in Figure 1(a). The new WTP will treat wastewater generated in the center and south areas of the city. Due to the low soil properties of some areas of the site, stone columns (SC) were realized in 2018 to support shallow foundations of different structures, internal roads and to mitigate consolidation settlement of clayey materials and liquefaction potential of sands and sandy silts (Luque, 2018).

3.1. In situ tests

Extensive geotechnical investigation was carried out at different stages: design stage (2014), start of construction works (2017), construction stage (2018), research studies (2019-2020). Field testing included: boreholes, SPT, CPTu, SDMT, MASW and REMI. The results presented in this study are referred to a trial site located nearby a clarifier as shown in Figure 1(b), where CPTu and SDMT tests were performed between 2019 and 2020 in natural (NS) and in treated (TS) soil up to 16-20 m depth. NS soil testing are identified as CPTu1_NS and SDMT1_NS, while surveys after SC installation are detected as CPTu2_TS and SDMT2_TS. Additional information regarding the NS condition were obtained from borehole and SPTs (SPTP3_NS) and CPTu test (CPTu14_NS) performed during the WTP construction (Figure 1(b)).

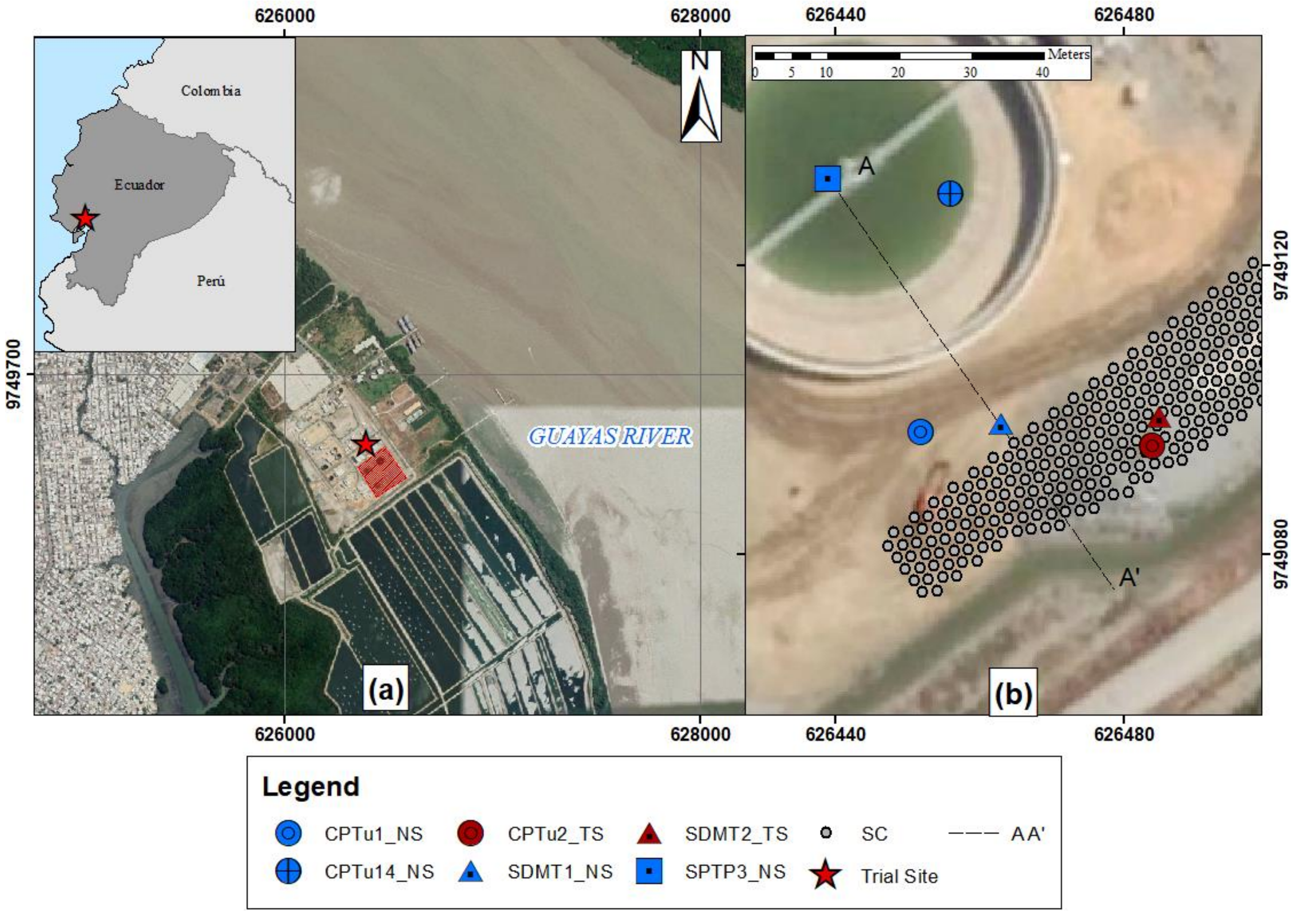

Figure 1. (a) Location of the wastewater treatment plant. (b) Location of in-situ tests and SCs

For the execution of CPTus and SDMTs the shallow compacted fill layer (≈ 0.6-0.8 m thick) was removed, to prevent damage on the geotechnical equipment. Collected data was used to define a subsoil model of the trial site, as reported in the geotechnical section of Figure 2. Table 1 summarizes the basic information of the in-situ tests used for verifying the ground improvement effectiveness. The ground water table (GWT) fluctuations at the trial site results strongly influenced by the tide of the Guayas river, in accordance with the measurements of the tides database INOCAR (2021).

Table 1. Summary information of the in situ tests at the trial site

Field test	Depth (m)	GWT depth* (m)	Test date
SPTP3_NS	19.0	2.0	2017
CPTu14_NS	20.8	2.7	Feb-18
SDMT1_NS	20.4	3.4	Aug-19
SDMT2_TS	20.6	3.4	Aug-19
CPTu1_NS	17.6	3.8	Aug-20
CPTu2_TS	19.0	3.8	Aug-20

*Note: Measured from the ground surface post filling.

