Comparative analysis of DMT, CPT and DPH for soil characterization of granular Rhine soil

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ABSTRACT

This paper presents the results of direct and indirect field investigations carried out as part of a detailed soil investigation of granular Rhine soil in Germany. After a brief overview of the project and geological conditions in the project area, the results of the field investigations are shown in a compressed and comprehensive manner. Focus of the presented results are the DMT, which are a novelty for projects in Germany. The presented results clearly indicate the challenges regarding the interpretation of the test results in the mainly dense to very dense sands and gravels, as especially the results of the DMT tend to scatter a lot. Despite the interpretation challenges, a comparative analysis of the CPT and DMT is carried out showing possible relations between dilatometer modulus E_d, friction angle φ_{DMT}, corrected cone resistance q_t and relative density I_D. The relations are discussed, and limitations are presented. Afterwards, a strength-based correlation between CPT and DMT to determine the effective friction angle as a function of the cone resistance is presented indicating reasonable results for the investigated soils in the project area. The paper finishes with a discussion of limitations of the DMT and conclusions.

Keywords: CPT; DMT; gravel; empirical correlation.

1. Introduction and Geological conditions

The investigation is located in the area of the *Upper Rhine Graben* (ORG; German: "Oberrheingraben"), which is located between the Black Forest in the south-east, the Odenwald in the north-east and the Palatinate Forest in the west. The general subsoil conditions are predominantly characterized by floodplain sediments followed by quaternary sands and gravels with varying relative densities down to depths of around 25 m below ground level.

A long section through a 3D subsoil model from the project area is shown in Fig. 1. The subsoil, generally speaking, is very homogeneous. It mainly consists of fluvial sediments (silts, silty sands, sandy silts) followed by dense to very dense sands and gravels. Partially loose and medium dense sand and gravel layers and lenses are present. With increasing depth, clay and silt interlayers occur, which are more pronounced on the east side of the project area (cf. range between 600 m to over 1000 m in Fig. 1). To investigate the subsoil conditions in detail, several field investigations in the form of deep drillings incl. SPT, DPH tests, CPT and DMT were performed. These tests were basis for the 3D subsoil model (cf. Fig. 1) and are described and analyzed in detail in the following sections.

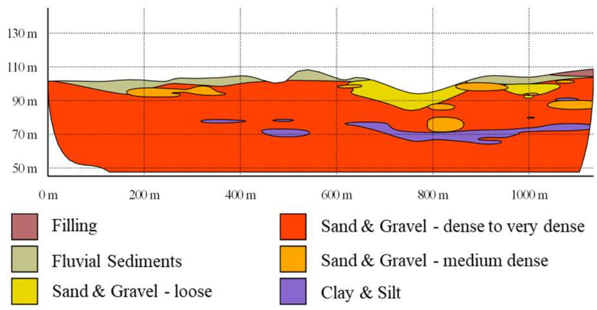


Figure 1. Long section through the subsoil in the project area

2. Field investigations

2.1. Deep Drillings

To investigate the soil in the project area, 29 boreholes were drilled on both sides of the Rhine with maximum depths of 40 m. The drilling method had two stages. A dry percussion drilling or down-the-hole (DTH) technique was used to get samples of the soil. After sampling the soil to a certain depth, a casing was put in place with rotary drilling to prevent the borehole walls from collapsing.

The percussion drilling equipment includes a pneumatic hammer, a driving assembly and a sampling tube. The hammering action is created by the drilling rig, which applies a series of impacts to the driving assembly. This impact force is transferred to the sampling tube. As the sampling tube drives further into the ground, it collects the soil samples in its chamber. The samples are usually kept inside the tube during the drilling process.

Percussion drilling offers versatility, cost-effectiveness, rapid progress, sample recovery, and minimal site disturbance. Disadvantages of percussion drilling include limitations in depth control, noise and vibrations, and suitability for certain formations. Percussion drilling, due to the dynamic impacts of the hammering device, causes more disturbance to the soil and makes it challenging to obtain high-quality undisturbed samples, making it a major disadvantage of this method (Patel 2019).

