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# How to Determine the Phreatic Surface in a Dike During Storm Conditions with Wave Overtopping: A Method Applied to the Afsluitdijk

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**Abstract.** The most famous dam of the Netherlands, the Afsluitdijk, is in need of renovation. Hydraulic conditions for the year 2050, for some aspects even 2100, must be considered during the renovation of the dam originally constructed 1927-1933. A combination of sea level rise due to climate change and a change in the applied statistical methods result in design conditions that are, at some locations, meters higher than those used a century ago. In a mission to retain the required level of safety while keeping expenses at a minimum, several innovative designs are considered; one of which is to allow very large overtopping discharges by making the dam resilient to erosion. However, calculations have shown that the overtopping values can be over 10 litres per second per meter dam, and the effect of this overtopping on the Afsluitdijk is unknown.

In Dutch national design guidelines for dikes, schematizations are proposed for the phreatic surface in dikes during storm conditions. Wave overtopping is mentioned to have a rising effect on the phreatic surface, but clear methods to determine the magnitude of this effect are not prescribed. The guidelines suggest assuming full saturation of the dam in severe overtopping conditions, which has a detrimental effect on the slope stability. Recent studies and large scale infiltration tests on the Afsluitdijk show that only a very limited amount of water infiltrates in the dam and that the assumption of full saturation is much too conservative in case of the Afsluitdijk.

In the present study, a finite-element model (Plaxflow) is used to model the phreatic surface in the Afsluitdijk during storm conditions, taking into account the effect of wave overtopping on the phreatic line. Parameters were determined by in-situ and laboratory investigations. To verify the model, calculations were calibrated on standpipe measurements. Wave overtopping is modelled by defining infiltration boundaries based on experiments and sensitivity analyses are conducted. The result of the study is a phreatic surface that can safely be used for the calculation of the slope stability of the levee during predicted storm conditions for the year 2050.

**Keywords.** Afsluitdijk, levee, phreatic surface, wave overtopping, FEM

## 1. Introduction

The 32 km long Afsluitdijk, which connects the north-eastern part of the Netherlands with the north-western part, protects a long shoreline along the IJsselmeer (former Zuiderzee) against high water levels and, with that, reduces the risk of flooding. The dike, that was built between 1927 and 1933, has to retain the required level of safety that has been set by the Dutch water law: the function of the dike needs to be fulfilled during storms with return periods up to 10,000 years. Nevertheless, the safety of the current structure is not sufficient for extreme conditions with a probability of occurrence of 1/10,000 per year: an upgrade is necessary.

Several studies on strengthening of the dike have been performed. One of the innovative designs that should keep expenses to a minimum, is to allow very large overtopping discharges by making the dike resilient to erosion.

### 1.1. Wave Overtopping

Wave overtopping is assumed to have a rising effect on the phreatic surface (e.g. TAW, 2004). Along with other failure mechanisms, both inner and outer slope stability need to be checked according to Dutch codes and guidelines. Both can be highly influenced by the phreatic surface in the dike. Therefore the assumed phreatic surface during storm conditions is an important aspect in dike design.

Dutch national design guidelines (e.g. TAW, 2001, Rijkswaterstaat, 2012) propose standard schematizations for phreatic lines. However, clear methods to determine the effect of wave overtopping on the phreatic surface are not prescribed and a validated calculation model is currently not available (TAW, 2004). According to the guidelines, the dike should therefore conservatively be assumed to be fully saturated during storm conditions. The consequence: insufficient stability of the inner slope of the dike and subsequently the requirement to reinforce the inner slope of 32 km dike.

In 2009, wave overtopping simulations have been performed on the Afsluitdijk to investigate its erodibility (Deltares, 2010). Overtopping discharges up to 75 litres per second per meter dike have been simulated over a period of 30 hours. Additionally, an infiltration test was performed over a dike length of 30 m wherein the inner slope was overflowed with a constant discharge during 56 hours. Measurements of the water pressures and phreatic surface in the dike during both these simulations indicated that a full saturation of the dike during storm conditions is not likely to occur. However, tests were performed on one location only, and wave overtopping and infiltration was not combined with a high sea water level that may occur during a storm. The test results can therefore not be extended to draw conclusions regarding the saturation of the dike.

### *1.2. Aim of the Research*

The aim of this study is to obtain a realistic phreatic surface schematization for the Afsluitdijk during storm conditions. It is expected that a more realistic view of the phreatic surface will lead to less reinforcement of the 32 km inner slope of the dike. The outer slope stability of the dike was found to be sufficient, even when taking into account the conservative phreatic line as described in the guidelines. Therefore, an improved schematization will not influence reinforcement investments required along the outer slope.

