Correlation of cone resistance with undrained strength of some very soft to hard cohesive clays

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ABSTRACT: The cone resistance of five soil types from different locations in Germany has been compared with undrained shear strength. The types of soil include sludge or viscous mud and marine clay a the bank of the river Elbe in Hamburg, the normally consolidated lacustrine soft soil deposit around the lake Costance, plastic clay and argilaceous clay stone in its weathered and intact condition at the Tunnel offenbau site, and the overconsolidated tertiary clay around the city of Noerdlingen. Although the correlation of the cone resistance with the undrained strength is not new to the geotechnical profession, the paper contributes to the already existing data bank and shares the local experiences on particular soil types.

1 INTRODUCTION

1.1 *General*

The correlation of the cone resistance with the undrained strength is not new in the geotechnical engineering. A huge numbers of references are available in the literature. For example, a guidance and interpretation of the cone penetration test can be found in Lunne et al. (1996). The correlation between the cone resistance and the undrained strength without considering the pore pressure is based on the formula suggested by Terzaghi (1943):

$$q_c = N_k \cdot c_{cu} + \gamma \cdot z \tag{1}$$

where q_c is the cone resistance, N_k the cone factor, c_{cu} is the undrained shear strength, γ is the total unit weight of soil, and z is the depth of penetration. Equation 1 may be rewritten as:

$$c_{cu} = \frac{(q_c - \sigma_{vo})}{N_k} \tag{2}$$

where σ_{vo} is the total overburden pressure. Equation 2 can also be found in Lunne et al. (1996) and the draft standard European Norm “Geotechnical Design by Field Testing“. In Germany, the approximation of the undrained strength from the cone resistance is not a common practice. The working group for offshore and water way structures (EAU 1996) suggests the following relationship between the cone resistance and the vane shear strength:

$$\tau_{fv} = \frac{q_c}{N} \tag{3}$$

where τ_{fv} is the uncorrected vane shear strength and the factor N = 12, 14 and 20 for soft, normally consolidated and overconsolidated cohesive soils respectively. Joerß (1998) also suggested N_k (Eq. 2) values of 20, 25, 20, and 15 for clay in general, Lauenberger clay, lacustrine clay and boulder clay in northern Germany respectively, however, he reported a large scatter of data and very low coefficient of correlation in some cases. If one uses the statistical method to approximate the best fit line, the linear equation does not necessarily pass through the origin (see Joerß 1998). In such case Equation 2 may be modified as:

$$c_{cu} = N_{cu} + \frac{(q_c - \sigma_{vo})}{N_k} \tag{4}$$

where N_{cu} is an intercept which depends on the type of soil and degree of the data scatter.

In this paper, an attempt is made to find a correlation of the cone resistance with the undrained strength of several soil types in Germany and contributes to the already available data bank. The type of the soils included in the study are the normally consolidated lacustrine soft soil deposit around the lake Costance, overconsolidated tertiary clay around the city of Noerlingen, the plastic clay and argilaceous claystone at the site of the Tuunel Offenbau, and the sludge or mud and young marine clay along the bank of the river Elbe in Hamburg. The description of the sites are presented in the next section.

The term c_{cu} in Equations 2 & 4 is alternatively used in this paper for the undrained strength of the unconfined compression test or pocket penetrometer and uncorrected field vane test (τ_{fv}).

2 THE SLUDGE AND MARINE CLAY DEPOSIT AT THE BANK OF THE ELBE RIVER

For the purpose of land reclamation for the Airbus extension project at the bank of the Elbe river in Hamburg-Finkenwerder, locally known as "Muehlenberger Loch", a number of soil exploration was carried out (Kempfert + Partner 1998-2002). The exploration were conducted both before and after the construction of the dike enclosing the site. The dike was up to 8.8 m high. The upper part of the underground consists of mainly sludge (up to 14 m deep), but peat, young marine clay were also encountered overlaying the bearing sand lyer. A typical soil profile is shown in Figure 1. The soil profile and the result of the cone test are obtained from the soil exploration at the worst location after the construction of the dike, whereas the other data are adopted from both before and after the construction of the dike and does not necessarily correspond to the soil description given in Figure 1. Although the data scatter of the water content, the consistency limits and the undrained shear strength are relatively large, there is a general tendency of decreasing the water content and increasing the field vane shear strength with depth at an average. The vane strengths are not corrected according to Bijerrum correction factors. This is purposely done to leave free room for the engineers to decide on the reduction factor according to the local condition.

