The Vibro-Penetration Testing (VPT) and its application to the control of blasting compaction in very loose sandy dumps

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ABSTRACT

During the so-called vibro-penetration test (VPT), a vertical harmonic excitation force drives a rod with a conical tip into the ground. For the evaluation of the VPT resistance, an energy-corrected number of vibration cycles N_{z10} for 0.10 m penetration is used. In order to determine N_{z10}, the dynamic penetration resistance, the tip and shaft resistance, the tip acceleration, and the depth of the penetrometer tip are continuously recorded and processed according to a mechanical vibro-penetration model. Calibration chamber tests, field tests, and Finite Element Analyses, which were carried out to validate the assumed penetration mechanism and to investigate the influence of the state variables (density and effective stresses) and the machine parameters (static moment, frequency, and static load) on the penetration resistance. As the force and displacement of the penetrometer are determined at the tip, an equivalent spring stiffness, which is correlated with the soil stiffness, can be calculated over the driving depth. Since vibro-penetration resistance is closely related to the soil behavior under cyclic shearing, the VPT should be more appropriate than monotonic tests to characterize the ground response to repeated dynamic and alternating loading, e.g. pile drivability, ground compaction, and soil liquefaction susceptibility. Compared with the cone-penetration test (CPT), the VPT equipment is much lighter, the execution time is shorter, and in-situ investigations of medium-dense to dense cohesionless soils at large depths are feasible. In this contribution, VPT and CPT are used to investigate the effect of blasting compaction in very loose dumps from opencast mines in Lusatia, Brandenburg, Germany. It is concluded that the VPT results are reproducible. In addition, N_{z10} shows a clear correlation with the state variables and the VPT records before and after blasting compares well with the results of CPT.

Keywords: vibratory driving, dynamic penetration testing, field testing, compaction control

1. Introduction

The mechanical behaviour of simple cohesionless grain skeletons depends on granulometric properties, such as grain shape, grain mineral and grain size, as well as on the state of the material, which for non-cemented soils is defined by the density and stress. The in-situ state of stress is usually determined from the overburden and empirical estimations of the earth pressure K_0. Since undisturbed sampling of granular materials is difficult and costly, the density is usually determined indirectly by means of in-situ tests.

Among other techniques, penetration tests provide a reliable and affordable basis for the empirical or analytical estimation of the in-situ density. Dynamic penetration tests as SPT (Standard Penetration Tests) and DP (Dynamic Probing) are easy to carry out and require a relatively simple equipment permitting frequent tests. However, the blow count is strongly influenced by variations of the energy delivered by the hammer, energy losses as well as by other factors like rod friction and hammer type. On the other hand, static penetration tests, which have advantages regarding simplicity, continuous data recording and reproducibility, show difficulties to overcome large penetration resistances in medium dense and dense granular soil deposits, especially at large depths.

In the vibro-penetration test (VPT), a vibratory hammer and the weight of a vibration-isolated bias mass generate the penetration force. As in conventional dynamic penetration tests, the driving resistance in the VPT is in a first approximation, inversely proportional to the driving velocity. A change of the machine parameters of the vibratory hammer, equivalent to a change of the blow energy in impact-based dynamic penetration tests, and the increase of the rod length and friction resistance with increasing testing depth, may alter the energy available to displace the soil at the tip. Such alterations, which are neither related to variations of soil properties nor to changes of the soil states over depth, may cause ambiguous results if one uses penetration rate as a measure of penetration resistance without taking into account the driving energy actually released at the tip. To overcome this problem, the energy spent to drive the tip is evaluated continuously during the VPT. Using a relationship between the amount of energy spent for driving the tip and the global velocity of penetration, a corrected driving velocity for a reference driving energy can be calculated, which is only depending on the density and the state of stress (Cudmani and Manthey, 2019). This is the main advantage of VPT with respect to SPT and DP. The VPT-prototype used in the field investigations is shown in Section 2. Two qualitatively different types of penetration modes, called cavitation and no-cavitation modes (Cudmani, 2001, Cudmani et al. 2000), which must be considered for the proper interpretation of the tests results, are described in section 3 and the method to evaluate vibro-penetration resistance is presented in section 4. The very loose saturated sandy dumps in the region of Lusatia in the eastern part of Germany, which are strongly susceptible to static liquefaction and must be compacted by blasting, are described in section 5. In section 6, the results of the VPT and cone-penetration test (CPT) used to check the success of the compaction are presented. The main conclusions and the perspectives for the further development of the VPT are summarized in section 7.

