Proceedings of the XVII ECSMGE-2019
Geotechnical Engineering foundation of the future
ISBN 978-9935-9436-1-3

doi: 10.32075/17ECSMGE-2019-0073

XVII European Conference
ECSMGE 2019
Reykjavik
on Soil Mechanics
and Geotechnical Engineering

Determination of SHANSEP parameters by laboratory tests and CPTu for probabilistic model-based safety analyses

Détermination de paramètres SHANSEP par essais en laboratoire et CPTu pour analyses à base de modèle probabilistes de sécurité

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ABSTRACT: Since January 2017 the assessment of macro-stability for primary dikes in the Netherlands must be made according to the new rules, called WBI2017. In WBI2017, the strength of soils has to be determined based on the Critical State Soil Mechanics (CSSM) theory and specifically the Stress History and Normalised Soil Engineering Properties (SHANSEP) method for the undrained soil layers. Consequently, laboratory tests have to be conducted and interpreted to determine the parameters for (semi) probabilistic macro-stability calculations. After the parameters are determined, the stress history profile of the soil can be obtained from the undrained shear stress and the measured cone resistance from the CPTu. This paper covers the determination of the soil parameters needed for the dike strengthening project Krachtige IJsseldijken Krimpenerwaard (KIJK) for the undrained soil layers. This project is located between the cities Krimpen aan den IJssel and Gouderak in the province of Zuid-Holland, in the Netherlands.

RÉSUMÉ: Depuis janvier 2017 l'évaluation de macro-stabilité pour des digues primaires dans les Pays-Bas doit être faite selon les nouvelles règles, appelées WBI2017. Dans WBI2017, la force des sols doit être déterminée basée sur la théorie de The Critical State Soil Mechanics (CSSM) et spécifiquement le Stress History and Normalised Soil Engineering Properties (SHANSEP) la méthode pour les couches de sol non drainées. C'est le contraire à la vieille règle d'évaluation, où la force des sols a été déterminée selon le critère d'échec de Mohr-coulomb utilisant les paramètres drainés. Par conséquent des essais en laboratoire doivent être conduits pour déterminer les paramètres pour des calculs de macro-stabilité (semi) probabilistes. Après que les paramètres soient déterminés, l'histoire de stress du sol peut être obtenu de la du contrainte de cisaillement non drainé et la résistance de cône mesurée CPTu. Ce papier couvre la détermination des paramètres des sols nécessaires pour renforce la digue au projet Krachtige IJsseldijken Krimpenerwaard (KIJK). Ce projet est localisé entre les villes Krimpen aan den IJssel et Gouderak dans la province de Zuid-Hollande, dans les Pays-Bas.

Keywords: SHANSEP; Parameter determination; Laboratory test; CPTu, Probability.

1 INTRODUCTION

Traditionally the slope stability of primary dikes in the Netherlands is calculated using a semi-probabilistic method with Bishop's model or the Uplift-Van model using Deltares software, D-Geo Stability. The calculations are made under drained conditions with the strength of the soil according to the Mohr-Coulomb failure criteria.

The required soil parameters, the friction angle and cohesion parameters, are determined in the laboratory at a strain level of 2% to 5%. An important aspect in the design of a dike improvement is to account for uncertainties in strength parameters of the dike body and the subsoil.

Since January 2017 the assessment of macro-stability for primary dikes in the Netherlands must be made according to the new design rules, called WBI2017 (MIM, 2017). The requirements for dike design will be based on the risk of flooding instead of the chance of exceeding a critical water level.

In WBI2017 for macro-stability, the strength of soils has to be determined based on the Critical State Soil Mechanics (CSSM) (Schofield and Wroth, 1968) theory and specifically the Stress History and Normalised Soil Engineering Properties (SHANSEP) (Ladd and Foott, 1974) method for the undrained soil layers.

Consequently, new soil parameter sets for the macro-stability calculations are required, which will be used in the (semi) probabilistic calculations for the design optimization of the dike. These parameters are determined at a strain level of about 25% for clay and 40% for peat.

