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Insights from the Eemdijk full-scale failure test - triaxial strength parameters of organic soils

Arny (H.J.) Lengkeek
Witteveen+Bos, Deventer, The Netherlands, arny.lengkeek@witteveenbos.nl

ABSTRACT: The Netherlands is inherently challenged by water, as a large part of the country lies below sea level and several major rivers in North-western Europe cross the country. The top elevation of existing earthen dikes is incrementally raised over time. However, raising a dike requires a stable base. A specific challenge regarding dike reinforcement in the Netherlands includes the presence of soft subsoil conditions at many dikes. These soft soils often consist of organic clays and peats. This paper focuses on parameter interpretation and modeling challenges of organic soils. The models are validated on the full-scale failure test. (In Dutch: 'Eemdijk damwandproef'), initiated by the Dutch Flood Protection Programme. The Eemdijk full-scale failure tests provides valuable insights on the triaxial strength parameters of organic soils.

KEYWORDS: SHANSEP, Slope failure, Finite element method, Dyke, Levee.

1 INTRODUCTION

The Netherlands is inherently challenged by water, as a large part of the country lies below sea level and several major rivers in North-western Europe cross the country. Subsidence, sea level rise, and the increase of rain intensity and river discharge due to climate change further challenge existing flood defenses to maintain required levels of safety. To do so, the top elevation of existing earthen dikes is often incrementally raised over time. However, raising a dike requires a stable base. A specific challenge regarding dike reinforcement in the Netherlands includes the presence of soft subsoil conditions at many dikes. These soft soils often consist of organic clays and peats.

This paper is based on the PhD research (Lengkeek, 2022) and focuses on improving on the global stability assessment of dikes in the Netherlands and modeling challenges of organic soils. The models are validated on the full-scale failure test. (In Dutch: 'Eemdijk damwandproef'), initiated by the Dutch Flood Protection Programme.

2 TEST SET-UP EEMDIJK GROUND DIKE

2.1 *Full-scale tests*

The Eemdijk full-scale failure test consists of three tests:

- sheet pile pull-over test (PO-test)
- ground dike test (GD-test)
- sheet pile reinforced dike test (SPD-test)

This paper only addresses the ground dike (GD). The GD-test is used as a reference case and to optimize the SPD-test conditions. The PO-test is used to investigate the sheet pile properties and the soil structure interaction until and beyond failure. The GD-test is performed on a newly constructed embankment. The embankment consists of two parallel sections of 60 m length with a maximum height of 5.5 m, see left side of Figure 1. The core material consists of sand and the cover layer consists of firm clay with a slope of 1V:1.7H. The center area between the two dikes is about 2 m lower and contains a clay cut-off wall to create two compartments with controllable water levels. The Eemdijk full-scale failure test enables to investigate failure conditions in a controlled manner. The experimental details are presented in Lengkeek (2022).

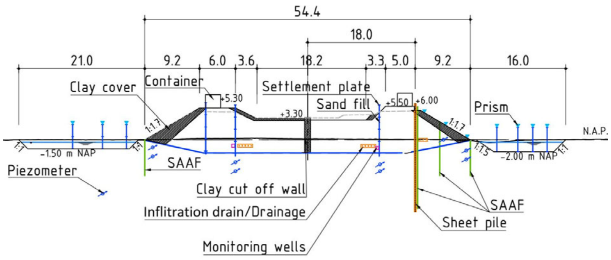

Figure 1. Cross section of ground dike (left) and sheet pile dike (right) with monitoring instrumentation.

3 PARAMETER DETERMINATION

3.1 *Field tests*

This section presents some of the geotechnical investigations and critical aspects of parameter determination. All tests are carefully examined as well as the procedures to determine the parameters. The aim is to derive the best estimate and upper bound parameters for the back-analysis, but also to improve the criteria to derive the ultimate strength parameters. The proposed methods are applicable to dike engineering projects.