2. VPT-prototype

The VPT-prototype consists of a vibratory driver, a guiding frame, driving rods and an instrumented tip as illustrated schematically in Fig. 1. The driving unit consists of a vibrator, bias mass, isolation springs and a hydraulic clamp for connecting the vibrator and the rods. The vibrator generate vertical forces by means of counter rotating eccentric masses, the rotation of which is induced by a hydraulic motor connected to a 30 kW hydraulic supply. The machine parameters controlling the magnitude of the dynamical force and the vibration amplitude are the static moment of the eccentric masses with respect to its rotation axes (S_v), its rotation angular frequency (ω) and the vibrating mass (m). The centrifugal force generated by the counter rotating masses is given by:

$$F_0\,(t) \;=\; \omega^2 \cdot S_v \cdot sin(\omega t) \qquad (1)$$

In addition, the static force P_s results from the mass of the rods and the device (vibrator + variable bias mass + isolation springs). Therefore, the total vertical force is $F(t) = P_s + F_0(t)$. The vertical displacement amplitude of the vibrator in the air can be estimated with the expression:

$$\hat{u}_{vib} \approx \frac{S_v}{m_{vib}} \qquad (2)$$

Where $m_{vib} = m_{rod} + m_{vibrator}$ hanging from the isolation springs. In the VPT-Prototype, the maximum force amplitude is $\hat{F}_0$ = 80 kN and mass $m_{vibrator}$ = 150 kg. Operating frequencies and displacement amplitudes of the vibrator can be varied from 25 Hz to 100 Hz and 5.3 mm to 1.3 mm, respectively. The isolation springs were designed for a maximum bias weight of 150 kg. The latter can be varied by means of the counterweight system shown in Fig. 1. The guiding frame of the prototype with a total weight of about 10 kN is necessary to guide both the vibrator and the rods in the vertical direction (anchoring of the frame is not required!). The driving rod consists of 3.20 m long stainless steel tube segments having an outer diameter of 36 mm and a wall thickness of 20 mm. The rod segments weights about 0.2 kN. Vibration resistant screw joints prevent a loosening of the segment connections during driving.

The geometry and instrumentation of the VPT-probe are similar to those of standard CPT-probe. The penetrometer tip has a diameter of 36 mm and ends in a cone with an apex angle of 60°. The friction sleeve, the area of the penetrometer shaft upon which the local side friction resistance is measured, is located immediately above the cone and has a length of 133 mm with an area of 15042 mm². Separated load cells are used to measure the soil resistances at the tip and the friction sleeve. The local motion of the penetrometer tip is recorded by an accelerometer at the tip above the load cells. In addition, an electrical rope drum gauge attached to the bias mass measures the global penetration of the probe. The working frequency of the vibrator is determined by tracking the position of one of the eccentric masses by a magnetic gauge. After amplification, filtering and digitalization, the five electrical signals are acquired with a rate of 4,000 samples/s. More details of the device are given in Cudmani (2001) and Cudmani and Manthey (2019).

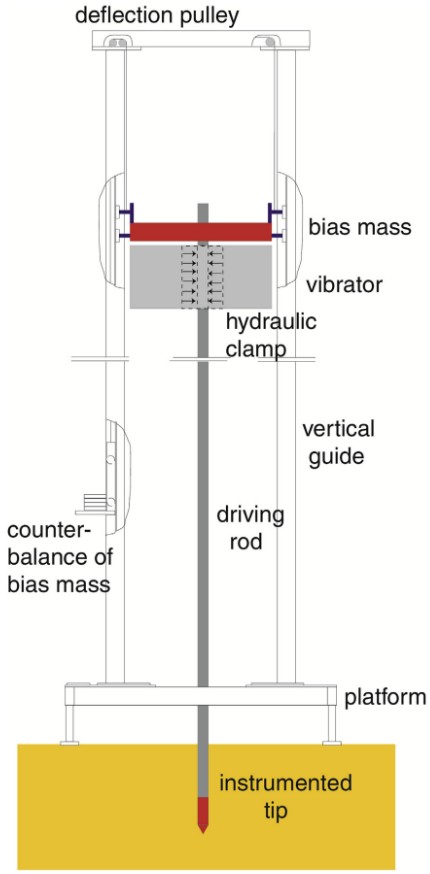

Figure 1: Layout of the VPT-prototype.

3. Evolution of the tip resistance

As shown by experiments and calculations (e.g. Cudmani, 2001; Dierssen, 1994), the rate of penetration during vibratory driving is strongly influenced by the soil resistance at the tip. Therefore, understanding the evolution of the tip resistance is a prerequisite for evaluation of the penetration resistance. Based on the analysis of full-scale vibratory driving tests with an instrumented closed-ended steel pile and the VPT carried out in a calibration chamber (Cudmani, 2001; Cudmani and Manthey, 2019), two possible evolutions of the tip resistance with the tip displacement shown schematically in Fig. 2 were identified.

In the so-called *cavitation mode* (top), beginning at the maximum force in point 1 (phase I), four well-defined phases can be identified within a loop. The pile moves upward and the tip remains in contact with the underlying soil. After a relatively small upward displacement (point 2) the contact between the soil and the pile is lost, since the pile moves faster at this point than the soil.

