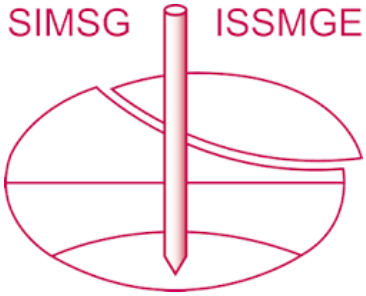
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The paper was published in the proceedings of the 6th International Conference on Geotechnical and Geophysical Site Characterization and was edited by Tamás Huszák, András Mahler and Edina Koch. The conference was originally scheduled to be held in Budapest, Hungary in 2020, but due to the COVID-19 pandemic, it was held online from September 26th to September 29th 2021.

Compressibility of granular soils from CPTU and DMT

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ABSTRACT: The compressibility of a granular soil is evaluated, based on CPTU and DMT data. A concept is described how the cone resistance can be adjusted with respect to the mean effective stress in order to reflect soil strength and stiffness independent of depth. This stress adjustment makes it possible to estimate the tangent modulus number and the constrained modulus, based on cone resistance measurement. The constrained modulus as measured by DMT was converted into tangent modulus number. Thus, it is possible to compare the tangent modulus number and constrained modulus obtained from CPT and DMT measurements, respectively. The results of extensive CPTU and DMT investigations from an ISSMGE test site, composed of sand and silty sand, were analyzed to determine the stress conditions and compressibility. Empirical values of the modulus number, published in the literature, are generally representative for normally consolidated soils. In the present study, a concept is presented how the effect of stress history (pre-loading) on the modulus number and the tangent modulus can be taken into account. The constrained modulus derived from CPTU and DMT results are compared and show good agreement.

Keywords: in situ tests; cone penetration test; dilatometer; tangent modulus method; preconsolidation.

1 Introduction

The results of extensive field investigations, which were carried out at the Bolivian Experimental Site for Testing Piles (B.E.S.T.), are reported. The site is located 24 kilometers North-east of the city of Santa Cruz de la Sierra, Bolivia. In connection with the third Bolivian International Conferences on Deep Foundation (C.F.P.B.), Technical Committee 212, "*Deep Foundations*" of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE), a series of full-scale loading tests was carried out on piles and a pile group. A particular aspect of the field tests was to investigate the potential effect of an enlarged pile base (Expander Body), installed at the toe of different bored pile types. A series of pile installation and pile loading tests were carried out. The results of these investigations are documented in [1]. The scope of the pile testing program has been reported in [2].

In order to establish the geotechnical site conditions, a comprehensive field campaign was carried out comprising a variety of in-situ tests. The results of the static and dynamic penetration tests were compiled and analyzed in [3]. Also, different types of seismic test were carried out. The results were reported in [4].

The results of extensive cone penetration tests with pore water pressure measurement (CPTU) and the flat dilatometer (DMT) are presented in the present study. Emphasis is on the determination of compressibility (soil modulus) and stress history of granular soils (sand and silt), which is required for settlement analyses. A reliable method of settlement analysis is the tangent modulus method, which will be described in detail. A critical step in settlement analyses is the selection of realistic input parameters, such as preconsolidation stress and modulus.

2 Geotechnical Setting

The geology of the area is characterized by an almost 100 m deep sedimentary river basin, created by the Piray River and its tributaries. The upper about 20 m thick soil deposit is composed of fine to medium sands with intermittent layers of silt, clay or clayey sand. The stress history of the soil deposit is complex, as it is affected by a sedimentation-erosion-sedimentation process. The first about 5 to 6 m are composed of loose to medium dense silt and sand, overlying a 6 to 7 m thick layer of silt and sand. At about 11 m depth follow layers of silty clay on top of an about 1 m thick layer of compact sand. Below about 12 m depth, the soil deposit consists of layers of compact to dense silty sand and loose sand. The groundwater table varies seasonally and is located about 0.5 m below the ground surface.

Geotechnical foundation concerns in granular soils (silt, sand, and gravel) are usually governed not by stability considerations, but by total and differential settlement restrictions. The most important aspect of a settlement analysis is the selection of realistic input parameters. It is generally accepted that estimating settlement of granular soil is an approximated process. The geotechnical literature provides only limited guidance on how to estimate the compressibility and stress history of granular soils from in-situ tests. A major reason for this is the difficulty to obtain undisturbed samples that can be tested in the laboratory. Thus, the compressibility of granular soils must either be estimated based on empirical data, or the interpretation of in-situ tests. In the absence of reliable data, over-simplified concepts are frequently used to assess both soil compressibility and stress history. When deformation properties of granular soils are determined, based on in-situ tests, it is nevertheless important to use a consistent approach based on stringent concepts.

The accuracy of settlement calculations is usually not determined by the employed analytical method used, but by the selection of relevant input parameters. The focus of the present paper is to outline a method for estimating in-put parameters for settlement analysis of granular soils based on CPT and DMT.

3 Interpretation of CPT

The CPT - and variations thereof, such as the CPTU (with pore water pressure measurement) or the SCPT (with seismic downhole test) is a widely used method for the assessment of strength and deformation properties of granular soils. An important advantage of the CPTU is that it measures three independent parameters: cone resistance, q_c, sleeve resistance, f_s, and pore water pressure, u. The measured cone resistance, q_c, is usually corrected for pore water pressure, u,

$$q_t = q_c + u\,(1 - a_c) \qquad (1)$$

where: q_t = cone resistance corrected for pore-pressure on the cone shoulder and a_c = net area ratio (the subscript "c" was added to prevent confusion with the modulus factor, a, see below).

3.1 Stress adjustment

The cone resistance is influenced by depth and, thus, by the effective confining stress. Massarsch [5] proposed a stress adjustment factor, C_M, to take into account the effect of mean effective stress, σ'_m, on the cone resistance measured in sandy soils, Eq. (2)

$$C_M = \left(\frac{\sigma_r}{\sigma'_m}\right)^{0.5} \qquad (2)$$

where C_M = stress adjustment factor ≤ 2.5
σ_r = reference stress = 100 kPa
σ'_m = mean effective stress.

The stress-adjusted cone resistance, q_{cM} can now be calculated, Eq. (3)

$$q_{cM} = q_c\, C_M = q_c \left(\frac{\sigma_r}{\sigma'_m}\right)^{0.5} \qquad (3)$$

It should be noted that the stress adjustment using the mean effective stress introduces the stress history of the soil deposit. This is in contrast to the frequently used vertical effective stress [6, 7]. In the opinion of the authors it is preferable to use an approximate procedure to estimate the mean effective stress by an approximate procedure rather than neglecting the effect of stress history.

3.2 Stress history

The stress-adjusted cone resistance, q_{cM}, reflects soil behavior independent of depth and thus reflects fundamental soil behavior. The method relies on knowledge (or assumption) of the mean effective stress (i.e. horizontal effective stress), which is expressed in Eq. (4)

$$\sigma'_m = \sigma'_v\left(\frac{1+2K_0}{3}\right) \qquad (4)$$

where σ'_m = mean effective stress
σ'_v = vertical effective stress
K_0 = at-rest earth stress coefficient.