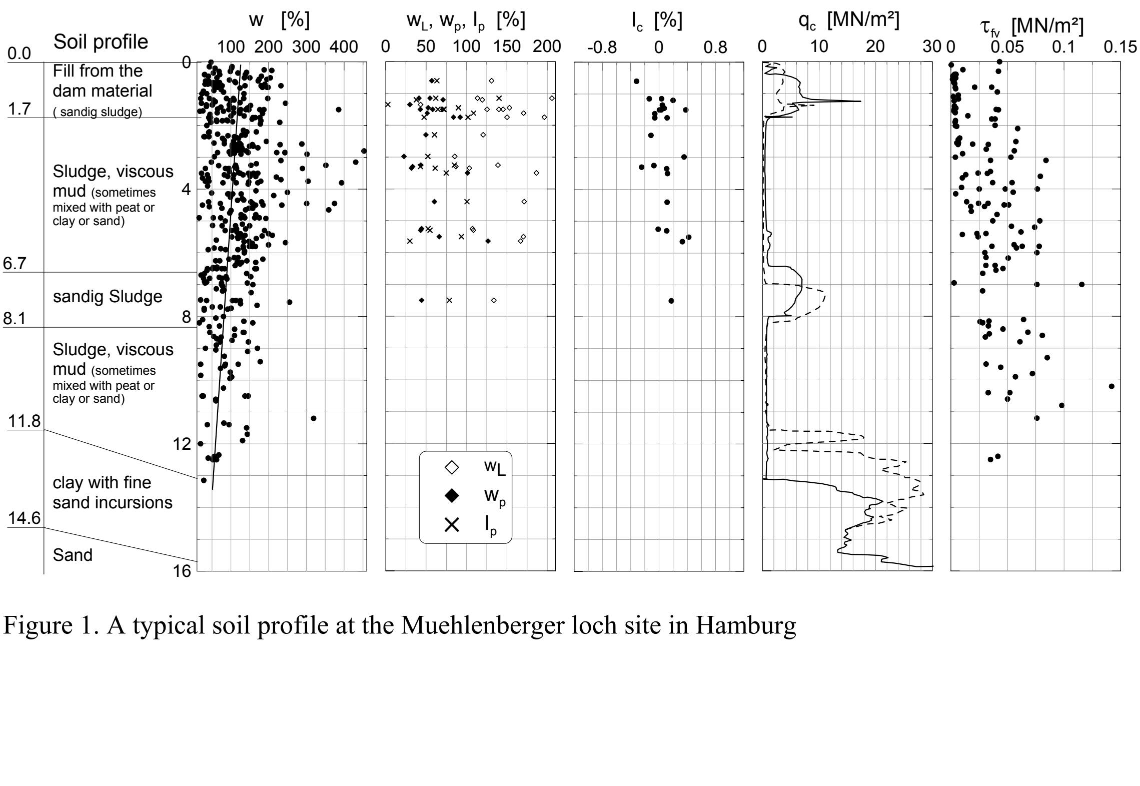


Figure 1. A typical soil profile at the Muehlenberger loch site in Hamburg

For the purpose of statistical analysis the soils are grouped into two. The predominately sludge some times mixed with peat, marine clay and sand in one group, and the young marine clay, peat and mixture of both on the other group. Figure 2 shows the correlation of the cone resistance with the field vane strength of the sludge. In both cases the surcharge pressure due dam material was considered in calculating the vertical stress σ_{vo}. As shown in Figure 2, the values of the cone factor N_k and the intercept N_{cu} in Equation 4 are 10.3 and 0.0129 respectively. Note that the value of N_{cu} is in MN/m². In this case a very low coefficient of determination $R^2 = 0.378$ was achieved. On the other hand, If one let the best fit line to pass through the origin similar to Equation 2, with the assumption that the undrained shear strength is zero when the net cone resistance (q_c - σ_{vo}) is zero, one may arrive at a relatively better correlation but still low coefficient of determination $R^2 = 0.692$, and the corresponding $N_k = 7.6$.