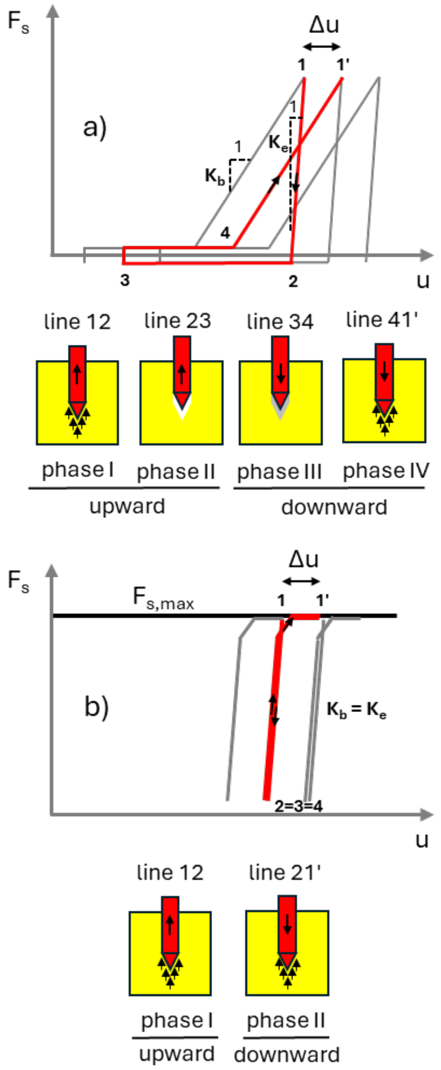

Figure 2: Idealization of mechanical soil response at the pile tip: a) cavitation b) no-cavitation mode of penetration.

The pile and the underlying soil uncouple and a cavity forms underneath the tip (phase II). From point 3 (motion reversal) the pile tip moves downward without contact with the soil until point 4 (phase III), which does not coincide with point 2. Between point 2 and 4 there is no contact between tip and the soil and therefore, the force-displacement curve does not give any information about the soil response. After the contact is restored, the soil resistance is mobilized (phase IV). At the end of a loop (point 1') the pile has been driven the amount Δu.

The force-displacement curve for the cavitation mode is governed by three quantities:

- the inclination K_b of the downward displacement line 4-1'
- the inclination $K_e >> K_b$ of the upward displacement line 1-2
- the penetration per loop Δu

For computing purposes, it is useful to express the penetration Δu as a function of the plastic deformation ratio β_p, which is defined as:

$$\beta_p = \frac{\Delta u}{u_4 - u_{1'}} \quad (3)$$

where u_4 and $u_{1'}$ are the displacements at the end of phases III and IV (Fig. 2a). The value $\beta_p = 0$ corresponds to quasi-elastic and $\beta_p = 1$ to fully plastic response.

The no-*cavitation mode* is shown in the bottom of Fig. 2. As can be seen, the contact between the pile tip and the underlying soil is not lost during the upward motion phase (phase I) and the soil resistance is mobilized immediately after the motion reversal (points 2, 3 and 4 coincide). During the downward motion phase, the stiffness is almost the same as during unloading until the maximum tip resistance is achieved. Thereafter, the downward motion continues at constant tip resistance until the next reversal of motion at point 1' takes place. The no-cavitation mode is characterized by the following quantities:

- the inclination $K_b \approx K_e$ of the downward and upward displacement lines
- the limit resistance $F_{s,max}$
- the penetration per loop Δu

As shown by Cudmani (2001) and Cudmani and Manthey (2019), the occurrence of the cavitation and no-cavitation mode of penetration depends on the parameter setting and the initial state of the soil. Exemplarily, Fig. 3 shows the occurrence of cavitation and no cavitation for different initial relative densities I_d and mean effective pressures p_0 at the same machine parameters.

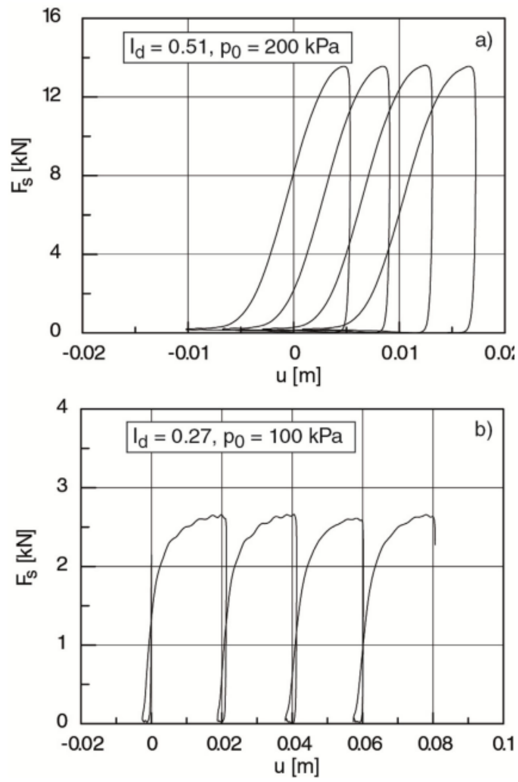

Figure 3: VPT in the laboratory: a) cavitation b) no-cavitation mode of penetration.

However, the occurrence of the no-cavitation mode requires the achievement of the maximum tip resistance in penetration loop, which for the used equipment is only possible at shallow depths and loose state of the soil. Therefore, the most common penetration mode for vibratory driving is the cavitation mode.