In normally consolidated sand, K_0, be estimated from the relationship proposed in [8]

$$K_0 \sim 1 - sin\,(\phi') \qquad (5)$$

where ϕ' = effective friction angle.

A typical value of K_0 for uncompacted sand ($\phi' \approx 33°$) would be 0.43. The effective friction angle of overconsolidated (preloaded) sands can be estimated from the expression derived in [9]

$$\phi' = 17.6° + 11.0 \cdot log\left(\frac{q_t/\sigma_r}{\sqrt{\sigma'_v/\sigma_r}}\right) \qquad (6)$$

where ϕ' = effective friction angle
q_t = stress adjusted cone resistance
σ_r = reference stress (100 kPa).

In overconsolidated soils, it is important to estimate the preconsolidation stress, σ'_p. Mayne et al. [6] have proposed the following general equation for all types of natural soils, including sands, silts, clays, and mixed soil types

$$\sigma'_p = 0.33\,(q_t - \sigma_{v0})^{m'} \qquad (7)$$

where q_t = stress corrected cone resistance
σ_{v0} = vertical total stress
m' = grain size parameter, which increases with fines content and decreases with mean grain size (clean quartz sands: $m' \approx 0.72$, silty sands: $m' \approx 0.8$, clays: $m' \approx 1.0$).

It should be pointed out that although Eq. (7) gives only an approximate estimate of the preconsolidation stress, it is recommended to use an approximate value rather than neglecting the preconsolidation effect. The overconsolidation ratio, *OCR*, can now be calculated from the preconsolidation stress, σ'_p, Eq. (7) and the vertical effective stress, σ'_v

$$OCR = \frac{\sigma'_p}{\sigma'_v} \qquad (8)$$

Massarsch and Fellenius [10] have suggested the following relationship between the horizontal effective stress ratio (K_1/K_0) and the overconsolidation ratio, *OCR*,

$$\frac{K_1}{K_0} = OCR^{\beta} \qquad (9)$$

where K_0 = horizontal stress coefficient of normally consolidated soils
K_1 = horizontal stress coefficient of overconsolidated (compacted) soils
β = empirical coefficient.

The horizontal stress coefficient is necessary for estimating the mean effective stress, σ'_m. Based on calibration chamber (CC) tests, [11] recommended $\beta = 0.42$ and [12] $\beta = 0.45$. In [13] a range from 0.38 to 0.44 for medium dense sand is suggested. In [14] a conservative value, $\beta \approx 0.48$ is proposed. Thus, the horizontal stress coefficient, K_1, can be estimated from the following relationship, accounting for the stress history

$$K_1 = K_0\, OCR^{0.48} \qquad (10)$$

Now, the mean effective stress, σ'_m, can be calculated from Eq. (4), using K_0 for normally consolidated soils or K_1 for overconsolidated soils.

4 Interpretation of DMT

A relatively recent in-situ method is the flat dilatometer, DMT, introduced by Marchetti [15]. Guidelines for the DMT equipment and interpretation of test results have been issued by ISSMGE Technical Committee 16 [15]. For a detailed description of the DMT, recent developments in data interpretation, and practical application of results, refer to the geotechnical literature, e.g., 3rd DMT Conference Proceedings [16].

The test procedure is to advance the dilatometer blade into the ground. Readings are taken at depth intervals of 200 mm by inflating a membrane by 1.1 mm and taking pressure readings. These "raw" pressure readings are corrected and subsequently converted into two pressure values, p_0, and p_1. A key characteristic, which distinguishes the DMT from other in-situ methods, is its ability to measure parameters that reflect the stress conditions in the horizontal direction. This fact has important consequences for the evaluation of soil stiffness (modulus), which is affected by stress history. While the CPT strains the soil to failure, the DMT strains the soil to an intermediate level, thus more realistically reflecting soil deformation properties.

From the derived p_0- and p_1-values, the following DMT index parameters are calculated

$$I_{DM} = \frac{p_1 - p_0}{p_0 - u_0} \qquad (11)$$

$$K_D = \frac{p_0 - u_0}{\sigma'_{v0}} \qquad (12)$$

$$E_D = 34.7(p_1 - p_0) \qquad (13)$$

where I_{DM} = material index (nomenclature modified in order to avoid confusion with the density index, I_D)
K_D = horizontal stress index
E_D = dilatometer modulus
u_0 = hydrostatic pore water pressure
σ'_{v0} = vertical effective stress
p_1 = stress applied at start of expansion
p_2 = stress applied at end of expansion.

5 Tangent modulus method

The tangent modulus method for settlement analyses was first proposed by Ohde [17] and Janbu [18] and is described in detail in [19]. The tangent modulus (constrained modulus) is the ratio between a change of stress and the change of strain induced by that stress change

$$M_t = \frac{d\sigma}{d\varepsilon} = m\,\sigma_r \left(\frac{\sigma'_v}{\sigma_r}\right)^{(1-j)} \qquad (14)$$

where M_t = tangent modulus
$d\sigma$ = change of stress
$d\varepsilon$ = change of strain
m = modulus number (dimensionless)
σ_r = reference stress (equal to 100 kPa)
σ'_v = vertical effective stress
j = stress exponent.

Integrating Eq. (14) yields the following general relationship for determining the strain, ε, of a soil layer resulting from an increase of stress

$$\varepsilon = \frac{1}{m\,j}\left[\left(\frac{\sigma'_{v1}}{\sigma_r}\right)^j - \left(\frac{\sigma'_{v0}}{\sigma_r}\right)^j\right] \qquad (15)$$

where σ'_{vo} = vertical effective stress prior to loading
σ'_{v1} = vertical effective stress after loading.

5.1 Modulus Number from Empirical Data

The most important aspect of the tangent modulus method is the selection of realistic input parameters, that is, the stress exponent, j, and the virgin modulus number, m and re-loading modulus number, m_r.

Based on data by [18], values for m and j according to soil type of coarse-grained soil were published in [19]. Table 1 shows the typical range and average value of j and m, respectively. Note that two cases of the stress exponent (0.5 and 1.0) are given for granular soils.

Table 1. Typical stress exponent and modulus numbers for granular soils [19].

Soil Type	Stress exponent j	Range m	Average m
Till, very dense to dense	1	1,000 – 300	650
Gravel	1	400 - 40	220
Sand			
dense	1	400 – 250	325
compact		250 – 150	200
loose	0.5	150 – 100	125
Silt			
dense	1	200 – 80	140
compact		80 – 60	70
loose	0.5	60 – 40	50

In a more recent paper, Janbu [20] updated typical values of the modulus number, m, for normally consolidated silt and sand (j = 0.5). In [20] the modulus number was presented as a function of porosity, n, herein converted to the more widely used void ratio, e. Figure 1 shows the modulus number, m, derived as a function of void ratio, e, for silt and sand and different degrees of density. Also indicated is the approximate range (and lower/upper boundaries) of modulus numbers for the respective soil category (sand and silt), according to the classification used in Table 1.

