Characterizing the Stiffness of Munich`s Tertiary Soils using the TEXAM Pressuremeter

Caractérisation de la rigidité des sols tertiaires de Munich à l'aide du pressiomètre TEXAM

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ABSTRACT

This contribution presents the results from TEXAM pressuremeter tests (PMT) conducted at a pile testing site in Munich, Germany. The aim of the PMT testing campaign is to improve the characterization of the tertiary subsoil. The underground conditions at the test site reflect Munich's typical stratrigraphy, consisting of a cover layer of quaternary gravels, followed by tertiary sediments. The tertiary sediments are highly heterogeneous consisting mainly of very dense sand and very stiff to hard clay layers and interlayers, which can be partially cemented. Identifying the different layers and quantifying their mechanical properties, particularly the soil stiffness, are the keys for a sustainable and efficient design of geotechnical structures. At the testing site in Freiham, the TEXAM PMT was used to investigate comprehensively the mechanical behavior of the tertiary sediments in-situ. The investigations were conducted in six boreholes up to a depth of 40 m. Two drilling methods were utilized: water core drilling (WCD) and rotary mud drilling (RMD). As expected, the RMD technique provided better borehole stability compared to the WCD. Nevertheless, at the depths where the borehole was stable for both drilling techniques, the test results were similar. The results show that the TEXAM pressuremeter can capture the variation in stiffness of the tertiary layers in a precise manner. In addition, the pressuremeter limit pressure and stiffness show a strong correlation with small-strain stiffness data obtained from cross-hole tests. The shear stiffness decay with shear strain amplitude can be realistically estimated by evaluating the shear stiffness from unloading-reloading loops from PMT and combining these results with the seismic measurements.

RESUME

Cette contribution présente les résultats des essais pressiométriques TEXAM (PMT) réalisés sur un site d'essais de pieux à Munich, en Allemagne. L'objectif de la campagne d'essais PMT est d'améliorer la caractérisation du sous-sol tertiaire. Les conditions souterraines du site d'essai reflètent la stratrigraphie typique de Munich, composée d'une couche de couverture de graviers quaternaires, suivie de sédiments tertiaires. Les sédiments tertiaires sont très hétérogènes et se composent principalement de sable très dense et de couches et intercalaires d'argile très raides à dures, qui peuvent être partiellement cimentées. L'identification des différentes couches et la quantification de leurs propriétés mécaniques, en particulier la rigidité du sol, sont les clés d'une conception durable et efficace des structures géotechniques. Sur le site d'essai de Freiham, le TEXAM PMT a été utilisé pour étudier de manière exhaustive le comportement mécanique des sédiments tertiaires in situ. Les études ont été menées dans six trous de forage jusqu'à une profondeur de 40 m. Deux méthodes de forage ont été utilisées : le carottage à l'eau (WCD) et le forage rotatif à la boue (RMD). Comme on pouvait s'y attendre, la technique RMD a permis d'obtenir une meilleure stabilité du trou de forage que la technique WCD. Néanmoins, aux profondeurs où le trou de forage était stable pour les deux techniques de forage, les résultats des tests étaient similaires. Les résultats montrent que le pressiomètre TEXAM peut capturer la variation de la rigidité des couches tertiaires de manière précise. En outre, la pression limite et la rigidité du pressiomètre présentent une forte corrélation avec les données de rigidité à faible déformation obtenues à partir d'essais transversaux. La décroissance de la rigidité de cisaillement avec l'amplitude de la déformation de cisaillement peut être estimée de manière réaliste en évaluant la rigidité de cisaillement à partir des boucles de déchargement-rechargement du PMT et en combinant ces résultats avec les mesures sismiques.

Keywords: in-situ testing; pressuremeter testing; TEXAM PMT; tertiary soils; drilling method.

1. Introduction

The expansion of the subway network in Munich, Germany, is increasing the demand for more efficient and sustainable design of underground temporary and permanent structures. To increase efficiency of pile design for foundations and deep excavations, an extensive pile testing campaign was conducted at a site near Munich (Rebstock et al., 2024). The goal of the investigation was to determine realistic values of the skin friction and base resistance for Munich tertiary soils, which are currently assumed conservatively in the design.

The key to achieve a robust, efficient and sustainable design is the proper geotechnical characterization of the ground, including the determination of the subsoil stratification and a realistic determination of the mechanical properties of the strata, particularly the non-linear variation of soil stiffness with shear strains (Figure 1). Particularly, for the serviceability limit state, which is usually more relevant than the limit state for underground structures in tertiary soils, information of the decay of the stiffness over the full range of shear strain is often required.

Generally, the relationship $G-\gamma$ can be evaluated by means of laboratory and field tests. The main limitation of laboratory tests is the unavoidable soil disturbance during sampling, which causes the stiffness obtained in the field to be considerably higher than in the lab for the same stress state. While seismic-based methods (e.g. cross-hole test) can only capture the shear stiffness for very small strains in the quasi-linear regime, the pressuremeter test (PMT) delivers information about the soil stiffness in the medium to large strain range.