The geotechnical boreholes revealed the following lithology:

- Layer 0 – Topsoil
- Layer 1 – Fillings
- Layer 2 – Flood sediments characterized by irregular alternations of cohesive and non-cohesive soils with a mean thickness of approx. 3.0 m.
- Layer 3 – Fluvial sands and gravels, which could be divided based on the SPT, CPT, DPH, DMT into

a) Loose (mainly) sands and (partially) gravels with a maximum thickness of approx. 12 m

b) Medium dense sands and gravels with a maximum thickness of approx. 10 m

c) Dense to very dense sands and gravels which were encountered down to the maximum investigation depth

- Layer 4 – Clay and silt interlayers with a maximum thickness of approx. 6.5 m

Due to their small thickness and irregular occurrence, Layer 0 and Layer 1 are not relevant for the scope of this paper and will be neglected in the following sections.

2.2. Indirect methods

2.2.1. CPT

In order to fulfill the project objectives, a total of 33 cone penetration tests, which included pore water pressure measurements (CPTu), were conducted in accordance with the DIN EN ISO 22476-1 standard, reaching a maximum depth of approx. 21 m below ground level and were stopped as soon as the maximum capacity of the CPT rig of 100 kN was reached.

The CPTu were carried out due to their advantages including, but not limited to, efficient and user independent data collection, continuous profiling of soil properties, versatility in different soil types, and cost-effectiveness. All CPT could be carried out within 2 days. Though, it should be noted, that the limitations of the CPTu include interpretation challenges, potential soil disturbance, depth limitations, and equipment constraints.

In Fig. 2 the corrected cone resistance q_t (cf. Eq. (1) with $\alpha = 0.85$) according to (Robertson 2016) of all CPT are shown as well as a mean value with respect to their absolute depth. Furthermore, a typical soil profile obtained from one borehole (cf. section 2.1) is depicted as well for reference. The mean q_t line shows that the cone resistance is generally increasing with depth. While in the flood sediments of Layer 2 values of $q_t \approx 3$ MN/m^2 were measured, the cone resistance is immediately increasing as soon as Layer 3 is reached. The mean q_t line shows values between 10 MN/m^2 to 50 MN/m^2 already indicating the high density and resistance this soil provides. However, there is a strong scattering of the measured cone resistance visible (5 MN/m$^2 \leq q_t \leq$ 80 MN/m^2), indicating not just loose and very dense layers but also very gravelly (partially maybe stony) areas.

$$q_t = q_c + (1 - \alpha) \cdot u_2 \qquad (1)$$

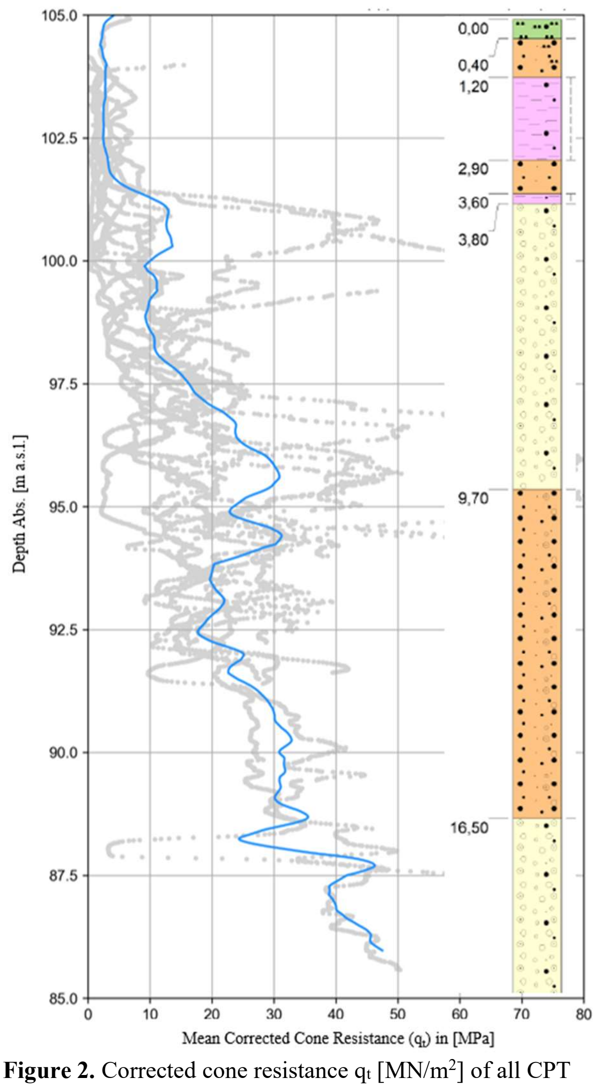


Figure 2. Corrected cone resistance q_t [MN/m^2] of all CPT (grey) incl. a mean over all tests (blue line) with respect to their absolute depth [m a.s.l.]