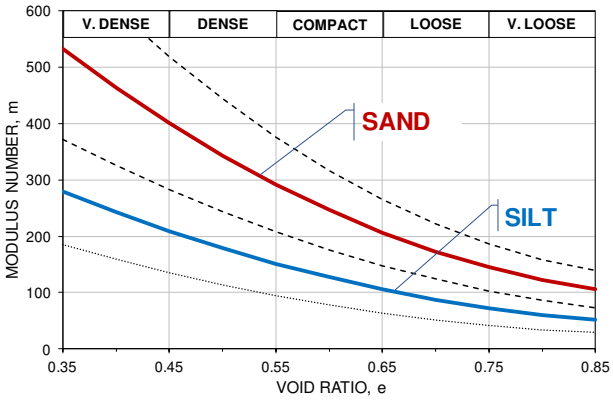


Figure 1. Typical values of modulus numbers for normally consolidated sand and silt [20].

5.2 Effect of pre-loading on modulus number

The empirical values of the modulus number proposed in Table 1 serve as a guidance for different soil types. Figure 1 shows typical values of the modulus number for normally consolidated sand. However, it is well known that stress history (pre-loading) affects the modulus number. As a result, the modulus number can increase significantly. Unfortunately, only limited factual information is available in the literature, quantifying the effect of pre-loading. Massarsch and Fellenius [14] re-analyzed results of laboratory compression tests on sand reported in [17]. The modulus number, m was measured during virgin loading as well as during unloading, m_u. The modulus number ratio, m_u/m, as a function of the virgin modulus number, m is shown in Fig. 2

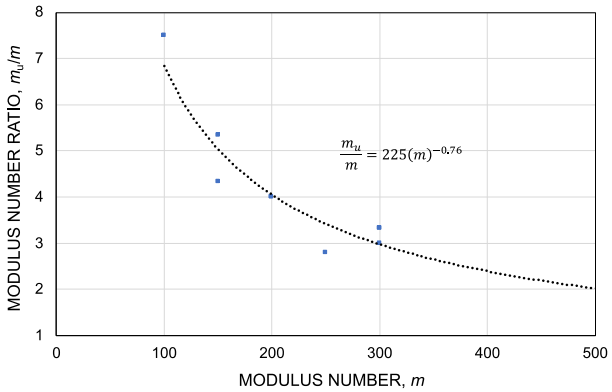


Figure 2. Modulus number ratio as a function of the modulus number during virgin loading, based on [14].

During unloading, the modulus number, m_u, is significantly higher than at virgin loading, m. The modulus number ratio increases with decreasing initial modulus number. The correlation between the unloading ratio, m_u/m and the virgin loading modulus number, m, shown in Fig. 2 can be expressed by the following equation

$$\frac{m_u}{m} = 225\, m^{-0.76} \qquad (17)$$

In very loose sand (m = 100), the unloading modulus ratio, $m_u/m \approx 7$. In case of an initially denser sand (m = 300) the unloading modulus ratio is lower, $m_u/m \approx 3$. Thus, the effect of pre-loading should be considered, especially in loose, normally consolidated sand.

There is a difference between the unloading modulus and the reloading modulus. However, for most practical purposes, this effect can be neglected.

5.3 Modulus number from CPT

Massarsch [5] proposed a correlation between the modulus number, m, for granular soils to the stress-adjusted cone resistance, q_{cM}, according to Eq. (3)

$$m = a\left(\frac{q_{cM}}{\sigma_r}\right)^{0.5} \qquad (18)$$

where m = modulus number
a = empirical modulus factor
q_{cM} = stress-adjusted cone resistance
σ_r = reference stress (100 kPa).

The modulus factor, a, reflects soil type and varies within a relatively narrow range for each soil category, as indicated in Table 2.

Table 2. Modulus factor, a, for different soil types [10].

Soil Type	Modulus Factor, a
Silt, organic soft	7
Silt, loose	12
Silt, compact	15
Silt, dense	20
Sand, silty loose	20
Sand, loose	22
Sand, compact	28
Sand, dense	35
Gravel, loose	35
Gravel, compact	40
Gravel, dense	45

An important benefit of determining the modulus number from CPT data is the fact that the cone resistance reflects the variation of soil type and stiffness with depth continuously as opposed to determining soil type and soil layer boundaries from intermittent sampling.

5.4 Modulus number from DMT

The DMT has become standardized. Marchetti et al. [16] suggested that a vertical, drained, constrained modulus, M, can be estimated from the dilatometer modulus, E_D, as follows.

$$M = R_M\, E_D \qquad (19)$$

with $I_{DM} < 0.6$: $R_M = 0.14 + 2.36 \log K_D$
$I_{DM} > 3$: $R_M = 0.5 + 2 \log K_D$
$0.6 < I_{DM} < 3$: $R_M = R_{M,0} + (2.5 - R_{M,0}) \log K_D$

with
$R_{M,0} = 0.14 + 0.15\,(I_{DM} - 0.6)$
if $K_D > 10$: $R_M = 0.32 + 2.18 \log K_D$
if $R_M < 0.85$: assume $R_M = 0.85$.

where M = vertical, drained, constrained modulus;
R_M is a correction factor based on empirical data [21]
I_{DM} = material index
K_D = horizontal stress index
E_D = dilatometer modulus
u_0 = hydrostatic pore water pressure.

The modulus number, m, can then be estimated according to Eq. (14). By rearranging terms, the following relationship is obtained.

$$m = \frac{M_t}{\sigma_r}\left(\frac{\sigma'_v}{\sigma_r}\right)^{j-1} \qquad (20)$$

In the case of normally consolidated sand, assuming that j = 0.5, the modulus number, m, is obtained from

$$m = M_t\left(\frac{1}{\sigma_r \sigma'_v}\right)^{0.5} \qquad (21)$$

For the case of compacted sand, assuming that j = 1, the following simple relationship is obtained

$$m = \frac{M_t}{\sigma_r} \qquad (22)$$

Thus, the modulus number, m, can be readily obtained by dividing M (in units of kPa) by 100, i.e., the reference stress, σ_r, (in units of kPa).

5.5 Stress Exponent

The stress exponent, j, in Eq. (14) defines the curvature of the load-compression relation and is based on soil type and stress conditions, which are relatively easy to estimate. Typical values of the stress exponent were recommended in [19], as summarized in Table 1. For dense sand and gravel or glacial tills (overconsolidated soils), the stress exponent is usually 1.0, which represents a linear response (elastic) to load. For loose silt and sand, j is typically 0.5, but decreases with decreasing grain size. Although j goes toward 0.25 in silty soils, in practice, it is usually satisfactory to assume $j = 0.5$ also here.