The site investigation comprises 48 Cone Penetration Tests (CPT) and 18 boreholes with sampling prior to the embankment raising. The stratification is based on the CPT interpretation and borehole log description. The variation in level of the layer boundaries is typically +/-20 cm for each investigation point. Figure 2 presents the initial stratification and Soil Behavior Type (SBT) classification based on (lengkeek, 2024). The measured parameters qc, Rf and u2 are plotted on the left. The groundwater level (u0-line) is based on the fit of the hydrostatic u2-measurements in the sand layers. The soil behaviour index (IB) is presented in the middle, where it is shown by shading whether the layer is contractive or dilative. The classification per 2 cm, the stratification per 20 cm and the drainage conditions are presented in the bars on the right side.

Additionally, 6 CPTs are executed after raising the embankment. Figure 3 presents the stratification after raising the embankment to 4.5m CD. Reference level CD is equal to NAP. The generalized soil profile based on these CPTs prior and post to raising the embankment are presented in Table 1.

The values between brackets [] in Table 1 are the total settlements at the top of the layer in meters, the delta settlement in a specific layer in meters and the strain in each layer. The total settlement is 1.1 m and the strains between 15 (in clay) to 32% (in peat) in less than a year. All levels are rounded off at 0.1 m, which is about the accuracy of a CPT given the size of the cone and sleeve, hence the margin of the strains is typically +/-5%.

Table 1. Stratification at the ground dike.

Layer	Initial level (m CD)	Final level (m CD)	Stratification and SBT type
1		4.5	Sand fill, medium, SD
2	0.0	-1.1	Clay (unsaturated), firm, CD [z=1.1m/Δ=0.1m/ε=20%]
3	-0.5	-1.5	Organic Clay, soft, OCC [z=1.0m/Δ=0.2m/ε=15%]
4	-1.8	-2.6	Peat, soft, PC [z=0.8m/Δ=0.8m/ε=32%]
5	-4.1/-4.3	-4.3	Sand, loose, SC
6	-4.8	-4.8	Sand, medium, SD

The classification methods L24R10 and L24R16 (Lengkeek, 2024) are based on Robertson (2010) and Robertson (2016) classifications, with adjusted and additional SBTs for peat and organic clay, including additional Bq-based criteria to separate organic soils from mineral soils. These new classification methods correctly classify all layers, including organic clay and peat. It also consistently classifies the peats in both cases, although the effective stresses at -3m CD increase from 11 kPa to 96 kPa after raising the embankment. It is noted that in Figure 2 it seems like there is a transition layer between -4.1 and -4.3m CD which is not or slightly present in Figure 3. Apparently, the boundary effects in the Peat layer at low stresses are more significant than at high stresses. The drainage conditions are shown in the right bar and based on Schneider (2008). These remain the same except for the SC layer at about -4.5 m CD. Apparently, the excess pore pressure is less significant at higher stresses. Furthermore, the groundwater level increased from -0.4 m CD (regional phreatic) to +0.4 m CD (local phreatic in the embankment), mainly due to different surface drainage and evaporation conditions in the large embankment.

These new CPT-based classification methods used here show that it is possible to correctly classify the type of layer despite the difference in stress and state. This is an important starting point for the modelling of the embankment and the back-analysis of the failure test. It is also crucial in the assessment of dikes which are raised on soft soils or hundreds of years and hence have compressed layers below the dike linked to very soft layers beside the dike.

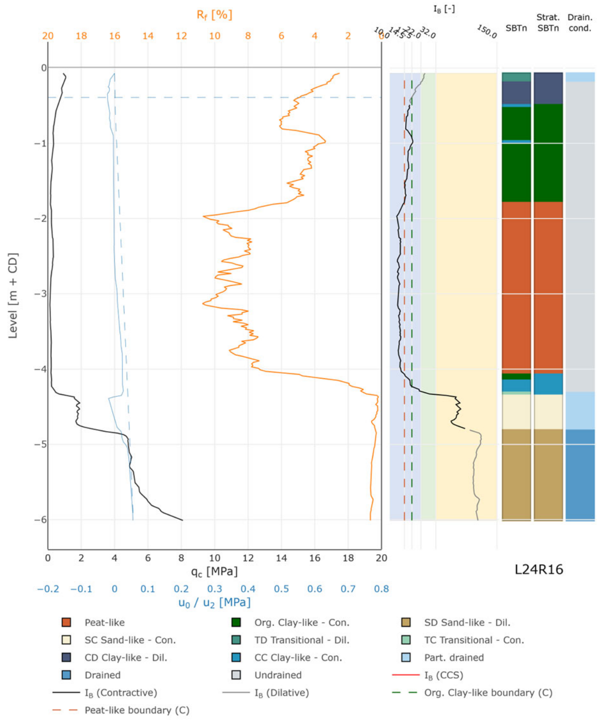

Figure 2. Prior CPT LKMP83 with SBT classification, stratification and drainage state (Lengkeek, 2024).

