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Innovative Finite Element based Design Methodology of Structurally Reinforced Dikes in the Netherlands

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ABSTRACT The protection of land against flooding is a problem of major concern in the Netherlands. Due to new and more strict safety regulations, a large number of existing dikes has to be reinforced. While ground solutions remain the most straightforward way to improve the strength of dikes, the installation of structural reinforcement in the dike is necessary in some cases due to lack of space or for the preservation of cultural heritage. In the design of a structurally reinforced dike, a probability is assigned to both geotechnical failure and structural failure. It does not only depend on the strength characteristics of the soil but also on the deformations. The classic Limit Equilibrium calculations are therefore not sufficient and more advanced Finite Element calculations are required. For this purpose, the Dutch research institute Deltares has published a technical report that provides a uniform framework for the Finite Element based design of dikes. This contribution presents the KIS project in the Netherlands, where parts of a 17.5 km long dike needs to be reinforced to prevent failure caused by the uplift phenomenon. Different solutions are applied, but focus will be given to the installation of a diaphragm wall in the center of the dike. The diaphragm wall does not eliminate the uplift phenomenon, but is designed to act as an independent water barrier. The application of the innovative design methodology is discussed. Analysis of a typical result allows to identify some of the key parameters which are investigated in a sensitivity analysis.

1 INTRODUCTION

The protection of land against high water levels is a historical problem of major concern in the Netherlands where 9 million inhabitants live below sea level. Rising water levels, decreasing ground surface levels, new technological understandings, and increasing economic value of the lowlands justify the need for higher and stronger dikes.

Due to new and more stringent safety regulations, a large number of existing dikes has to be reinforced. In this context, the dike at the left bank of the river Lek between the towns of Kinderdijk and Schoonhovenseveer (KIS) has to be strengthened to raise the safety against overall failure (WSRL 2015).

The main stability problem in the KIS project is caused by the well-known uplift phenomenon, which is the dominant problem for over 50% of the dikes in the Netherlands (Van et al. 2005). Uplift occurs when soft impermeable layers with very low unit weight overly a permeable sand stratum that is in contact with the adjacent river (see Figure 1). During high water levels in the river, the pore water pressure in the sand layer increases and, consequently, the effective stresses at the layer interface decrease, eventually to zero. The loss of effective stress at the interface provokes sliding of the inner slope over the sand layer and finally instability of the dike.

In the KIS project, three different techniques are used to improve the stability of the dike. The first technique consists of the installation of a soil blanket at the toe of the dike that prevents uplift by increasing the effective vertical stresses at the polder side.

The second technique is the installation of an open pile wall at the toe of the dike. As the sand layer is still in contact with the river, uplift of the soil at the polder side is not prevented, but additional support to the dike is provided by the pile wall such that the combination of the dike and the wall is able to retain the high water levels in the river.

The third technique is the installation of a diaphragm wall in the center of the dike. It does not prevent uplift, but is designed to resist a high water level independently.

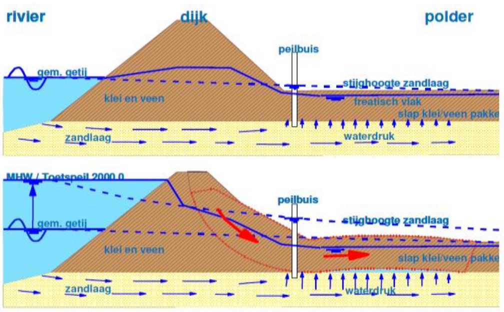

Figure 1. Illustration of the uplift phenomenon.

The first technique is the most classical as it only involves the installation of additional soil, but it requires enough free space behind the dike. This is not always available at a reasonable cost due to a high building density near dikes or due to the presence of cultural heritage. The installation of pile walls or diaphragm walls therefore becomes an economic alternative for dike reinforcement. These solutions require a more complex design methodology, however.

In the traditional design of dikes, instability is related to geotechnical failure and so-called Limit Equilibrium (LE) methods are used to determine the safety factor of the dike. In the available software packages, different failure mechanisms are implemented including the uplift phenomenon, e.g. in the Uplift-Van module in the D-Geo Stability software (Van et al. 2005).

In structurally reinforced dikes, instability can be related to geotechnical failure as well as to structural failure. The assessment of structural forces strongly depends on the displacements in the dike, however. Therefore, the classic LE methods are not suited anymore and a Finite Element (FE) approach is required for the design.