6 Geotechnical investigations

In the following, a novel concept of interpreting the results of CPTU and DMT is applied to the extensive data from the B.E.S.T. investigations. The aim of this study was to determine geotechnical parameters important for settlement analyses, such as soil compressibility (constrained modulus) and stress history (preconsolidation stress and overconsolidation ratio).

The geotechnical testing program included both in-situ and laboratory tests. The in-situ testing program comprised different types of tests. However, only the results of 15 CPTUs and 6 DMTs are included in the present study. Supplementary laboratory tests included: grain size distribution, water content, and plastic and liquid limits tests. For details, reference is made to [3, 4]. The in-situ tests were carried out along an about 80 m long and 6 m wide area. Although the area was essentially level, some height variations did occur, which affected to some extent the evaluation depth readings of the CPTU and DMT data.

The distance between the test points was 5 m. The lateral distance between the CPT and DMT in each test area was 1.1 m. A limited number of tests had to be eliminated from the analysis due to malfunctioning of the measuring system. Although the tests were carried out to greater depth, this study is restricted to 12 m depth, where granular soils dominated. All raw data of the field investigations can be downloaded from the web site of the 3rd C.F.P.B conference: http://www.cfpbolivia.com.

7 Results of CPTU investigations

The results of nine CPTUs were included in this study. The CPTU data were evaluated according to the concepts outlined above. The cone resistance corrected for pore water pressure effects, q_t, the sleeve resistance, fs, the friction ratio, F_R, and the pore water pressure, u, are shown in Fig. 3.

The results of the cone resistance and of the sleeve resistance show a relatively homogeneous soil profile, with a deposit of loose silty sand ($q_t < 5$ MPa) down to about 6 m depth, followed by a stiffer layer with cone resistance values varying between 5 and 15 MPa. The large friction ratio in the silty soil and the large excess pore water pressure indicate the presence of fine-grained layers.

From the measured cone resistance, q_c, the stress-adjusted cone resistance, q_{CM}, was determined according to Eq. (3), as shown in Fig. 4a. The effective friction angle was calculated according to Eq. (6) and is shown in Fig. 4b. The friction angle was used to estimate the horizontal stress coefficient, K_0.

An important step in the assessment of compression properties of soil deposits is the determination of the preconsolidation stress, σ_p'. Figure 5a shows the vertical effective stress and the preconsolidation stress as determined according to Eq. (7). It is interesting to note that the soil deposit is overconsolidated by a stress margin of about 200 – 300 kPa, with the exception of an intermediate layer between 3 and 6 m depth. This effect can also be detected from the inspection of the stress-adjusted cone resistance, cf. Fig. 4a.

The overconsolidation ratio, OCR, is shown in Fig. 5b. High OCR values are obtained close to the ground surface but decrease with depth. Below about 3.5 m, OCR varies in the range of 2 – 4. However, at shallow depth, very high OCR values are obtained, due to the low vertical effective stress. At shallow depth, OCR is very sensitive to calculation errors. Figure 5 confirms the advantage of expressing preconsolidation in terms of the stress margin (difference between the preconsolidation stress and the vertical effective stress) rather than OCR. In the overconsolidated layers below about 3 m depth, the stress margin is about 150 – 200 kPa and almost constant with depth.

In order to determine the constrained modulus from CPTU data, at first the modulus factor, a, according to Table 2 needs to be estimated. This is usually done based on inspection of the cone resistance and the friction ratio according to Fig. 3a and c. The assumed values of the modulus factor, a, and the modulus number determined according to Eq. (16) are shown in Fig. 6a and b.

Based on the information shown in Fig. 5 (preconsolidation stress) and Fig. 6b (modulus number) it is possible to carry out a conservative settlement analysis. However, it should be pointed out that the modulus numbers shown in Fig. 6b are for normally consolidated conditions and are thus conservative. As has been pointed out above, pre-loading (overconsolidation) has a significant effect on the modulus number and thus also on the constrained modulus. The following procedure was used to determine the modulus number taking into account the pre-loading effect: a) select all data with a friction ratio < 1.5 (granular soils); b) for these data, choose values with an estimated OCR > 4; c) apply Eq. (14) to calculate the modulus number for the case of pre-loaded layers. Figure 7a shows the modulus number values considering the pre-loading effect, cf. Fig. 6b.

Finally, the constrained modulus has been calculated based on Eq. (14). In order to eliminate the effect of fluctuation in calculated values, and to facilitate comparison with the constrained modulus determined from DMT, an average of m-values was determined over a depth interval of +/- 0.2 m. The so determined constrained modulus is shown in Fig. 7b. It is important to note the significant effect of pre-loading on the estimated constrained modulus. A similar effect will probably also

occur in the fine-grained soil layers (silt and clay). However, for such soils it is recommended to determine the pre-loading effect based on laboratory oedometer tests.