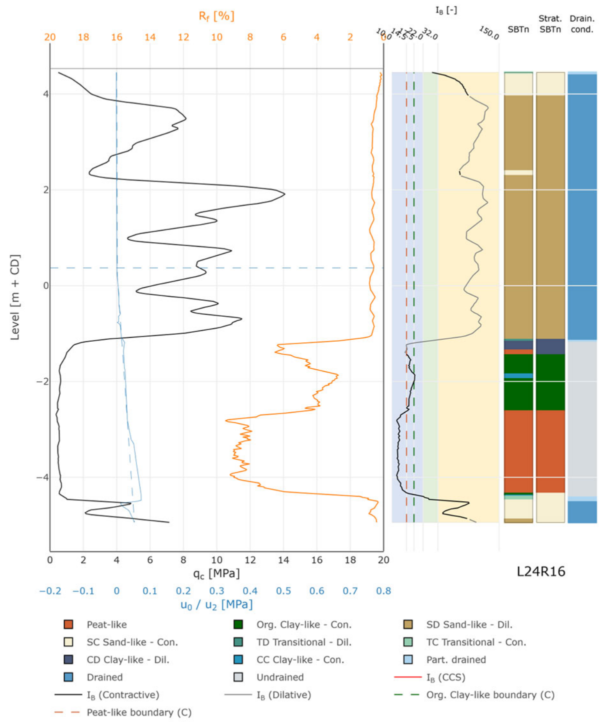

Figure 3. CPT LKMP83C in embankment with SBT classification, stratification and drainage state (Lengkeek, 2024).

3.2 *Laboratory tests*

An extensive laboratory test programme was undertaken comprising classification tests, compression tests (K0CRS), anisotropic consolidated undrained Triaxial tests (CAUC) and

Direct simple shear tests (DSS). No triaxial extension test are performed because of the mixed experiences in previous testing campaigns on organic soils. The tests were initially performed for organic clay layer 3 and peat layer 4. After completing the investigations, for specific CPTs a sensitive soft clay layer '3a' was identified. The few tests on this thin (<0.5 m) and local (<30 m) layer all show very low strength parameters. On the other layers, limited tests have been performed as these are less relevant for the back-analysis.

The advantage of CAUC tests is that all stress components are measured, whereas in the DSS this is not the case. The selection of the ultimate strength for CAUC tests, however, is more complicated than for DSS tests. This is related to the applied corrections and large variation in stress paths. CAUC tests require membrane and filter corrections. For these tests this implies that the deviator stress is reduced by approximately 2 kPa, which is still relevant for low strengths of typically 10 kPa at in-situ stress levels.

Furthermore, a geometrical correction is normally applied, to triaxial tests related to the average cross section area as a function of the vertical strain. There are two methods defined in (Head, 1998) to calculate the geometrical correction (area correction). The first method is called the 'barreling' correction. In an undrained test it is assumed that the volume remains unaltered. This method accounts for the increasing area with increasing axial strain. It is assumed that the increase in area results in a decrease of stress compared to the original area and diameter. This method has been applied to all samples of Eemdijk by the laboratory as default method. The second method is called the 'slip plane' correction. When failure occurs along an inclined plane, the effective overlapping area of the elliptical surfaces decreases with increasing strain. It is assumed that the decrease in area results in an increase of stress. The correction works in fact opposite of the 'barreling' correction.

The photos of the Eemdijk CAUC test samples show in most cases a combination of both failure mechanisms, sometimes more 'barreling' and sometimes more 'slip plane'. The difficulty with applying these corrections is that the 'barreling' correction should start from the initial strain, whereas the 'slip plane' correction should start from the point the slip begins.