Similarly, $N_k = 28.7$ and the intercept $N_{cu} = 0.0191$ with very low $R^2 = 0.242$ were obtained according to Equation 4 for the second group of soils, whereas a value of $N_k = 14.1$ and $R^2 = 0.752$ were calculated according to Equation 2 (see Fig. 3).

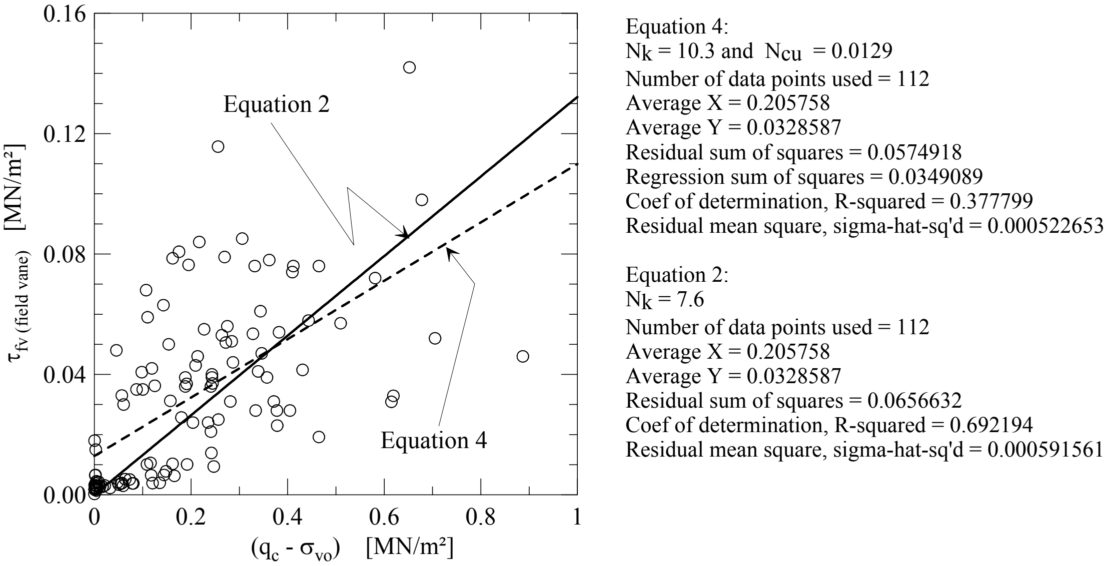


Figure 2. The correlation of the cone resistance with the field vane shear strength of sludge soil deposit at the Muehlenberger loch site in Hamburg.