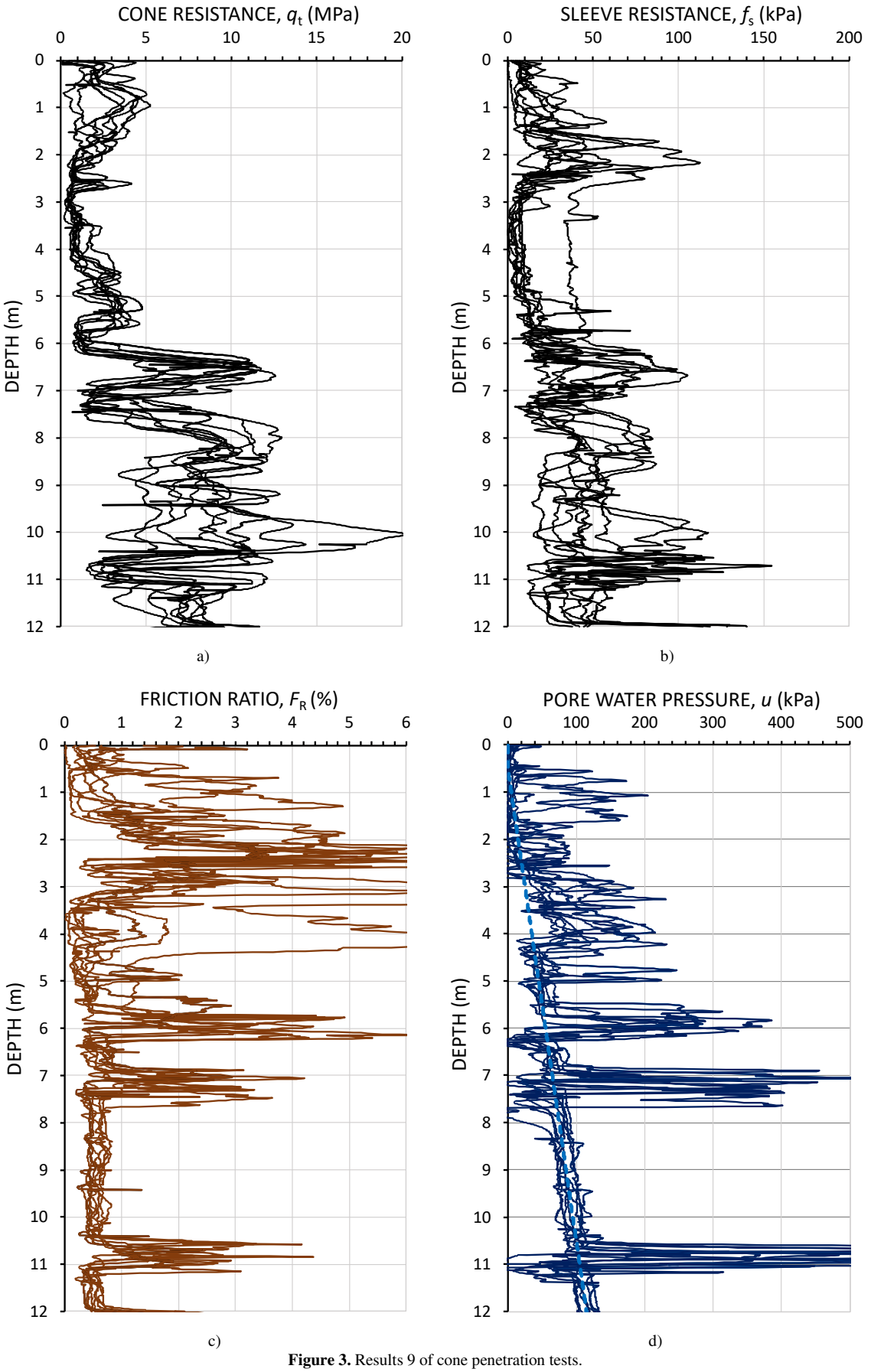
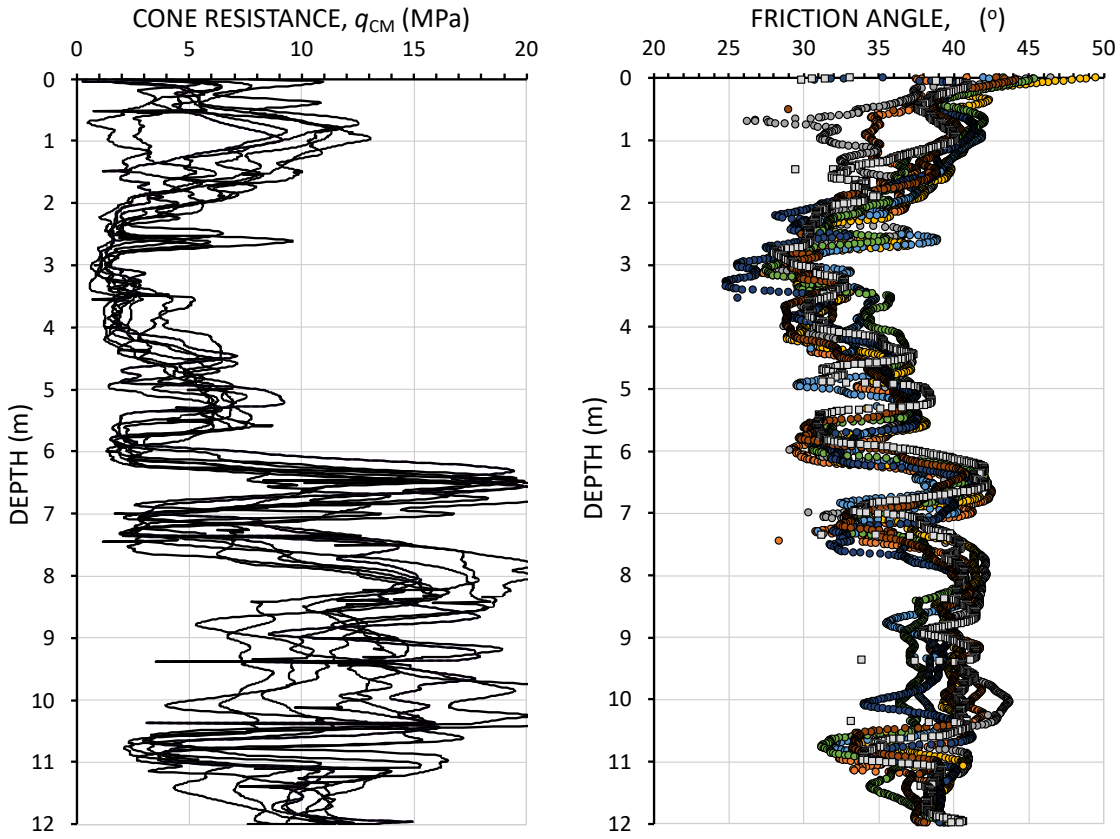
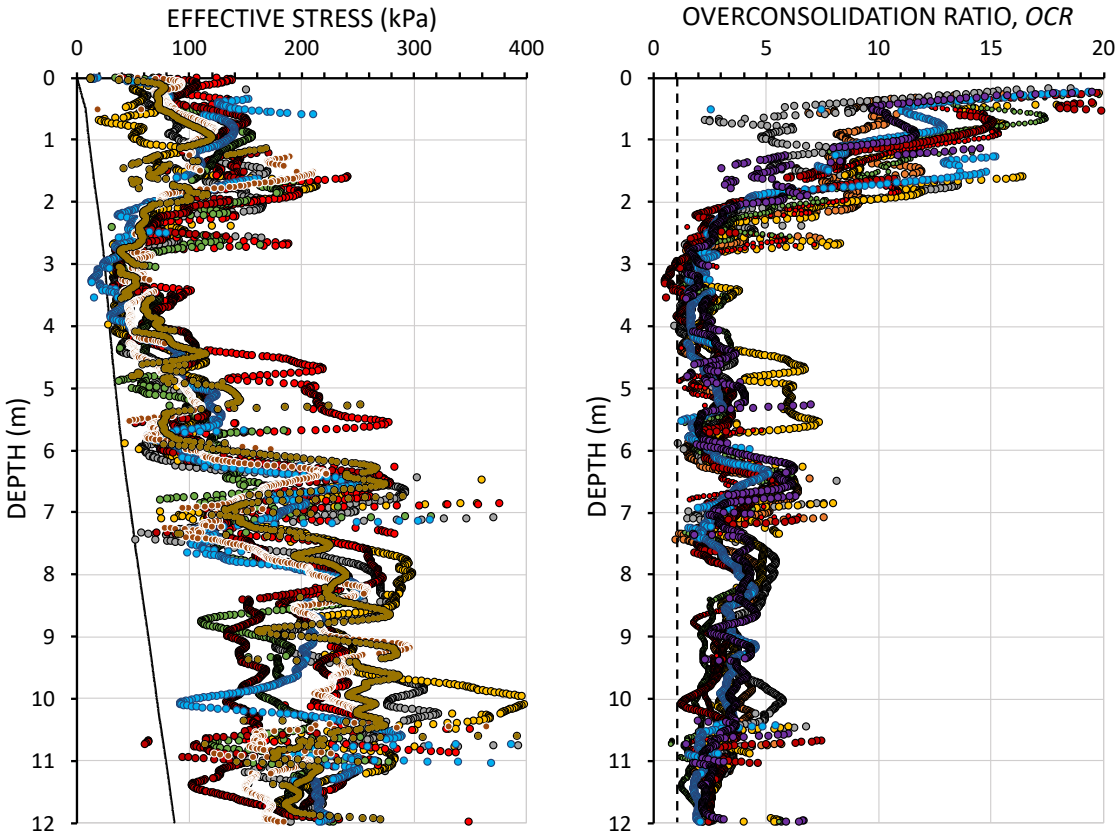


Figure 3. Results 9 of cone penetration tests.