Cudmani (2000) provided a soil mechanical justification for the occurrence of cavitation and no-cavitation modes. In the cavitation mode, the stretching of the soil during the upward motion leads the soil near the pile tip to the critical state, and there the memory of the material on the previous deformation history is almost completely swept out. Therefore, only the actual density and the state of stresses, which do not strongly change from one loop to the next, governs the soil response after the reversal of motion. The stiffness during reloading is smaller than during unloading. On the contrary, in the no-cavitation mode the soil is not completely unloaded during the upward motion phase and the shear deformation is not large enough to erase the material memory. In this case, the soil shows a typical "unloading-reloading" behavior.

4. Evaluation of penetration resistance

In analogy to dynamic penetration tests, the penetration resistance in the VPT is defined as the number of cycles N^*_{z10} of the excitation force which are necessary to drive the probe $\Delta z = 0.10\ m$ into the soil. Considering that $\Delta u = v_{glob}/f$, with v_{glob} according to Fig. 4 and f the driving frequency, N^*_{z10} is given by

$$N^*_{z10} = \frac{\Delta z}{\Delta u} = \frac{0.10 \cdot f}{v_{glob}} \qquad \textbf{(4)}$$

Nevertheless, N^*_{z10} is not an objective quantity since v_{glob} is influenced not only by the state of the soil, but also by the setting of the machine parameters, the length of the rod, the shaft friction, and the energy losses in the guide-vibrator-rod system. This is demonstrated in Fig. 5 which shown exemplarily, the evolution of the tip resistance and the velocity of the tip ($\dot{u}$) as a function of the tip displacement for different static loads (top) and different initial densities (bottom). As can be seen, the penetration velocity is not only affected by the density, but also by the change of the static load. Therefore, for an objective evaluation of the VP-resistance the energy actually released at the tip must be considered.

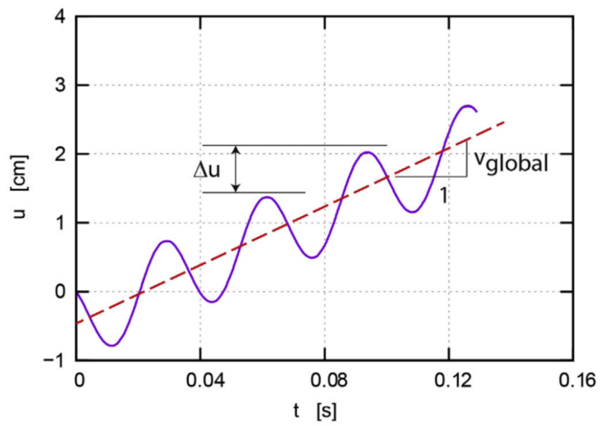

Figure 4: global and local velocity during vibratory driving

For the SPT and DP, it was found that the blow count N^* ($N^* \sim 1/\Delta u$) is inversely proportional to the energy released at the tip, i.e.

$$N^* \cdot W_{blow} = const. \qquad \textbf{(5)}$$

Where $W_{blow} = \eta \cdot G \cdot h$ (G: weight of the falling mass, h: falling height and η: impact efficiency factor). This relationship indicates that impact driving is a no-cavitation penetration process.

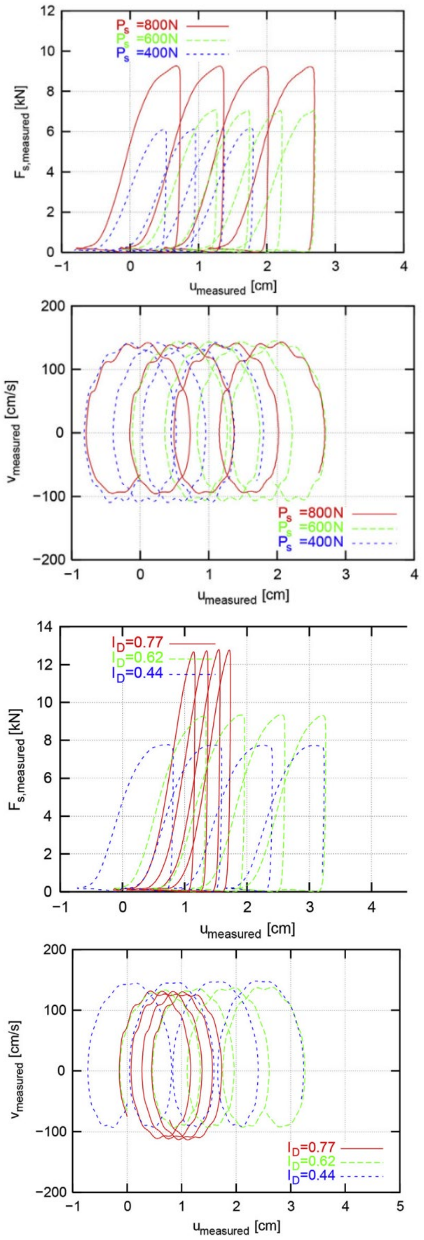

Figure 5: Typical evolution of the tip resistance and the velocity ($\dot{u}$) with the tip displacement for different static loads and the same initial density (top); different initial densities and the same machine parameters (bottom).

In fact, as shown in Fig. 6, a change of energy spent at the tip per cycle W_z, which is determined by the area enclosed in one force-displacement loop, induces a proportional change of Δu ($\Delta u \sim N^*_{z10}$).