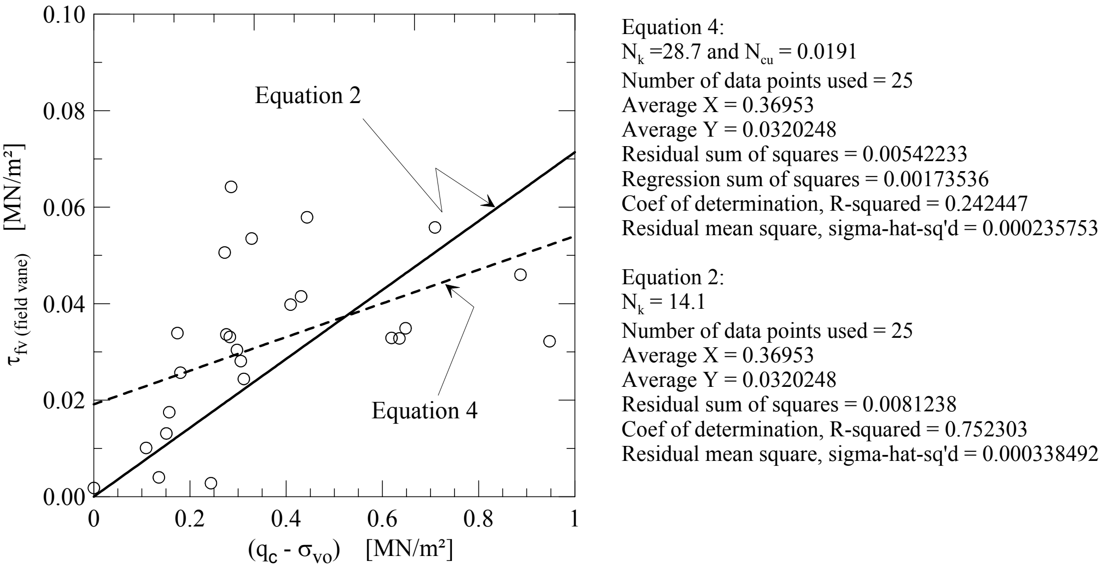


Figure 3. The correlation of the cone resistance with the field vane shear strength of marine clay and peat deposit at the Muehlenberger loch site in Hamburg.

3 THE LACUSTRINE SOFT SOIL DEPOSIT IN THE CITY OF CONSTANCE

The lacustrine deposits are mainly distributed around the Lake Constance and the South Bavarian Lakes. They are post glacial sediments and their thickness is estimated to be over 20 m. Figure 4 shows a typical soil profile and the cone penetration result from Seeuferhaus project (Kempfer + Partner 2002) in the city of Constance. Most of the data on the water content, the consistency limits, the liquidity index and field vane strength in Figure 4 are collected from this project, however, few data from soil exploration of one more project in Constance (Kempfert et al. 2001) are also added. The water content distribution shows a decreasing tendency with depth as would expected. Note that the vane strengths are not reduced according to Bijerrum correction factors for the reason explained in Section 2.

