

# Back-analysis of Eemdijk full-scale failure test on sheet pile reinforced dike

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**ABSTRACT:** The Netherlands is inherently challenged by the sea as a major part of the country lies below average sea level. 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 an extension of its base, which is frequently restricted by the presence of existing buildings and other spatial constraints. These dikes can be reinforced by alternative means such as a sheet pile wall. This paper focuses on improving on the global stability assessment and modeling challenges of dike reinforcement using sheet piles. The models are validated on the full-scale failure test (In Dutch: ‘Eemdijk damwandproef’), initiated by the Dutch Flood Protection Programme. The back-analysis confirms the validity of the constitutive models and close approximation of the field measurements. Despite the good fit, it is concluded that there is still room for improvement.

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

## 1 INTRODUCTION

The delta of the Netherlands is inherently challenged by water. Subsidence, sea level rise, and the increase of rain intensity and river discharge due to climate change further challenge existing river dikes 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 an extension of its base, which is frequently restricted by the presence of existing buildings and other spatial constraints. These dikes can be reinforced by alternative means such as a sheet pile wall. This paper is based on the PhD research (Lengkeek, 2022) and focuses on improving the global stability assessment and modeling challenges of dike reinforcement using sheet piles. The models are validated on the full-scale failure test. (In Dutch: ‘Eemdijk damwandproef’), initiated by the Dutch Flood Protection Programme.

## 2 SHEET PILE REINFORCED DIKE TEST

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)

The focus of the Eemdijk full-scale failure test is on the SPD-test. 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 and SPD-test are 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 Figure 2. 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, which are representative for a sheet pile reinforced dike as schematically shown in Figure 1. The primary loading and reduction of strength follow from the increased water pressures (A). The secondary loading follows from surcharge (C). The sheet pile wall (B) is unanchored and embedded in the deeper sand layers. The corresponding failure mechanism as shown on the right side is a combination of active/passive wedges and a slip circle. In practice also shorter sheet piles supported by inclined anchors can be used to reinforce dikes.

The sheet pile wall configuration consists of a staggered wall with alternating panels of 18 m and 9 m length. The panels consist of triple U-profiles. A staggered wall is selected because a continuous wall was expected to be too strong to fail. All panels are connected by the sheet piles interlocks. The structural properties of the sheet piles are presented in Lengkeek (2022). The monitoring instrumentation on the sheet pile wall comprises of glasfibers (GFSG), inclinometers (SAAF) and Prisms.

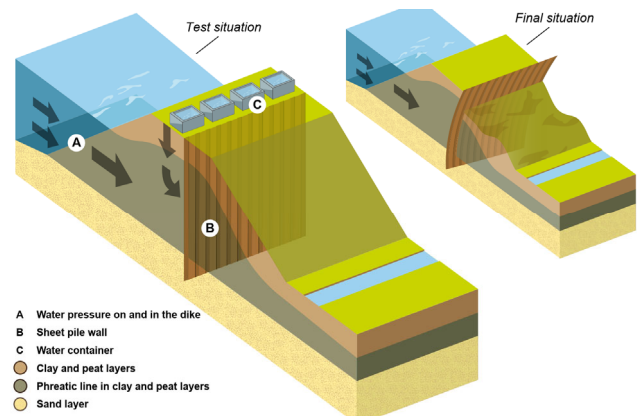


Figure 1. Schematic loading conditions of a sheet pile reinforced dike [Figure courtesy Bredeveld et al. (2019)].

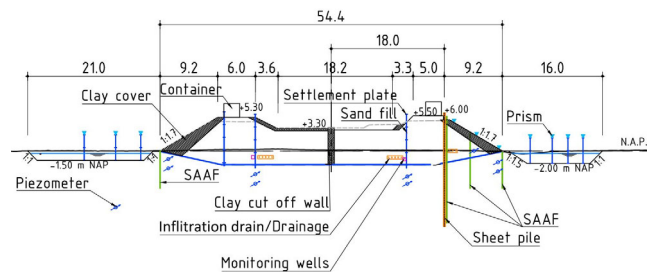


Figure 2. Cross section of ground dike (left) and sheet pile dike (right) with monitoring instrumentation.