2.2.2. DPH

During the investigation campaign 11 Dynamic Probing Heavy tests (DPH) according to DIN EN ISO 22476-2 down to a maximum depth of approx. 15 m below the ground level were performed. The DPH were conducted using a standard DPH-crawler, and the counts were manually recorded by the operator.

Advantages of the DPH method include its versatility in various soil conditions, rapid progress, and the ability to provide insights into soil stratification. Limitations of DPH include limited depth accuracy (inclination cannot be measured), soil disturbance, and the reliance on manual counting by the operator. Measuring only the total count of blows per 10 cm in dynamic probing may lead to overestimating soil parameters at greater depths, as it reflects the total energy along the rods rather than specifically on the tip.

Fig. 3 shows the results of one DPH carried out in the project area. In Layer 2 the blow count ranges from $N_{10} = 1$ to 5 in the cohesive soils, $N_{10} \approx 10$ in the non-cohesive soils respectively. As soon as the DPH reaches Layer 3, the blow count increases to $N_{10} \geq 5$. The DPH within layer 3a and 3b show average blow counts predominantly ranging from $N_{10} \approx 5$ to 20, with a general increase observed with depth. Occasionally, localized higher blow counts were recorded, which can be attributed to embedded stones. Within layer 3c, consistent blow counts exceeding $N_{10} > 30$ were obtained, with a general increase observed with depth. In opposition to the measured cone resistance of the CPT, the blow count of the DPH is only increasing as soon as the cone penetrates into the dense to very dense sandy gravels, demonstrating one of the disadvantages of the DPH measuring only one absolute resistance.

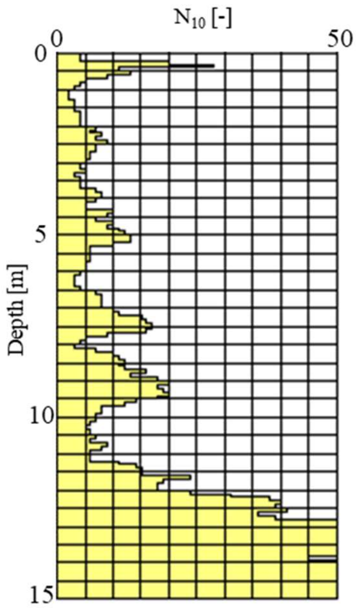


Figure 3. Results of one DPH carried out in the project area

2.2.3. DMT

A total of 15 Dilatometer Marchetti tests (DMT) were performed following the DIN EN ISO 22476-11:2017-08 standard, with a maximum depth of approx. 22 m below ground level.

Advantages of the DMT include its ability to provide in-situ testing of soil properties, especially soil stiffness, efficiency and user independent data collection, versatility across various soil types, and the provision of continuous data along the depth of the test and cost-effectiveness. All DMT were carried out within 2 days. Furthermore, according to (Schmertmann 1984), less soil disturbance occurs during blade insertion in comparison to CPT. Limitations of the DMT include its limited depth range compared to other methods such as Menard-Pressuremeter or CPT. Additionally, inexperienced users may encounter issues with equipment preparation, leading to inaccurate data acquisition. Furthermore, the interpretation of the results may be challenging for unexperienced users. Also, there are geometric constraints that may restrict the use of DMT in certain soil types or formations, mostly in gravels and medium to hard rocks.