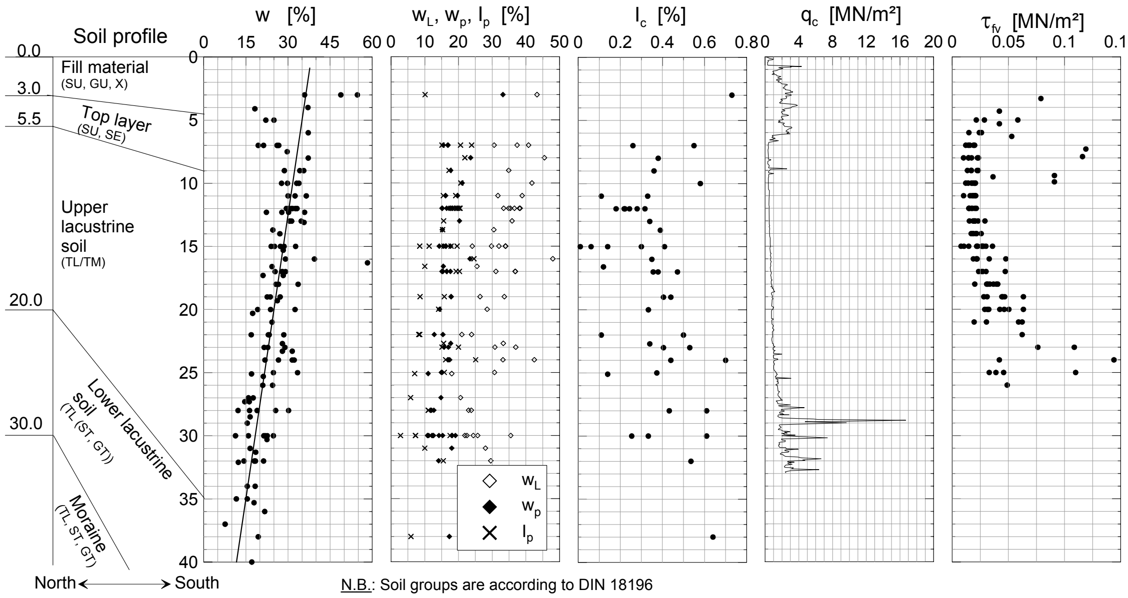
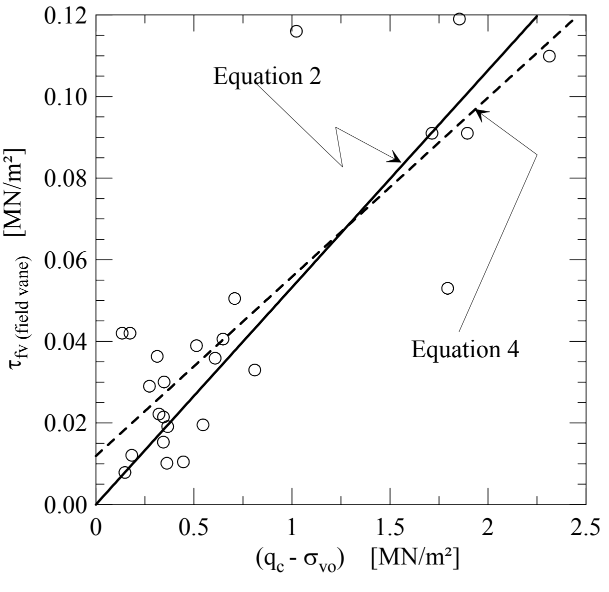


Figure 4. A typical soil profile at the Seeuferhause site in the city of constance.



Equation 4:
N_k = 22.8 and N_{cu} = 0.0119
Number of data points used = 25
Average X = 0.727448
Average Y = 0.0438488
Residual sum of squares = 0.0086196
Regression sum of squares = 0.0194069
Coef of determination, R-squared = 0.692448
Residual mean square, sigma-hat-sq'd = 0.000374765

Equation 2:
N_k = 18.8
Number of data points used = 25
Average X = 0.727448
Average Y = 0.0438488
Residual sum of squares = 0.0101494
Coef of determination, R-squared = 0.866621
Residual mean square, sigma-hat-sq'd = 0.000422892

Figure 5. The correlation of the cone resistance with the field vane shear strength of lacustrine soft soil deposit in the city of Constance.

The correlation of the cone resistance with the field vane strength are given in Figure 5. The number of the field vane shear strength measurements does not correspond to the number of data in the correlation, because only data from the vane test immediately near the cone penetration test were considered. As shown in Figure 5, the values of the cone factor N_k and the intercept N_{cu} in Equation 4 are 22.8 and 0.0119 respectively ($R^2 = 0.692$). On the other hand, If the fit line is forced to pass through the origin (Eq. 2), one may arrive at better correlation with $R^2 = 0.867$. The corresponding $N_k = 18.8$.

4 THE CLAY AND CLAY STONE AT THE TUNNEL OFFENBAU SITE

As part of the new high speed railway line between Nürnberg and Ingolstadt in southern Germany, two pile load tests were conducted at two different locations at the site Offenbau Tunnel (Kempfert + Partner &Spotka und Partner 2002). In the frame work of the pile load test project, a site exploration was made which include four cone penetration tests and one borehole at each location. Undisturbed samples were retrieved and their unconfined compression strength were determined in laboratory. As shown in Figure 6, the underground is made of predominately soft to hard, silty, sandy clay and clay stone. The clay stone was found in weathered and intact condition, where the degree of weathering decreases with the depth. The distribution of the natural water content also shows a decreasing trend with the depth, whereas no pattern can be observed on the undrained shear strength. The typical cone resistance diagram shows an increase of the resistance with depth as would expected.

