Settlement estimations for buildings founded on saturated silty sands from CPT and DMT results

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ABSTRACT: The estimation of building settlements founded in saturated sandy and silty soils is a relevant part of foundation design. Settlement calculation methodologies are still based on SPT and plate-load tests. SPT results can have significant dispersion and soil stiffness estimations are obtained from correlations without a physical meaning and valid only for particular soils and geology conditions. Moreover, plate-load tests are normally limited to shallow depths. CPT and DMT can obtain reliable results in engineering units. These tests are operator independent and the equipment is truly standard worldwide. Results obtained using CPT and DMT equipment for an urban renovation project with buildings founded on saturated sands and silts in Concepción city in Chile, are presented. These results allow estimations of static and liquefaction-induced settlements obtained from calculation methods developed for CPT and DMT.

1 INTRODUCTION

Bearing capacity and settlements are key components in the analysis and design of foundations. Bearing capacity of shallow foundations is usually calculated adopting the Terzaghi procedure which has been extended for different conditions (geometry, loading). However, in silty sands the design becomes controlled in most of the cases by the allowable settlement and not by bearing capacity. Therefore, estimations of settlements are crucial in almost every project. Settlement estimation involves usually the determination of the soil stiffness by means of an operational deformation modulus related to the design load range, which can be obtained from laboratory tests such as triaxial tests. Alternatively, in situ tests can be carried out instead or to complement the information from laboratory. In situ testing can have benefits in terms of measuring directly in the soil without the hassles of extracting, transporting, storing and the preparation of samples. A traditional in situ test carried out in projects is the standard penetration test SPT. However, there are concerns about its reliability and result interpretation for soil stiffness due to poor reproducibility and lack of continuity (Robertson, 2012). In situ load plate test can provide a scaled footing load-displacement response. However, its applicability is limited to surface or shallow depths.

Cone penetration test CPT is versatile and reliable since results can be faster and continuously obtained without major operator influence as with SPT (Lunne et al., 1997). A CPT based method to estimate static settlements of shallow foundations in sand requires the soil modulus of deformation E_s, which is normally obtained from the cone tip resistance.

The flat dilatometer test DMT is another in situ test which has had a particular success in the estimation of settlements of shallow foundations (Schmertmann, 1986; Schnaid, 2009; Marchetti, 2015). The settlement estimation is mainly based on the determination of the 1D dilatometer modulus M_{DMT}. Schmertmann (1986) has shown 16 case histories for different structures on sand, silt, clay, peat and mixtures of these soils where the use of M_{DMT} led to an average and standard deviation values of the estimated/measured settlement ratio s_e/s_m of 1.18 and 0.36 respectively, with s_e/s_m varying from 0.71 to 2.23 for measured settlements between 3 mm and 2.85 m. If only sand and silt and their mixtures were considered (9 cases), s_e/s_m average and standard deviation are 1.10 (0.73 – 1.34) and 0.21 for settlements between 3 and 58 mm. Moreover, other researchers have collected more cases with favourable performance of the DMT (Hayes, 1986; Monaco et al., 2006; Failmezger et al., 2015). For that reason, settlement studies based on CPT have often been benchmarked against results from the DMT (Kaggawa et al., 1996; Lehane and Fahey, 2004).

This work focuses on the Aurora de Chile urban recovery project located at the Bío Bío north riverbank in the city of Concepción, Chile, which started in 2016. The project considered the construction of 8 four-storey buildings of with 128 flats and 78 two-storey houses. As part of the geotechnical site investigation, one DMT and two CPT tests were carried out. This offers the opportunity for the comparison of settlement estimations based on results from CPT and DMT. The soils found in the project area correspond to fluvial deposits mainly of sands and silts and a mixture of them. Static settlements are determined from CPT and DMT results considering 1D deformation modulus for each soil layer and different vertical load increments. Liquefaction-induced settlement analysis based on the Ishihara (1996) chart and the liquefaction potential are performed. The latter follows the modified Seed and Idriss (1971) simplified method for CPT (Youd and Idriss, 2001) and DMT (Monaco et al. 2005; Marchetti et al. 2013).

2 STATIC SETTLEMENT ESTIMATION

Static settlement can be calculated based on a 1D compression modulus also referred to as oedometric or constrained modulus M which is commonly obtained from consolidation tests. This modulus M corresponds to the slope of a straight line between two points in the 1D effective vertical stress - vertical strain curve, which is therefore valid for that stress increment applied by a structure.