### 3 BACK-ANALYSIS

#### 3.1 Introduction

This section presents the results of the back-analysis of the SPD. The performance of the four FEM back-calculations is compared with the monitoring results and with each other. The back-analysis is compared to the measurements at specific monitoring instrumentation locations, see Figure 3. The locations are identified by [A, ..., M].

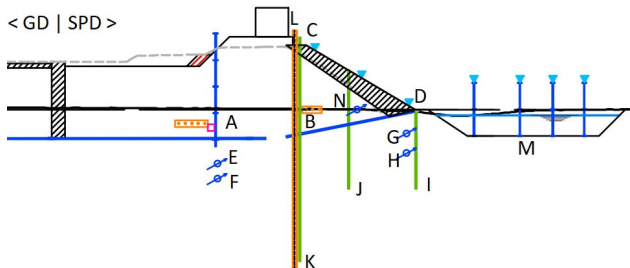


Figure 3. Cross section of the SPD with locations of monitoring instrumentation.

#### 3.2 Construction phase

The embankment and subsoil showed significant construction induced displacements, vertical up to 1.0 m and horizontal up to 0.4 m without failure. These deformations are in line with expectations given the embankment height of 5 m on soft subsoil. The construction induced deformations prior to the test occurred over a long period (6 to 9 months) with partial consolidation.

The back-analysis is performed with FEM program Plaxis. The construction phases of all FEM back-calculations are performed with the Soft Soil Creep (SSC) model for the Holocene layers. The hydraulic parameters of the SSC model are adjusted such that during and in particular at the end of construction pore pressures match the measurements, as this determines the actual stress state and strength during the test phase.

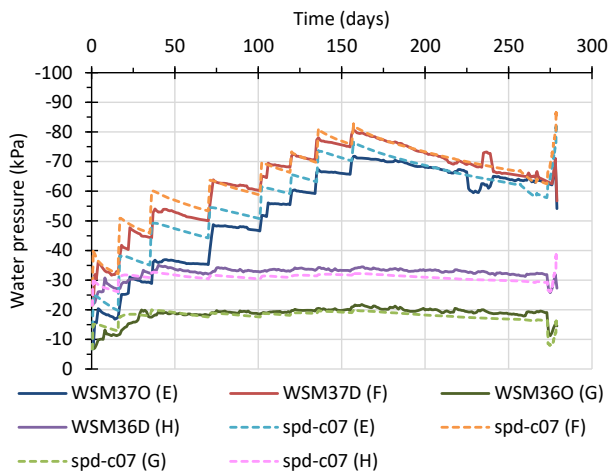


Figure 4. Measurements (WS..) and back analysis (spd-c07) of pore pressure, at 1.5 m depth in the clay and 3.0 m depth in the peat, at the toe and crest.

Figure 4 presents the pore pressures during construction phase and failure test phase. The comparison of the construction phase is only shown for FEM back-calculation c07, as the construction phases are the same for all FEM back-calculations. The solid lines are the measurements and the dashed lines the back-analysis results. Based on these figures it is concluded that

the excess pore pressures are correctly modelled. The initial response deviates as is often seen with embankment on organic soils. For the back-analysis it is more to get the correct excess pore pressures at the start of the full-scale failure test.

#### 3.3 Failure mode

The total duration of the failure test was six days, see Table 1. The total deformations in the first five days are small, generally less than 5 cm and only start to increase significantly in the last day. The maximum water level prior to failure was +5.0 m NAP. Failure started to be visible with active wedge cracks on the crest behind the sheet pile the day before failure.

Based on the evaluation of the SPD measurements it is concluded that failure of the slope (see Figure 5) occurs prior to the buckling failure of the sheet pile. The latter is considered the point where the integrated system of the sheet pile reinforced dike fails.

The rate of deformation increased rapidly after 10 cm displacement, up to a horizontal displacement at the crest of about 60 cm and at the toe of about 35 cm. At this deformation the sheet pile failed which caused a collapse of the SPD and ultimately a flooding. The total duration of the collapse was about 7 minutes. The deformations stopped after the top of the sheet pile was deflected to almost 45 degrees.