In a similar manner, a relationship between N^*_{z10} and the mechanical work per cycle W_z for the cavitation mode can be found using the mechanical model shown in Fig.

2. Assuming that for a given tip geometry and soil properties the quantities K_b, K_e and β_p depend only on the soil state, the relationship between N^*_{z10} and W_z for the cavitation mode is:

$$N^{*}_{z10}{}^{2} \cdot \left(-\beta_p{}^2 + 2\beta_p\right) \cdot W_z = const. \qquad (6)$$

The derivation of this equation is shown in Cudmani and Manthey (2019). The N_{z10} for a reference energy E_r can be determined as a function of the values of N^*_{z10}, W_z and β_p determined in the test:

$$N_{z10} = N^{*}_{z10} \cdot \sqrt{\frac{(-\beta_p{}^2+2\beta_p)\cdot W_z}{E_r}} \qquad (7)$$

The maximum kinetic energy of the vibrator in the air, $E_r = ½\ m_{vibrator}\ (\hat{v}_{vib})^2$ for a frequency f = 25 Hz, a $m_{vibrator}$ = 150 kg, a static moment S_v = 0.81 kg*m was adopted as reference energy for the evaluation of VPT.

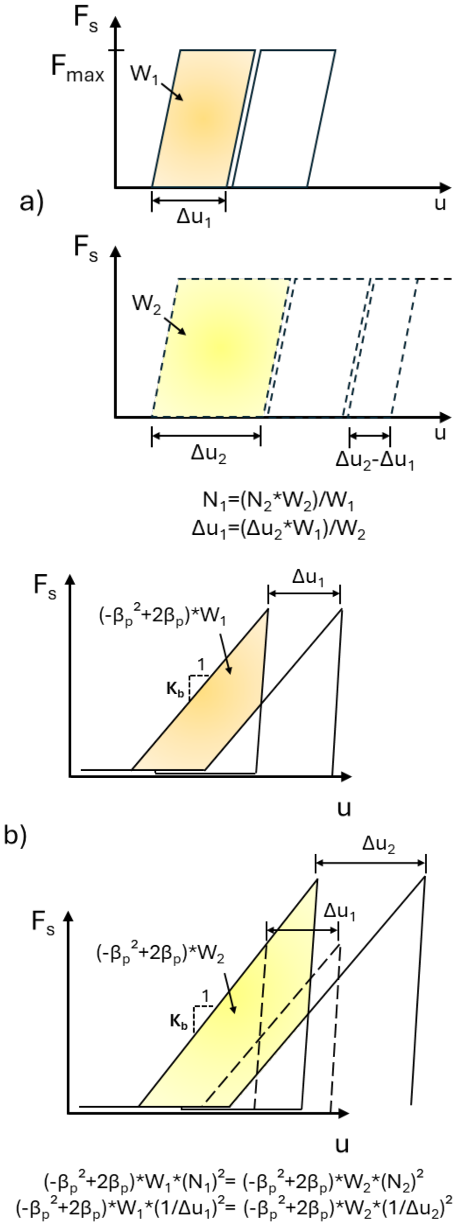

Figure 6: Relationship between $N^*_{z10}(1) = N_1$, $W_z(1) = W_1$ and $N^*_{z10}(2) = N_2$, $W_z(2) = W_2$ for the no-cavitation mode (a) and the cavitation mode (b).

Assuming harmonic motion of the vibrator, the amplitude of the velocity can be determined from equation (5) $\hat{v}_{vib} = 2\pi f\ \hat{u}_{vib} = 2\pi f\ \frac{S_v}{m_{vibrator}}$. The resulting reference energy is E_r = 54 kN.

Cudmani and Manthey (2019) investigated the dependence of K_b, K_e and β_p on the machine parameters experimentally in a calibration chamber and by means of numerical simulations with the Finite Element Method and a hypoplasticity model. It was found that assuming the cavitation mode of penetration, the influence of the frequency and the static load on penetration rate can be satisfactorily neutralized by using the normalized penetration resistance N_{z10}. On the contrary, the influence of the static moments on the penetration rate cannot be completely eliminated by eq. 7. Therefore, in our investigations a constant value S_v = 0.81 kg*m was used. In future applications of the VPT, this value shall be fixed for comparison purposes.

For the calculation of W_z, four periods of the tip force $F_s(t)$ and the tip velocity $v_s(t)$ were considered in each evaluation depth. The velocity of the tip was obtained from the sum of the global driving velocity and the local velocity which results from the integration of the tip acceleration $a(t)$. The mechanical work W_z results from the integration of the current power $P(t)$ from t_0 to $t_0 + 4T$ ($T = 1/f$):

$$W_z = \frac{1}{4}\int_{t_0}^{t_0+4T} P(t)\ dt = \frac{1}{4}\int_{t_0}^{t_0+4T} F_s(t)\cdot v_s(t)\ dt \qquad (8)$$

Theoretically, equation (7) is valid for any driving velocity provided that $v_{glob} > 0$ (if the probe sticks N^*_{z10} goes to infinity and the product $\beta_p \cdot W_z$ approachs zero). Practically, there is a minimal driving velocity, which depends on the test equipment, below which the normalization loses reliability, similar to any. For the VPT-prototype, the minimal reliable driving velocity is 0.0005 m/s. Penetration resistances below this value shall be interpreted as refusal. Equations (6) and (7) apply only to the cavitation mode, which is the most common mode of penetration, as mentioned before. Although the evaluation of VPT in connection with the no-cavitation mode is possible using equation (5), vibro-penetration resistances obtained with different penetration modes are not convertible. To enforce the occurrence of the cavitation mode in very loose soils, adjustment of the static load, alternatively of the static moment is recommended.