2.1 *CPT-based settlement prediction*

The CPT test consists in the continuous penetration through the soil of a standard steel bar with a conical tip at a constant rate of 20 mm/s. An electronically instrumented cone with load cells allows the measurement of the cone tip resistance q_c and sleeve friction resistance f_s. The use of a piezocone also allows the continuous measurement of the pore water pressure u. The CPT equipment and test procedures are standardized (ASTM D5778, 2020; EN ISO 22476-1, 2012). The CPT results can provide a detailed record for the evaluation of the ground stratigraphy and geotechnical properties (Lunne et al., 1997; Robertson, 2009a). Settlement estimations for footings in sand can be carried out by methods that use the drained elastic modulus E. 1D constrained modulus M are generally used for the estimation of long term consolidation settlements. However, for stresses below the preconsolidation stress, it can be assumed that M is approximately constant and possible to correlate with the net cone resistance (q_t - σ_{v0}) by means of DMT-CPT relationships represented in the Q_{tn}-F_r chart (Robertson, 2009a, 2009b).

$$M = \alpha_M(q_t - \sigma_{v0}) \quad (1)$$

where q_t is the corrected tip resistance, σ_{v0} is the in situ vertical stress and α_M is a modulus factor:

If $I_c > 2.2$: $\alpha_M = Q_{tn}$ for $Q_{tn} \leq 14$ (2a)

$\alpha_M = 14$ for $Q_{tn} > 14$ (2b)

If $I_c < 2.2$: $\alpha_M = 0.03[10^{(0.55Ic+1.68)}]$ (3)

where I_c is a soil behaviour type SBT index that can represent the SBT zones in the Q_{tn}-F_r chart, Q_{tn} is the normalized tip resistance and F_r is the friction ratio. I_c represents the radius of a concentric circle:

$$I_c = [(3.47\text{-}\log Q_{tn})^2 + (\log F_r + 1.22)^2]^{0.5} \quad (4)$$

$$Q_{tn} = [(q_t - \sigma_{v0})/p_a](p_a/\sigma'_{v0})^n \quad (5)$$

$$F_r = [f_s/(q_t - \sigma_{v0})] \times 100\% \quad (6)$$

where n is an exponent which varies according to the soil type (Robertson, 2009a). α_M increases from soft soils to dense granular soils with a division around the middle for $I_c = 2.2$. Expression (9) is used to calculate static settlements, where M is obtained with (1) to (6).

2.2 *DMT-based settlement prediction*

The flat dilatometer test DMT provides subsurface information through two horizontal pressure readings in a circular membrane. The DMT is recognized for being a suitable test to acquire information related to the stiffness of the soil. Moreover, it is also sensitive to the soil stress history, therefore, it can provide reasonable estimations of parameters such as the coefficient of lateral earth pressure K_0, over-consolidation ratio OCR and 1D constrained modulus M (Marchetti et al., 2001; Schnaid, 2009; Marchetti, 2015). M can be estimated according to the following expression:

$$M_{DMT} = E_D R_M(K_D, I_D) \quad (7)$$

where E_D is the dilatometer modulus, determined by the elasticity theory for the 60 mm diameter membrane displacing 1.1 mm.

$$E_D = 34.7(p_1 - p_0) \quad (8)$$

where p_0 and p_1 are the corrected lift-off and full expansion pressures, respectively. R_M is a correction factor applied to E_D, which is a function of the horizontal stress index K_D and the material index I_D; therefore, it is calculated according to the soil type. Static settlements are calculated using the following 1D relationship:

$$S_{DMT} = \sum \frac{\Delta\sigma_v}{M_{DMT}} \Delta z \quad (9)$$

where $\Delta\sigma_v$ is the stress increment applied by buildings or embankments in the middle of a layer of thickness Δz, which can be adjusted using the theory of Boussinesq for deep layers.

3 IN SITU TESTING RESULTS

Fig. 1 shows the q_t and f_s variation with depth as well as the soil behaviour type SBT interpretation for the CPT profiles. The CPT1 (Fig.1a) and CPT2 (Fig. 1b) tests reached 22 and 17.5 m, respectively. The groundwater was detected at approximately 5.5 m depth from the ground level. In general, sand layers from fluvial deposits are detected with maximum thicknesses of 10 m mixed with silt and clay lenses with thicknesses that vary between 0.8 to 1.5 m.