To determine the deformations prior to slope failure, the ‘moment of failure’ needs to be defined. This is defined as the moment in time when progressive failure under constant loading is permanent (irreversible). This moment is based on the evaluation of the measurements presented in Figure 6. From this evaluation, the moment of failure is set to a horizontal displacement rate of 1 cm per hour at the toe of the dike. This moment coincides with a sharp increase in displacement rate, and with the occurrence of visual cracks in the slope. The same rates and moment of failure is found for the ground dike test (GD).

Table 1. Stratification at the ground dike.

SPD-test phase	Start time	day	duration
Excavation ditch -2 m NAP, 7 m width	3/12/18 8:00	273	24:00
Excavation ditch -2 m NAP, 15 m width	3/13/18 8:00	274	9:00
Water level fill to +2 m NAP	3/13/18 17:00	274	24:00
Water level fill to +3 m NAP	3/14/18 17:00	275	17:00
Water level fill/basin to +4.2 m NAP	3/15/18 10:00	276	22:00
Water level container 1 m	3/16/18 8:00	277	8:00
Water level container 2 m & slope +1.0 m NAP	3/16/18 16:00	277	18:00
Water level fill/basin to +5.0 m NAP	3/17/18 10:00	278	3:00
Water level ditch -0.9 m NAP & slope +1.5 m NAP, start slope failure	3/17/18 13:00	278	3:00
Failure sheet pile, collapse, end SPD-test	3/17/18 16:00	278	

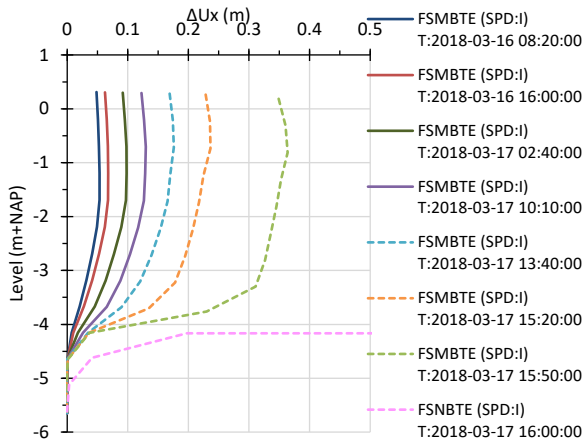


Figure 5. Inclinometer measurements at the toe (SAAF location I). The final eight measurements are presented starting 1 day prior to failure. The last measurement is as after failure and not reliable due to excessive deformations.

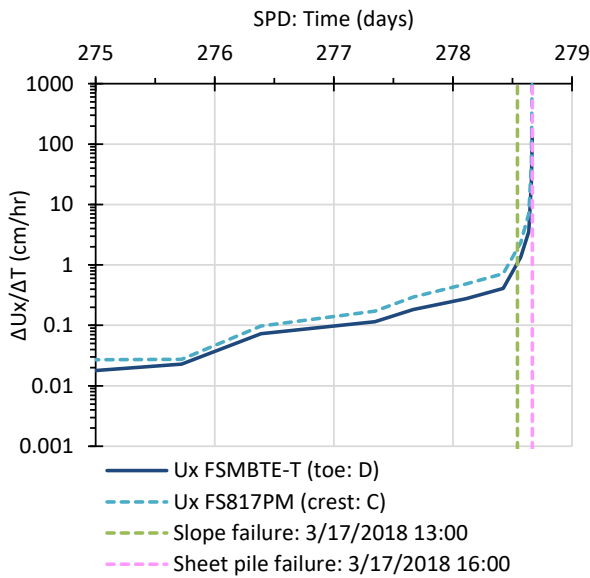


Figure 6. Horizontal displacement rate of the SPD during the last days of failure test. The displacement rate at the crest and toe are derived from the top of the inclinometer measurements.

Figure 7 presents the phase displacements of FEM back-calculation c07. The deformation mode complies well with the reconstructed failure mode as presented in Figure 8. The reconstructed failure mode is based on all the measurements and post failure 3D radar scans and excavations, as illustrated in the photos of Figure 9 and 10. Figure 8 shows that on the passive side the exit point of the slip circle is pushed up more than 2 m. At the toe, the failure surface is at the boundary between the peat layer and sand layer. Towards the sheet pile there seems to be no single failure surface, but a failure zone indicated between the two dashed lines. The maximum inclination of the sheet pile is 42 degrees. The failure point of the sheet pile is approximately at -3.5 m NAP. On the active side an area of active wedges developed.