Moreover, it is questionable whether the CAUC test should be interpreted based on large strains and updated geometry and stresses, whereas the numerical simulations are not performed with a large strain analysis. Besides, vertical stress correction is based on the change in area, but does not account for eccentricity, stress rotation and accompanying radial and tangential stresses. Therefore, it is decided to perform two CAUC interpretations for this research, one with the default geometrical correction by the laboratory ('barreling'), and one without any geometrical correction.

Figure 4 shows a comparison for three samples with and without geometrical correction. The effect of the geometrical correction on the undrained shear strength and normalized undrained shear strength ratio (S) is significant, whereas the friction angle is affected to a lesser extent. The S-ratio is defined in equation 1:

$$S = \left[\frac{s_u}{\sigma'_{v,con}}\right]_{nc} \qquad (1)$$

The best estimate strength parameters for the back-analysis of the Eemdijk test are based on the average ultimate value including geometrical correction. The upper bound strength parameters for the back-analysis of the Eemdijk test are based on the peak strength without geometrical correction, while maintaining the default membrane and filter corrections.

The ultimate strength as applied for design of dikes should in line with the Dutch guidelines (POVM, 2020) be selected at 25% strain. Defining the ultimate strength at 25% axial strain has two disadvantages. On one hand, some NC samples show significant softening, even when the pore pressures are stabilized, resulting in a low undrained shear strength and S-ratio. On the other hand, some OC samples show significant hardening, even with negative excess pore pressures, resulting in a high undrained shear strength and S-ratio.

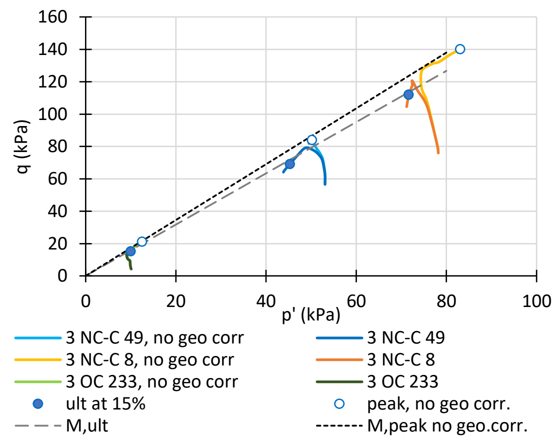

Figure 4. CAUC results organic clay layer 3: comparison of stress paths for three samples with and without geometrical correction.

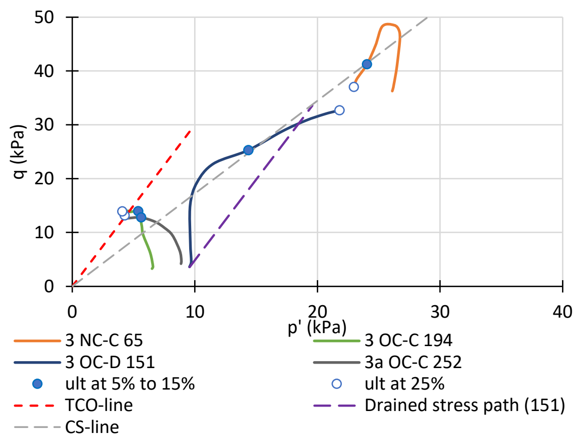

Figure 5. CAUC results organic clay layer 3: examples of strongly deviating stress paths.

Examination of the CAUC test of clay layer 3 indicates that 3 out of 9 tests show significant deviating stress paths. These 3 tests have a significant effect on average and standard deviation. After careful evaluation of the tests, it is proposed to define the default ultimate strength for a CAUC test more in line with international practice at 15% axial strain value, see sample 65 in Figure 5. In addition, two requirements are applied. The additional conditions are presented in Table 4 and elaborated with the help of Figure 5.

Sample 194 and 252 are consolidated at in-situ stress and show contractive behavior ending up in zero horizontal effective stresses. For such contractive stress paths, it is proposed to select the strength before the tension cut-off line is reached [σ'3>0].

Sample 151 is also consolidated at in-situ stress level and shows significant dilative behavior with continuous hardening along the M-line resulting in negative excess pore pressures. For such dilative stress paths, it is proposed to select the strength before the excess pore pressure become negative [Δu<0].