For the cohesive soils (e.g. peat and clay) the undrained shear strength of the soil is determined by means of an CPTu, according to (Lunne, Robertson, Powell, 1997).

$$s_u = \frac{q_t - \sigma_v}{N_{kt}} \quad (1)$$

Where s_u (kPa) is the undrained shear strength, q_t (kPa) the corrected cone resistance, σ_v (kPa) the total stress and N_{kt} (-) empirical cone factor.

The strength profile is now based on the SHANSEP principle according to (Ladd and Foott, 1974)

$$s_u = \sigma'_v \cdot \mathrm{S} \cdot (OCR)^m \quad (2)$$

Where s_u (kPa) is undrained shear strength, σ'_v (kPa) the effective stress, S (-) the undrained shear strength ratio for OCR = 1, OCR (-) the overconsolidation ratio and m (-) the strength increase component. The OCR can be determined by combining equations (1) and (2).

For the non-cohesive soils (like sand) the parameters used in the calculations are the angle of internal friction (ϕ') and the cohesion (c').

The D-Geo Stability software has the option in the input menu to specify for each cohesive layer yield stress points to determine the soil shear strength. The yield stress describes the maximum historical vertical pressure and is calculated as:

$$\sigma'_y = \sigma'_v \cdot \mathrm{OCR} \text{ or } \sigma'_y = \sigma'_v + \mathrm{POP} \quad (3)$$

With σ'_y (kPa) the yield stress and POP (kPa) is the Pre-Overburden Pressure. The relation between POP and OCR can there-fore be written as:

$$OCR = \frac{\sigma'_v + POP}{\sigma'_v} \quad (4)$$

Depending on the software used for the stability calculation, for e.g. DGeo-Stability or PLAXIS, σ'_y or OCR must be used as an input parameter to calculate the undrained shear strength s_u with the SHANSEP principle (Simanjuntak et al., 2018).

To determine the reliability index β with probabilistic calculations, input parameters S, m, and σ'_y are described with log-normal distribution functions with the mean and standard deviation of the parameters determined by the laboratory results. For details on the performed probabilistic calculations reference is made to (Bakker et. al., 2019) and (Simanjuntak et al., 2019).

2 FIELD INVESTIGATION

For the use of the correlations to determine the undrained shear strength with the cone resistance, the accuracy of the measured cone resistance (q_c), the sleeve friction (f_s) and the pore pressure (u) must be very high. The used cone and test procedure should meet the requirements according to a type 1 cone as described in NEN-EN-ISO 22476-1.

In total 74 CPTu‘s with a class type 1 cone are made in the toe or hinterland of the dike. In the crest of the dike 80 CPTu‘s are executed with a class type 2 cone. This because of the dike material with a higher resistance which can cause damage to the very sensitive type 1 cone.

Besides the CPTu‘s also a total of 80 bore holes where conducted to get undisturbed soil samples for a good soil description and to test in the laboratory.

3 GEOTECHNICAL PROFILE

For the project a parameterset is determined based on the laboratory investigation. To do so first a geotechnical profile is determined with a general description of the soil layers.

In a previous stage of the project a study was made of the geological origin from the different soil layers in the project area. In total seven different Holocene soil layers were distinguished.

In the following stage this is, due to more soil samples and a better understanding of the soil, reduced to six layers, the peat layers Hollandveen and Basisveen and the clay layers “klei met plantenresten” (clay with organic material), klei antropogeen (anthropogenic clay) and Kreftenheye (clay deposit above Pleistocene sand).

Besides the natural deposits there is the layer “dikes-material”, which is the clay layer of which the dike is made of. Based on this description a geotechnical profile is made across the dike. A distinction is made for the soil layers underneath the dike and next to the dike. The geotechnical profile was made for the crest of the dike and for the toe of the dike.

4 CONDUCTED LABORATORY INVESTIGATION

4.1 Static triaxial tests

In total 102 single stage CAU triaxial tests are conducted on clay samples. The tests are performed in accordance to procedures described in the WBI2017. The samples are trimmed to dimensions of approximately 100 mm height and an internal diameter 50 mm.

Following the K_0-value recommendations provided in WBI2017 the clay samples were anisotropically consolidated under $K_0 = 0.45$ for the silty clay samples (saturated volume weight > 14 kN/m^3) and $K_0 = 0.35$ for the organic clay samples (saturated volume weight < 14 kN/m^3).