3.2. Geotechnical profile

Figures 2 and 3 summarize the geotechnical profile of the natural soil using borehole and lab testing, SPT, CPTu and SDMT tests (CPTu14_NS, CPTu1_NS, SDMT2_TS and CPTu2_TS are projected on the cross-section of Figure 2). Beneath the shallow fill, the soil is variable but four clearly defined layers can be observed. The first layer is approximately 2 m thick and varies from silt to clay, as described by the soil behavior index (I_c) profile that intercalates between 2.6 and 3.4, by the material index (I_D) values that are between 0.2 and 1.1, and by USCS classification (ASTM D2487-11, 2011) that refers to non-organic clays with high plasticity, the fines content FC, estimated using the samples retrieved from borehole SPTP3_NS varies from 60 to 99%, while the plasticity index (PI) ranges between 42% and 67%. Underlying this layer, loose to medium dense sand mixtures (2MPa < q_t < 8 MPa; 2 < K_D <9, 5 < $(N_1)_{60}$ < 16) are present until a maximum depth of ≈ 10 m. These non-plastic sands and silty sands (SP-SM according to USCS) are mainly characterized by I_c < 2.6 and I_D > 1.2 with FC between 6 and 26%. A lens, of silt mixtures (2.6 < I_c < 3.0, 0.6 < I_D < 1.1) of variable thickness is present within the sandy layer between ≈ 7 and 10 m depth. Finally, below 10-11 m depth, normally to moderately overconsolidated clays, according to OCR approximation by Marchetti *et al.* (2001) are encountered (FC ≈ 67-99% and PI ≈ 43 - 67%), associating the following DMT and CPTu parameters: 2.2 < K_D < 3.3, with 3.1 < I_c < 3.9, 0.2 < q_t < 2.0 and 0.2 < I_D < 0.6.

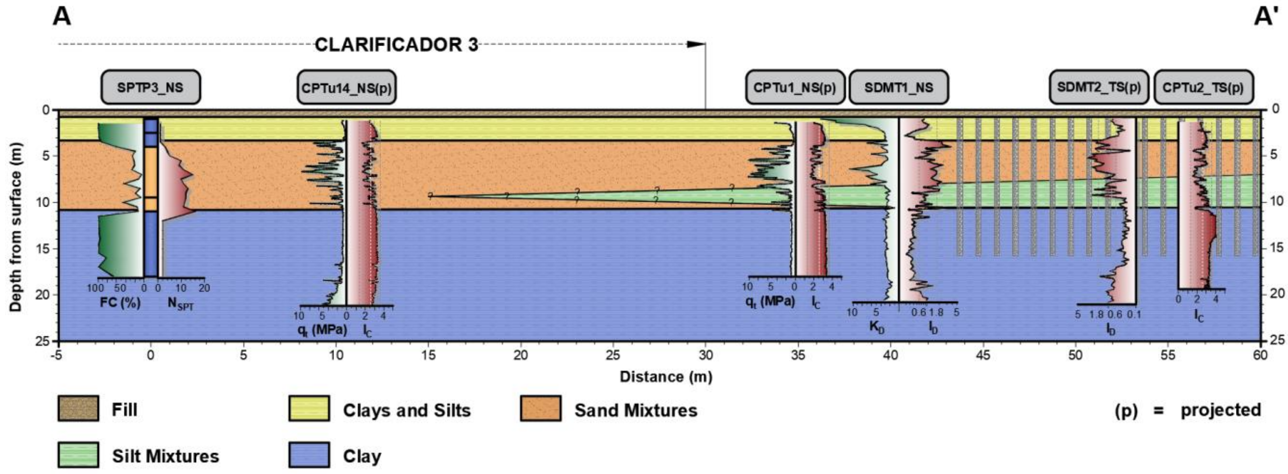

Figure 2. Geotechnical Section

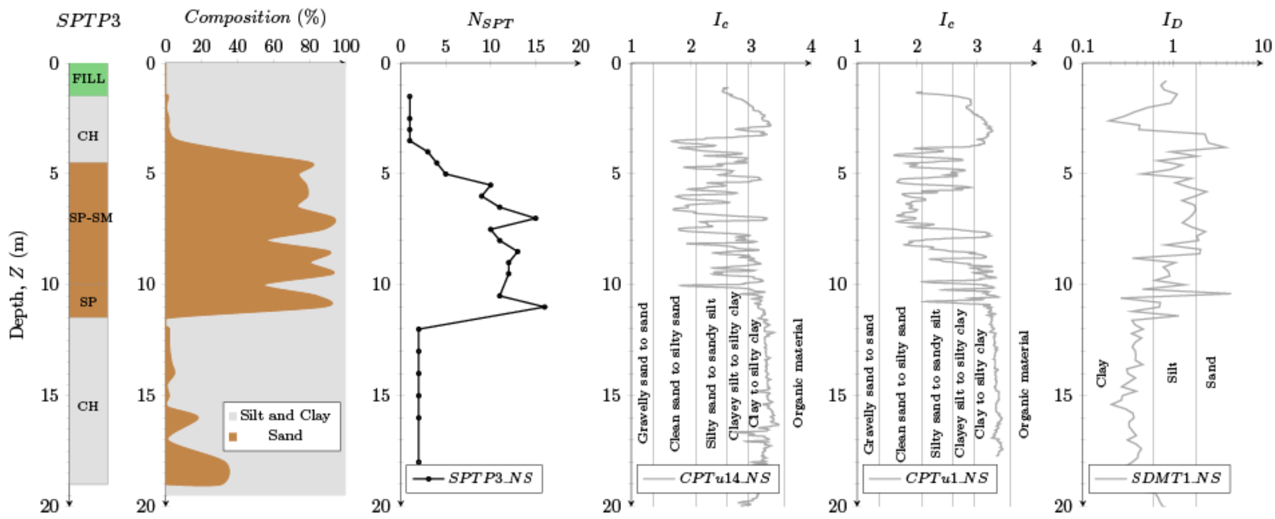

Figure 3. Borehole, FC, N_{SPT}, I_c, I_D profiles from in situ tests

3.3. Ground improvement

There are two similar vibratory methods to densify the soil: vibro-compaction (also referred as vibroflotation) and vibro-replacement. Both use identical equipment for the installation, but they use different material to fill up the voids generated during the vibration (compaction). Extensive literature agrees that vibro-compaction is more effective for clean sands with low silt content, and the use of sand is recommended to be used as backfill. On the other hand, vibro-replacement is preferable to soils with a higher FC and stone is used instead of sand as backfill (Mitchell and Wentz, 1991; Mitchell, 2008).

The selection of stone columns (SC) built by vibro-replacement as ground improvement method for the project was motivated to counteract the effects of liquefaction in silty sand to sandy silt soils, characterized by soil behavior type index I_c < 2.6 and material index I_D > 1.2, and to reduce the settlement due to the consolidation of clayey materials (Luque, 2018). In the study area, these deposits are located in a layer between ≈ 3 to 10 m of depth, overlaying a clay deposit (Figure 3). The 15 m-long and 0.55 m-diameter SC were installed in different zones of the project area in a staggered arrangement with 2 m-spacing between columns.