The PMT is basically the radial expansion of a cylindrical cavity with a finite length. To carry out the tests, a cylindrical probe, consisting of an inflatable membrane that expands radially against the walls of the borehole, is inserted into the ground and expanded using a pressurized fluid (or gas). During the expansion, the injected fluid volume (corresponding to the cavity volume) and corresponding pressure are measured. Depending on the insertion method, three types of PMT are available: pushed-in PMT, pre-bored PMT and self-boring PMT (SBP). The SBP is the most complex, expensive and technically demanding device, but produces the lowest disturbance (Schnaid, 2009). On the contrary, the push-in PMT is much easier to install, but produces the largest disturbance of the surrounding soil. In addition, pushing the probe into the subsoil is only feasible in soft to stiff soils. The pre-bored PMT, which is applicable to all soil types, requires the drilling of a borehole, which must be stable until the probe is inserted. The disturbance of the soil is largely influenced by the drilling technique, the employed drilling tools and the soil sensitivity. By employing a proper drilling technique and drilling tools, disturbance can be reduced in less sensitive soils and the results are comparable to those obtained with the SBP.

As a part of the pile testing campaign, extensive field and laboratory investigations have been conducted to characterize the geotechnical subsoil conditions, among them seismic tests (Csuka et al. (2024)). The aims of this contribution is to describe the tests with the TEXAM probe, to evaluate and to interpret the experimental results. The test campaign had three specific goals: 1) to investigate the influence of the drilling technique on the results and determine whether a water core drilling (WCD) or a mud rotary drilling (MRD) are appropriate drilling techniques for the Munich tertiary soils; 2) to assess the ability of the PMT to identify soil layers with similar particle size distribution and mineral composition, but different stiffness and mechanical properties due to different formation history and geological process, e.g. diagenesis; 3) to evaluate the decay of the shear stiffness with the shear strain in the full strain range by combining the results of PMT and cross-hole tests.

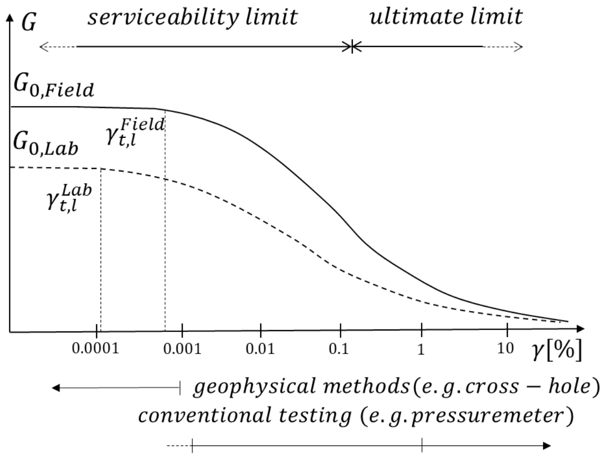

Figure 1. Non-linear stiffness decay in field and laboratory.

2. Site description and methods used

2.1. Geological set-up

Figure 2 provides an overview of the pile testing site, showing the locations of three exploratory boreholes, BP1-3, two boreholes used for seismic testing, SK1-2, and the six boreholes used for PMT testing.

The geological conditions at the site are given by the boreholes BP1 to BP3. In the Munich region, two main geological formations are typically encountered. The cover layer, which is called the "Munich gravel plain", consists of medium dense to very dense quaternary sandy, silty gravels with depths up to around 15 m. The quaternary formation is underlain by the tertiary soils of the "Upper Freshwater Molasse", which extend to depths of several hundred meters. The tertiary soils consist of alternating fine-grained (clayey silts, silty clays and clays) and coarse-grained (silty sands and sands) soil layers with interbedded soil layers and lenses. Generally, the sands are very dense, and the clays are stiff to hard. Due to slowly diagenetic processes, some tertiary soils are classified as soft rocks. In addition to the preload due to strong erosion of their surface (Pelz et al., 2009), the tertiary soils can present a pronounced variability of stiffness and shear strength due to local particle bonding (so-called "Kalkkonkretionen") and weak zones with pre-defined shear zones (Hänisch surfaces) caused by deformation process, e.g. induced by tectonics. The results of the seismic-based investigations (Csuka et al., 2024) and the observations during the pile installation process confirmed this variability. In Figure 7 the results of the seismic-based testing are presented. Two very hard layers were identified at around -26 m and -34 m depth.