Both normally consolidated samples (OCR = 1) and samples at situ stress conditions (OCR > 1) are tested.

4.2 Direct Simple Shear tests

The Netherlands has presently no standard to perform DSS test, so the 62 DSS tests were conducted in accordance with a, by the laboratory in-house developed, method based on the ASTM D 6528-07.

Both normally consolidated samples (OCR = 1) and samples at situ stress conditions (OCR > 1) are tested. The samples were subjected to DSS testing under two different directions of loading; perpendicular or parallel to the dike.

The tests where preformed with a shearing rate of 8% per hour.

4.3 Oedometer test

For the determination of the stiffness of the soil and the yield stress 160 oedometric tests are conducted in accordance to the protocol of the WBI2017. The test results are interpreted according to Koppejan, NEN-Bjerrum and the abcmethod. The yield stress is determined from the stress-strain curve with the Casagrande method.

5 DETERMINATION OF GEOTECHNICAL PARAMETERS

5.1 Introduction

For the (semi)probabilistic calculations the average and characteristic values of the soil parameters are determined according to:

$$x_{gem} = \frac{\sum_{x=1}^{x=n}(x_i \dots x_n)}{n} \qquad (5)$$

$$\sigma_x = \sqrt{\frac{\sum_{x=1}^{x=n}\left(x_i \dots x_{gem}\right)^2}{n-1}} \qquad (6)$$

$$x_{kar} = x_{gem} - \sigma_x * t_{0,05;n-1} * \sqrt{\frac{1}{n} + (1-\alpha)} \qquad (7)$$

With x_{gem} the average value of the parameter, x_{kar} the chacacteristic 5% lower limit, n is the number of tests, σ_x is the standard deviation of parameter x, $t_{0,05;n-1}$ is 5% value of the Student-t distribution and α is parameter depending of use is made of local or regional data. In this project a value of 0,75 is used, according to WBI2017, because all the data was derived from samples retrieved from the total project area (length approximately 11 km). For local data a value of 1 can be used.

To avoid the presence of negative values in the probabilistic calculations the lognormal distribution of S and m parameters is used. This results in an average value of the parameters of μ(log) with a standard deviation σ(log) (where μ(log) stands for mean and σ(log) for standard deviation of the correspondent Probability Density Function).

During the analysis of the results of the clay taken underneath and next to the dike, as well as from the clay layers with different geological origin, it was found that there was no distinction in the parameters S and m. So, for the statistical analysis all the results for the clay layers are combined. The exception was the result from the test on dikes-material. For the layer Hollandveen there was also no distinction in the results from the samples underneath and next to the dike. For the deeper peat layer (Basisveen) there were not enough test, so the results were combined with the result from Hollandveen. This is a conservative approach.

For the determination of the necessary parameters distinction is made in three different soiltypes: 1) Clay, 2) Peat, 3) Dikes-material.

5.2 Assessment of undrained shear strength ratio S

The undrained shear strength ratio S is determined from test results of the triaxial and DSS tests performed on the soil samples tested under normally consolidated conditions (OCR=1). For the clay samples from the triaxial test, S has been calculated at 25% axial strain. In the case of peat samples from the DSS tests, the result at 40% radial strain is used. S is determined according to:

$$\mathrm{S} = \left(\frac{s_u}{\sigma'_{vc}}\right)_{NC} \qquad (8)$$

Where s_u (kPa) the undrained shear strength, σ'_{vc} (kPa) the effective stress at the end of consolidation.

In Figure 1 the undrained shear strength results from the normally consolidated samples is plotted against the vertical effective stress. The derived mean values of the undrained shear strength ratio S and the standard deviation are shown in Table 1.