5. Investigation of the blasting compaction in sandy dumps

5.1. Introduction

The Lautasia region in the eastern part of Germany was a major lignite reservoir. The lignite deposits with thicknesses of decametres lie at depths varying from about 50 m to several hundred meters below the former ground surface. The soil covering the deposits consists mainly of fine and medium, rounded uniform quartz sands. The lignite was mined in open pit mines, the main steps being:

1. Groundwater lowering up to the bottom of the lignite deposit

2. Excavation up to the deposit and mining of the lignite
3. Refilling of the mine with formerly overlying sediments
4. Restoration of the natural ground water level

Because of the dumping procedure, the soil properties and the rising of the ground water table, an extremely loose nearly saturated sand fill resulted, which is very susceptible to spontaneous liquefaction. Owing to the very large remediation volume, blasting compaction is used to stabilize the dumps. The initial density as well as the changes due to compaction are the key quantity to assess the liquefaction susceptibility before and after blasting. The density and degree of saturation of the soil can be measured from undisturbed soil specimens sampled by soil freezing. Nevertheless, this procedure is expensive, time consuming and its practical execution is difficult. On the other hand, field testing offer a more economical alternative to investigate the initial state of the soil and the improvement induced by compaction. In Lausatia, CPT are usually used to assess the success of blasting compaction. However, there is a necessity to develop alternative field testing methods with lighter equipment and comparable quality, allowing the access to zones of the dumps, which cannot be accessed with the heavy CPT vehicles.

5.2. Test field for the investigation of blasting compaction

The test site was located in Kleinkoschen near Senftenberg, state of Brandenburg. The dump belongs to the former Sedlitz-Skado-Koschen open pit mine complex (Fig. 7). A typical soil profile together with results of a CPT are shown in Fig. 8. It consists of three layers with a thickness of about 42 m. The two top layers consist mainly on very loosely deposited fine and medium quartz sands with q_c varying between 2.0 to 3.0 MPa. The third layer has some content of fine cohesive material and shows a slow increase of q_c with depth.

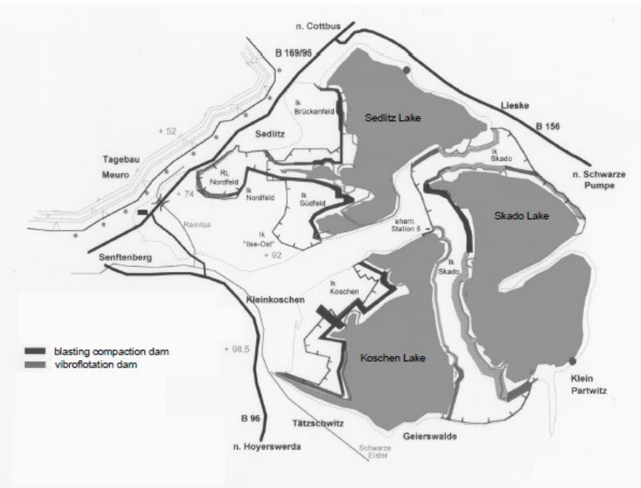

Figure 7: Sedlitz - Skado - Koschen remediation complex.

The three layers are separated by working planes, which were compacted during the refilling works by the heavy machinery required to dump the fill. The ground water table is in 5 m depth.

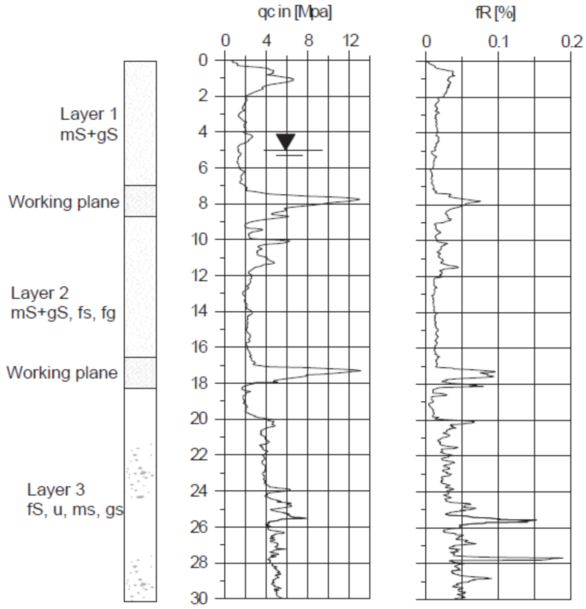

Figure 8: Typical soil profile and CPT results at the test field.