### *1.3. Geometry of the Dike*

The typical profile of the Afsluitdijk is constant along the length of the dike. During construction a triangular core of loam (originally a morene deposit from the North sea (Thijsse, 1972)) was used as primary material for closure of the estuary, supplemented by a body of sand after the closure was established. This body of sand is subsequently covered by a layer of loam. On the outer side (seaside) of the dike, the loam is covered by basalt blocks. On the inner side (lakeside) the loam is covered by clay and grass (figure 1).

## 2. Methods

The research is divided into a literature study, in-situ investigations and numerical modelling. The aim of the in situ investigations was to obtain the parameters necessary for the numerical modelling. Important aspects are:

- hydrological and geotechnical properties of both the dike materials and the subsurface.
- geometry of the dike, thickness of the layers of dike materials.

The numerical modelling combines all aspects to determine the phreatic line. Hydraulic storm conditions, such as water levels and wave overtopping, are implemented in the model.

### *2.1. In Situ Investigations*

CPT's and borings were used to determine the geometry and layering of the dike, including the

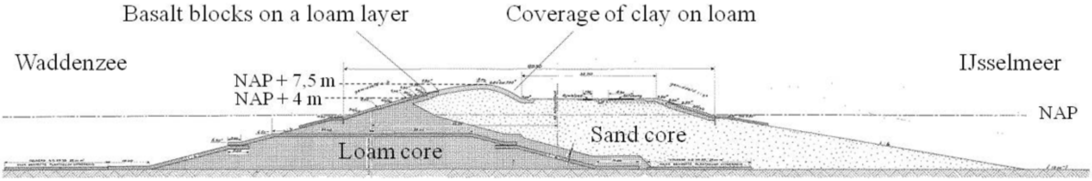


**Figure 1.** Typical profile of the Afsluitdijk. Average height of the crest is approximately NAP[1] +7.5 m, average top level of the loam core is approximately NAP[1] + 4 m. (Directie zuiderzeewerken, 1931, recent soil investigations).

Sievings were performed on samples of the sand core of the dike to determine the hydraulic conductivity of the sand by correlations. Properties of the loam core and coverage are determined by classification tests (such as Atterberg limits) and triaxial tests. The permeability of the loam is determined by double ring infiltrometer tests and Aardvark tests.

Standpipes were already installed in the past on 10 locations on the dike. The hydraulic head in the sand core was measured in four arrays , on the dike crest, on the inner slope, between the two lanes of the highway and on the lakeside of the dike. The head was measured every minute to register possible effects of tidal oscillations in the Waddensea, lake level variations and precipitation.

Double ring infiltrometer tests were carried out using a set of two steel rings with a diameter of 57 (outer ring) and 32 cm (inner ring) and a height of 25 cm. Both rings were placed 10 cm into the surface of the dike and filled with water. The water level in the outer ring was kept at the same level as the inner ring. The rate at which the water level in the inner ring lowered was recorded over time. Based on these measurements the permeability of the subsoil was determined. Since the double ring infiltrometer set was positioned at shallow depth, only the permeability of the top soil layers could be determined.

Aardvark tests were used to determine the permeability of the loam layers: a logger was placed in a borehole, and the water head in the borehole was kept at a constant level. The permeability of the soil was determined using the loss of water over time and the dimensions of the borehole.

Double ring infiltrometer tests and Aardvark tests were performed at ten locations along the Afsluitdijk. Each location contained at least two test points: one on the crest of the dike and one on the inner slope. Five locations were selected for one additional test point on the outer slope. Locations were chosen based on available data from standpipes and previous soil investigations.

Statistical analyses were performed on the results for the permeability. Data was categorized based on material (loam / clay), location on the dike (outer side / inner side / crest of the dike) and method (double ring infiltrometer / aardvark). 5% non-exceedance values ($\Gamma = 0.75$) were determined.

## 2.2. Numerical Modelling

Numerical models were set-up in the software program Plaxflow to determine groundwater flow using non-linear, time dependent equations solved using a finite element method (FEM). To relate the saturation to the pressure head, the Van Genuchten model was used as the hydraulic model (Plaxis, 2010).

Hydraulic conditions during a storm (water levels, wave overtopping volumes) were determined based on statistical calculations combined with values presented in TAW (2009). The resulting hydraulic boundary conditions are presented in figure 2.