The resulting parameters for layer 3 are presented in Table 2. The ultimate friction angle for the organic clay is typically 39°, similar to the DSS value shown in Table 3. The ultimate S-ratio for organic clay is typically 0.42, similar to the DSS value shown in Table 3. The upper bound effective strength parameters are about 5 to 10% higher, mainly caused by applying the peak strength criterion. The upper bound undrained shear strength parameters are 20 to 25% higher, mainly caused by not applying the geometrical barreling correction.

Table 2. CAUC results organic clay layer 3.

Strength criterion	ϕ' (°)	S=[s_u/σ'_{vy}] (-)
ultimate (ε=15%)	38.8	0.42
peak & without geometrical correction	42.0	0.52

Table 3. DSS results organic clay layer 3.

Strength criterion	ϕ' (°)	S=[s_u/σ'_{vy}] (-)
ultimate (ε=40%)	38.3	0.41
peak (ε≈25%)	35.6	0.42

From the DSS tests it can be concluded that the NC samples show contractive behaviour, and the OC samples show a slightly dilative response. The peak strength is reached at about 25% shear strain and the amount of softening is marginal. The normally consolidated S-ratio and effective friction angle are determined from the NC samples by Ordinary Least Squares (OLS) regression through the origin, in line with the model application (no cohesion). The S-ratio follows from the ultimate shear stress and vertical consolidation stress. The effective friction angle follows from the ultimate shear stress and ultimate normal stress. The definitions of the friction angles used here are presented in equation (2) and (3):

$$\sin\phi'_{tx} = \left[\frac{t}{s'}\right]_{nc} \;;\; (t = s_u) \tag{2}$$

$$\sin\phi'_{dss} \cong \left[\frac{\tau}{\sigma'_v}\right]_{nc} \;;\; (\tau = s_u) \tag{3}$$

Multiple definitions exist for the friction angles from DSS tests, as the boundary conditions are not exactly known. However, the selected definition provides consistent results with numerical simulations of soil tests.

The new proposed criteria result in more realistic strength parameters, both drained and undrained, with less variation. The latter also results in less conservative characteristic values. It is recommended to use the criteria as summarized in Table 4 for ultimate strength parameters determination based on triaxial tests.

Table 4. Recommended criteria for Triaxial ultimate strength parameter selection.

Stress path	Primary ε_u (%)	Secondary ε_{max} (%)
Contractive behavior at failure (NC samples)	15	σ'3 > 0 before the TCO-line is reached
Dilative behavior at failure (OC samples)	15	Δu > 0 before the zero excess pore pressure line is reached

4 PARAMETER DETERMINATION

4.1 *Field tests*

In this section it will be investigated by which parameter-set the stability of the ground dike test (GD) is best approached. The back-analysis is compared to the measurements at specific monitoring instrumentation locations, see Figure 6. The locations are identified by [A, …, I].

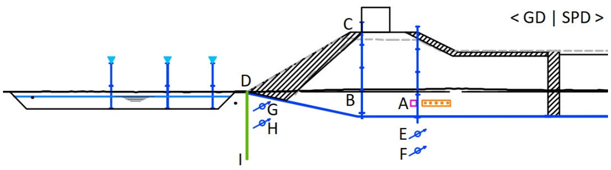

Figure 6. Cross section of the GD with locations of monitoring instrumentation.

4.2 *FEM calculations*

All phases of the back-analysis are modelled in a large deformation analysis (Updated Lagrange) with pore pressure update, to account for the large initial deformations prior to failure test phase. The construction calculation phases consist of a mixture of alternating undrained effective stress calculations (in which the loading is instantaneously applied and excess pore pressures are calculated) and consolidation calculations (in which excess pore pressures are dissipated in a certain time interval). The failure test calculation phases are all considered undrained without intermediate consolidation during these seven days.

Various constitutive models have been analyzed in the back-analysis. Two of these back-analysis models are reported in this paper. The basic parameter set for each constitutive model is based on the best estimate (average) values derived from the laboratory test results and numerical simulations, as presented in (Lengkeek, 2022).