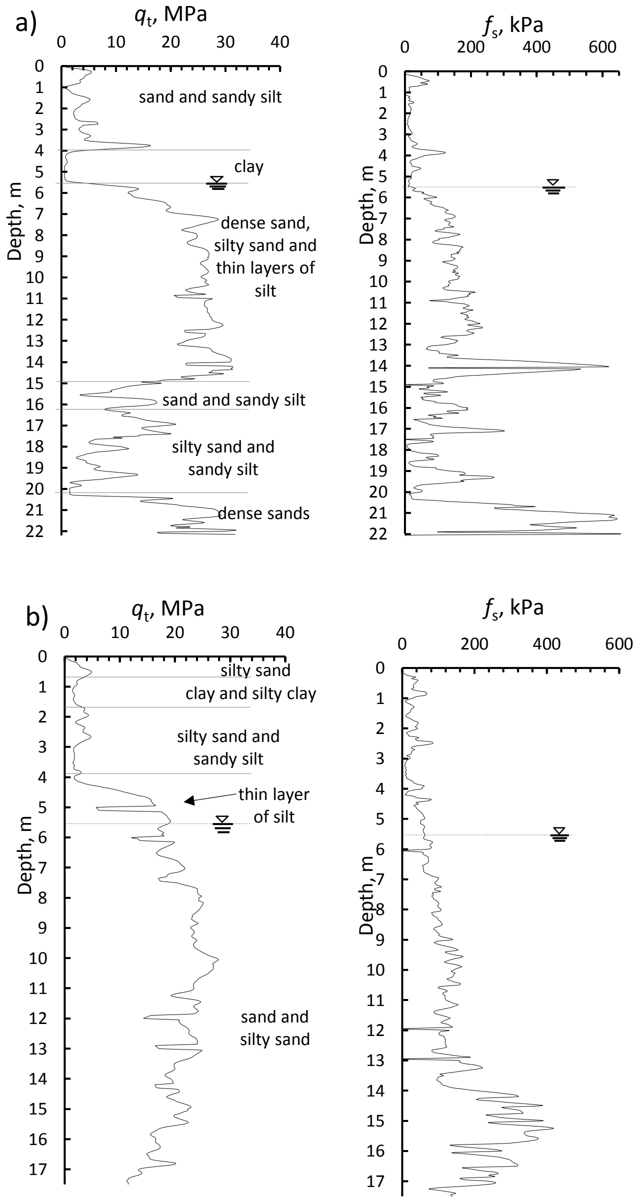

Figure 1. CPT profiles of q_t and f_s: a) CPT1 and b) CPT-2

The DMT test reached a depth of 10 m and the variation of I_D and K_D with depth are shown in Fig. 2. The stratigraphy is comparable to the CPT1 profile, both coinciding in the presence of a soft layer of clay approximately 1.5 m thick at 4 m depth. It is worth noting that when $K_D = 2$ soils are normally consolidated.

In Fig. 3 some of the soil parameters that are calculated based on the measurements from both test equipment are compared. Similarities can be seen between DMT and CPT1 for the profiles of unit weight γ and friction angle ϕ', where ϕ' is around 35° in the first 4 m and 40° below 6 m. In Fig. 4, the 1D compression modulus *M* shows low values in the first 3 m, and in the clay layer even lower values between 2 and 6 MPa according to the DMT and between 2 and 12 MPa according to CPT1.