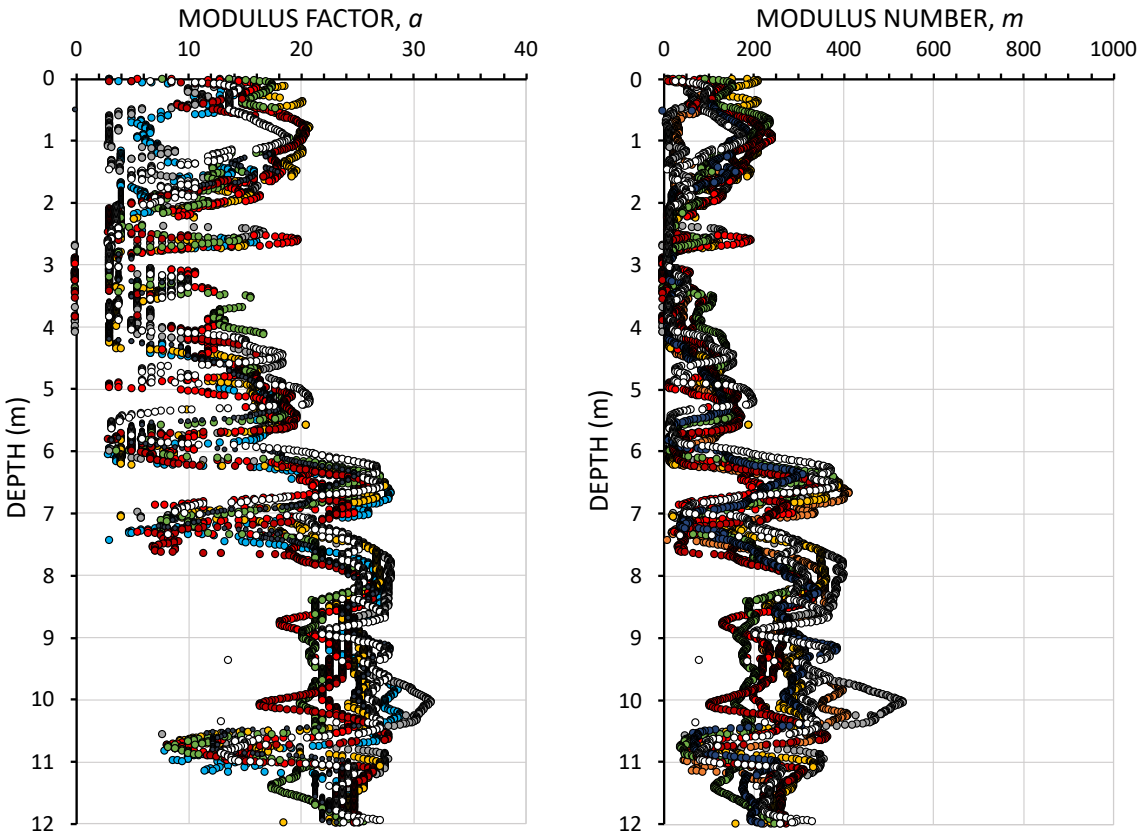
a) b)

Figure 4. Stress-adjusted cone resistance and derived friction angle.



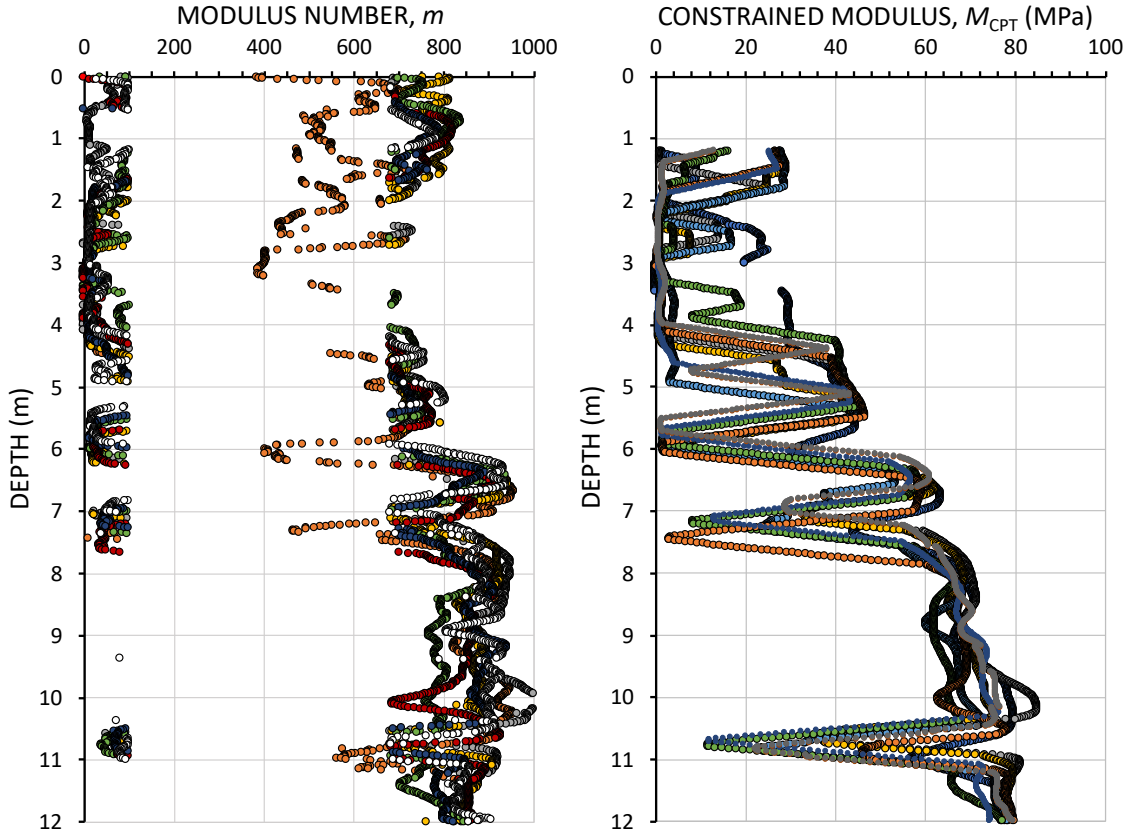
a) Preconsolidation stress, vertical effective stress (blackline) b) Overconsolidation ratio

Figure 5. Determination of stress history.



a) b)

Figure 6. Modulus factor and modulus number for normally consolidated soil, determined from CPTU data.



a) Modulus number adjusted for pre-loading effect b) Constrained modulus including pre-loading effect

Figure 7. Modulus number and constrained modulus, determined from CPTU data, with adjustment to pre-loading effect according to Eq. (17).