The deformation mode is similar for all four FEM back-calculations. The failure of the sheet pile (exceedance of the maximum bending moment capacity followed by a plastic hinge) occurred in all FEM back-calculations at the same level followed by a global failure of the dike.

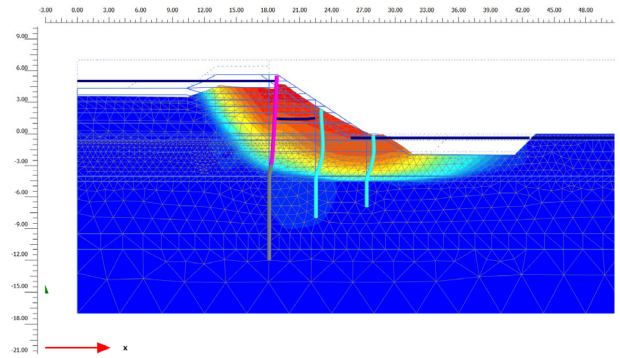


Figure 7. FEM calculation results with shaded plot of phase displacements prior to failure (red is large, blue is small). The magenta line is the continuous sheet pile and the grey line is the staggered part. The turquoise lines are the modelled inclinometers.

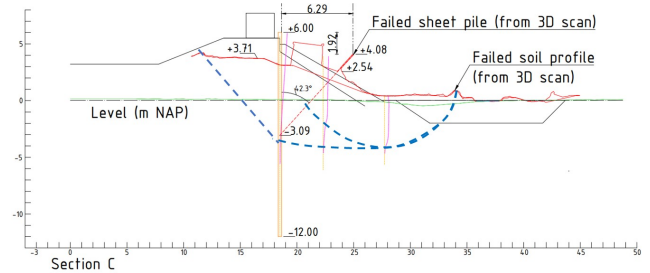


Figure 8. Reconstructed failure surfaces from 3D scans and surveys.



Figure 9. Photo at the moment of failure (16:00 hr.), showing the pushed-up slip circle, the sheet pile deflection and overtopping.



Figure 10. Left Side view of the SPD failure mode after draining the ditch, with clear indication of the exit point of the slip circle. Right Top/back view of the sheet piles after failure and local excavation, the dashed line indicates the inclination of the sheet pile. All interlocks of sheet piles are still intact.

### 3.4 FEM calculations

All phases of the back-analysis are modelled in Plaxis 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. Four 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.

After the construction stage, the constitutive models are switched to another constitutive model to simulate the failure phase. The following FEM back-calculations are performed and 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 1.15.
- **c03**: best estimate parameters and SHANSEP NGI-ADP model with anisotropic strength. The DSS strength is the same as in FEM back-calculation c02, but the Active loading strength is based on the Triaxial strength without geometric correction and the Passive loading strength is assumed to follow the ratio A : D : P is 1.0 : 0.8 : 0.67.
- **c09**: best estimate parameters and Creep-SClay1S effective stress model with anisotropic strength. The strength parameters are fitted both on the Triaxial strength without geometrical correction and the DSS strength.

The strength parameters of FEM back-calculation c07 are increased by a factor of 1.15. The primary reason to do this is to be able to calculate all loading stages and reach failure at the correct maximum loading condition. Nonetheless, a higher stability can be justified by two following two contributions:

- In reality failure of dikes is three dimensional, this is clearly seen in the aerial view of the GD failure. The ‘3D effect’ is not accounted for in a 2D analysis but can be simulated by increased strength properties. The 3D factor is in the range of 1.1 to 1.2. The back-analysis of the GD-test shows that a factor of 1.15 is required to achieve the required stability. A factor 1.2 would overestimate the equivalent strength and would not result in failure at the maximum loading stages.
- The second reason to perform back-analysis c07 with a higher strength is to verify whether this factor is lower than the ratio between the ‘Triaxial strength without geometrical

correction’ and the ‘DSS strength’. This factor is about 1.25, which is higher than the required factor of 1.15 to reach the same stability in the back-analysis. This has led to the other two anisotropic strength FEM back-calculations, c03 and c09, with explicit modeling of DSS and Triaxial compression strength.