Table 1. Undrained shear strength ratio from test results

Soil type	**Test**	**μS**	**σ**
Clay	Triaxial	0,32	0,02
Dikes-material	Triaxial	0,37	0,02
Peat	DSS	0,39	0,02

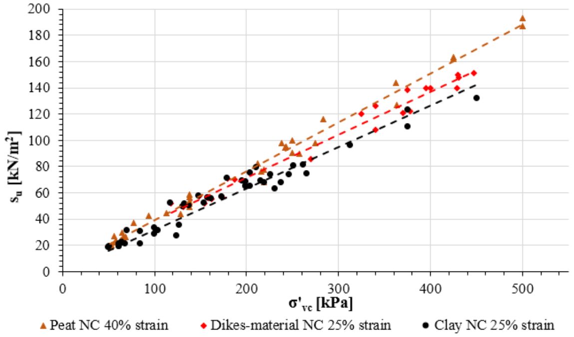

Figure 1 Undrained shear strength vs effective stress

5.3 Assessment strength increase component m

The parameter m was calculated with the abc-isotache stiffness parameters b (slope of the virgin loading line from the oedometer test) and a (slope of the unload-reload line from the oedometer test) according to:

$$m = \frac{b-a}{b} \tag{9}$$

For the determination of parameter a an unloading step of 50% of the previous loading step is used after which the sample is loaded again to the load before unloading (OCR = 2).

In Figure 2 the isotache a and b parameters from the oedometer tests are plotted against the saturated volumetric weight. Figure 3 presents the calculated factor m. In Table 2 the determined mean values of the factor m are presented and the standard deviation.

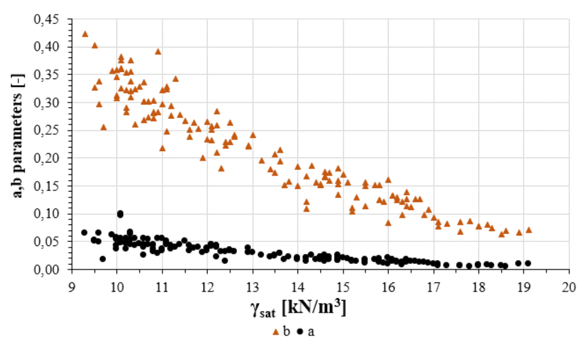

Figure 2 a, b parameters vs saturated volumetric weight

Table 2. Factor m from test results

Soil type	Test	µm	σ
Clay	oed	0,88	0,01
Dikes-material	oed	0,91	0,02
Peat	oed	0,85	0,02

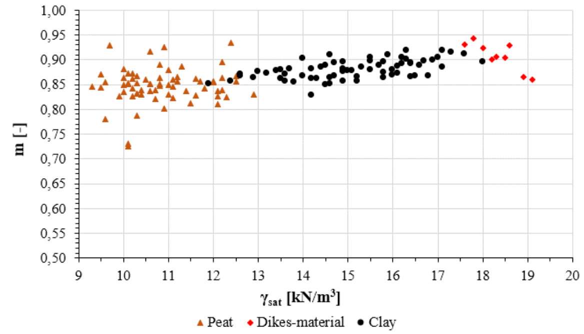

Figure 3 Factor m vs saturated volumetric weight

5.4 Assessment of the cone factor N_{kt}

For each soil sample the cone factor N_{kt} is determined to correlate the undrained shear strength s_u from the triaxial or DSS test with the netto cone resistance, q_{net} (MPa) from CPTu which are located close to the tested samples.

$$N_{kt} = \frac{q_{net}}{s_u} = \frac{q_t - \sigma_v}{s_u} \tag{10}$$

The analysis of the data is done according to the method mentioned in WBI2017. Hereby the mean value of the correlation factor N_{kt} is determined by linear regression. This is done by searching for the N_{kt} value for which the sum of the squares of relative residues has a minimum according to:

$$\min\left(\sum_i \left(s_{u;i} * \frac{N_{kt}}{q_{net;i}} - 1\right)^2\right) \tag{11}$$

The result of the analysis is shown in Figure 4. In this figure the N_{kt} value is the slope of the lines. In the figure are also shown with the dotted lines the 5% upper and lower limits for the N_{kt} values.

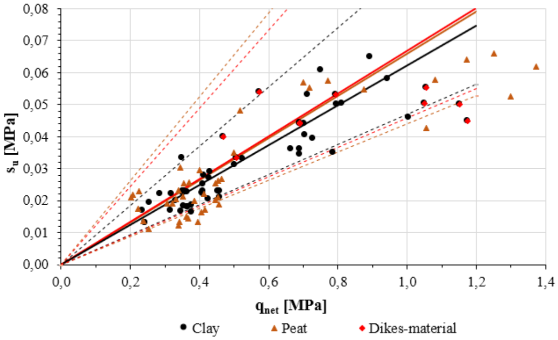

Figure 4 Cone factor vs undrained shear strength

The mean value for N_{kt} for each soil type is shown in Table 3. For the soil layers above the phreatic level, the mean value of the N_{kt} is multiplied with a factor of 3 to account for the higher measured q_c values due to drying out of the soil. The table also gives the calculated variation coefficient for N_{kt}.