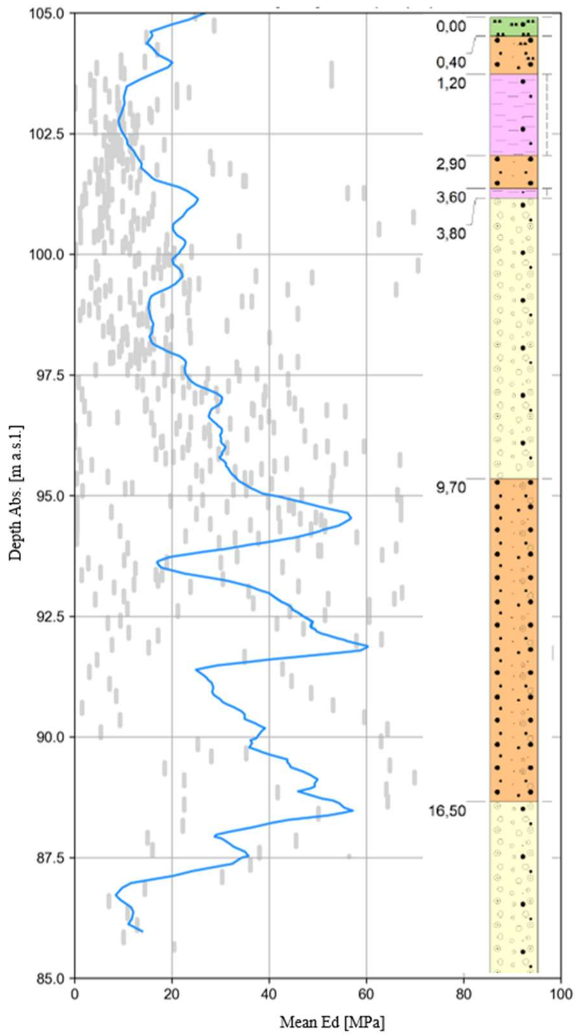


Figure 4. Dilatometer modulus E_d [MN/m^2] of all DMT (grey) incl. a mean over all tests (blue line) with respect to their absolute depth [m a.s.l.]

The results of the conducted DMTs are summarized in Fig. 4, showing the dilatometer modulus E_d of all conducted DMT as well as a mean line over all tests with respect to their absolute depth. In the flood sediments, the dilatometer modulus is homogeneously in a range of $E_d \approx 10 - 20$ MN/m^2 with relatively low scattering. As soon as the DMT reaches Layer 3 containing sands and gravels, the DMT results scatter a lot showing dilatometer moduli between $E_d \approx 1$ MN/m^2 and $E_d \approx 100$ MN/m^2. Based on the observations in the field and in comparison to the determined sieving curves, the DMT results in the sandy areas are more plausible than in the gravelly areas. The limitations can be attributed to the inherent geometry of the DMT equipment. Due to the small area of the membrane with 6 cm in diameter, DMT measurements are not reliable for medium to coarse gravel. During the field investigations, the membrane got even damaged several times due to the sharp contact with large, high-strength gravel particles.

Further results of the DMT, not shown in Fig. 4, indicating an average vertical constrained modulus of $M_{DMT} \approx 20$ MN/m^2 with a mean friction angle of $\varphi_{DMT} \approx 35°$ for the non-cohesive and $M_{DMT} \approx 4$ MN/m^2 with a mean undrained shear strength of $s_{u,DMT} \approx 12$ kN/m^2 for the cohesive flood sediments. Based on the authors experience with similar soils, the determined values based on the DMT are plausible.

In the sandy areas of Layer 3, mean constrained moduli of $M_{DMT} \approx 20$ MN/m^2 (loose), $M_{DMT} \approx 45$ MN/m^2 (medium dense) to $M_{DMT} \approx 130$ MN/m^2 (dense to very dense) were obtained with a mean $\varphi_{DMT} \approx 30°$ for the loose and $\varphi_{DMT} \approx 35°$ for the medium dense to very dense sands. For the loose sand the determined values are similar to the experience of the authors with comparable soils. However, for the medium dense to very dense sands, the friction angle is slightly lower, and the constrained modulus is significantly higher compared to the authors experience. Thus, neglecting the influence of the gravel content on the measured values, the DMT results would lead to a slightly lower soil strength but a higher soil stiffness applicable for the foundation design. As foundation design may increasingly be based on the serviceability load situation, i.e. settlements, the DMT enable an optimized design of the foundation with in-situ measured stiffness moduli compared to moduli determined based "on experience".

3. Comparative analysis

3.1. General

Despite the limitations of the different field investigations described in section 2, a comparative analysis between the CPT and DMT was carried out. In order to partially overcome these limitations, the most unplausible data was filtered out before the analysis. Thereof the following values were excluded:

- $q_t < 7.5$ MN/m^2 in combination with $\varphi_{DMT} > 35°$
- $q_t < 15$ MN/m^2 in combination with $\varphi_{DMT} < 30°$
- $q_t > 35$ MN/m^2

After the filtering process, the following combinations were compared and correlations by means of a linear or logarithmic regression function were determined through the method of least squares:

- Corrected cone resistance q_t [MN/m^2] and dilatometer modulus of the DMT E_d [MN/m^2]
- Corrected cone resistance q_t [MN/m^2] and friction angle determined by the DMT φ [Deg]
- Relative density determined with the CPT I_D [-] and dilatometer modulus of the DMT E_d [MN/m^2]