As the stability assessment of structurally reinforced dikes with the FE method is a relatively new development, the Dutch research institute Deltares has published a report (Bakker et al. 2011) in order to create a uniform approach. A probabilistic based approach is applied to prove the safety of the dike: a probability of failure is therefore assigned to structural and geotechnical failure modes. Based on this report, a design methodology has been created specifically for the KIS project. Further development of this methodology should result in a uniform design guideline in the future.

In the present paper, the results will be presented for the design of a diaphragm wall in the center of the dike. Paragraph 2 elaborates on the corresponding design methodology. In paragraph 3, some results are presented based on a realistic reference case. These results are interpreted in order to have a good understanding of the failure mechanism and to allow for an identification of the key parameters. In paragraph 4, the influence of some of these parameters is treated qualitatively and quantitatively. Finally, a conclusion is presented in paragraph 5.

2 DESIGN METHODOLOGY

Diaphragm walls in the center of the dike are called type I constructions, for which a specific design methodology has been made (Havinga and Larsen 2013). This methodology is briefly discussed in the following.

As the assessment of the safety of the dike is performed with an LE model, an FE model of the existing dike (without wall) is first made which can be benchmarked with the classic LE model.

Constructions in the center of the dike are unanchored and act as an independent water barrier. Both inward and outward stability are controlled, for Ultimate Limit State (ULS) and for Serviceability Limit State (SLS), so that a total of four calculations is performed. A detailed scheme is provided for each calculation where the weight of the dike, the increase of the pore water pressure, the traffic load, etc. are applied in different steps in the FE model. The aim is to simulate the stress state in the soil as realistic as possible.

Table 1 shows the calculation scheme for the ULS of the inward stability. In step U2d, the final stress state is obtained with characteristic values, and in step U3b the soil parameters are switched to design values. Next, the strength parameters of the soil are reduced until failure in a so-called ‘Safety’ calculation in Plaxis (steps U3a and U3c in Table 1). In SLS calculations, no strength reduction is done.

Table 1. Calculation scheme for the ULS of inward stability.

Step	Description	Condition
U1a	Initial situation, no weight of dike.	drained
U1b	Add weight of dike.	drained
U1c	Add phreatic water level daily conditions.	drained
U2a	Add pore water pressure daily conditions in sand layers.	drained
U2b	Installation of diaphragm wall.	drained
U2c	High water level and corresponding pore water pressure in sand layers.	drained
U2d	Add traffic load.	undrained
U3a	Strength reduction step with characteristic values.	undrained
U3b	Change to design values.	drained
U3c	Strength reduction step with design values.	drained

For each of the limit states, design criteria are imposed in terms of a minimum safety factor and a maximum displacement. In the remainder of the paper, the focus will be on the ULS for the inward stability as this case will govern the geotechnical stability and the structural design. The corresponding safety factors are given in Table 2. For structural design, an additional safety factor of 1.15 is applied to the moments calculated in the FE model.

Table 2. Safety factors for ULS inward stability

Geotechnical stability	$\gamma_{EEM;g} = 1.27$
Construction strength	$\gamma_{EEM;i} = 1.17$

A particular problem in the modeling of the inward stability is that the inner slope may become unstable at a lower factor of safety than specified in Table 2. Sliding of the inner slope will lead to the loss of weight and to a reduction of the strength properties of the soil at the passive side of the wall. Both effects will reduce the safety factor and are accounted for by modeling a residual profile where two thirds of the height between the dike and the polder level is removed (see Figure 2). Note that failure of the inner slope (and possibly houses at this location) is accepted as a small damage in the safety philosophy; above all, an entire breakthrough of the dike that would lead to a much larger damage needs to be prevented.

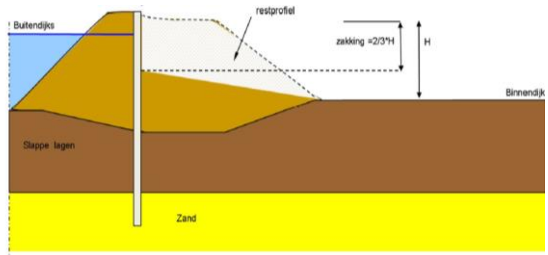

Figure 2. Residual profile for the inward stability.