For the SC installation by vibro-replacement the primary equipment consists of a vibrating probe with an inner rotating eccentric mass around the horizontal axis (a scheme is shown in Figure 4(a)), which penetrates the soil by its self-weight and vibrations. Once it reaches the specified depth, the probe is lifted and the generated empty space is filled with stone; the probe is lowered to the deposited material to force the stone into the surrounding soil, forming a stone column (an illustration of this process is shown in Figure 4(b)) (Mackiewicz and Camp, 2007).

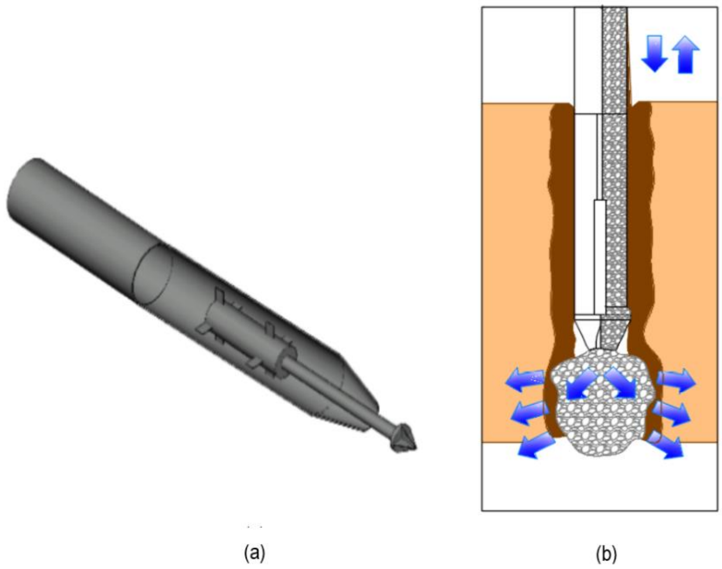

Figure 4. (a) Schematic of the vibrating probe. (b) Illustration of the insertion of the vibrating probe and the fill up of the resulting void with stone

4. Results

4.1. Soil improvement

Figure 5 shows the variation of the CPTu parameters in natural soil (NS) and treated soil (TS), estimated according to Robertson and Cabal (2015). The relative density (D_R) and the effective friction angle (φ') estimations are based on the correlations proposed by Kulhawy and Mayne (1990) and Jefferies and Been (2006) respectively. I_c profiles present a very slight variability of the soil before and after treatment, which makes quite comparable the data within the depth of the SC improvement where the silty sand to sandy silt layer ($I_c < 2.6$) is located. However, for some depth intervals between 4 and 10 m, q_t (≈ 4.2-6 m, 8-9 m, 9.6-10.4 m depth), and D_R (≈ 4.5-6 m, 8-9 m depth) values in the NS are somewhat higher than in TS. This is observed when the I_c increases in the TS behaving more like a fine grained soil. The φ' profile in TS presents a slight increase throughout the entire $I_c < 2.6$ layer. Figure 5 also compares the CPTu-φ' profiles with ones estimated from DMT (Marchetti *et al.*, 2001) and SPT (Kulhawy and Mayne, 1990), and the CPTu-D_R values with the ones evaluated from SPT (Skempton, 1986); Skempton (1986) tested five types of sand and proposed different a and b values for each type of sands, D_R values are estimated with a= 27, b= 28 up to 8.5 m depth, from that depth a= 38, b= 50 is used. Friction angle (φ') profiles from DMT and CPTu follow the same trend for NS and TS while for the SPT in the NS, the friction angle is overpredicted from ≈ 7 to 10 m depth. The D_R SPT-based values in the NS are in good agreement with the related CPTu ones from ≈ 6 to 8 m depth, while between 8 and 11 m depth, SPT-based method overpredicts the relative density. The SPT-based overestimations of D_R and φ' can be attributed to the lens of silt mixtures present detected only by CPTu and SDMT located in the proximity of the SC arrangement, while the SPT-based values where measured in the layer composed by the sand mixtures.