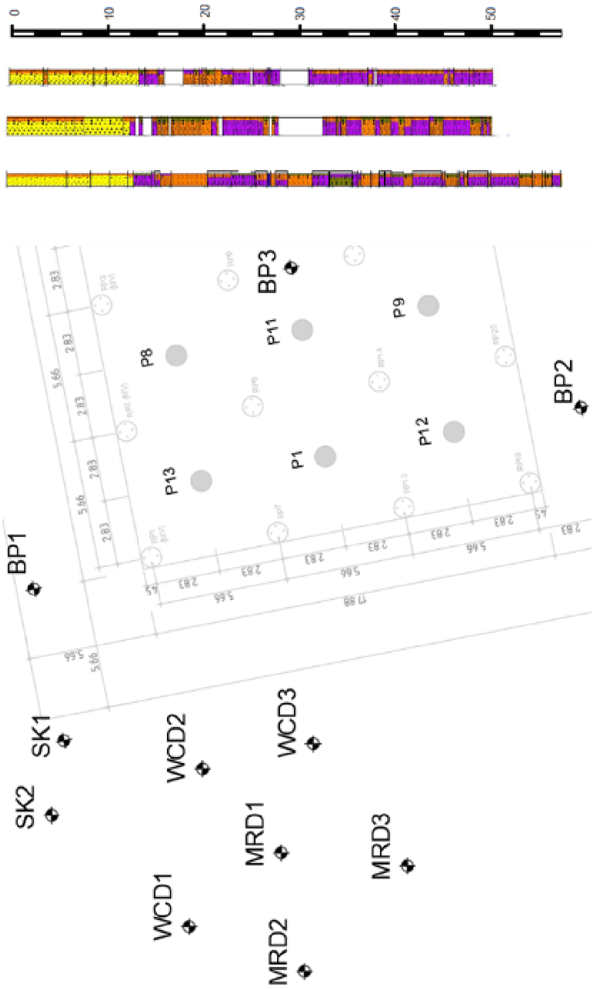

Figure 2. Site overview with the location of the six boreholes for PMT testing, WCD1-3 and MRD1-3, the two boreholes used in seismic testing, SK1-2, and geological profile at the exploratory boreholes BP1-3. The colors in the geological profiles represent: yellow - quaternary gravel, orange - tertiary sand and violet - tertiary clay.

2.2. TEXAM pressuremeter

Due to geological conditions, economic reasons and time constrains to carry out the tests, schedule reasons, the pre-bored PMT technique was selected among the three techniques available.

The pre-bored TEXAM PMT of the company Roctest was selected for the tests (Figure 3). It is a hydraulic monocell PMT with a volume of 1,714 cm³ and a maximum working pressure of 10,000 kPa. In the TEXAM pressuremeter, the expansion is conducted by injecting a fluid volume into probe, while the pressure results from the soil response. The injected water volume is controlled precisely by a piston, which is manually activated with a screw jack. A read-out unit consisting of a D/P BOX unit operated with an Android tablet is used to control the PMT test and for real-time data visualization. The TEXAM unit's rugged construction proved beneficial, ensuring reliability during testing. The easy-to-change membrane of the probe minimized time delays during PMT testing, even though several probes burst during testing due to local soil inhomogeneity.

The pressure readings were recorded only 15 seconds after injecting a given water volume to allow pressure equalization in the system. The raw data were corrected with two pressure-volume functions: the pressure loss , and the volume loss correction. The pressure loss function considers the inflation resistance of the probe in the air. The maximum pressure to inflate the probe in the air of 40 kPa at the maximum volume of 1,400 cm³ is significantly lower than the pressures attained during the field tests. The volume correction considers the compressibility of the probe. To determine it, the probe is inflated inside a calibration and the change of volume as a function of the pressure is determined. The used TEXAM has a system stiffness of around 100 kPa / cm³.

The main assumption for the interpretation of the PMT results is that the probe expands uniformly. This requires a cylindrical cavity and homogenous soil conditions around the probe. Inhomogeneity, e.g. due to soil layers with different stiffness, cause non-uniform expansion of the PMT membrane, which can lead to membrane bursting. This happened several times during the PMT campaign.

Figure 3. TEXAM pressuremeter unit

2.3. Drilling methods

The boreholes required for inserting the probe were drilled using two techniques. The first method used at the boreholes WCD1-3 is water core drilling (WCD). This is a standard drilling method in the region, as it allows for obtaining core samples for laboratory testing. The second method used at the boreholes MRD1-3 is mud rotary drilling (MRD), which is the recommended drilling method for the soil conditions at the site according to ASTM D4719 – 2020, DIN EN ISO 22476-4. The schematic description of the drilling steps for the two techniques is shown in Figure 4.

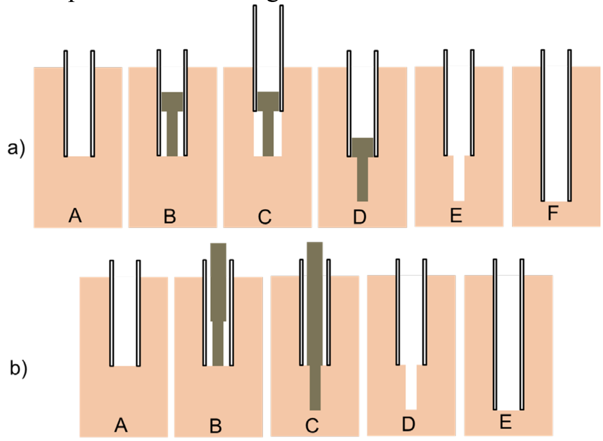

Figure 4. Drilling sequence of the pre-bored borehole for a) water core drilling and b) mud rotary drilling methods.