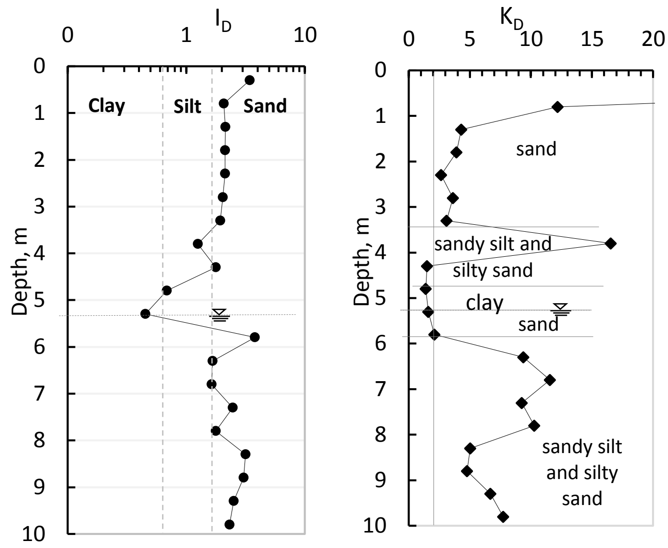

Figure 2. I_D and K_D variation with depth from DMT results

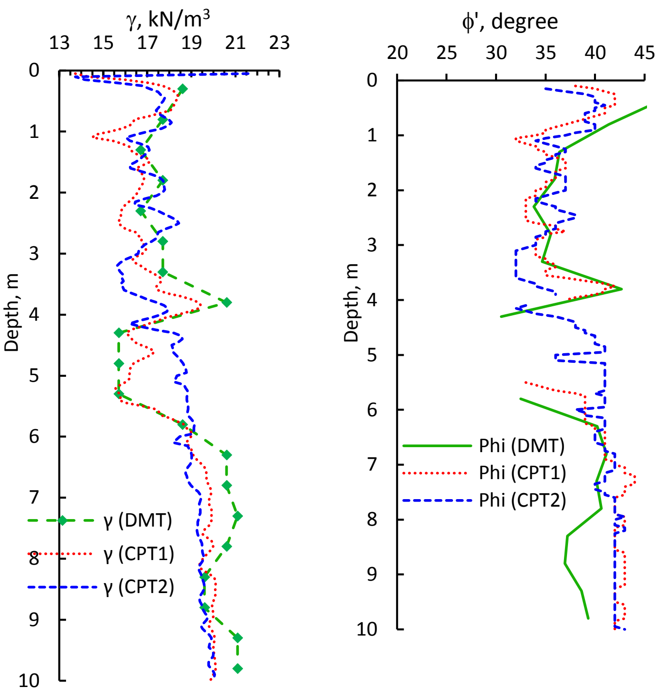

Figure 3. Comparison of the variation with depth of the unit weight γ and angle of friction ϕ' from DMT and CPT data

Below 6 m, *M* increases with much larger values with better agreement between CPT and DMT for values in the order of 180 and 200 MPa. Fig. 4 also shows the M_{DMT}/M_{CPT1} ratio, where it can be observed that CPT1 tend to overestimate *M* respect to DMT.

4 STATIC SETTLEMENT CALCULATIONS

Settlements are calculated with the method previously explained in 2.1 and 2.2 considering a 2 m wide square footing and founded 2 m below ground level. An increasing sequence of vertical load is considered as the stress increment $\Delta\sigma_v$. Fig. 5 shows the estimated settlements for each CPT and DMT test. It can be observed that DMT settlement estimations tend to be higher than those estimated with CPT.

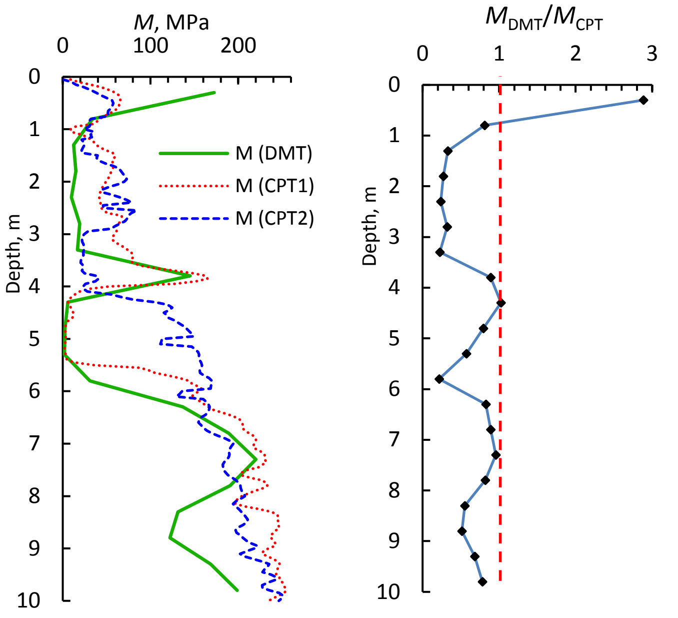

Figure 4. Variation with depth of the constrained modulus *M* determined from DMT and CPT and the *M* ratio