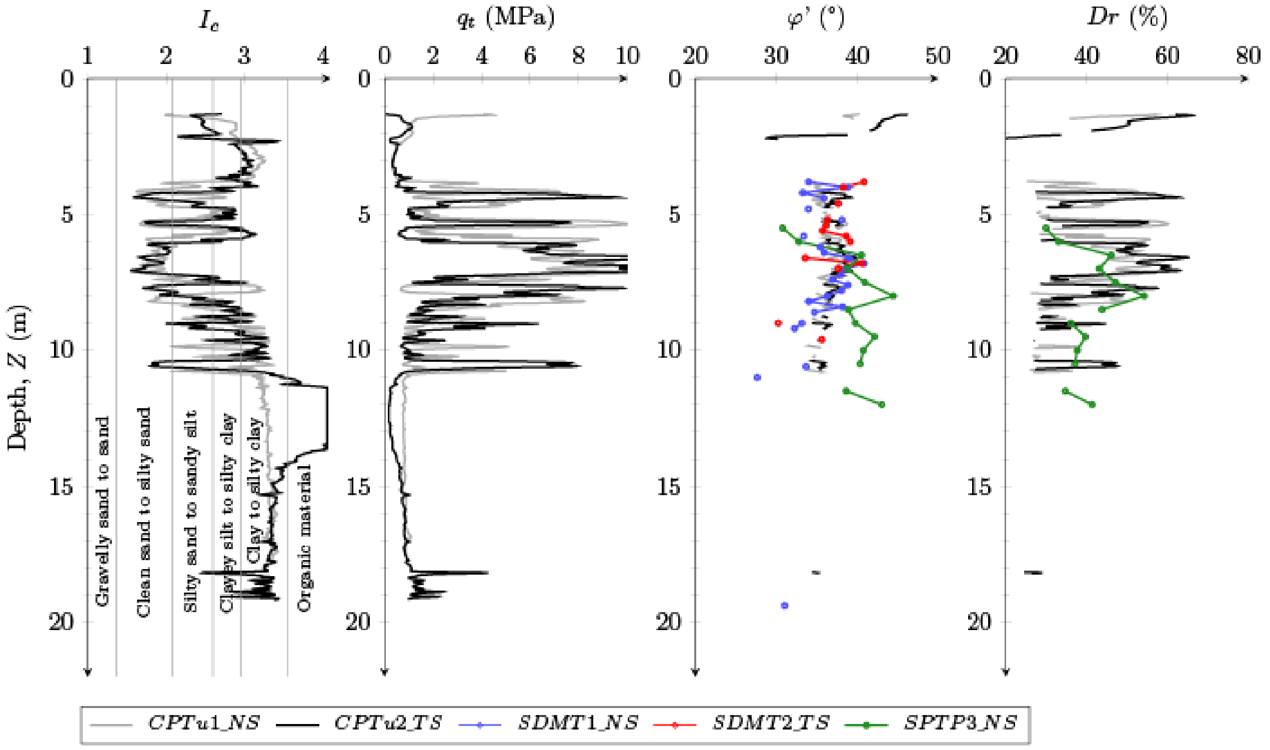

Figure 5. Results (pre and post treatment) I_c, q_t, φ', D_R

Figure 6 presents the plots of the DMT parameters which were calculated using the Marchetti *et al.* (2001) formulae. The GWT location was well determined by the equilibrium pore pressure, u_0, obtained from the third DMT pressure reading (p_2) into the sandy layers. A certain lateral soil heterogeneity is distinguishable in the NS and TS, I_D profiles between ≈ 6 to 8 m depth: the TS exhibits a fine grained soil behavior, considering the lower I_D values (0.3 < I_D < 1.2 corresponding to silty clay to silt), while the NS of the same layer results mostly silty-sandy (1.2 < I_D < 2.3). This response helps to understand why for the same depth interval the horizontal stress index K_D and the constrained modulus M are much lower despite the SC installation. The effectiveness of the treatment results then much more noticeable from ≈ 2 to 6 m depth, by M and K_D profiles. In this depth range (2 to 6 m) the I_D ranges between 0.2 to 3.8 indicating clayey and sandy soils, and K_D increased 52% after treatment. The shear wave velocity V_S also provides some increase after improvement, but limited between 4 and 6 m.

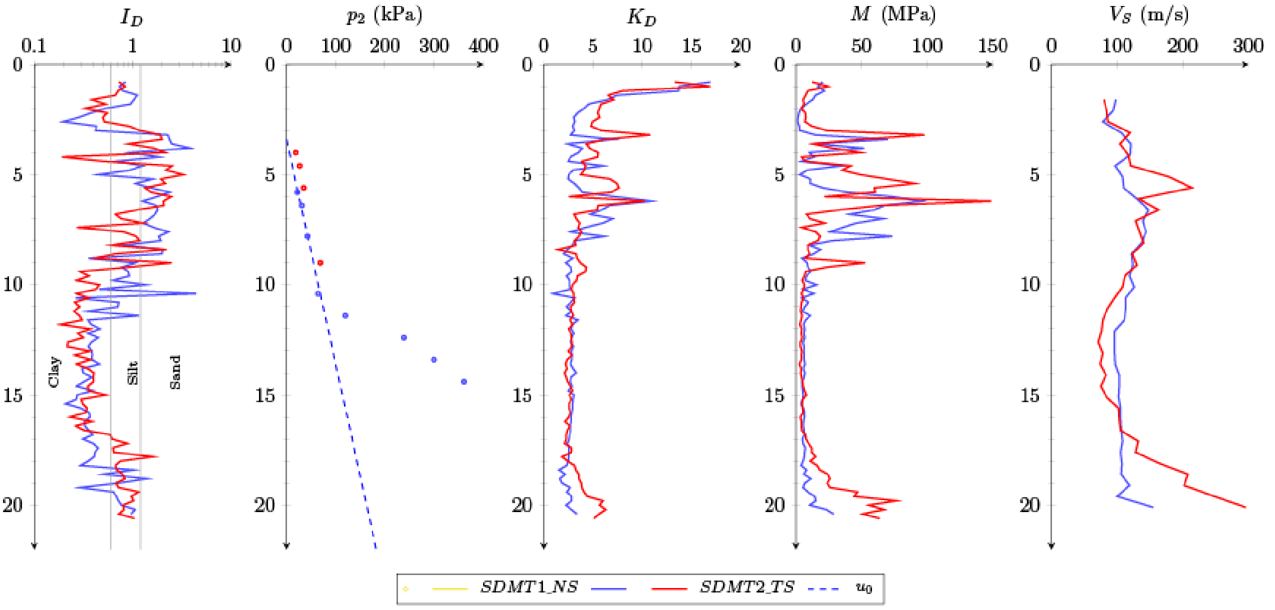

Figure 6. SDMT results (pre and post treatment) I_D, p_2, K_D, M, V_S

The analysis of CPTu and DMT combined parameters are displayed in Figure 7 to monitor ground improvement effectiveness. The profile of the ratio M/q_t, which is limited to silty sand to sandy silt soils (i.e. $I_c < 2.6$ and $I_D > 1.2$) exhibit a 65 % increase after the treatment. The estimation of the over-consolidation ratio OCR and of the in-situ earth pressure coefficient K_0 was performed both in fine-grained and incoherent soils. In particular, for $I_D < 1.2$ OCR and K_0 were estimated only by DMT using Marchetti *et al*. (2001) formulae, while for sandy layers ($I_c < 2.6$ and $I_D > 1.2$) the combined CPT-DMT approach was used according to Equation 2 from Monaco *et al.* (2014) for OCR and to Equation 1 from Hossain and Andrus (2016) for K_0. The OCR and K_0 profiles detect the effectiveness of the SC treatment between ≈ 2.6 and 6.6 m depth. Below 6.6 m the trend in the NS and TS is the same despite the SC installation up to 15 m, due to the presence of the cohesive layer.