The rope core drilling method is used for WCD (Figure 4a). After reaching the investigation depth (step A), the drill bit (76 mm) used to drill the PMT test cavity is lowered inside the borehole casing using a rope (step B). After that, the borehole casing is lifted to connect the drill bit (step C). Once connected, the drilling begins by rotating the borehole casing (step D). Subsequently, the drill bit is removed, and the PMT test is performed (step E). After finalizing the PMT test, the borehole casing is lowered to the depth of the subsequent investigation (step F). In the MRD (Figure 4b), after reaching the investigation depth (step A), the drill bit (74 mm) is lowered using drilling rods (step B). The test cavity is drilled using the drilling rods (step C). After drilling the PMT test cavity, the drill bit is removed, and the PMT test is carried out (step D). In the last step, the borehole casing is lowered to the subsequent investigation depth (step E). The primary differences between the two drilling techniques are the method of drill bit rotation, the drill bit itself and the drilling fluid (water in the WCD and mud in the MRD). In total, 39 PMT tests were successfully conducted using these two drilling techniques.

3. Results and interpretation

As in the cylindrical cavity expansion problem, the evolution of the experimental radial pressure as a function of the cavity volume or cavity radius depends on the mechanical behavior of the surrounding soil. The more complex the behavior of the soil, the more difficult is the theoretical interpretation of the experimental results of a PMT. Assuming that the behavior of the soil around the pressuremeter can be described by an elastic-ideal plastic constitutive model, the experimental pressure-volume response depends only on the stiffness and the shear strength of the material. In this case, the stiffness and the shear strength parameters of soil can be determined by comparing the experimental and theoretical curve. However, in real soil, several issues make the theoretical interpretation of PMT results difficult:

1) elastic-ideal plastic material models can only roughly describe the behavior of real soils, which is non-linear and state-dependent.

2) the strain and stress fields surrounding the probe are not necessary homogeneous as assumed in the cavity expansion theory

3) the drainage conditions during the test are often not clearly defined, while solutions for elastic-ideal plastic material behavior assume either drained or undrained conditions.

4) the disturbance caused by drilling, the size of the initial gap between the membrane and the cavity wall and the limitations of the devices regarding maximum volume and pressure.

To overcome some of these limitations, numerical methods using advanced constitutive models for the soil can be used (Schorr et al., 2024). Nevertheless, in this contribution, we will use empirical methods, which are conventionally used to assess and interpret the results of PMT in practice.

The result of the PMT and its interpretation are shown exemplarily in Figure 5. The diagrams show the determination of the pressuremeter modulus E_{pmt} and the pressure limit (P_L). E_{pmt} is computed from the quasi-linear part of the pressure-volume expansion curve (Figure 5a) in the region after the probe membrane comes in full contact with the borehole wall (point P_0) and the onset of nonlinear deformation according to:

$$E_{pmt} = 2(1 + \upsilon)(V_0 + V_m)\frac{\Delta P}{\Delta V} \quad (1)$$

where υ is the Poisson ratio, V_0 is the initial volume of the probe and V_m is the mean volume of the linear part. For the interpretation, a Poisson ratio υ = 0.3 was assumed.

Although the determination of the pressuremeter modulus E_{pmt} has been established historically due to the work of Menard for the semi-empirical design of shallow foundations (Schnaid, 2009), the shear modulus G_{pmt} can be directly computed from the test data using:

$$G_{pmt} = \Delta P/(2\Delta\varepsilon_c) \quad (2)$$

where $\varepsilon_c = \Delta R/R$ and R is the radius of the cavity and $\Delta R = R - R_0$.

Besides the stiffness moduli, information regarding the shear strength of the soil can be obtained from the ultimate limit pressure P_L. One standardized interpretation method of P_L is the 1/V method, according to ASTM D4719 – 2020. Figure 5b shows an interpretation of P_L for the test shown in Figure 5a exemplary. The inverse of the volume expansion is plotted against the pressure. Given that enough plastic strains developed during the test, the 1/V curve is linearly extrapolated, and the intersection with the limiting value $1/V_{lim}$ gives P_L. V_{lim} is defined as the volume at which the cavity expanded twice its initial size.

As can be seen in Figure 6, the variation of the pressuremeter moduli E_{pmt} and the shear wave velocity from cross-hole tests from Csuka et al. 2024 over the depth show a good correlation. Both the cross-hole and the PMT show their largest values at the depths of 26 and 34 m below the ground surface, where the two hard fine-grained tertiary soil layers were identified during pile installation. The lowest values of E_{pmt} and V_s are observed in the alternating layers of sands and clays between 15 m and 23 m and in the tertiary sand layer between 30 m and 32 m.