The relative density I_D was determined using an empirical relationship according to Eq. (2a) of (Jamiolkowski et al. 2003), documented in (Schneider et al. 2008) applying a $p_{ref} = 100$ kN/m^2:

$$I_D = 0.35 \cdot \ln(q_{c1n}/20) \tag{2a}$$

with

$$q_{c1n} = (q_t/\,p_{ref})/\left(\sigma'_{v0}/p_{ref}\right)^{0.5} \tag{2b}$$

3.2. Relation between q_t and E_d

In Fig. 5, the relationship between the corrected cone resistance q_t and the dilatometer modulus E_d is shown with respect to the encountered soil layers. In general, a linear relationship between q_t and E_d (cf. Eq. (3)) can be observed. This relationship is less pronounced for the dense to very dense sands and gravels, because especially in this layer the gravel content is high. As already mentioned in section 2, the measurements of the DMT tend to show an increased scattering with increasing gravel content as the DMT is operating beyond its designated boundaries. In the flood sediments and the loose to medium dense sands and gravels however, there is a satisfactory match between the linear trendline according to Eq. (3) and the measured values. With a correlation coefficient of $R^2 = 0.63$, Eq. (3) is considered satisfactory for practical engineering use (Poenaru, 2023).

$$E_d = 2.15 \cdot q_t + 1.42 \tag{3}$$

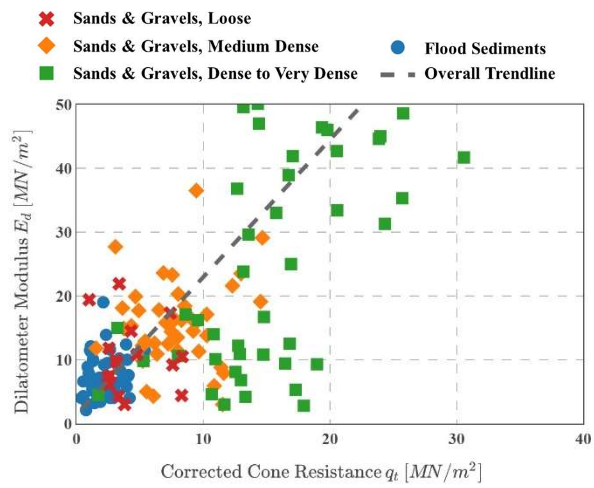
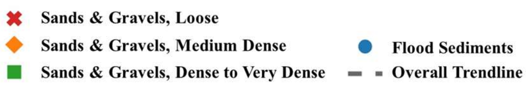


Figure 5. Relation between q_t and E_d

3.3. Relation between q_t and φ_{DMT}

Fig. 6 shows the relation between the corrected cone resistance q_t and the friction angle determined by the DMT φ_{DMT}. A general, trivial trend is apparent, i.e. with an increase of cone resistance, the friction angle is increasing as well, demonstrating at least the plausibility of the data. Though, the overall picture is dominated by the scatter in the data. Applying a logarithmic regression function to the data leads to the trendline curve depicted in Fig. 6, which is, according to the authors opinion, not satisfactory and will underestimate the real strength of the gravelly soil as outlined in section 2.2.3. The authors' opinion is further supported by the low regression coefficient of $R^2 = 0.16$.



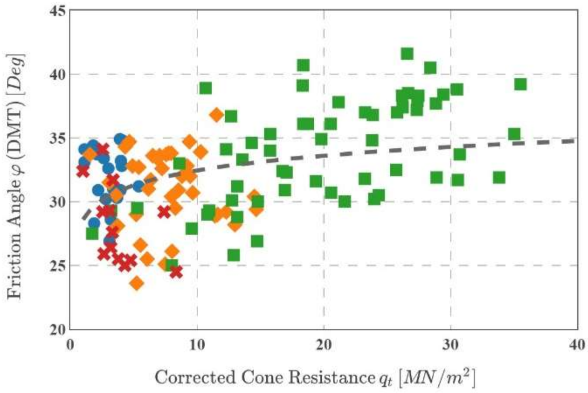


Figure 6. Relation between q_t and φ_{DMT}

3.4. Relation between E_d and I_D

The relation between the dilatometer modulus E_d and the relative density is shown in Fig. 7. A logarithmic relation according to Eq. (4) between both values can be seen. Within $I_D \leq 0.6$, the measured values are close to the overall trendline. Though for $I_D > 0.6$ the data points and the trendline are partially highly deviating. Despite the deviations and the low correlation coefficient ($R^2 = 0.36$), the relation between I_D and E_d with the logarithmic function of Eq. (4) is considered satisfactory for engineering use. In fact, Fig. 7 is showing that in dense to very dense conditions the dilatometer modulus and thus the stiffness of the soil can increase a lot compared to soils in a loose or medium dense state.