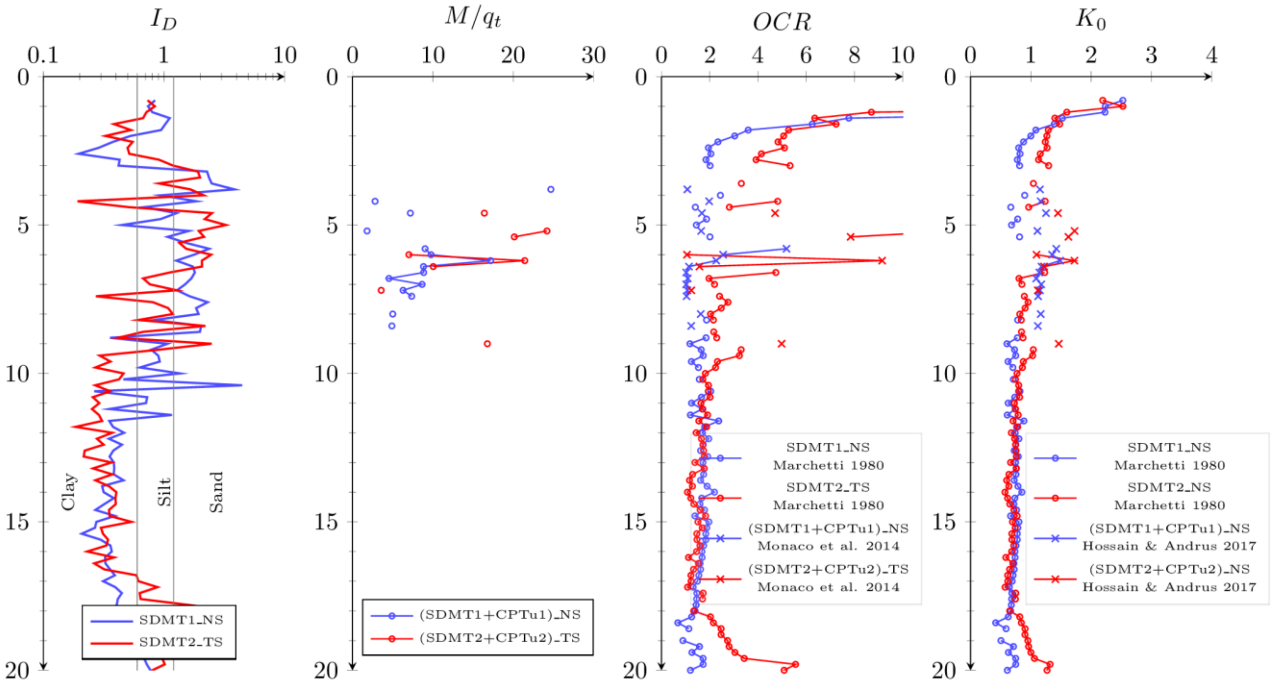

Figure 7. DMT and CPTu combined interpretation (pre and post treatment) M/q_t, OCR, K_0

Table 2 summarizes the average test results of the single and combined parameters from CPTu and SDMT in the layer where the increase was better noticed and for $I_c < 2.6$ and $I_D > 1.2$, that is approximately between 3.2 and 6.6 m depth. The improvement was calculated by relating the difference between TS and NS to NS results, expressed as a percentage. The CPTu conventional indicators of improvement show an increment of 6% for q_t and 7%for D_R, while for the SDMT parameters; K_D increased 22%, M twice as K_D and V_S 26%. The M/q_t ratio seems to be more sensitive to the improvement than K_D and M, being 65% higher in the TS. For the combined CPTu and SDMT parameters, K_0 increased just 15% while OCR increased 98%.

Table 2. Summary of average parameters (pre and post treatment) between 3.2 and 6.6 m depth: q_t, D_R, K_D, M, V_S, M/q_t, OCR, K_0

	q_t (MPa)	D_R (%)	K_D	M (Mpa)	V_S (m/s)	M/q_t	OCR	K_0
NS	5.00	42.49	5.04	43.55	121.20	10.02	3.08	1.27
TS	5.30	45.70	6.17	61.85	152.80	16.54	6.11	1.47
Increase (%)	6.00	7.55	22.42	42.02	26.07	65.06	98.37	15.75

4.2. Variation of the liquefaction susceptibility due to the ground improvement

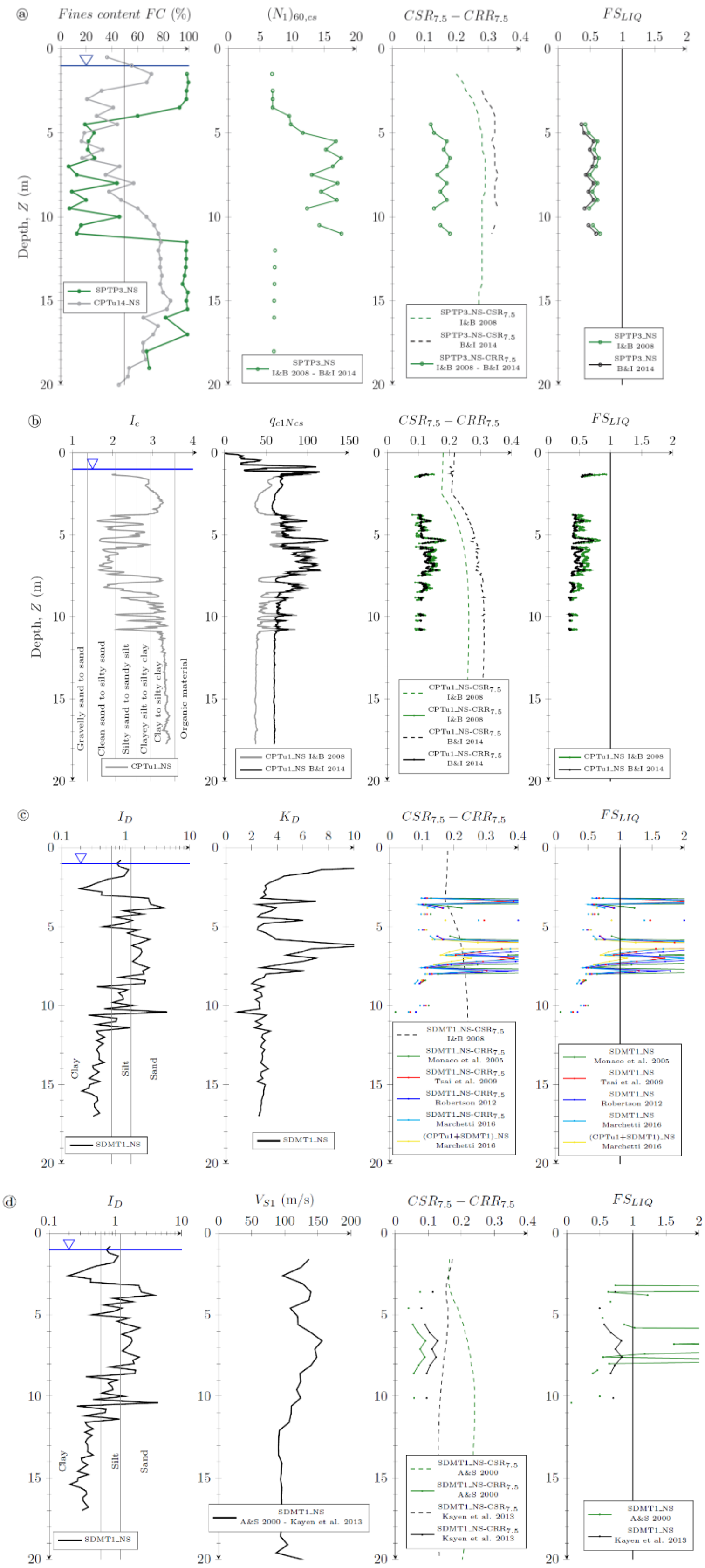

Figure 8. Liquefaction assessment before treatment (NS) for: (a) SPT; (b) CPTu; (c) DMT and CPT+DMT; and (d) V_S methods