In the "weak" upper layer, between 15 m and 23 m, the cavity wall was often unstable and collapsed when the WCD technique was used. In these cases, the PMT could not be conducted. Similar observations were made at other sites where pre-boring with the WCD technique was employed for other borehole tests. In contrast, the boreholes remained stable, and the PMT was consistently successfully carried out using the MRD technique. We believe that the slurry played a crucial role in supporting the cavity wall in coarse-grained soils, where particle bonding is absent.

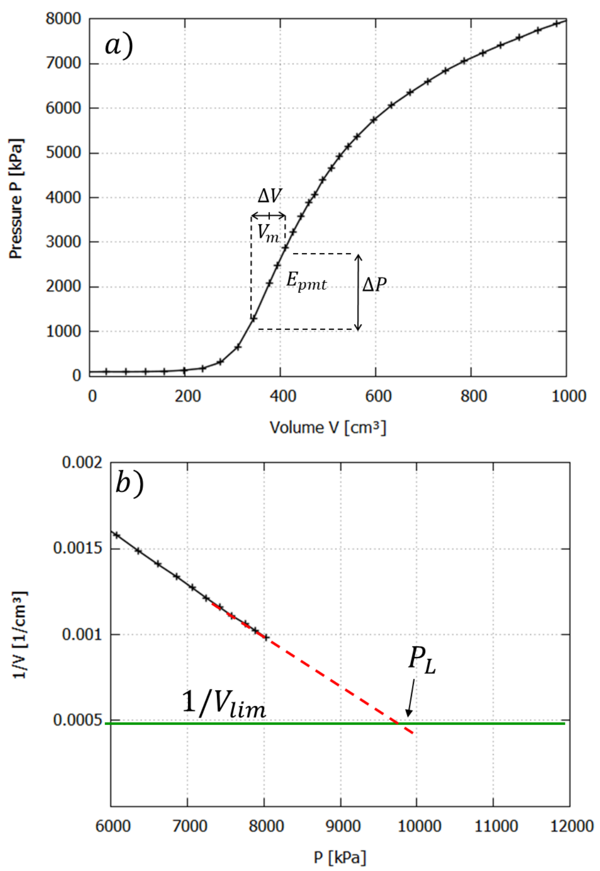

Figure 5. Example of a PMT test and interpretation of a)PMT modulus E_{pmt} and b) PMT pressure limit P_L.

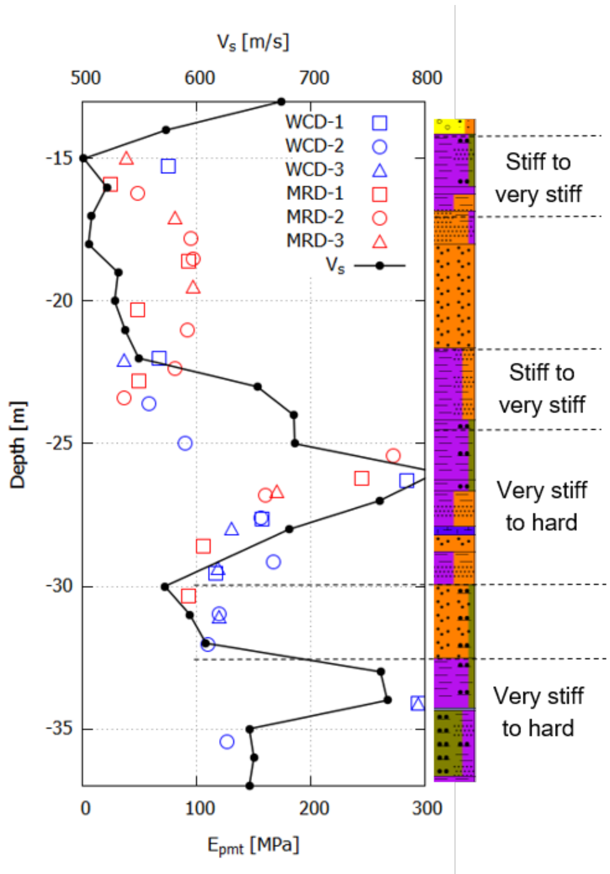

Figure 6. Comparison of E_{pmt} with the shear wave velocity V_s over the depth.