$$\log E_d = 0.84 \cdot I_D + 0.78 \quad (4)$$

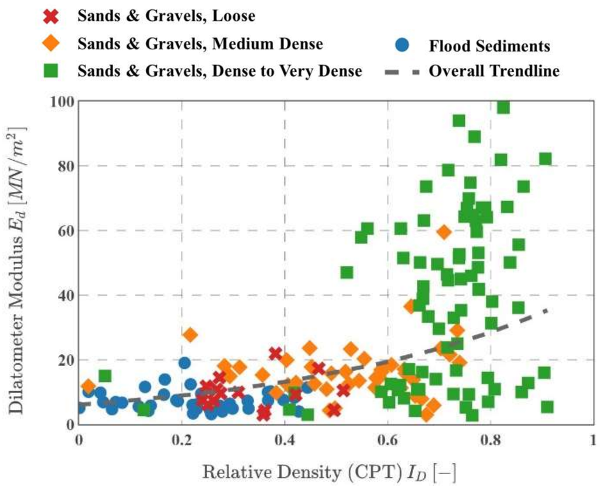


Figure 7. Relation between I_D and E_d

As, at least in Germany, soil stiffness often is determined by experience, it might often be underestimated compared to the in-situ stiffness. Though, in order to satisfy the high demands on the serviceability limit sate, sophisticated settlement calculations with 3D finite element models are becoming more and more important and making it necessary to determine in-situ soil stiffness as acutely as possible.

4. Strength-based correlation between CPT and DMT

The comparative analysis in section 3 has shown possible relationships between the CPT and DMT. Following, a strength-based correlation between these two tests is proposed. As section 3 has shown a high influence of the DMT results on the gravel content, only DMT values measured in the sandy areas of soil layer 3 were considered to determine the relationship between the friction angle φ_{DMT} and the corrected cone resistance q_t according to Eq. (5). Considering only the sandy areas of layer 3, Eq. (5) provides with a correlation coefficient of $R^2 = 0.60$ satisfactory results for engineering practice.

$$\varphi_{DMT} = 5.5 \cdot \ln(q_t) + 19 \quad (5)$$

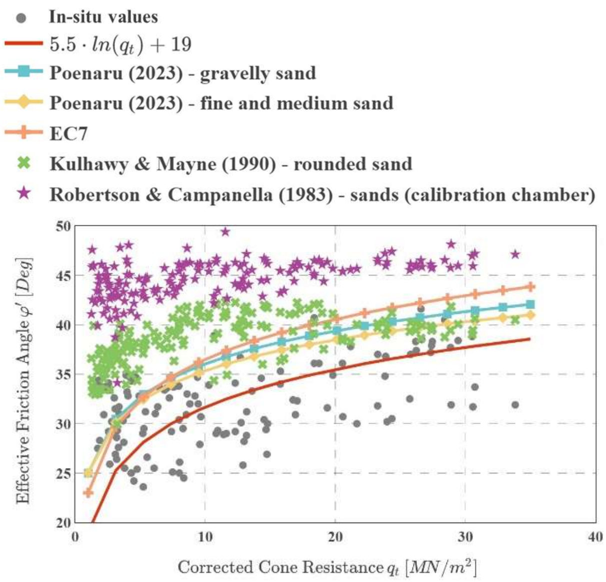


Figure 8. Correlations from literature and proposed correlation between q_t [MN/m²] and φ' [Deg]

Fig. 8 illustrates the logarithmic variation in correlations between q_t and φ, including the newly developed correlation and literature-based correlations between the friction angle and the cone resistance. Also, the in-situ values, identical to Fig. 6, are depicted in the background of the figure (grey dots) to visually compare the robustness of the different correlations.