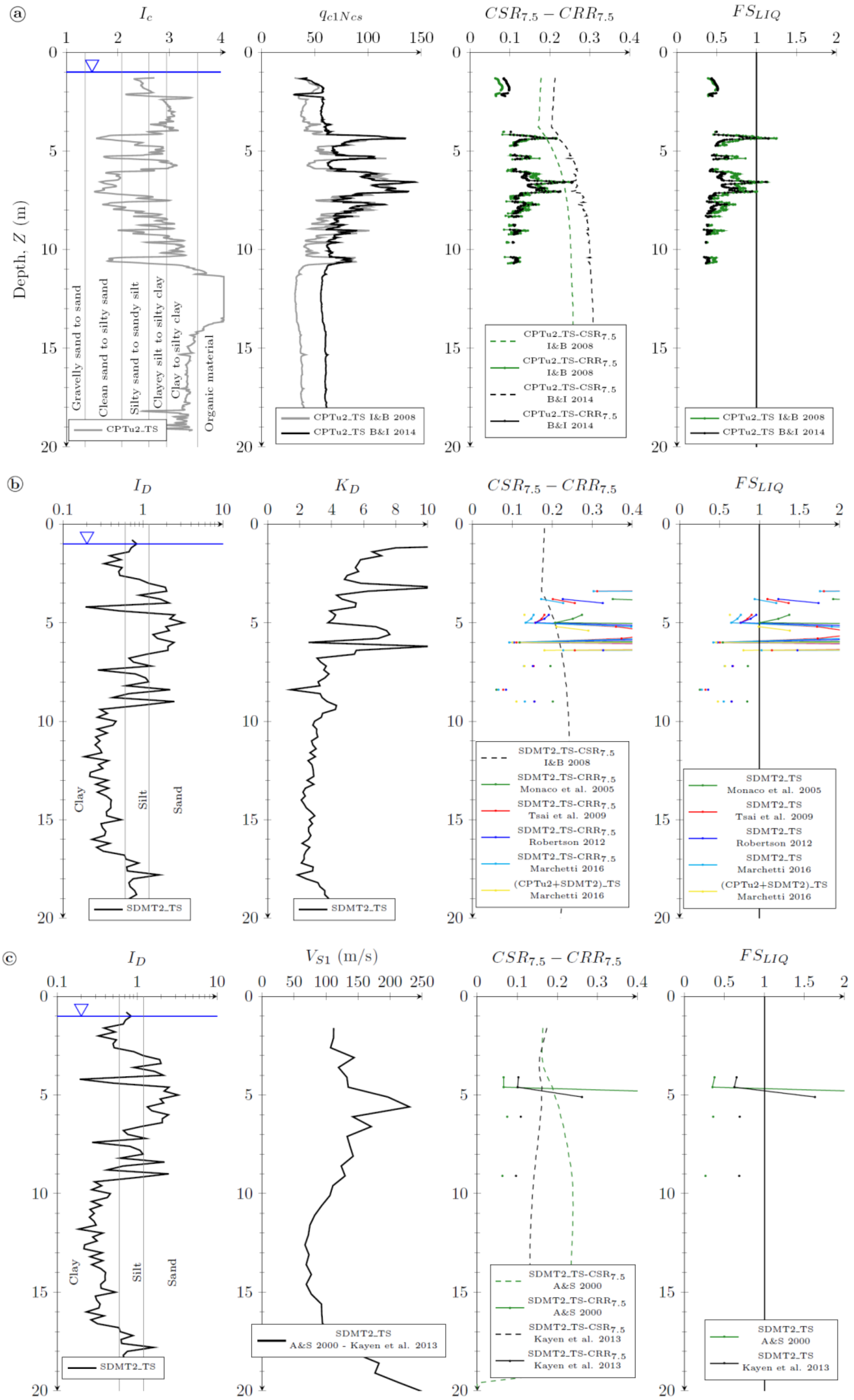

Figure 9. Liquefaction assessment after treatment (TS) for: (a) CPTu; (b) DMT and CPT+DMT; and (c) V_S methods

For evaluating the effectiveness of the ground improvement, a liquefaction assessment was performed in natural and treated soils. The stress-based approach based on the simplified procedure from Seed and Idriss (1971) was used for the liquefaction analyses, where the cyclic stress ratio at 7.5 magnitude ($CSR_{M=7.5}$) was obtained according to the ground response analysis performed by Geoestudios (2014). In particular, the design ground motion of the trial site corresponds to an earthquake with a moment magnitude (M_w) equal to 6.7 and a peak ground acceleration (PGA) of 0.34 g, with the ground water table (GWT) during earthquake assumed at 1.0 m below the surface level. These values were then used to evaluate $CSR_{M=7.5}$ based on the simplified procedure. The magnitude scaling factor (MSF), as well as the shear stress reduction coefficient (r_d) were estimated from Idriss and Boulanger (2008) and Boulanger and Idriss (2014) for the SPT, CPTu and DMT data; and from Andrus and Stokoe (2000) and Kayen *et al.* (2013) for the V_S data. The cyclic resistance ratio ($CRR_{M=7.5}$) was evaluated using correlations with the equivalent clean-sand normalized cone resistance $(q_{c1N})_{cs}$ and the equivalent clean-sand corrected standard penetration resistance $(N_1)_{60,cs}$ for the CPTu and SPT data respectively Idriss and Boulanger (2008), Boulanger and Idriss (2014); with the horizontal stress index K_D for the SDMT data Monaco *et al.* (2005), Tsai *et al.* (2009), Robertson (2012), Marchetti (2016); with the combination of $(q_{c1N})_{cs}$ and K_D Marchetti (2016); and with the corrected shear wave velocity V_{S1} Andrus and Stokoe (2000), Kayen *et al.* (2013). FC values estimated from correlations with I_c from CPTu (Suzuki *et al.*, 1998) were used in the CPTu, CPT+DMT and V_S liquefaction analyses, while FC laboratory measurement were used into SPT assessment. As shown in Figure 8(a), the two FC profiles results in reasonable agreement, corroborating the choices of the liquefaction analyses.