The undrained strength of the soil from each borehole were matched with the corresponding average value of the four cone resistance at the respected depth. Though, the number of data are very limited, an attempt was done to correlate the cone resistance with the undrained strength from unconfined compression test as shown in Figure 7. The values of the N_k and N_{cu} according to the Equation 4 are 157.2 and 0.0543 respectively ($R^2 = 0.296$), whereas N_k according to Equation 2 is found to be 89.3 with a better value of $R^2 = 0.773$.

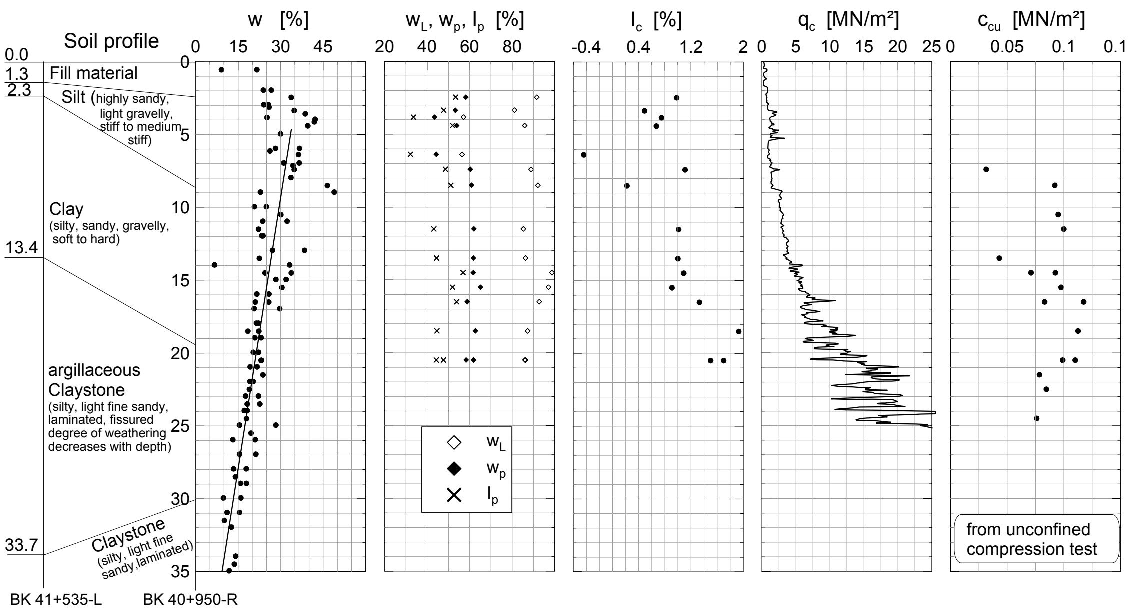
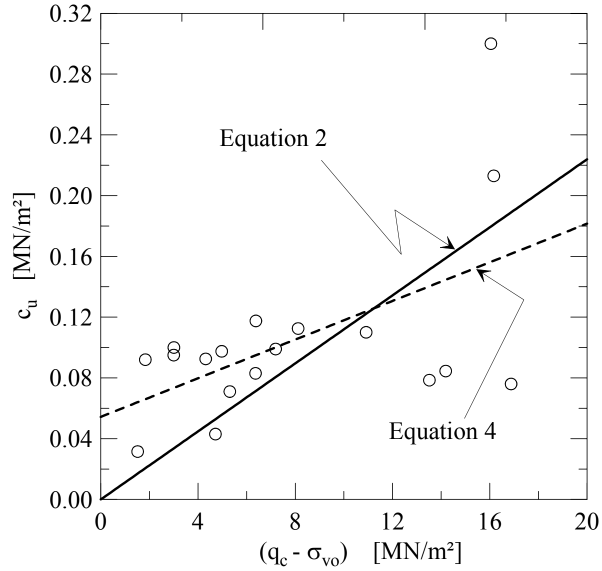