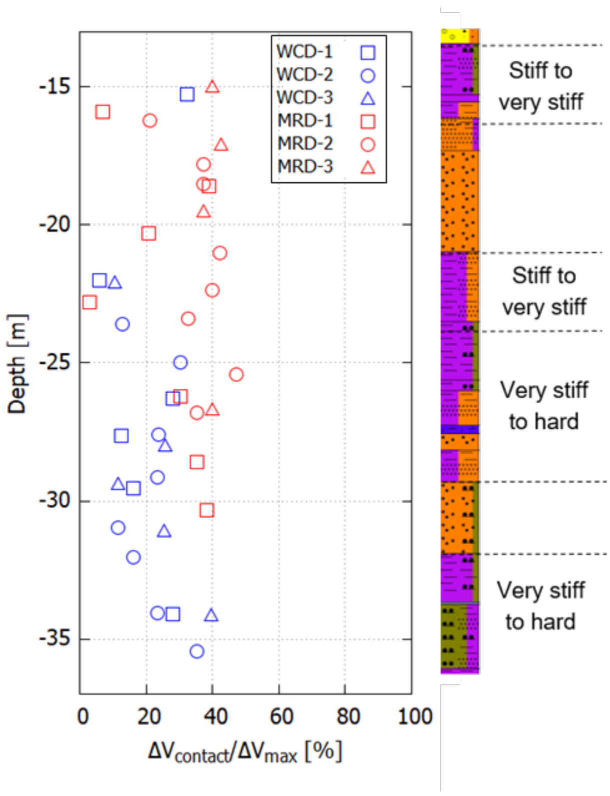

Figure 7. Expansion required to close the initial gap between the borehole wall and the membrane at the beginning of the PMT in dependence of depth and drilling technique ($\Delta V_{contact}$: expansion required to close the gap, ΔV_{max}: maximum admissible pressuremeter expansion)

At other depths, the results obtained using both drilling techniques showed good agreement. By comparing the PMT expansion curves from both techniques, we found that the boreholes created with the MRD technique were consistently larger than those created with the WCD technique (see Figure 7). This result was unexpected, as the drill bit used with the MRD technique was smaller (74 mm) compared to the one used with the WCD technique (76 mm). We hypothesize that the drill bit is better guided in the WCD technique, as it is rotated by the casing, which is itself guided by the surrounding soil.

Figure 8 shows the limit pressure P_L against the pressuremeter modulus Epmt. The experimental values fall within the range found in the literature for very dense sands and hard clay (Briaud, 2013). A good correlation between these two quantities can be observed. Such correlations are useful for PMT tests to estimate PL in the cases in which the initial borehole diameter is too large, and the non-linear part of the expansion curve, which is required to determine PL, cannot be achieved during the test. The ratio Epmt/PL is plotted with depth in Figure 9. Generally, the experimental values fall within the range found in the literature for very dense sands and hard clay. The lower values in the diagram, which correspond to the upper tertiary layers, are on and partially below the lower bound from the literature. According to (Schnaid, 2009), such low values could result from disturbance during installation rather than reflecting the true in-situ conditions.

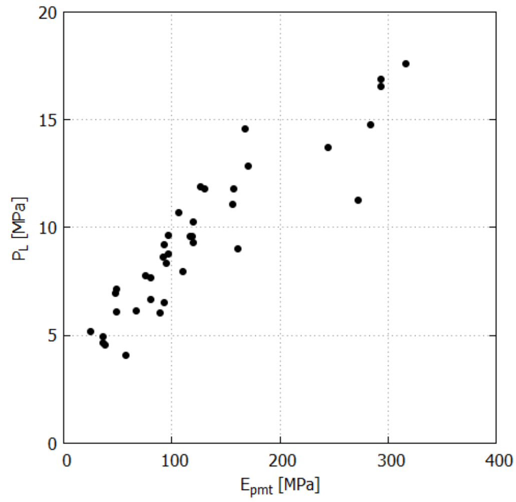

Figure 8. Comparison of the pressuremeter modulus E_{pmt} against the limit pressure P_L.

Correlations between the pressuremeter modulus E_{pmt} and the shear wave velocity V_s can be found in the literature (Akkaya et al., 2019; Cheshomi & Khalili, 2021). However, a more meaningful comparison can be made between the pressuremeter and small-strain shear modulus. Equation 2 shows that assuming elastic material behavior, the shear modulus G can be directly determined from the pressure-expansion curve. This large-strain shear modulus G_{pmt} derived from PMT can be compared to the small–strain shear modulus G_0 derived from cross-hole tests. In Figure 10, the relationship between G_{pmt} and G_0 is shown together with values obtained from the literature (Akkaya et al., 2019; Cheshomi & Khalili, 2021). A nonlinear increase of the pressuremeter shear modulus G_{pmt} with G_0 is observed. This increase may be attributed to two factors. Firstly, the soil disturbance during drilling tends to be lower in harder soils. Secondly, the strain levels achieved during testing harder layers are lower than in soft layers and thus the achieved G/G_0 values are higher. Assuming ideal elastic behavior for very hard layers, the relationship G/G_0 should tend to the unity for very large stiffness.

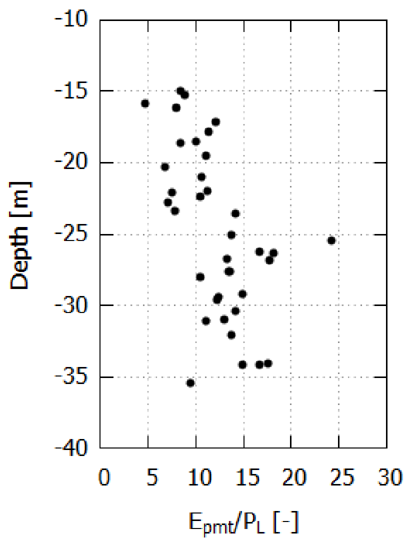

Figure 9. E_{pmt}/P_L ratio plotted with depth.