Before discussing the liquefaction performance indexes on the different methods, it is important to observe how the different classification and resistance parameters vary with depth before and after treatment (Figure 8 and 9 respectively). For the natural soil, Figure 8(a) shows that for SPT method, there is a liquefiable layer approximately between 4 and 11 meters of depth, being that in this layer FC is lower than 50% and $(N_1)_{60,cs}$ lower than 20. Idriss and Boulanger (2008) and Boulanger and Idriss (2014) approaches provide same profiles of $(N_1)_{60,cs}$ and $CRR_{M=7.5}$ while they result slightly different in terms of $CSR_{M=7.5}$ and hence of FS_{LIQ}. Figure 8(b) shows that for CPTu methods, the liquefiable layer is still between around 4 and 11 meters of depth (which matches well with the SPT method), being I_c lower than 2.6 and $(q_{c1N})_{cs}$ lower than 200. $(q_{c1N})_{cs}$, $CRR_{M=7.5}$ and $CSR_{M=7.5}$ are different for both methods but they have rather similar FS_{LIQ} along the liquefiable layer. On the other hand, Figure 8(c) shows that for DMT method, the liquefiable layer is between around 3 and 11 meters of depth, being I_D mostly higher than 1.2 and K_D mostly lower than 8. There is small difference in $CRR_{M=7.5}$

between the DMT and CPT+DMT methods and consequently in the FS_{LIQ} profiles. Figure 8(d) shows that for V_S method, the liquefiable layer is between 3 and 10 meters of depth, corresponding to I_D mostly higher than 1.2 and V_{S1} lower than 200 m/s. V_{S1}, $CRR_{M=7.5}$ and $CSR_{M=7.5}$ are different for both methods (Andrus and Stokoe, 2000; Kayen *et al.*, 2013), resulting in a FS_{LIQ} lower than 1.

As for after treatment results, Figure 9(a) shows no difference in the liquefiable layer thickness for CPTu assessment as pre-treatment corresponding case, but there are depth intervals (around 4.3 and 6.6 m) where the FS_{LIQ} results higher than 1. Figure 9(b) shows a decrease in the liquefiable layer thickness (between 4.5 meters and 9 m of depth), with thicker layers (between 5 and 6.5 m) over the FS_{LIQ} = 1 line than previously observed. Figure 9(c) also highlights a decrease in the liquefiable layer thickness, limited to few points between 4 and 9 m of depth.

The liquefaction potential index (LPI, Iwasaki *et al.*, 1978), the Ishihara-inspired liquefaction potential index (LPI_{ISH}, Maurer *et al*., 2015) which takes into account the thickness of the non-liquefiable capping layer (in this case H_1 = 2.5m), the liquefaction severity number (LSN, Tonkin and Taylor, 2013), and the liquefaction-induced vertical settlement (S, Zhang *et al.*, 2002) were calculated for all methods before and after ground improvement, considering the post-liquefaction volumetric strain (ε_v, Zhang *et al*., 2002) and the equivalent clean-sand normalized cone resistance $(q_{c1N})_{cs}$ from CPTu data. In particular, for the DMT and V_S methods, the $(q_{c1N})_{cs}$ values from CPTu1_NS were used for the NS and from CPTu2_TS for the TS, while for the SPT method, it was necessary to use the $(q_{c1N})_{cs}$ profile related to CPTu14_NS. A comparison between these liquefaction vulnerability indicators calculated before and after ground improvement by SPT, CPTu, DMT, CPT+DMT and V_S is presented in Table 3.

Table 3. Comparison of LPI, LPI_{ISH}, LSN and S in natural and treated soils in the WTP Las Esclusas.

		Mw = 6.7, PGA = 0.34g, GWT = 1m											
Method		LPI (NS)	LPI (TS)	LPI Change (%)	LPI_{ISH} (NS)	LPI_{ISH} (TS)	LPI_{ISH} Change (%)	LSN (NS)	LSN (TS)	LSN Change (%)	S (cm) (NS)	S (cm) (TS)	S Change (%)
SPT	I&B (2008)	18.22	-	-	10.75	-	-	29.81	-	-	14.03	-	-
	B&I (2014)	20.72	-	-	12.21	-	-	29.81	-	-	14.10	-	-
CPTu	I&B (2008)	13.46	12.05	10.5	7.96	4.13	48.2	21.35	32.82	-53.7	11.29	11.88	-5.2
	B&I (2014)	16.32	14.47	11.3	9.74	6.12	37.1	22.59	32.15	-42.3	11.83	12.18	-2.9
DMT	Monaco *et al.* (2005)	6.22	2.00	67.9	3.92	1.02	73.9	15.33	4.60	70.0	8.68	3.81	56.1
	Tsai *et al.* (2009)	7.77	3.15	59.4	4.70	1.44	69.3	18.50	8.92	51.8	10.45	5.89	43.6
	Robertson (2012)	7.79	3.01	61.4	4.67	1.45	69.0	17.83	7.94	55.5	10.01	5.44	45.6
	Marchetti (2016)	9.70	4.25	56.2	5.90	2.34	60.2	19.08	11.37	40.4	10.78	6.88	36.2
CPT + DMT	Marchetti (2016)	5.38	2.73	49.3	3.01	1.47	51.1	10.32	4.85	53.0	5.95	3.00	49.6
V_S	A&S (2000)	22.44	9.21	59.0	13.81	6.12	55.7	27.50	12.77	53.6	16.88	6.73	60.1
	Kayen *et al.* (2013)	10.20	4.75	53.4	5.80	3.21	44.7	27.32	12.93	52.7	16.76	6.82	59.3

Figures 10(a) and (b) (before and after treatment) show the different severity liquefaction indicators for all the methods. In general, all the indexes for the CPTu methods seem to provide different results in comparisons with all the other methods (Figure 10 and Table 3). The change (in percentage) was calculated by the difference between the treated soil value (TS) and the natural soil value (NS), divided by the NS value (i.e. positive values correspond to a gain of liquefaction resistance, and negative values indicate a loss of liquefaction resistance). LPI values from DMT methods show a larger decrement after the ground improvement (on average from 56% to 68%, moving therefore from high to low liquefaction potential) when compared with combined CPT+DMT (on average 49%, with LPI values similar to DMT approaches), and V_S methods (on average from 53 to 59%, moving from very high to high liquefaction potentials). On the other hand CPTu methods result in a slight decrement in LPI (on average 11%, remaining in the range of high liquefaction potential). Again, LPI_{ISH} values from DMT methods apparently provide a larger decrement (on average from

60 to 74%, reaching LPI_{ISH} ≈ 1-2 after treatment) when compared with CPTu methods (on average from 37 to 48%, reaching moderate and barely to high liquefaction potential), combined CPT+DMT (on average 51%, with LPI values similar to DMT approaches), and V_S methods (on average from 45 to 56%, detecting however after pier installations higher LPI ≈ 3-6). LSN decreases are very similar for all the methods (on average from 53 to 70%), except for the CPTu methods, which actually result in an increment in LSN (on average from -42% to -54%, corresponding to moderate to severe liquefaction after the treatment). However LSN absolute values from V_S approaches are generally higher than the ones obtained from DMT and CPT+DMT methods, moving from moderate (NS) to minor liquefaction (TS). For the settlement (S), the V_S methods show the larger decrement for before and after the ground improvement (on average 60%) compared to both DMT and combined CPT+DMT methods (on average from 36 to 56%). However, all these methods show that S varies approximately between 6 and 17 cm in NS and between 3 and 7 cm in TS. As with LPI and LSN values, CPTu methods result in a slight increase in settlement after the ground improvement (on average from -3 to -5%, or basically no change in settlement). Since SPT tests were performed at the site only on natural soil, there is no comparison between natural and treated soils for this method, but the liquefaction severity indexes were calculated before the improvement, resulting in a reasonable agreement with DMT, CPT+DMT, and V_S methods. The inconsistencies of CPTu liquefaction results may be mostly due to the small increase of CPTu parameters after the SC installation (see Table 2), nevertheless the I_c profiles are quite similar before and after treatment. Some further small differences may be related to the presence of a shallow liquefiable layer into CPTu profiles (depth interval ≈ 1-2 m), and to a soil lateral variability that reduces the thickness of the silty-sandy layer only for the SDMT2_TS test that was performed after treatment.