### 3.5 Model performance

Figure 11 to Figure 13 present the four back-calculations in comparison to the measurements. In Figure 11 the vertical displacements of the top of the sheet pile are presented. In Figure 12 the horizontal displacements at the toe are presented. In Figure 13 the horizontal displacements of the sheet pile are presented. Figure 13 shows that the location of the plastic hinge in the sheet pile at about -3.5 m NAP is well captured by the FEM back-calculations. The indication T=278.5 in the legend refers to day 278+12 hrs.

It is concluded that the back-analysis c02 overestimates the displacements at any moment in time, with a premature instability. The actual global stability is underestimated with the current approach and 2D modelling.

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. 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.

The performance of FEM back-calculation c03 with anisotropic strength is in between FEM back-calculation c02 and c07. Apparently, the increase in Active strength is more beneficial than the decrease in Passive strength.

The performance of FEM back-calculation c09 with the anisotropic Creep-SClay1S model is even slightly better than c07.

Both the vertical and horizontal construction induced displacements are well captured by the Soft Soil Creep (SSC) model. Alternatively, the CSC1S model can be used as it also accounts for creep. The HS model can be used, but it requires a correction of the compression stiffness to account for creep.

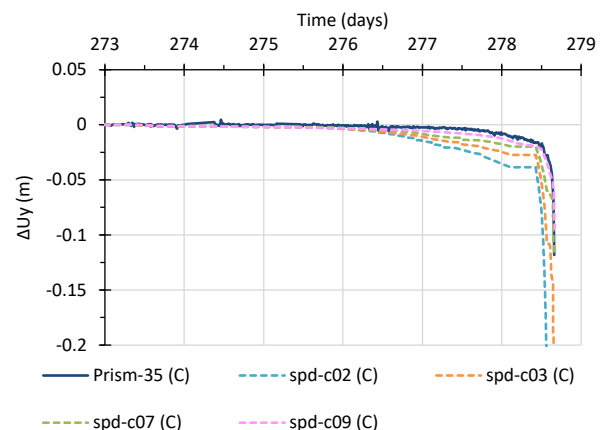


Figure 11. Measurements (Prism..) and back analysis (spd..) of vertical displacements at the crest (location C).

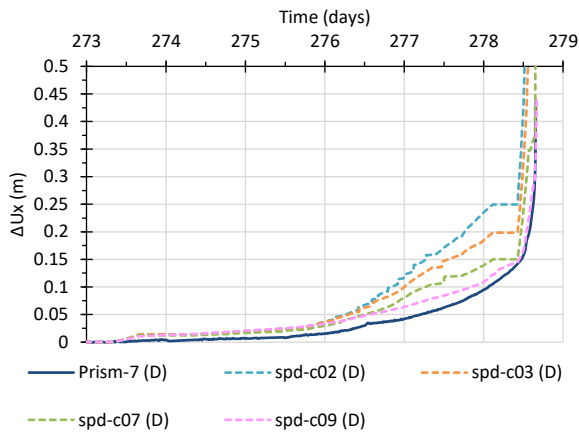


Figure 12. Sheet pile dike, measurements (Prism-7) and back analysis (spd..) of horizontal displacements, at the toe (location D).

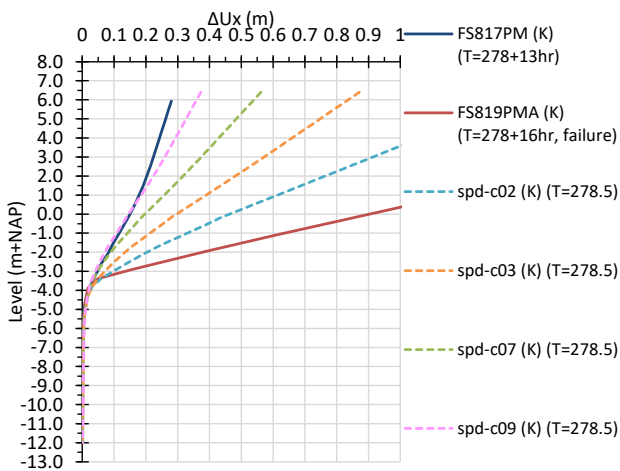


Figure 13. Measurements (FS..) and back analysis (spd..) of horizontal displacements of the sheet pile (SAAF location K) in last loading stage prior to failure with water level +5.0 m NAP.