The correlation established for layer 3 seems to represent a mean value for $q_t \leq 10$ MN/m² and an upper boundary for $q_t > 10$ MN/m² compared to all measured in-situ values. Compared to the literature-based correlations, Eq. (5) falls within the range of (Poenaru, 2023) and EC7, but marks a lower bound estimate considered to be on the safe side for the encountered granular Rhine soil in the project area.

The correlations of (Robertson & Campanella 1983) and (Kulhawy & Mayne 1990), both documented in (Robertson & Cabal 2022), seem to overshoot the measured in-situ values. This discrepancy could potentially be explained by the presence of more gravelly incursions that differentiate the current soil. Though, both correlations applied outside their calibrated boundaries, (Poenaru, 2023) noted that correlations derived from DMT tend to underestimate the internal friction angle compared to correlations based on CPT.

Based on the experience of the authors, the correlation of (Robertson & Campanella 1983) does overestimate the effective friction angle and should not be applied to soils comparable to the encountered dense to very dense gravelly Rhine sands and Rhine gravels. The correlation of (Kulhawy & Mayne 1990) tends to predict very unconservative effective friction angles within the encountered Rhine soils. However, the predicted values are reasonable based on the authors experience with comparable soils in the greater project region. As the (Kulhawy & Mayne 1990) values cannot be considered on the safe side within the encountered Rhine sands and Rhine gravels, they should be threated as an upper bound estimate and only applied with care.

5. Limitations & Conclusions

5.1. Limitations

The previously presented results, comparisons and correlation are due to certain limitations:

- The gravelly sands and gravels encountered in the project area are beyond the designed limits of DMT. DMT are mainly valid for cohesive soils and sands, i.e. a rough, initial filtering process of the results was performed to drop the most unreliable values from the analysis. Due to local variation some errors can be expected and the data must be interpreted with engineering judgement.
- There is only limited experience with DMT in German soils, most of which even is not documented in literature. With German soils, the experience of the authors was also limited to DMT carried out in mainly cohesive soils prior to the project described in this paper.
- Due to project requirements, the CPT, DMT and DPH could only partially be made prior to the deep drillings. A part of the indirect field investigations planned in the vicinity of the boreholes was made after the drilling was finished. Additionally, mandatory unexploded ordnance (UXO) investigations were conducted in the vicinity of the investigation points, as required for the investigated area. Soil disturbance, especially influencing the DMT results, may have occurred and might be an additional explanation for the scatter in the data.
- Not all investigation points were located via GPS in detail. If multiple investigations were performed at one point, only the center point was located via GPS and the investigations were carried out within a radius of approx. 0.5 m around the center. Thus, a maximum distance of 1.0 m between the investigation points to be correlated is possible. With increasing inhomogeneity of the subsoil, the accuracy of the presented relations will become more inaccurate. As the subsoil in the project area is in general very homogeneous, discrepancies are *only* expected within the soil layer boundaries, determined with the 3D subsoil model.

5.2. Conclusions

This paper presents the results of field investigations, mainly CPT and DMT, carried out in flood sediments and sandy and gravelly soil in vicinity of the river Rhine in Germany. Experience with DMT is rare in Germany, thus these DMT results are one of the first international published case histories demonstrating the feasibility but also challenges of DMT within coarse grained Rhine soils in Germany. Despite the dense to very dense sands and gravels, challenging for all indirect soil investigations, the CPT, DMT and DPH could be successfully pushed / driven into the soil and corresponding measurements be carried out.

Despite the limitations of the DMT being used outside of the calibrated soil, the measured data was reasonable after filtering the data by engineering judgement (cf. section 5.1). The filtered DMT data could then be compared to the CPT data showing reasonable relations between the dilatometer modulus E_d and the corrected cone resistance q_t as well as the friction angle φ_{DMT} determined by the DMT and the relative density I_D determined by the CPT. A reasonable stress-based correlation between the DMT and the CPT could be established only based on the data of sandy soils. This relationship allows to determine the effective friction angle as function of the corrected cone resistance for the investigated soils. The determined relationship as well as the in-situ values were compared with correlations from literature. While the DMT-based correlations tend to estimate a lower bound effective friction angle, the CPT-based correlations tend to be an upper bound or even overshoot the effective friction angle. Due to the lack of laboratory tests to determine the effective strength, only the authors experience with similar soils was used to assess the plausibility of the correlations.

Based on the presented relations and the correlation, it was possible to transfer results between CPT and DMT within the context of the project for the preparation of a 3D subsoil model, and for the determination of optimized soil mechanical parameters (strength and stiffness).

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