The investigation of blasting compaction was carried out in two stages. In the first stage, the compaction effect of single charges was investigated. In the second stage, the compaction caused by group charges was investigated. A detailed description of the blasting compaction program, the test field layout including the location of the blasting charges and of the field tests can be found in (Cudmani and Huber, 2000).

The field investigations included different field testing techniques. CPT and VPT were carried out before compaction, after single charge blasting compaction and after group charge compaction. In addition, Cross-hole and CPMT were performed before and after single charge compaction. Cudmani and Huber (2000) described the evaluation and interpretation of the CPT and CPMT results. Only the VPT results are analysed in this contribution.

6. VPT results

Fig. 9 shows a typical result of a CPT and a typical result of an adjacent VPT before blasting. The soil profile shown in Fig. 6 can be clearly identified.

The penetration resistance in the loose sand layers show values of N_{z10} (loose)≈2 ↔ q_c (loose)≈2 kN/m²), while in the work levels with medium to dense relative density N_{z10} (dense)≈18 ↔ q_c (dense)≈14 kN/m².

The increase of the penetration resistance from loose to dense is more pronounced for the VPT than for the CPT. A reduction of the VP-resistance from N_{z10} ≈5 above to N_{z10} ≈2 below the ground water is observed, which is not occurring in the CPT. This indicate that excess porewater pressure develops below the ground water table in the VPT. Nevertheless, a total decay of the resistance force at the tip, which would occur if effective stresses of the sand vanishes due to liquefaction, was not observed during the tests.

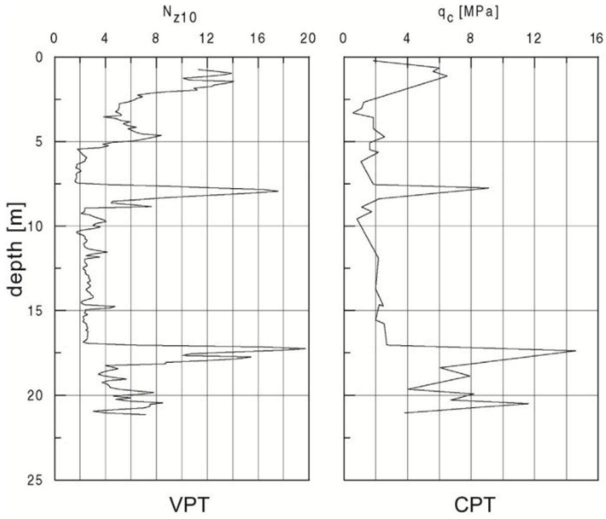

Figure 9: Comparison of VPT and CPT results for the inital state of the dump.

In addition, it is observed that the cone penetration and the vibro-penetration resistance increase below the deeper work level, in the third layer with a higher fine content.

Fig. 10, which shows the results of three VPTs performed side by side, demonstrates the good reproducibility of the VPT.

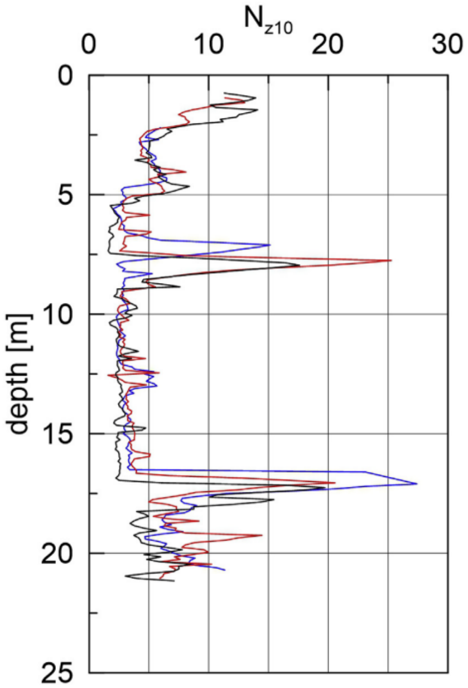

Figure 10: Results of three VPT at close locations before blasting showing repeatability of the VP-resistance.

The cone resistance and the vibro-penetration resistance over depth before and after blasting compaction are compared in Fig. 11. The CPT and VPT results on the left and on the right correspond respectively to a distance of 5 m and 8.5 m from a single blasting point B (see Cudmani and Huber, 2000 for more details about the location of the field test and the blasting point B).

As can be seen, both, the CPT and the VPT, show similar changes of the penetration resistance due to blasting compaction. At the distance of 5 m, an influence of the single and the group blasting is observed (diagrams on the left), while at the distance of 8.5, the compaction induced by the group blasting was significantly weaker than single blasting.