3 RESULTS

In this section, results are presented for the reference case of a dike where the uplift phenomenon occurs. The soil consists of light clay and peat layers, overlying a sand stratum with a higher pore water pressure. For the reason of brevity, the geometry, soil layering, hydraulic loads, etc. are not mentioned in the paper but are available upon request.

The reference case in this paper is loosely based on the conditions in the KIS project and is used as a qualitative example. Although realistic values are obtained, the absolute numbers are less important and the results can be interpreted in a more general way too. The FE calculations are performed with the Plaxis software.

3.1 Geotechnical stability

The geotechnical stability of the dike is assessed by means of a strength reduction step in Plaxis (see Table 1). In order to get a good understanding of the governing parameters, the failure mode of the soil is investigated. First, Figure 3 shows the displacements of the soil at full loading (design values) before strength reduction. Note that the shape of the dike at the polder side is determined by the residual profile. The displacements are mainly concentrated in the soil around the diaphragm wall and show the typical active and passive soil wedges. A number of plastic points has been generated in these areas, but due to stress redistribution equilibrium is found.

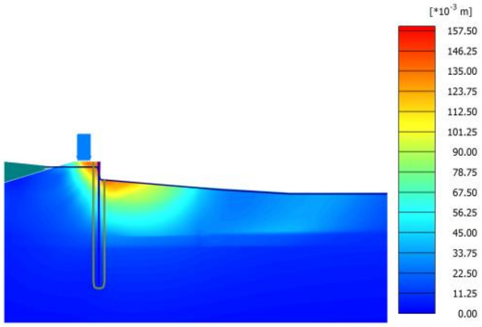

Figure 3. Soil displacements |u| [m] at full loading with design values and before strength reduction.

The situation in Figure 3 is the starting point for the strength reduction calculation with design values (step U3c in Table 1). By reducing the strength of the soil with a safety factor γ_{EEM}, additional displacements will be generated. Figure 4 shows the safety factor as a function of the displacement |u| of the top of the wall. First, a monotonically increasing safety factor is found with a relatively small but increasing displacement. As the safety factor increases, more and more points in the soil will reach the plastic state. Finally, in the horizontal part of the curve, a failure surface has fully developed when equilibrium is no longer found and the displacements infinitely increase while the safety factor has reached the maximum value. The reached safety factor has to meet the criterion for geotechnical stability, as specified in the design guideline (see Table 2).

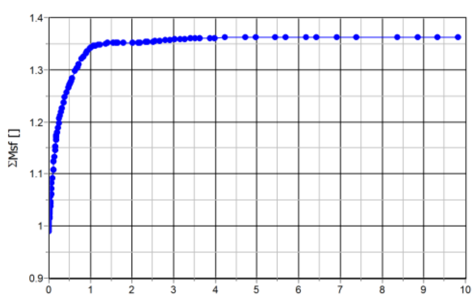

Figure 4. Safety factor vs. displacement of wall top during strength reduction.

Figure 5 shows the soil displacements at failure (maximum value of the safety factor). At the active side, a slightly curved slip surface is obtained. At the passive side, a more complex slip surface is obtained. The complex shape of the slip surface has a large influence on the safety factor and illustrates the need to model structurally reinforced dikes with the FE approach.

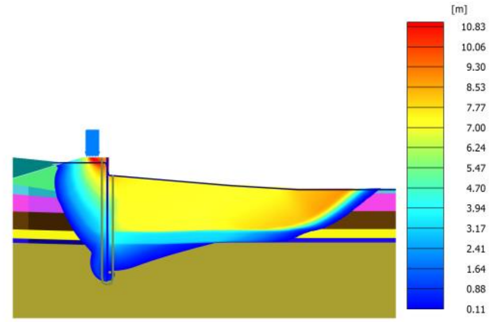

Figure 5. Soil displacements |u| [m] at full loading with design values and at end of strength reduction.

Generally, two different parts are distinguished at the passive side of the slip surface. In the upper layers, large displacements are observed up to a large distance from the wall. Due to the pore water pressure in the sand layers (yellowish colors in Figure 5), a low frictional resistance is mobilized at the interface between the soft layers and the sand. The soil will therefore slide as far as the location where the smallest passive resistance is found, i.e. at the lowest ground surface level. This results in the long shape of the slip surface, which is typically found in case of the uplift phenomenon. In the lower (sand) layers, the displacements are concentrated around the wall where a passive soil wedge is found.