The clay and peat layers are modelled by the Soft Soil Creep model during construction and switched to the SHANSEP NGI-ADP model during the test phase. The SHANSEP NGI-ADP model is the prescribed model in Dutch guidelines for FEM modeling of dikes (POVM, 2020), whereby the prior effective stress state is used to derive the undrained shear strength, as used in the NGI-ADP model, according to the SHANSEP equation.

The following FEM back-calculations are here compared with the actual measurements of both failure tests:

- c02: best estimate parameters and SHANSEP NGI-ADP model with isotropic strength. Ultimate strength parameters are determined in line with dike design guidelines in The Netherlands.
- c07: best fit parameters and SHANSEP-NGI-ADP model with isotropic strength. Best fit is based on a strength increase such that in 2D the maximum loading conditions can be calculated without premature failure. The strength of the Holocene layers is increased with a fit factor of 1.15. The increase of 15% is still less than the upper bound undrained shear strength parameters which are 20 to 25% higher, mainly caused by not applying the geometrical barreling correction.

4.3 *4.3 Model performance*

This section presents the results of the back-analysis of the GD. The performance of the FEM back-calculations are compared with the monitoring results and with each other. Figure 7 presents the phase displacements of FEM back-calculation c07. The deformation mode complies well with the reconstructed failure mode based on observations as presented in Figure 8. The reconstruction is based on the monitoring measurements and photo’s in Figure 9 and 10. The deformation mode is similar for all four back-analyses with a failure surface in the soft clay layer 3, more specific in sensitive soft clay layer 3a, just above the peat layer 4. This shows that peat can have high strength.

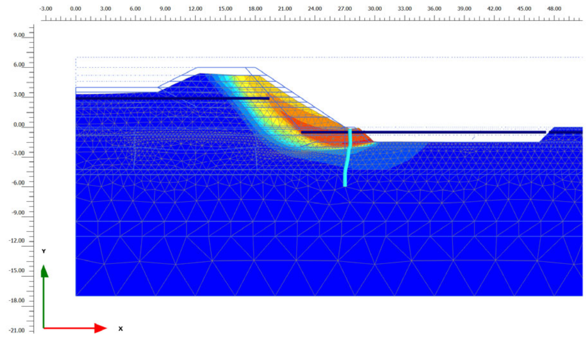

Figure 7. Ground dike, phase displacements prior to failure. The shading is an indication of the relative displacement (red is large, blue is small). The turquoise line is the modelled inclinometer.

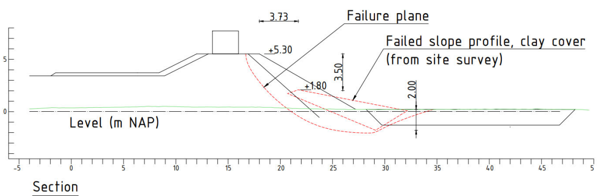

Figure 8. Reconstructed failure surface of the GD with deepest point at 2 m depth just above the peat layer 4.

Figure 9. Aerial view of the Eemdijk test embankment with the failed slope of the GD on the left side (Lengkeek, 2022).

Figure 10. Left: Failure surface at the embankment in the sand layer 1a indicated by the dashed lines. Right: failure surface at the toe in soft clay layer 3a just above peat layer 4 (Lengkeek, 2022).

Figure 11 to Figure 13 present the back-calculations in addition to the measurements. The water pressures are presented in Figure 11. The settlement of the crest is presented in Figure 12. The horizonal displacements at the toe are presented in Figure 13.

It is concluded that the back-analysis c02 overestimates the displacements at any moment in time, with a premature instability. With the increased strength properties of FEM back-calculation c07, failure occurs in the correct stage with the correct loading and with a slight overestimation of the displacements.