Wave overtopping is not a feature within Plaxis and was therefore modelled as infiltration. Models were set up for a wave overtopping discharge of 10 l/s/m. Deltares (2010) concluded during the overtopping simulations that average overtopping discharges within the simulated range (10-30 l/s/m) resulted in the constant presence of a water layer of at least 1 cm on the slope of the dike.

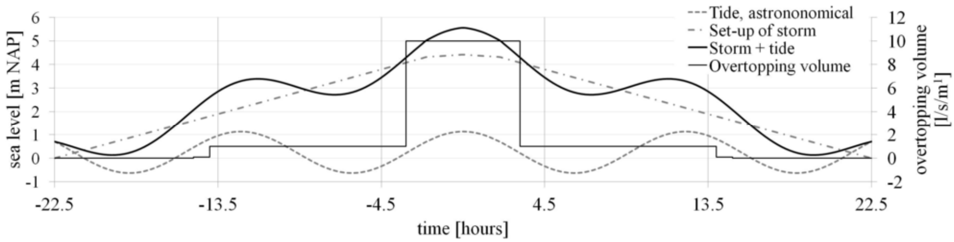


**Figure 2.** Superposition of astronomical tide and set-up of the storm, including wave overtopping volumes in time.

A hydraulic head of 1 - 5 cm is modelled in Plaxflow. The overtopping discharge of 10 l/s/m is assumed to be able to infiltrate in the retaining part of the dike (length: 16 m), that forms the main sea defence.

The following stages of numerical modelling were used to verify the models, and to obtain a reliable phreatic surface:

1. Standpipe measurements are used as calibration of the model, by comparing measurements and corresponding sea water levels with numerical results.
2. The phreatic surface is analysed with and without wave overtopping.
3. The phreatic surface during storm conditions is determined with expected (average) and conservative (5% non-exceedance) values.

## 3. Results

### *3.1. Standpipe Measurements*

The highest sea water level was measured during the north-western storm in December 2013. The sea water level is plotted in figure 3 together with the standpipe data at the centre of the dike. Response of the water level in the standpipes is minimal: an average response of 5 cm in water level in the standpipes is found, compared to a change in sea water level of approximately 4.5 m. A clear response to the sea water level is observed in the standpipe on location 10-2.

### *3.2. Hydrological Parameters*

The permeabilities determined during the in-situ investigations (clay, loam) and laboratory tests (sand) are presented in table 1. The permeabilities of the subsurface are calibrated based on numerical analysis of the standpipe measurements.

### *3.3. Numerical Model*

The sensitivity of the hydraulic and Van Genuchten parameters for the different soil types is checked through a comparison of numerical data with standpipe measurements, under the same sea and lake water level conditions. The permeability of the loam core is relatively low, and therefore variation within the range of determined values (0.02 - 0.05 m/day) does not have a significant influence on the calculated head at the standpipe locations. Variations in the permeability of the soil layers below the dike core result in either (figure 4):

- a good agreement of the groundwater head but an overestimate of the measured time lag.
- a good estimate of the time lag in combination with an overestimated ground water head.

No satisfying results have been obtained during the present research.

Expected (average) input values for hydraulic conductivities result in a phreatic surface as shown in figure 5. Conservative (5% non-exceedance) values result in a phreatic surface as shown in figure 6. The inflow of water in the dike proceeds mainly from the lakeside, due to wave overtopping and due to the higher hydraulic conductivity of the cover layers on the lake side of the dike. The inflow through the loam layers, from the seaside, is marginal.