Figure 6. The soil profile at the pile load test locations in Tunnel Offenbau (Nuernberg - Ingolstadt Los Nord)



Equation 4:
N_k = 157.2 and N_{cu} = 0.0543
Number of data points used = 18
Average X = 8.02357
Average Y = 0.105361
Residual sum of squares = 0.0445447
Regression sum of squares = 0.0187717
Coef of determination, R-squared = 0.296475
Residual mean square, sigma-hat-sq'd = 0.00278401653

Equation 2:
N_k = 89.3
Number of data points used = 18
Average X = 8.02357
Average Y = 0.105361
Residual sum of squares = 0.0597435
Coef of determination, R-squared = 0.772954
Residual mean square, sigma-hat-sq'd = 0.00351432

Figure 7. The correlation of the cone resistance with the undrained shear strength of the clay and clay stone at the site Tunne Offenbau (Nuernberg - Ingolstadt Los Nord).

5 THE TERTIARY CLAY IN THE SURROUNDING AREA OF NOERDLINGEN

A deep slope failure had occurred twice on a ring road embankment in the vicinity of the city Noerdlingen in southern Germany. To investigate the cause of the failure a new soil exploration was made in addition to the already existing exploration results (Kempfert 2002). A total of 4 boreholes and 4 cone penetration tests were carried out. As shown in Figure 8, the top layer consists a fill material composed of clayey, sandy and gravely silt. An incursion of the embankment material was also found in this layer due to the sliding of the slope deep in the foundation. Beneath the fill layer a loamy silt again composed of clay, sand and gravel was encountered overlaying the stiff to medium hard silty clay. The water content distribution in the tertiary clay layer shows no change with depth.

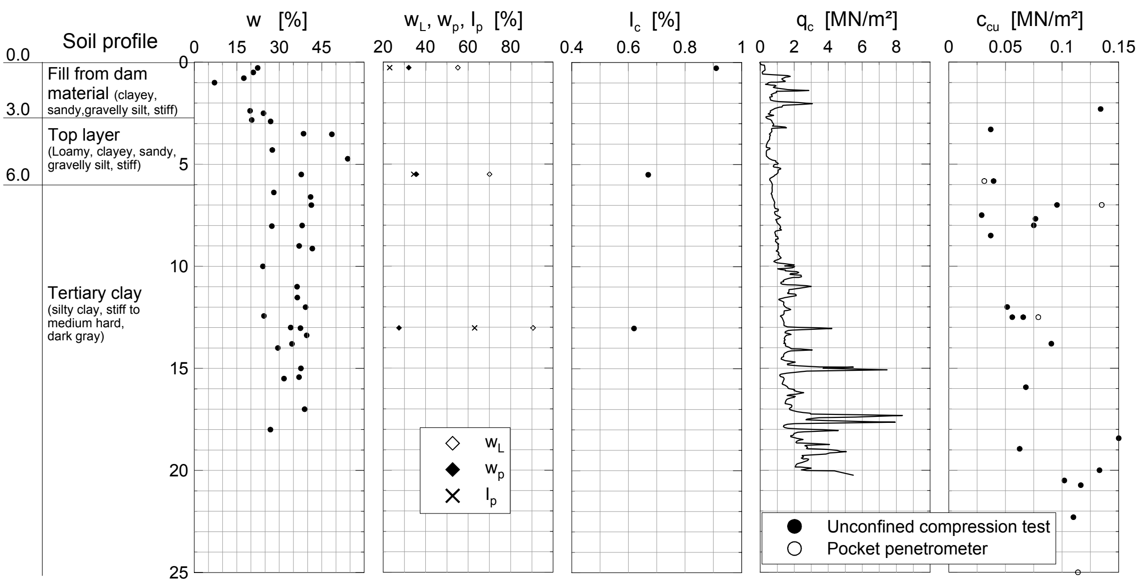


Figure 8. A typical soil profile at the collapsed ring-road site in the vicinity of the city Noerdlingen.