8 Results of DMT

The results of all six performed DMTs were included in this study. The analysis procedure started with determining the two pressure values, p_0, and p_1. Due to the variability of the soil deposit, relatively large variations of pressure values were obtained at some depths, Fig. 8. From the pressure readings, the material index, I_D, and the horizontal stress index, K_D, were then obtained. The results are shown in Fig. 9. At some depths, the scatter of data was significant, but the general trend agrees with the CPTU measurements. The material index in Fig. 9a suggests the existence in some test points of more fine-grained (clay/silt) layers down to about 5 m depth. This observation is confirmed by the relatively high excess pore water pressure readings in Fig. 3d).

The horizontal stress index is an important parameter for the calculation of the DMT modulus, E_D, and the constrained modulus, M_t, cf. Eq. (19). The constrained modulus, M_t, and the modulus number determined according to Eq. (21) are shown in Fig. 10a and b, respectively.

It is interesting to compare the modulus number determined from CPT (Fig. 6b and 7a) with the DMT modulus number (Fig. 10b). Note the difference in scale between the figures. There is good agreement between the modulus number determined from the CPTs and DMTs in the normally consolidated layer between 3 and 5 m depth. However, in the overconsolidated layers, the modulus number derived from the DMT is significantly higher than from the CPT for normally consolidated conditions. However, when the preconsolidation effect is accounted for, the modulus number from CPT (Fig. 7a) is in good agreement with the modulus numbers back-calculated from DMT. This comparison shows the importance of adjusting the modulus number for pre-loading effect according to Eq. (17) as shown in Fig. 2.

9 Conclusions

Calculation of settlement in granular soils is an important task as the design of foundations on such soils usually is governed by total and differential settlement. However, even when employing sophisticated analytical methods, often very crude assumptions are made when estimating soil compressibility and stress history. The concept of the tangent modulus method is simple and transparent and can be applied in all soil types.

In granular soils, the most reliable in-put data – although never perfect – can be derived from in-situ tests, such as the CPTU or the DMT. It is important to note the difference in soil testing between the two methods. The CPTU causes soil failure during probe penetration. Thus, deformation properties are only indirectly related to the measured cone resistance. The advantage, however, is that a relatively continuous soil profile is obtained. In the case of a DMT, the deformation properties of the soil adjacent to the blade can be measured without causing soil failure. Another important advantage of the DMT is that it measures soil properties in the lateral direction, which preserve the effect of stress history more realistically than the CPTU.

Extensive field investigations were performed on a relatively homogeneous tests site, consisting of sand and silt. The complex geology of the river deposit resulted in significant variations of geotechnical properties.

An important conclusion is that in spite of the relatively uniform site conditions, significant variations in soil properties could be observed by CPTU and DMT measurements. For important projects, it is recommended to compare results from different in-situ methods (CPTU and DMT). Moreover, the number of tests and their locations must be chosen reflecting the particular geological setting.

A method is described how the modulus number, m, can be derived from a stress-adjusted cone resistance. As this concept is based on tests on normally consolidated soils, significantly lower (conservative) values of the constrained modulus can be obtained in overconsolidated soils.

The modulus number is affected by pre-loading. This effect can be taken into account using Eq. (17). The unloading modulus number is about 3 to 7 times higher than the modulus number during virgin loading, cf. Fig. 2. The pre-loading effect is higher in loose soils with a low virgin modulus number.

The constrained modulus derived from CPT (Fig. 7b) and measured by DMT (Fig. 10a) are in good agreement, provided that the pre-loading effect is considered.

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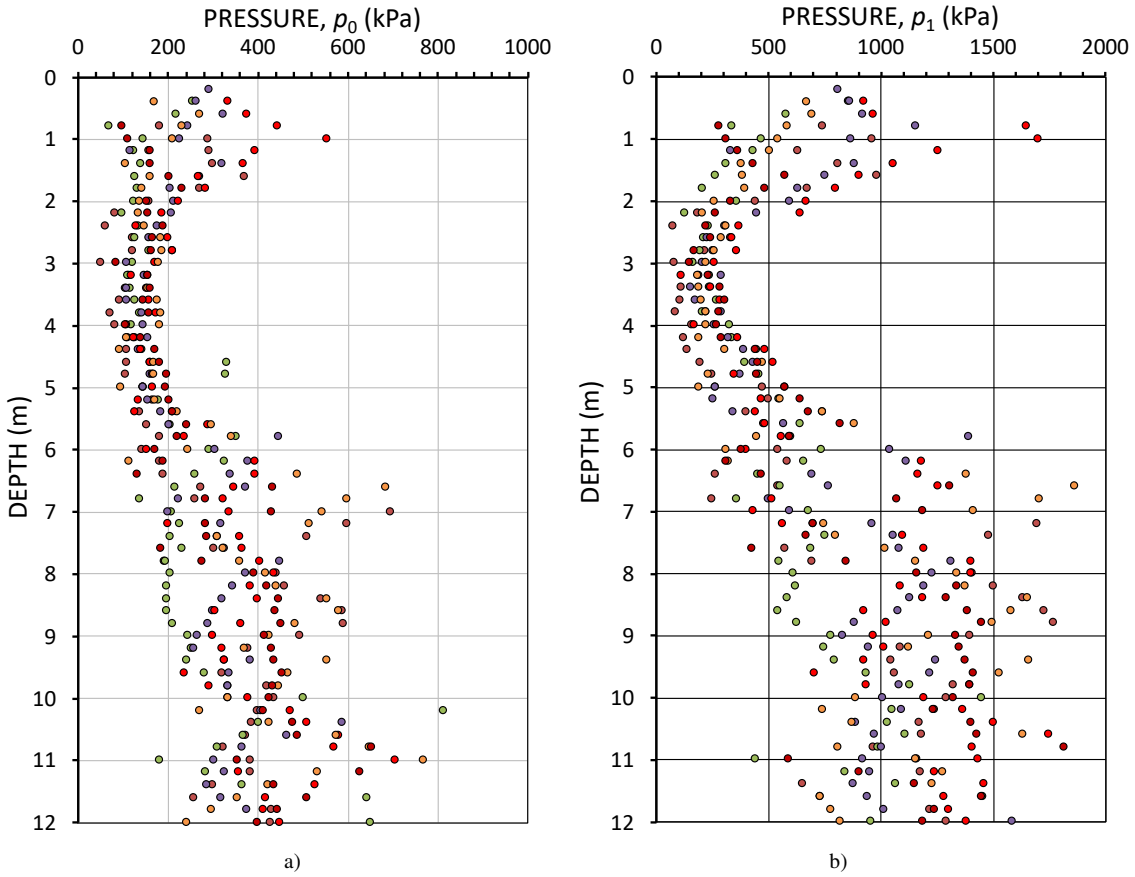


Figure 8. Pressure readings obtained from six DMT tests.