Table 3. Cone factor N_{kt} from test results

Soil type	μ_{Nkt}	VC_{Nkt}
Clay	16,1	0,20
Dikes-material	15,0	0,28
Peat	15,2	0,30

6 UNCERTAINTY IN UNDRAINED SHEAR STRENGTH AND YIELD STRESS

To perform the probabilistic calculations, it is also necessary to know, besides the mean values, the variation in the undrained shear strength and the yield stress.

6.1 Uncertainty in undrained shear strength

The uncertainty in the undrained shear strength is depending on two components. The first one is the uncertainty of linking the undrained shear strength from laboratory tests to a cone resistance. With this a transformation uncertainty is introduced. The second component is due to spatial variability.

$$VC_{gem} = \sqrt{VC^2_{transformation} + VC^2_{spatial}} \quad (12)$$

In which VC_{gem} (-) is the variation coefficient for the uncertainty in the undrained shear strength, $VC_{transformation}$ (-) is the variation coefficient for the transformation uncertainty and $VC_{spatial}$ (-) is the variation coefficient for the spatial variability.

6.1.1 Transformation uncertainty

Part of this uncertainty will be systematic, and a part will be random. The systematic part will be depending on soil type, stress level, degree of overconsolidation and of CPT equipment and of possible disturbance of the samples on which the laboratory tests is been carried out. The random part is averaged over the layer thickness.

$$VC_{transformation} = \sqrt{r_{sys} + \frac{1}{n_{m;dsn}}} * VC_{Nkt} \quad (13)$$

In which $VC_{transformation}$ (-) is the variation coefficient for transformation uncertainty, r_{sys} (-) the systematic part of the total uncertainty which cannot be averaged, $n_{m;dsn}$ (-) number of used measurements form the soundings.

If the layer thickness is large enough (> 2 to 3 times the vertical correlation length ~ 1 m) then $\frac{1}{n_{m;dsn}} \rightarrow 0$, so equation (13) becomes:

$$VC_{transformation} = \sqrt{r_{sys}} * VC_{Nkt} \quad (14)$$

6.1.2 Spatial uncertainty

Due to spatial variability, the properties of a soil layer of the same geological origin will differ from samples taken at different locations, although on average the properties will be about the same. The scale of these fluctuations in a layer are small, compared to the size of the slip plane. The extent to which the spread of a parameter averages along a slip plane is expressed with the variance reduction factor Γ.

$$VC_{spatial} \approx \sqrt{\Gamma^2} * VC_{regionaal} \quad (15)$$

and

$$\Gamma^2 \approx 1 - \alpha_v \quad (16)$$

With $VC_{regional}$ (-) variation coefficient spatial variability of the measured cone resistance, Γ (-) variance reduction factor and α_v (-) is the ratio between local and regional spreading. According to WBI2017 a value of 0,14 is used for $VC_{regional}$ and 0,75 for α_v.

6.2 *Uncertainty in yield stress*

To be able to determine the yield stress, for example in the middle of a soil layer, a variation coefficient for the yield stress, $VC_{\sigma y}$, must be derived. To determine $VC_{\sigma y}$ it can be assumed that the coefficient of variation in undrained shear strength (i.e., coefficient of variation of N_{kt}) can be divided into uncertainty in the parameters S and m and the yield stress with a fixed ratio. In the WBI2017 this fixed ratio is to be assumed 85% of VC_{gem}.

$$\mathrm{VC}_{\sigma y} = 0{,}85 * \mathrm{VC}_{gem} \quad (17)$$

The variation coefficients for the undrained shear strength and yield stress are shown in Table 4.