These correlations could be use in further geotechnical investigations to identify outliers. A feasible approach for conducting an economic geotechnical investigation could be to combine seismic methods with PMT. The cost of carrying out seismic measurements is much lower than PMT. Thus, one could start identifying soil layers using seismic methods and conduct the PMT at the selected layers of interest.

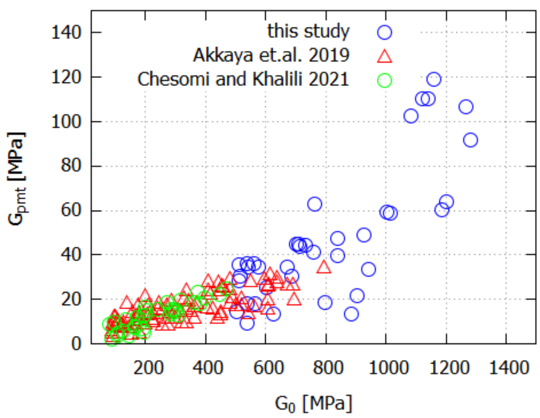

Figure 10. Comparison between the shear moduli obtained from PMT (G_{pmt}) and from cross-hole tests (G_0). G_0 was computed by assuming $\rho = 2$ g/cm³.

3.1. Non-linear shear modulus

Several authors have proposed to derive the decay of shear stiffness with the shear strain from unloading-reloading loops of PMT (Belloti et al. 1991, Jardine 1992, Bolton and Whittle 1999, Whittle and Liu 2013, Lopes 2020). They claim that using the unloading-reloading loops is more appropriate than the linear part of the first loading, which is strongly affected by the installation-induced disturbance. Furthermore, the cavity radius at strain reversal can be used as reference to compute the strain-dependent stiffness. According to (Schnaid 2009), combining shear wave velocity measurements and unloading-reloading loops from PMT is currently the most reliable method to assess the decay of the shear stiffness from field tests.

In this study, the evaluation of shear stiffness degradation with increasing shear strain from the PMT tests is carried out using the tangent modulus, as defined in Equation 2, in conjunction with unloading-reloading loops. The coresponding shear strain $\gamma\ (= \varepsilon_r - \varepsilon_\theta \approx 2\varepsilon_c)$ used to plot the non-linear tangent G_{pmt}/G_0 is calculated as the difference between the current strain and the reference strain γ_{ref} at the beginning of the unloading or reloading loop.

The pressure expansion curves for two PMT including unloading-reloading loops for a stiff to very stiff clay layer (depth of 23,3 m) and a sand layer (depth of 31 m) are shown exemplarily in Figure 11. Furthermore, the normalized tangent modulus G_{pmt}/G_0 for the two curves are plotted in Figure 12. The G_0 values were determined from the cross-hole results at the same depth of the PMT.

According to the elasticity theory, the mean pressure does not change during a cylindrical cavity expansion, since the radial and the hoop stresses respectively increase and decreases by the same value and the vertical stresses do not change during the expansion. This expectation is confirmed in the clay layer, in which the normalized shear stiffness G_{pmt}/G_0 does not show any significant dependence on the cavity pressure. On the contrary, a slightly increase of G_{pmt}/G_0 is observed in the sand layer, which can be attributed most probably to initial soil disturbance.

In Figure 13, the relationship G_{pmt}/G_0 as a function of the shear strain is for all PMT tests, separated in tertiary sand (blue circles) and tertiary clay (red circles), are shown. As can be seen, there is no significant differences of the degradation curves obtained for sand and clay. In addition, the experimental data for $\gamma > 0.001$ are in the range of degradation curves proposed by Vucetic and Dobry (1991) for fine-grained soils with different plasticity index, which are also plotted in the diagram. For $\gamma < 0.001$, the significant scatter of G_{pmt} and the deviations of the experimental data from the expected range result from the accuracy limitations of the TEXAM to measure the corresponding small volume and pressure changes during unloading.

4. Conclusions and outlook

In an extensive experimental campaign, the potential of the PMT technique to evaluate the mechanical properties of the Munich Tertiary soils was investigated. The campaign included 39 pre-bored PMTs using the TEXAM pressuremeter, installed with different drilling techniques across six neighboring boreholes.