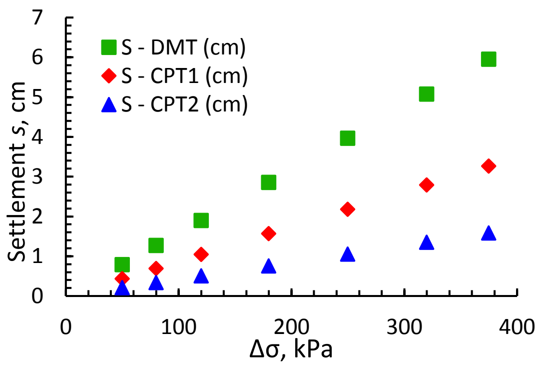

Figure 5: Shallow foundation static settlement estimated from DMT and CPT test results

5 LIQUEFACTION ANALYSIS

5.1 *Liquefaction occurrence*

Chile is a highly seismic country and in particular Concepción is a region where several mega-thrust earthquakes ($M_w \geq 8.0$) have occurred (in year 1570, 1657, 1730, 1751, 1835, 1939, 1960 and 2010). Moreover, the Aurora de Chile project site is next to Bío Bío River, the largest of Chile in terms of width and water flow volume. For that reason, it is important to assess the liquefaction potential of the saturated sandy ground and the associated settlements.

The procedure to assess liquefaction potential is mainly based on the simplified method by Seed and Idriss (1971) developed initially for SPT, from which the cyclic stress ratio CSR imposed by the earthquake can be estimated.

$$CSR = 0.65(a_{max}/g)(\sigma_{v0}/\sigma'_{v0})r_d \qquad (10)$$

where r_d is a stress reduction coefficient which account for the reduction of a_{max} with depth in absence of liquefaction. The expression used of r_d is too big to include it here (see (3) in Youd and Idriss, 2001).

For CPT data, a modified, although similar empirical procedure presented by Youd and Idriss (2001) has been adopted, where q_t is normalized and transformed to:

$$q_{t1Ncs} = K_c(q_t/p_a)/(p_a/\sigma'_{vo})^n \qquad (11)$$

where K_c is a grain characteristic factor that is a function of I_c and in this form a clean sand value q_{t1Ncs} is obtained, from which the cyclic resistance ratio CRR can be determined (Robertson and Wride, 1998).

$$CRR_{7.5} = 0.833(q_{t1Ncs}/1000) + 0.05 \quad \text{if } q_{t1Ncs} < 50$$
$$CRR_{7.5} = 93(q_{t1Ncs}/1000)^3 + 0.08 \quad \text{if } 50 \leq q_{t1Ncs} < 160 \qquad (12)$$

$$K_c = 1.0 \quad \text{for } I_c \leq 1.64$$
$$K_c = -0.403I_c^4+5.581I_c^3-21.63I_c^2+33.75I_c-17.88 \quad \text{for } I_c > 1.64 \qquad (13)$$

Subsequently, a liquefaction factor of safety FS_L can be obtained as:

$$FS_L = (CRR_{7.5}/CSR)\ MSF\ K_\sigma \qquad (14)$$

where MSF is a magnitude scale factor to account for earthquakes different from $M_w = 7.5$.

$$MSF = 10^{2.24}/M_w^{2.56} \qquad (15)$$

which results in MSF = 0.66 for M_w = 8.8. A correction factor for confining stresses higher than 100 kPa is determined as:

$$K_\sigma = (\sigma'_{v0}/p_a)^{f-1} \qquad (16)$$

where f is a factor related to the site conditions such as relative density DR, ageing and overconsolidation ratio OCR. In these analyses f has been assumed either 0.65 for DR > 60% or 0.75 for DR ≤ 60%.

For DMT, FS_L is also calculated using (14) with the same components as for CPT, except $CRR_{7.5}$ which is determined with the following expression (Monaco et al., 2005):

$$CRR_{7.5} = 0.0107K_D^3-0.0741K_D^2+0.2169K_D-0.1306 \qquad (17)$$

The liquefaction analyses assume M_w = 8.8 earthquake and a maximum acceleration a_{max} = 0.4g, which are actually the values recorded in the centre of Concepción during the 2010 earthquake and normally adopted in practice.