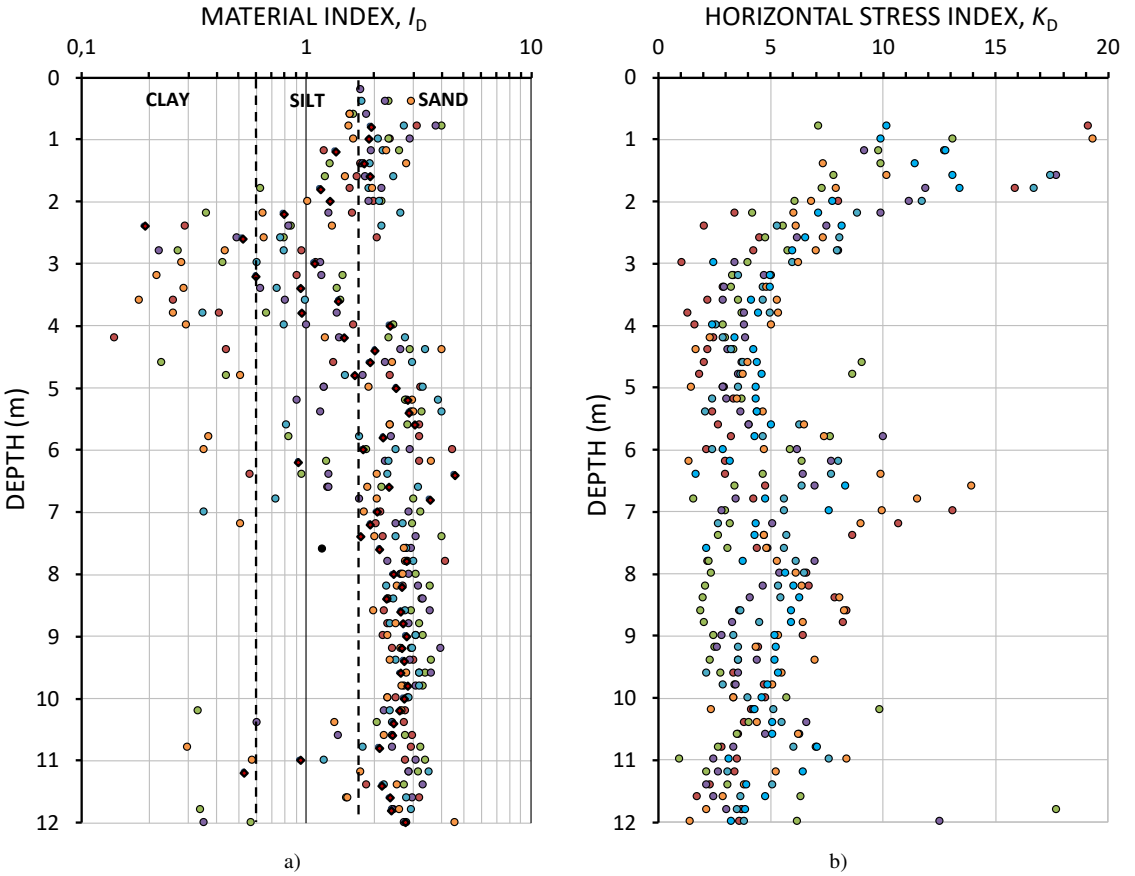


Figure 9. Material index and horizontal stress index from DMT measurements.

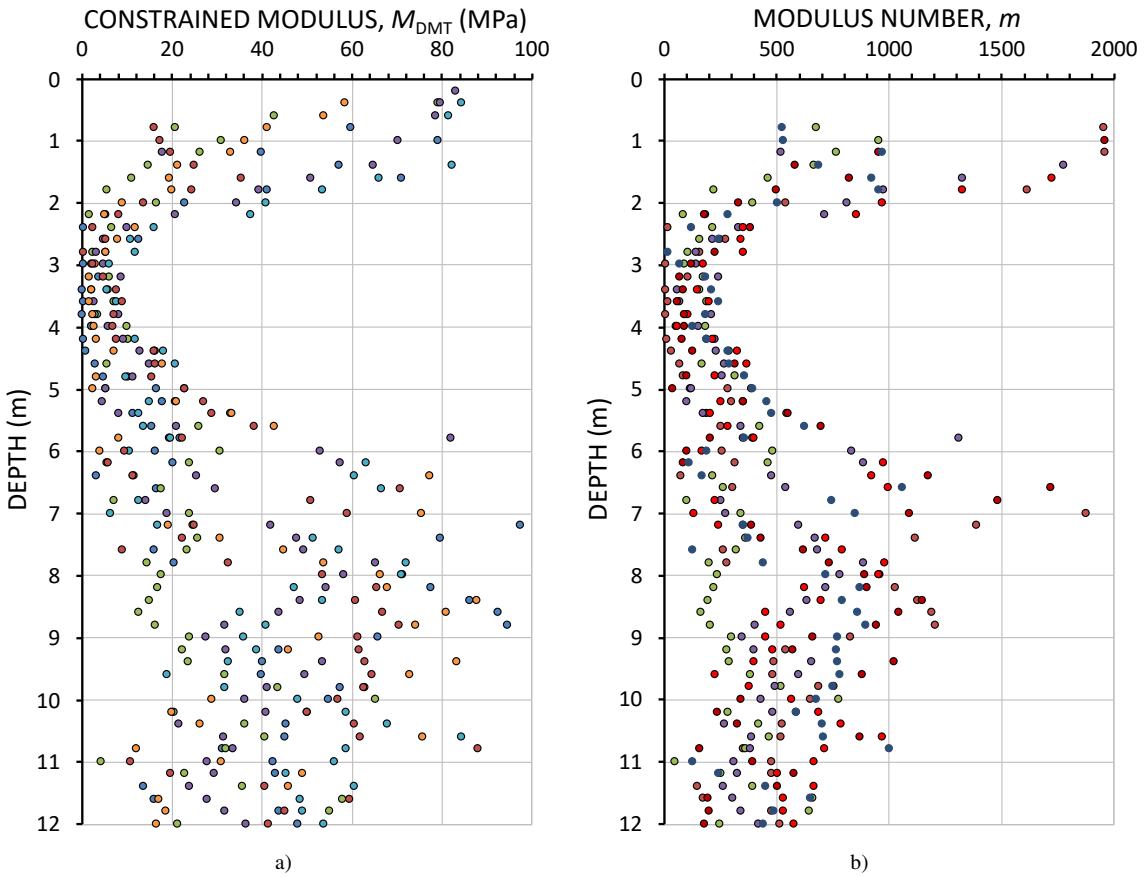


Figure 10. Constrained modulus and modulus number determined from DMT.

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