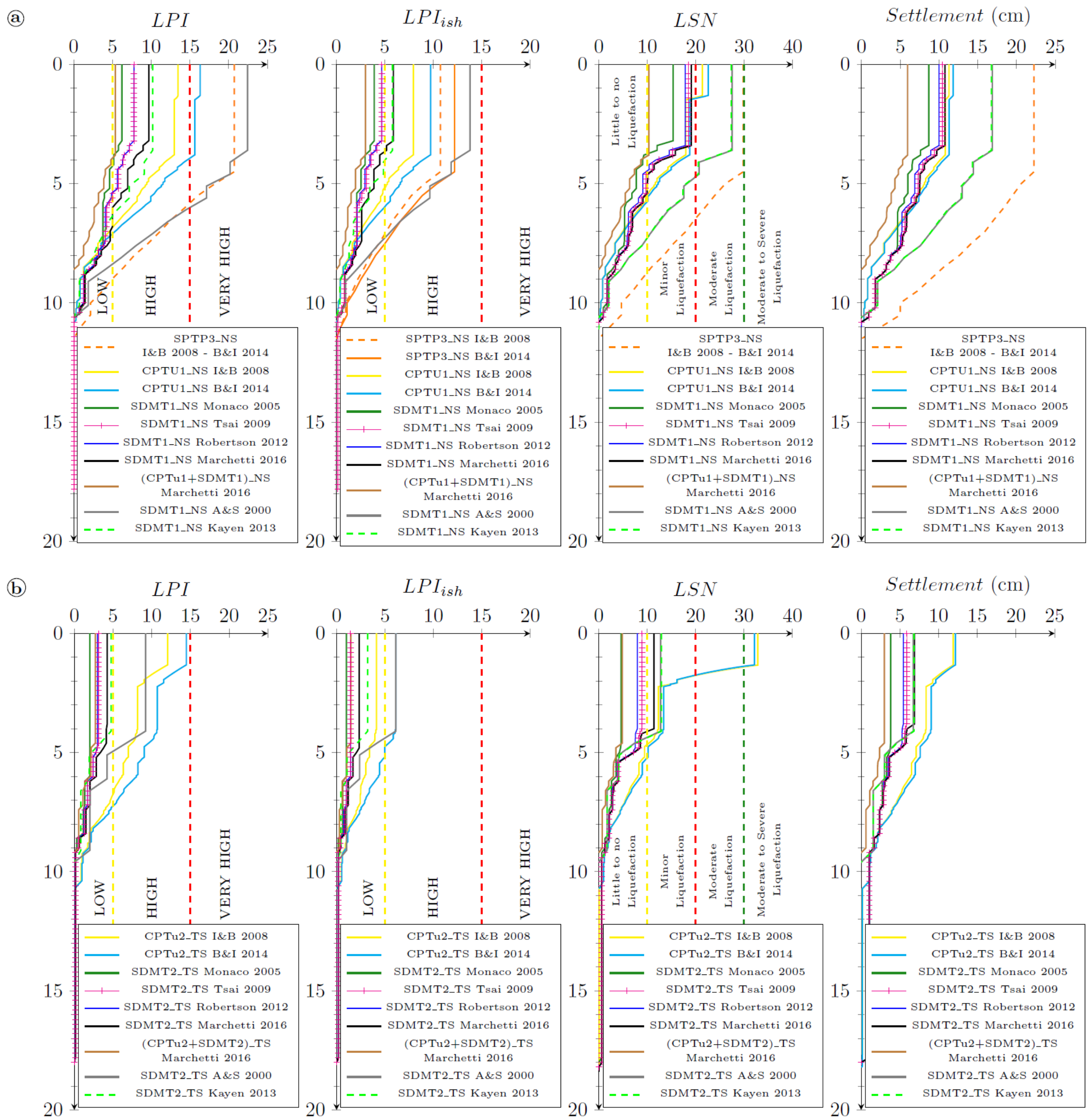

Figure 10. Liquefaction vulnerability indicators: (a) before and (b) after treatment for all methods

5. Conclusions

Despite the length of the SCs, the effectiveness of the treatment resulted noticeable only between 3.2 to 6.6 m depth, where the sand mixtures where detected by in situ tests. Below this layer a lens of silt mixtures, with higher FC (up to 46%) approximately between 7 to 10 m depth, and of a cohesive soil layer, from 10-11 m depth, were identified.

The evaluation of the soil improvement between 3.2 to 6.6 m depth was mainly detected through the use of the combined CPTu and SDMT parameters, with a 65% increment in M/q_t, 98 % increment in OCR and 15% increment in K_0. The relatively low increment in K_0 can be attributed to the high initial K_0 condition in NS ≈ 1.27, as noticed by Schmertmann (1985). In the CPTu based effectiveness assessment, q_t and D_R have a similar increase (6% and 7.5% respectively), although the NS and TS were related to quite homogeneous subsoil, as detectable looking at I_c. On the other hand, SDMT single parameters, K_D, M, V_S, provided a more evident SC improvement, even still limited, of 22%, 42% and 26% respectively.

As for liquefaction susceptibility the DMT, CPT+DMT and V_S analyses showed a higher sensitivity to the improvement when compared to the CPTu results. On average, the percentage of LPI, LPI_{ISH}, LSN, and S improvement for these DMT, CPT+DMT and V_S methods are equal to 49-68%, 51-74%, 40-70%, and 36-60%, respectively, while for CPTu they are equal to 10-11%, 37-48%, -42-(-54) %, and -3-(-5)%, respectively. The inconsistencies of CPTu liquefaction results may be mostly due to the small increase of CPTu parameters after the SC installation. SPT, CPTu and V_S, often result in higher absolute values than the other methods (DMT and CPT+DMT). Overall, SC implementation as ground improvement technique clearly shows a gain in liquefaction resistance that is better reflected by the DMT, CPT+DMT and V_S methods.

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