Figure 6 shows the variation with depth of the liquefaction factor of safety FS_L determined from the CPT and DMT results. The groundwater level is assumed to be at 2 m as an unfavourable condition during winter when the river water level is very high. A vertical line for $FS_L = 1$ separates where liquefaction is likely or not to occur. It can be observed that between 2 m and 3.5 or 4 m (where starts the clay layer) $FS_L < 1$ for CPT and DMT results, hence, liquefaction is highly likely to occur there. It is worth mentioning that liquefaction has been reported along the Bío Bío river promenade during the 2010 earthquake (Verdugo et al., 2010). Liquefaction is not expected in the clay layer as shown for CPT2 and DMT. However, $FS_L < 1$ for CPT1 due to the very low values of the clay tip resistance. It is important to bear in mind that liquefaction does not occur in clay.

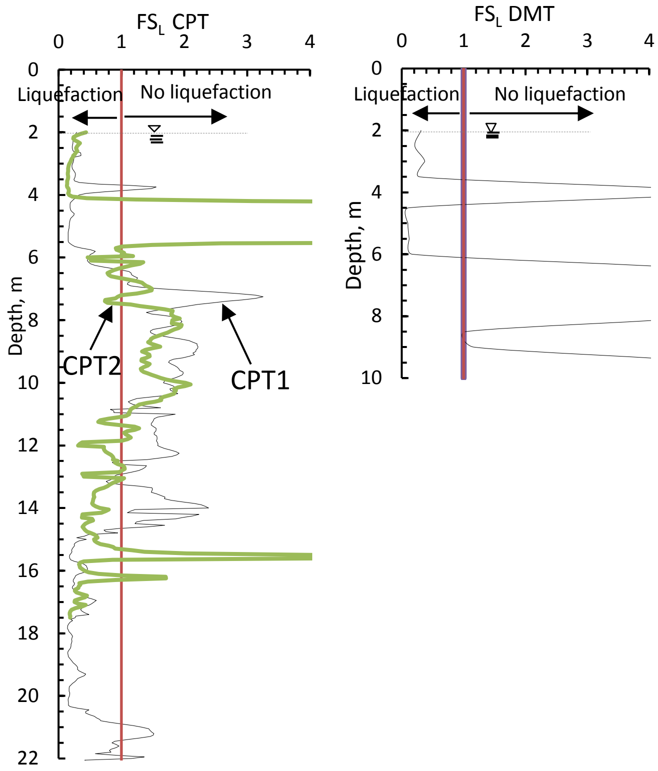

Figure 6: Liquefaction factor of safety obtained from CPT and DMT tests

Below the clay layer liquefaction may still take place down to 7.5 m according to the CPT results. Between 7.5 and 11 m $FS_L > 1$ for CPT results, although for DMT between 8.5 and 9 m, FS_L = 1.0 - 1.2, which is somewhat close to CPT2 results. This zone of no liquefaction may stop propagation of liquefaction occurring below. Indeed, below 11 m $FS_L < 1.0$ and according to CPT1 a layer of 6 m thick may liquefy.

5.2 *Liquefaction-induced settlements*

The estimation of free-field settlements caused by earthquake-induced liquefaction is carried out usually based on the soil incremental volumetric strain ε_{vol}. Dissipation of excess pore pressure densifies the soil which may result in important volume changes leading to settlements. A chart proposed to estimate ε_{vol} as a function of FS_L and N_1 from the SPT or alternatively q_c or DR can be used (Ishihara and Yoshimine, 1992; Ishihara, 1996). From CPT and DMT data this chart was directly used for $\varepsilon_{vol} = f(FS_L, q_t)$ and $\varepsilon_{vol} = f(FS_L, \mathrm{DR})$, respectively. The DR expressions used are a best fit to the DR-K_D plot by Reyna and Chameau (1991) for normally consolidated sands and the DR-K_D-OCR plot by Lee et al. (2011) for overconsolidated sands:

$$\mathrm{DR} = 50.66 \ln K_D + 7.95 \tag{18}$$

$$\mathrm{DR} = a \ln K_D + b$$

$$a = 1.344\mathrm{OCR}^2 + 11.334\mathrm{OCR} + 72.316$$

$$b = -0.6737\mathrm{OCR}^3 + 7.807\mathrm{OCR}^2 - 31.603\mathrm{OCR} + 46.989$$