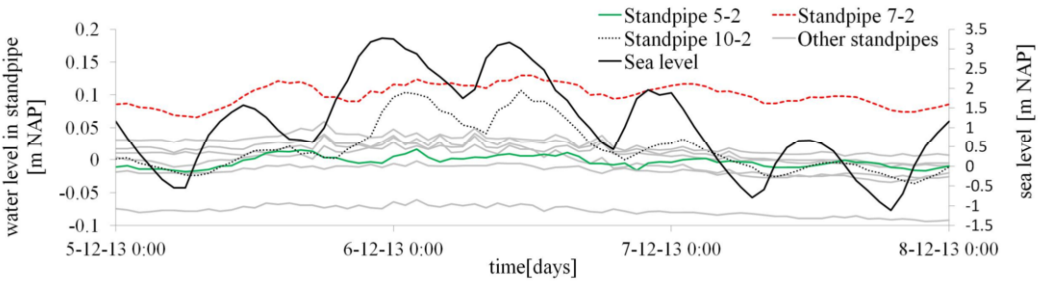


**Figure 3.** Standpipe measurements of the North-West storm in December 2013 (centre of the dike, sand core). The sea water level (black) increases from NAP -0.6 m to NAP +3.3 m within 24 hours. The majority of the standpipes (grey and green) hardly show any response to the sea level fluctuations (1 - 3 cm response in the core of the dike). Standpipe 7-2 (red striped) shows a limited response of 7 cm with a delay of 3 hours for the first peak water level and a delay of 2 hours for the second peak level, with respect to the sea level. Standpipe 10-2 shows a response of 13 cm with a delay of 1 and 2 hours with respect to the first and second peak water levels.

## 4. Discussion

### *4.1. Phreatic Surface: Factors of Influence*

Main factors of influence are identified during this research.

Permeability of the cover layers of the dike, is of major influence on the phreatic surface. The top layer is usually highly permeable, due to weathering and a high sand content, and therefore double ringinfiltrometer tests result in hydraulic parameters that overestimate the permeability of the total covering layer; deeper parts of the covering layer are usually better conserved, or even consist of a different material (like loam, in the case of the Afsluitdijk). More realistic permeabilities are obtained using Aardvark tests. The duration of these tests is an important factor: the permeability, in most cases, decreased with time, and a duration of the test of at least 2.5 hours is advised.

Standpipe measurements in combination with data of water levels on both sides of the dike are useful in calibrating the numerical models. Sensitivity of hydraulic parameters of the soil that are used in the model can be tuned, using aardvark and double ring infiltrometer measurement data. However, the calibration was not totally satisfying, as described in section 3.4, which resulted in a conservative selection of hydraulic parameters.

### *4.2. Modelling of Wave Overtopping*

Wave overtopping can be modelled by Plaxflow, by introducing infiltration in the 'water conditions' mode. An amount of time-dependent infiltration (m/day), and a maximum hydraulic head (m) can be set per section in the model. However, water is not conserved in the model: water that does not infiltrate in the model, does not flow to another section, but disappears.

Infiltration volumes can be calculated per section by subtracting the infiltrated volume of water from the total wave overtopping volume per section, but this is time-consuming. A model in which one volume of wave overtopping can be introduced (for example: 10 l/s/m), which can infiltrate in a certain amount of selected sections of the dike, would be more appropriate for the modelling of wave overtopping. For low overtopping discharges it is advised to compare the amount of infiltration into the dike with the overtopping discharge. If the infiltration is larger than the overtopping rate, clearly there will be room for optimizing the boundary conditions.

### *4.3. Implications for Geotechnical Risks*

Dutch guidelines conservatively propose fully saturated dikes if wave overtopping volumes > 0.1 l/s/m occur (TAW 2001, 2004, Rijkswaterstaat, 2012). This is due to the many uncertainties regarding the effect of wave overtopping on the phreatic surface and results in very low calculated slope stabilities, c.q. gentle slope designs. By performing tests on permeability of the cover layers and core materials of the dike, comparing standpipe data with numerical models, and using site and laboratory data to determine the build-up of the dike, the phreatic surface can be optimized while maintaining risks at an acceptable level. Meeting the safety requirements at lower costs is the result.

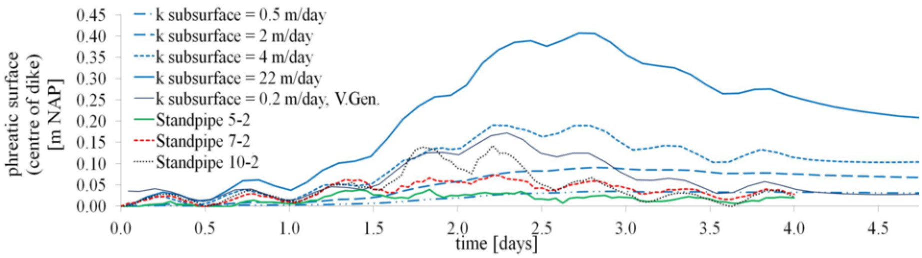


**Figure 4.** Standpipe measurements of the North-West storm in December 2013 (centre of the dike, sand core) plotted together with the phreatic surfaces obtained using FEM. The hydraulic parameters of the subsurface (soil below NAP -5 m) and have been varied. A very fast response results in a rapid rise of the phreatic surface in the dike: k subsurface = 0.2 m /day, V.Gen. shows that the phreatic surface in the FEM is already higher than the standpipe measurements, even with a very low hydraulic conductivity (0.2 m.day). High conductivities result in the correct shape of the phreatic surface, but the response is slower than the measured standpipe response.