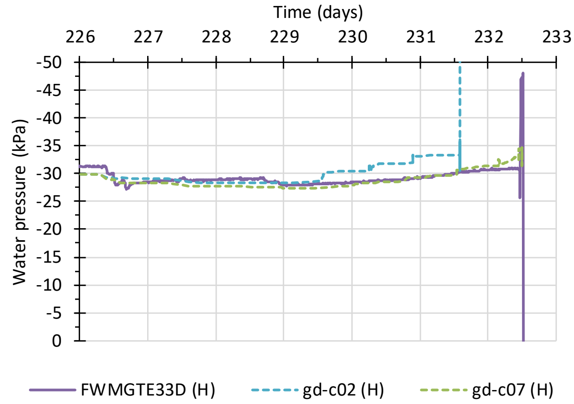

Figure 11. Pore pressure measurements (FW..) and back-analysis (gd..), at 3.0 m depth in the peat layer at the toe (location H).

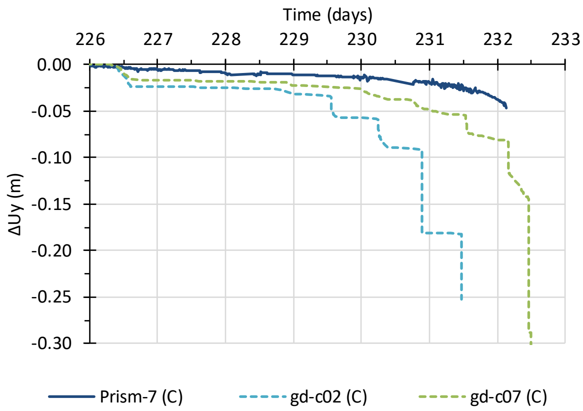

Figure 12. Measurements (Prism..) and back-analysis (gd..) of vertical displacements, at the crest (location C).

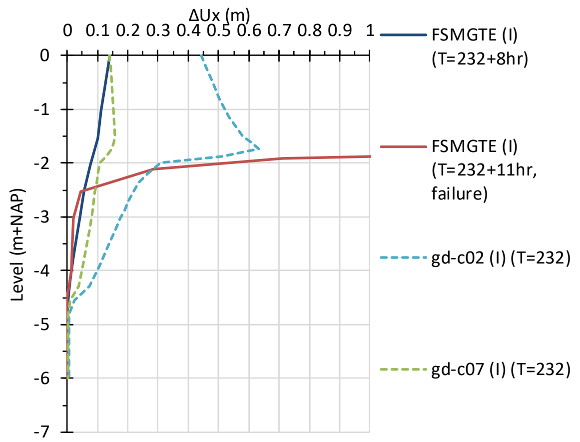

Figure 13. Inclinometer measurements (FS..) and back-analysis (gd..) of horizontal displacements over depth, at the toe. Last loading stage prior to failure with water level +2.5 m NAP. The location of the sliding surface at -2 m NAP is well captured by the FEM back-calculations.

5 CONCLUSIONS

The Eemdijk full-scale failure tests provides valuable insights through a detailed analysis of the deformations of dikes leading up to and beyond failure.

The new CPT-based classification methods used here show that it is possible to correctly classify the type of layer despite the difference in stress and state. This is an important starting point for the modelling of the embankment and the back-analysis of the failure test.

The dike design guidelines for the Netherlands currently prescribes the SHANSEP model for undrained analysis during a dike's functional lifetime.

The best fit isotropic parameters underestimated the stability and overestimated the displacements at all load stages. This corresponds to about 15% underestimation in strength of the soft soil layers.

Based on a careful examination of the CAUC test it is recommended to revise the current Dutch guidelines for the ultimate undrained shear strength determination, and to reexamine the applied geometrical corrections. It is recommended to use the 15% axial strain value as a basis for the ultimate value and to apply the following criteria to prevent unrealistic high or low undrained shear strength values:

For overconsolidated samples and contractive stress paths it is proposed to select the strength before the tension cut-off line is reached ($\sigma'3 > 0$) and the minor effective stress becomes zero.

For overconsolidated samples and significant dilative behavior resulting in negative excess pore pressures it is proposed to select the strength before the zero excess pore pressure line is reached ($\Delta u > 0$) and the excess pore pressures become negative.

It is shown that the CAUC undrained shear strength determined without geometrical correction can results in approximately 20% higher undrained shear strength. This results in a higher undrained shear strength for CAUC tests than for DSS tests, which is in line with the well accepted ADP concept. It is recommended to conduct further research on how and whether this geometrical correction should be applied.

6 ACKNOWLEDGEMENTS

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