The back-analysis of the pore pressures in the embankment fill and subsoil are well aligned with the measurements. This is crucial to obtain correct effective stresses, state and undrained shear strength in the SHANSEP model. The permeability parameters, although primarily based on the K0-CRS test, are iteratively determined as they are constitutive model dependent.

The SPD-test back-analysis confirms the location of the failure surface at the toe in peat layer 4 at 4 m depth. Below the slope and in front of the sheet pile, the failure surface changes to a failure zone over the height of the peat layer 4. Behind the sheet pile an active wedge occurs.

The back-analyses show the sheet pile deformations and structural forces are well captured by the FEM analysis where SSI is properly taken into account. Based on the measurements and the FEM back-analysis it is concluded that the complex failure mechanism and the SSI of a dike with sheet pile should not be modelled in a LEM analysis, which considers either a slip circle analysis of a slope, or an active/passive wedge analysis of a sheet pile wall.

The displacements following the back analysis models are all larger than the measurements during the field test. The vertical displacements of the back analysis of the SPD are better aligned as these are very low, due to the interaction with the sheet pile. FEM back-calculation c02 deviates the most due to early failure following the lower strength compared to the other

models. 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. For the SPD-test it is concluded that FEM back-calculation c09 (CSC1S) even performs better than c07.

The anisotropic strength models with the increased Active loading strength and decreased Passive loading strength (ADP), perform better than the isotropic strength model. The effect on the increased stability is about 10%. This would explain at least half the difference between the back-analysis and the actual measurements. This is an improvement compared with the current approach in the Netherlands. In order to implement this in dike design practice, the interpretation of the CAUC tests with respect to the geometrical correction should be re-examined. Applying a higher CAUC strength makes sense as this agrees with literature and consensus on anisotropic strength. The current applied geometrical correction, in particular when combined with 25% strain level seems too conservative.

A more extensive overview with comparison of the FEM back-calculations is presented in Lengkeek (2022). In general, it can be concluded that the direction of the deformation and the relative deformations match very well with the measurements.

#### 4 ROBUSTNESS OF SHEET PILE REINFORCED DIKE

The slope instability of the SPD causes a significant displacement of the inboard slope and deflection of the sheet pile. This is the first and initiating failure mechanism, and can be regarded as element failure, an exceedance of the geotechnical Ultimate Limit State (ULS). Before reaching a new equilibrium the second element failure occurred, by exceeding the ULS of the sheet pile strength. The second failure resulted directly in the system failure and lowering of the top of the sheet pile, and basically the complete crest.

An interesting finding from the Eemdijk full-scale test is that a continuous sheet pile placed in the crest, with sufficient length and embedment, also makes an important contribution to the stability of the dike after failure. Although the sheet pile fails, the tensile and shear force capacity of the sheet pile wall remains sufficient to create a system of two sheet pile parts, connected by a hinge. This keeps the lower part of the sheet pile in place, while allowing the top of the sheet pile to deflect and find equilibrium with the failed inboard slope. This creates a local crest lowering which acts as a weir as the deflected sheet pile top level was 1 m below the maximum water level. The lowering of the top of the sheet pile caused a flooding due to overflow in the middle section of the wall. The sheet pile wall acted as a weir and partial flooding occurred. The sheet pile wall prevented however a breach and uncontrolled flooding.

During the tests, all sheet piles remained interlocked, even after failure. All sheet piles are removed after the test. From this inspection it is concluded that only the outer panels on each side did not fail, all other sheet pile panels failed. The failure is caused by local buckling of the compression flanges. The total elongation at the top determined from the 3D scan is 1.8 m on a total length of 61 m, which is 3%. This elongation is caused by plastic yielding in the corners between flange and web. The required force to create yielding is typically 20 kN per vertical meter, based on sheet pile properties. The membrane action induced longitudinal tension force is thus limited by this strength. This capacity of the Larssen interlocks is typically 100 kN/m<sup>2</sup>, which is much larger. This explains the observation that all sheet piles remained connected by the interlocks. This is an important finding which is not considered in any guideline for the application of sheet piles along dikes. It also is an argument for installing continuous walls and for not placing walings in

unanchored walls, as they cannot take over the membrane action because of the rigidity in longitudinal direction.