Table 4. Variation coefficient undrained shear strength and yield stress

Soil type	**VC_{gem}**	**$VC_{\sigma y}$**
Clay	0,19	0,16
Dikes-material	0,25	0,22
Peat	0,27	0,23

7 STRENGTH AND STRESS HISTORY PROFILE FROM CPTU

A spreadsheet tool is developed to determine the strength and stress history profile based on the CPTu measurements and with the use of the equations (1) and (2). The input for the tool are the parameters from the previous paragraphs. With the tool a check is preformed whereby the soil profile is compared with the classification according to Been and Jeffries (MIM, 2017).

For the strength profile the mean values and the characteristic values for the undrained shear strength are calculated. The stress history profile is determined with the mean value of the parameters. In the middle of a soil layer the mean and the characteristic values of the yield stress are determined. The mean yield stress is determined as the average value off the yield stress calculated for each q_c measurement point within a soil layer.

With the mean and characteristic values of the soil parameters known the (semi)-probabilistic safety analyses can be performed. An example of the graphical output of the tool is given in Figure 5. In the stress history profile also, the result for the yield stress for two tested samples from a adjacent bore hole is plotted. A good agreement is found between the calculated average profile and the laboratory determined yield stress.

8 CONCLUSIONS

In this paper the determination of a parameter set based on conducted field and laboratory investigation for the KIJK project in the Netherlands is discussed. The determination was necessary because of the new rules according to WBI2017 for macro-stability, the strength of soils must be determined based on the Critical State Soil Mechanics theory and specifically the Stress History and Normalised Soil Engineering Properties method for the undrained soil layers. With the parameters both semi- as probabilistic calculations are made.

9 ACKNOWLEDGEMENTS

The work presented in this paper is part of the KIJK dike strengthening project in the Netherlands. The authors are grateful to Het Hoogheemraadschap van Schieland en de Krimpenerwaard for providing the data for this study and would also like to thank the anonymous reviewers for their comments to improve the quality of the paper.

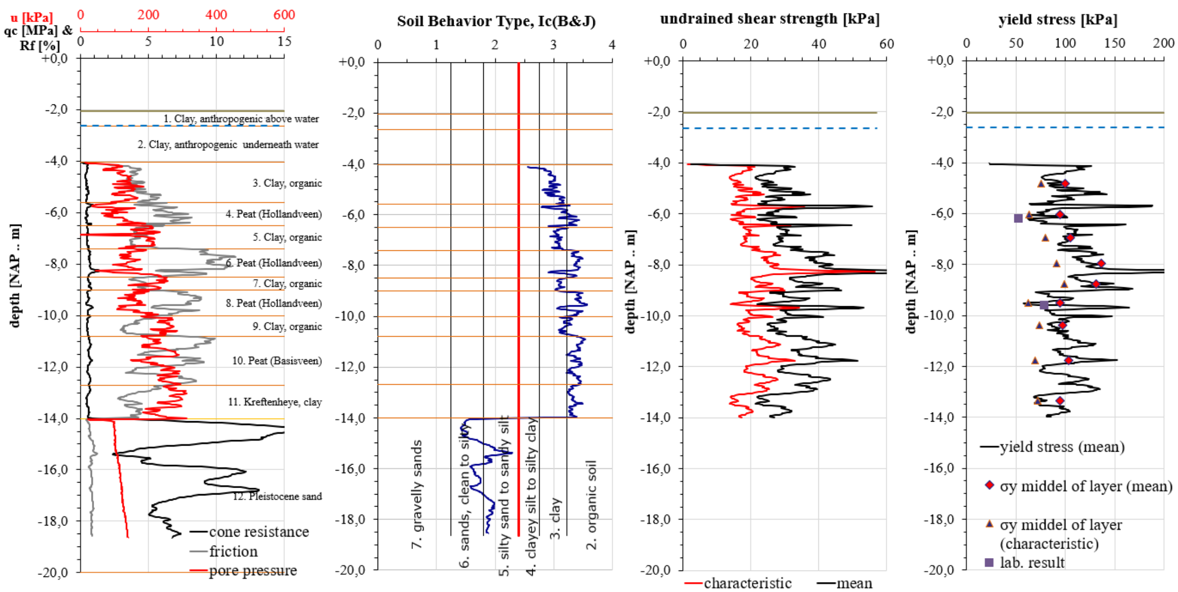

Figure 5 Interpretation of CPTu and strength and stress history profile

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