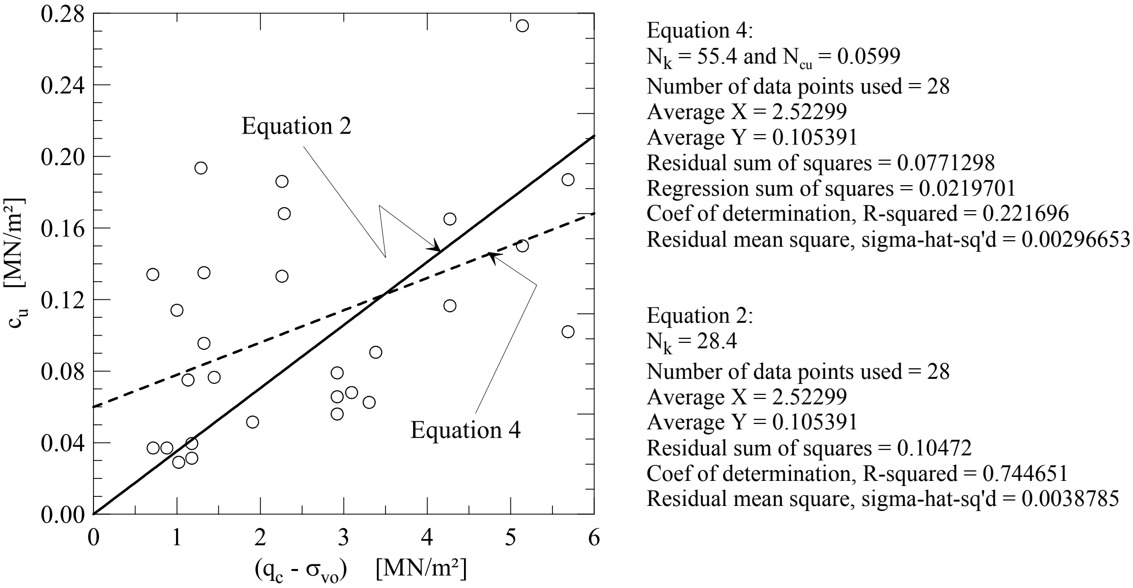


Figure 9. The correlation of the cone resistance with the undrained shear strength of the tertiary clay in the vicinity of the city Noerdlingen.

The undrained shear strength was determined from unconfined compression and pocket penetrometer tests on undisturbed samples. As shown in Figure 9, a very poor correlation ($R^2 = 0.222$) of the parameters according to Equation 4 was observed. In this case the factors N_k and N_{cu} are found to be 55.4 and 0.0559 respectively. On the other hand, a value of $N_k = 28.4$ and a better value of $R^2 = 0.745$ was determined according to Equation 2.

6 SUMMARY AND CONCLUSIONS

The authors are aware of the limited data available for statistical analysis, however, an attempt has been made to correlate the cone resistance to the field vane strength and undrained shear strength from unconfined compression test and pocket penetrometer for five soil types from different locations. It was observed that the correlation of the parameters according to Equation 4 provide very low coefficient of determination. On the other hand, letting the fit line to pass through the origin according to conventional Equation 2, with the assumption that the undrained strength is zero when the net core resistance is zero, has lead to a better matching of the parameters and a higher and reasonable coefficient of determination. The value of the cone factor N_k for the later case are summarized in Table 1 below.

Table 1. Summary of the N_k values according to the conventional Equation 2

Soil type	N_k	R^2
Sludge	7.6	0.752
Marine young clay	14.1	0.752
Lacustrine soft soil	18.8	0.867
Quaternary clay and clay stone	89.3	0.773
Tertiary clay	28.4	0.745

Due to the limited number of data, the value of N_k in Table 1 should be applied in practice with care, until it is supported by further data, although the achieved coefficient of determinations (0.745 to 0.867) are not as such unrealistic in soil mechanics. The authors believe that there are enough available data in practice on these type of soils and others and would invite all the responsible experts to contribute to this theme.

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