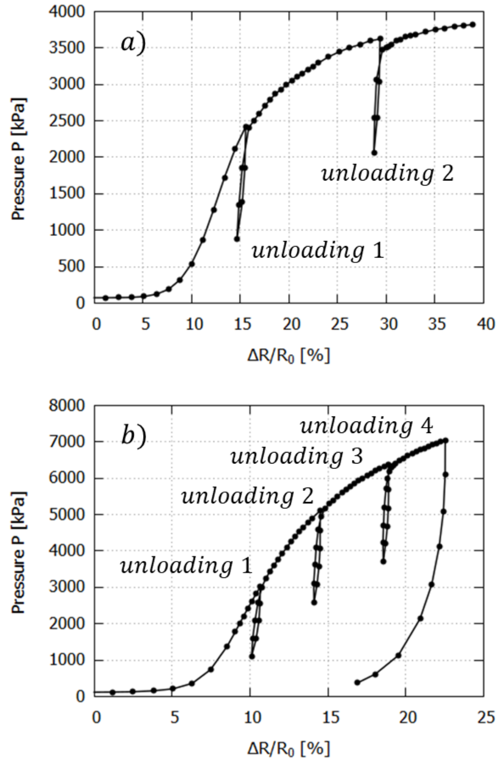

Figure 11. Example of data tests containing unloading-reloading loops for a) very stiff clay and b) sand layer

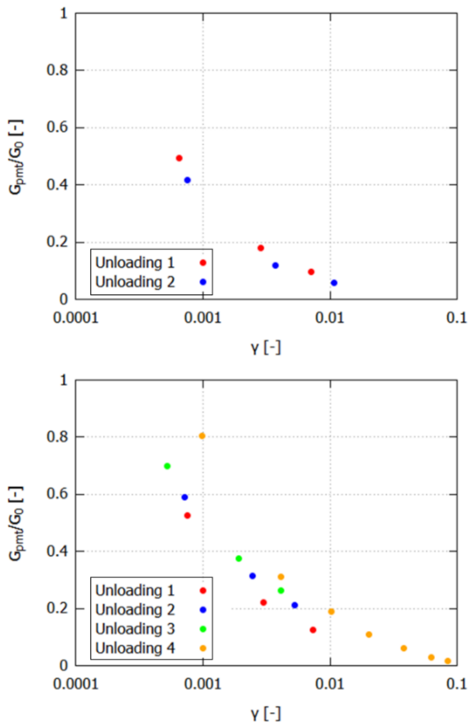

Figure 12. Normalized pressuremeter tangent modulus G_{pmt}/G_0 for the PMT tests shown in Figure 11.

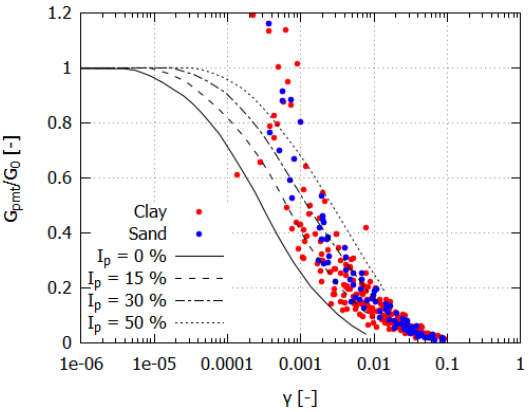

Figure 13. Normalized shear modulus.

The experimental pressure-expansion curves, pressuremeter modulus, and limit pressure fall within the ranges expected for hard soils and soft rocks, as reported in the literature. The stiff-to-very-stiff Tertiary clay layers exhibit the greatest variability in pressuremeter moduli, which may indicate higher susceptibility to disturbance during installation. The pressuremeter modulus and the small-strain shear modulus from cross-hole tests show a similar variation with depth. Thus, this testing campaign successfully demonstrated the capability of both methods to identify layers with significant differences in stiffness and shear strength, particularly weak zones in fine-grained soil layers, which can be crucial for the stability and serviceability of underground constructions. By combining PMT results with small-strain stiffness from cross-hole tests, the decay of shear stiffness with increasing shear strain in the Munich Tertiary soils could be estimated from the unloading-reloading loops of the PMT. The resulting degradation curves for Tertiary sands and clays do not show significant differences and fall within the range proposed by Vucetic and Dobry (1991) for fine-grained soils with varying plasticity indices.

The TEXAM pressuremeter produced smooth curves during both loading and unloading phases. Its robust probe and user-friendly control unit reduce testing and waiting times, enabling efficient execution. A current limitation of the device is the absence of local volume and pressure measurements. While the pressure-expansion curves derived from measurements at the whole system level are accurate enough to determine the pressuremeter modulus and limit pressure, they are insufficient to assess stiffness from unloading-reloading loops at shear strains $\gamma < 0.001$.

In summary, the potential of PMT to characterize the mechanical properties of Munich Tertiary soils was successfully demonstrated. When combined with seismic testing, it allows for the assessment of installation-induced disturbance and the evaluation of stiffness across a broad strain range. Moreover, performing seismic tests in advance can help efficiently identify target layers—such as weak zones—for PMT deployment. Building an experimental database of PMT and shear wave measurements will enable a more realistic characterization of the mechanical properties of Munich Tertiary soils, thereby supporting more efficient and sustainable geotechnical design for large underground infrastructure projects in the Munich area.

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