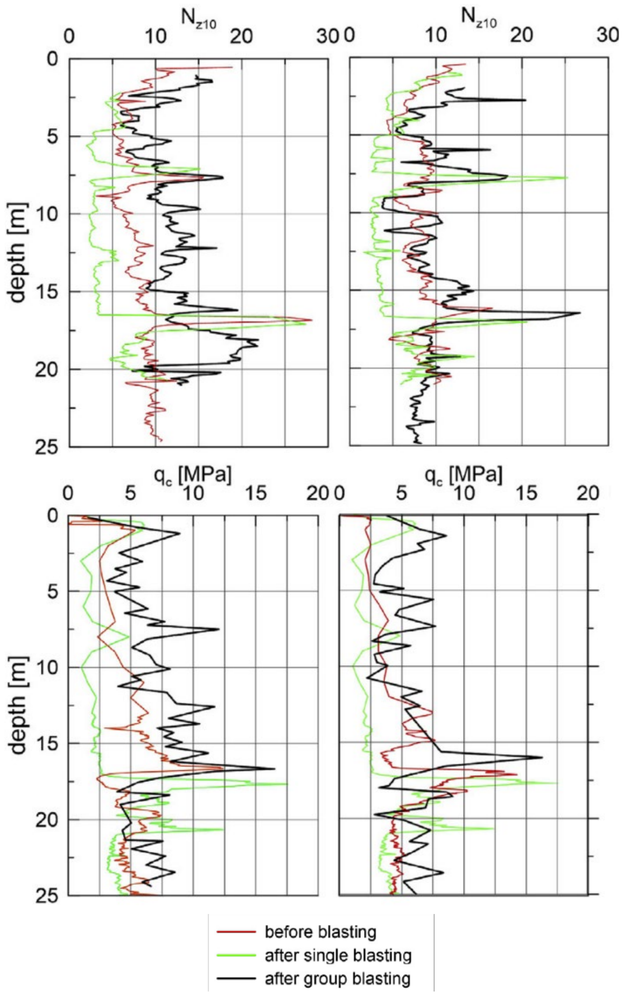

Figure 11: Results of VPTs and CPTs 5.0m (left) and 8.5m (rigth) from blasting point B before and after compaction.

Therefore, the VPT is able to capture the changes of the density with the same accuracy as the CPT.

7. Conclusions and outlook

A novel dynamic penetration testing method for the assessment of the in-situ state of granular soils based on the principles of vibratory driving has been described. The tip resistance governing the penetration progress is characterized by two different penetration modes, whose prevalence depends on the setting of the parameters and the soil state. The cavitation mode takes place when the contact between the tip and the soil is shortly lost during the upward phase of the tip motion. In contrast, in the no-cavitation mode the tip and the soil does not loose contact during driving. According to experimental and numerical results, the cavitation mode is the most common penetration mode for vibratory driving and therefore, the evaluation of the penetration resistance in the VPT assumes the occurrence of this penetration mode. This assumption is automatically controlled during the test evaluation.

Although it is used in the SPT and the DP, the inverse of the penetration rate is not an objective measure of the penetration resistance neither in these tests nor in the VPT. Changes of the machine parameters, friction resistance at the rods and energy losses in the system, induce changes of the penetration rate not related to variations of the soil state. To overcome this problem, an energy-normalized penetration resistance N_{z10} is used in the VPT. The quantities required for this normalization are evaluated at every penetration depth from the measurements of the tip resistance, the tip acceleration and the global penetration speed during driving.

Numerical and experimental investigations of the vibro-penetration process showed that although the proposed mechanical model and energy-correction can account for the effect of a change of machine parameters up to some extent, the dependence of N_{z10} on these changes cannot be fully eliminated. Therefore, to ensure reproducibility and avoid ambiguity of the test results, variations of the machine parameters in VPT should be kept as small as possible. In the field tests, the frequency was set to f =30 Hz and S_v = 0.81 kg m.

In the field, the VPT-results showed a good reproducibility and correlated well with the CPT-results. Advantages of the VPT over the CPT are the shorter duration, the considerably smaller static load required for penetration (about 20 times smaller) as well as the suitability for medium dense to dense cohesionless soils at larger depths. Additionally, the VPT provides the force-displacement response at the tip, which can be determined for every penetration loop and correlates with the soil stiffness, which is also a state-dependent soil property. However, the alternating loading of the soil during vibratory driving induces an accumulation of excess porewater pressures. Therefore, the penetration resistance above and below the ground water table will be in general different above and below the ground water table. This feature of VPT can be advantageous to investigate the static and dynamic liquefaction susceptibility liquefaction of soils depending on the density, the state of stress and the fine content.

VPTs differ from blow-based penetration tests in the mode of penetration since in these tests the probe penetrates in the no-cavitation mode, as the experimental results of SPT and DPT in the literature demonstrated. Naturally, the accuracy of the energy-corrected penetration-resistance from VPT is higher than the uncorrected blow counts from SPT and DPT, especially at larger depths in which the energy reaching the tip, e.g. due to the increasing mass and friction of the rod, can change considerably,

Since vibro-penetration is closely related to the soil behavior under alternating and cyclic shearing, the author believes that VPT should be more appropriated than monotonic tests to characterize the ground response to repeated dynamic and alternating loading, e.g. pile drivability, machine foundations, ground compaction and earthquake response. Further research will focus on developing and improving the VPT-device, e.g. implementing a pore pressure transducer at the tip, developing of empirical and semi-analytical correlations for estimating the relative density from VPT results and applying technique to ground response characterization for problems involving alternating and dynamic loading. Application of the VPT to sensitive low plasticity soils and tailings is also envisaged.

Acknowledgement

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