The maximum deformation of the failed sheet pile is thus stopped by the resistance of the failed soil mass in front of the sheet pile, which is pushed upward, combined by the membrane action in the sheet pile. In addition, 3D side friction effects are expected too. The Eemdijk experiment shows that a continuous cantilever sheet pile wall, placed in the crest, enhances the robustness.

In the context of codes and standards for structural design such as the Eurocode, the term robustness has been used to indicate the ability of a structural system to resist damage under extreme loads. In Stochino et al. (2019), robustness is described as the structure's ability to avoid disproportionate collapse due to an initial damage. Some recommendations they provided to enhance robustness are alternative load paths and redundancy at various levels, i.e., material level, member level, and system level. System failure requires retrogressive failure which ultimately results in a breach and flooding. The sequential failures can be assessed by a failure path analysis, for example a secondary failure surface near the outboard crest followed breaching due to seepage and erosion. The quantification, however, is associated with large uncertainties, both in terms of physical processes as probability of occurrence.

In the Eemdijk full-scale test it is observed that the SPD maintains to a large extent the ability to act as a flood defense, after initial failure of the slope as well as the sheet pile. Furthermore, in the GD-test it appears that the stability is governed by a local weak layer. A dike with sheet pile reinforcement penetrates such layers and a continuous wall allows for redistribution along the dike. Therefore, it can be concluded that dikes reinforced by a continuous sheet pile, with sufficient length and embedment, are more robust than earthen dikes.

## 5 CONCLUSIONS

The Eemdijk full-scale failure tests involves separate tests on a ground dike, as well as a combined test on a ground dike with sheet pile reinforcement. The sheet pile improves the stability and robustness of the dike. The maximum water level at the SPD is about 2.1 m higher than at the GD (+5.0 m NAP versus +2.9 NAP), which is a considerable increase. In the Eemdijk full-scale test it is observed that the SPD maintains to a large extent the ability to act as a flood defense, after initial failure of the slope as well as the sheet pile. Furthermore, in the GD-test it appears that the stability is governed by a weak local layer. A dike with sheet pile reinforcement penetrates such layers and a continuous wall allows for redistribution along the dike. Therefore, it can be concluded that dikes reinforced by a continuous sheet pile, with sufficient length and embedment, are more robust than earthen dikes.

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

Both the GD-test and SPD-test are back-calculated with the same set of input parameters and four FEM post-dictions are reported. This paper presented the results for the SPD-test. The back-analyses were able to reproduce the correct deformations and excess pore pressures for each stage by allowing consolidation. Therefore, the back-analysis of the test phase is performed with the SHANSEP NGI-ADP model based on undrained shear strength, as well as the Creep-SCLay1S model with effective stress models in an undrained analysis.

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

modelled with advanced models such as the SSC model to account for settlements. The back-analysis of the Eemdijk full-scale tests confirm that these models are also well suitable for the design of dikes with sheet pile reinforcement. It is concluded that all back-analyses show the correct failure modes for both dikes as well as the failure of the sheet pile.

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. Hence, the deformation of the sheet pile and the bending moment are overestimated too. The anisotropic strength models, with the increased Active loading strength, same Direct simple shear strength and decreased Passive loading strength (ADP), perform better than the isotropic strength model. The effect is an increased stability by 10% relative to the isotropic strength model. This would explain at least half the difference between the back-analysis and the actual measurements. The combined effect of the anisotropic strength and 3D failure mode explains the difference between the actual measured and back-calculated stability and displacements using best fit average parameters. The 3D failure mode includes side friction effects which increases the stability compared to a 2D failure mode.

The anisotropic constitutive models (SHANSEP-ani, CSC1S-ani) are capable to fit both the DSS and uncorrected CAUC test well, with one set of parameters. The back-analysis of the Eemdijk SPD-tests shows that these models perform better than the isotropic models. It is recommended to investigate the possibilities for application of anisotropic constitutive models for dike engineering projects, as this results in a more realistic assessment of the stability and a more economic dike design.

## 6 ACKNOWLEDGEMENTS

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