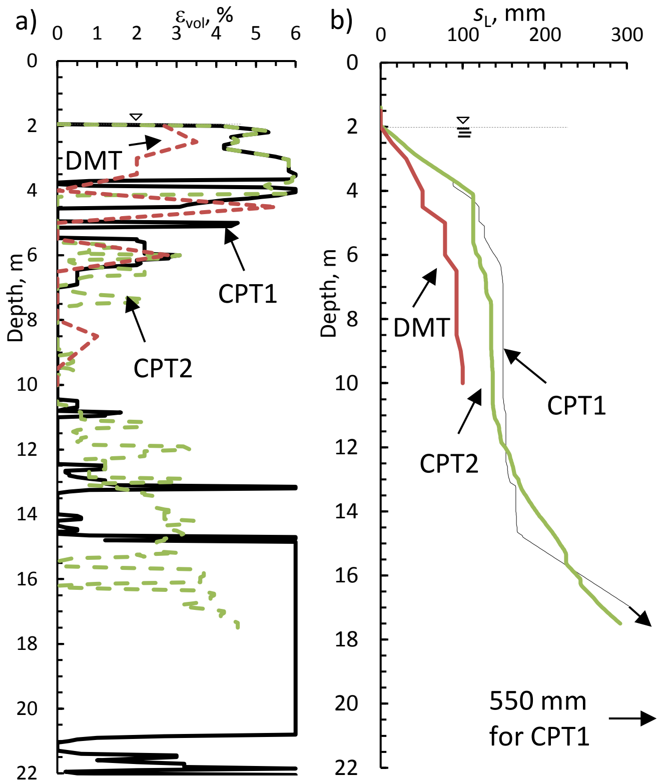

Figure 7: For CPT and DMT: a) incremental volumetric strain with depth and b) accumulated vertical displacement with depth

Figure 7a shows the variation with depth of the increments of ε_{vol} for each layer analysed from CPT and DMT data. It can be observed that in the first 2 m and below 15 m there are high values of ε_{vol}. The three curves are relatively close to each other denoting similar variation trends of ε_{vol}. Figure 7b shows the accumulated integration of ε_{vol} with depth which is the accumulated vertical displacement at each depth. It is clear to observe that DMT data lead to smaller settlements s_L compared with CPT data. s_L increase rate is higher in the first 4 m and then stabilises and increases again around 11 m and 15 m for CPT2 and CPT1, respectively. For the first 10 m, s_L is estimated to be around 100 mm for DMT, whereas for CPT1 $s_L \approx 150$ mm and $s_L \approx 130$ mm for CPT2. If the liquefied soil below 10 m manages to dissipate the excess pore pressure, s_L values may increase even more to values of 300 mm at 17.5 m (CPT2) and up to 550 mm at 22 m (CPT1).

It has been found that free-field liquefaction-induced settlement can become different from that occurring underneath buildings founded on shallow foundations (Bertalot, 2011). The overburden pressure imposed by a building applies large confining stresses in the soil which tend to modify settlements. Overburden increases settlements because of higher loads, but only until a certain value of bearing pressure owing to the reduction of excess pore pressure ratio with confinement (Bertalot et al., 2013). A chart has been developed to estimate building settlements s_L based on data that include the Concepción 2010 earthquake (Bertalot et al., 2013). s_L is determined based on the thickness of liquefied soil D_L, building width B and building bearing pressure q. For B = 15 m, q = 50 kPa, s_L/D_L = 0.1 and considering D_L = 2, 5 and 7 m results in s_L = 200, 500 and 700 mm, which are larger than those from free-field.

6 CONCLUSIONS

CPT and DMT tests were carried out in fluvial silty sands in Concepción Chile. A general good agreement was found among the results of soil profile interpretation, constrained modulus and resistance parameters. Static settlements estimated through the 1D M_{DMT} were approximately twice larger than those estimated with CPT data. Free-field liquefaction-induced settlements resulted in much larger than that from static analyses, from 100 mm (DMT) up to 150 mm (CPT) for the first 10 m. Including the confining stress imposed by a building increases even more the liquefaction-induced settlements.

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