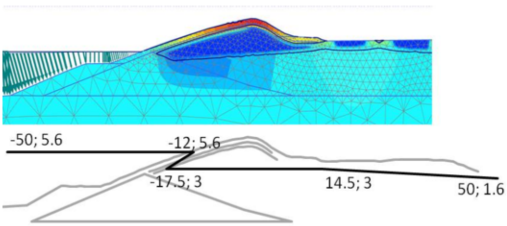


**Figure 5.** Phreatic surface during storm conditions with wave overtopping: FEM with expected, average values.

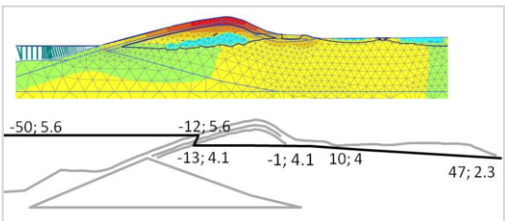


**Figure 6.** Phreatic surface during storm conditions with wave overtopping: FEM with conservative, 5% non-exceedance values. The maximum water level in the retaining part of the dike is NAP +4.1 m.

## 5. Conclusions

The phreatic surface in the Afsluitdijk during storm conditions is modelled. The main findings can be summarized as follows:

- A combination of in-situ tests and numerical modelling results in good, reliable results for the phreatic surface in the dike during storm conditions.
- Wave overtopping can be modelled, but the methods are restricted.
- Double ring infiltrometer tests can be used to determine the permeability of the top soil layers, but Aardvark tests are more suitable for determining the hydraulic conductivity of the deeper parts of the coverage layers of the dike.
- Standpipe measurements are suitable for validation of the models response to outside water level variations without overtopping.
- The modelling of the phreatic surface resulted in high savings of costs, because the overly conservative scenario of a saturated dike could be discarded for a more realistic scenario, which has a positive influence on the calculated dike slope stability.

## References

Deltares (2010). SBW Golfoverslag en Sterkte grasbekleding, Fase 3D Evaluatie Afsluitdijk, 1200259-010-GEO-0017.

Directie Zuiderzeewerken (1931). Bestekstekeningen van de Afsluitdijk.

Plaxis, (2010). Plaxis 2D user manual, reference manual and material models manual.

Rijkswaterstaat (2012). Handreiking Toetsen Grasbekledingen op Dijken t.b.v. het opstellen van het beheerdersoordeel (BO) in de verlengde derde toetsronde, 25-10-2012.

TAW (2001). Technisch Rapport Waterkerende Grondconstructies, Geotechnische aspecten van dijken, dammen en boezemkaden, ISBN 90-369-3776-0, P-DWW-2001-035.

TAW (2004). Technisch Rapport Waterspanningen bij dijken, ISBN-90-369-5565-3, DWW-2004-057.

Thijsse, J.Th. (1972). Een halve eeuw Zuiderzeewerken, H.D. Tjeenk Willink, ISBN 90 01 86070 2.

**Table 1.** Results permeability tests

| Material | Location | Method | Permeability [m/day] | | | Amount of tests |
|---|---|---|---|---|---|---|
| | | | average | 5% non - exceendance | SD | |
| Loam | Outer side (seaside) | Aardvark | 0.02 | 0.07 | 0.03 | 5 |
| | | Double ring infiltrometer | 0.02 | 0.04 | 0.01 | 5 |
| | Crest | Aardvark | 0.02 | 0.06 | 0.03 | 10 |
| | Inner side (lakeside) | Aardvark | 0.02 | 0.05 | 0.02 | 10 |
| | | **Total loam** | **0.02** | **0.05** | **0.02** | **30** |
| Clay | Crest | Double ring infiltrometer | 0.8 | 2.2 | 0.9 | 10 |
| | Inner side (lakeside) | Double ring infiltrometer | 2.3 | 5.4 | 2.0 | 10 |
| | | **Total clay** | **1.6** | **2.2** | **1.7** | **20** |
| Sand (dike) | Centre of the dike | Sieving | 2 | 22 (max) | n/a | |
| | | Literature | 2 | 22 (max) | n/a | |
| Subsurface | n/a | Analysis standpipes | n/a | 4 | n/a | |