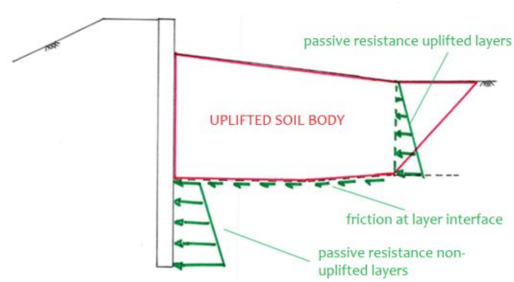

Figure 6. Schematic representation of the passive resistance.

The passive side of the slip surface is shown schematically in Figure 6. This representation allows to estimate the passive resistance and safety factor and determine the required depth of the wall.

3.2 *Strength of the structural diaphragm wall*

The structural forces are determined in ULS at a certain safety factor (see Table 2). This means that for soil parameters reduced with this safety factor, the structure still has to be able to resist the acting forces. The safety factor for construction strength is lower than the safety factor for geotechnical stability (otherwise, the FE model would not be able to find equilibrium), but is higher than 1. This means that the result will be situated in the increasing part of the curve in Figure 4. Figure 7 shows the soil displacements at the corresponding safety factor.

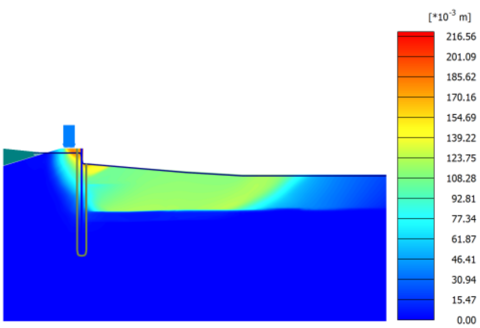

Figure 7. Soil displacements |u| [m] at full loading with design values and at safety factor corresponding to construction strength.

While the absolute value of the soil displacements in Figure 7 is low compared to the values in Figure 5, it is clear that the upper part of the slip surface has already been generated. The larger displacements in the upper layers mean that the relative stiffness will be low and the support provided to the wall will be relatively small. In the lower layers, the effective stress and, hence the passive resistance, is higher. Due to this stress distribution, the wall will act as a cantilever beam with a clamping point somewhere in the sand layers and with a theoretical cantilever length that includes the soft layers. This results in large moments in the wall, in particular because the diaphragm wall is unanchored. The resulting moment is shown in Figure 8. The moment in the upper part of the wall corresponds to a cantilever moment with an increasing moment up to a large depth.

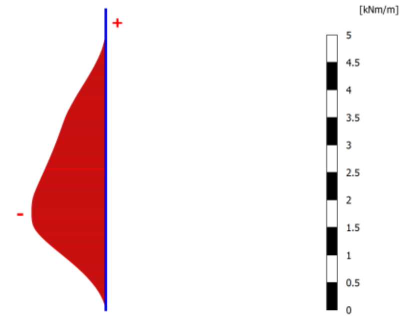

Figure 8. Moment M [MNm/m] in the wall (ULS)

A remarkable point in the current design methodology is the difference between the SLS moments and the ULS moments. Although a slightly different calculation scheme and characteristic soil parameters are used, the SLS displacements generally resemble the situation in Figure 3 where the upper part of the slip surface has not developed yet. Hence, a larger support will be provided to the wall and a much smaller (fictitious) cantilever is obtained. This results in a large difference between the moments in ULS and SLS. This effect is more pronounced for unanchored structures such as the diaphragm wall, where an ULS/SLS ratio up to 10 is commonly found. Further research is ongoing to determine if such ratios are realistic or more an unexpected consequence of the use of partial material factors applied at the source.

4 SENSITIVITY ANALYSIS

A good understanding of the failure mechanism discussed in the previous paragraph allows to assess the qualitative impact of a number of parameters on the design and to identify the key parameters. This paragraph discusses three parameters that are important in the design. The influence of these parameters is determined quantitatively with the FE model.

4.1 *Depth of sand layer*

It is clear that the depth of the sand layer plays a crucial role in the design. A deeper sand layer corresponds to a thicker soft layer with low effective stress. The passive resistance and, hence, the safety factor will be lower. Or, alternatively, a deeper wall will be required to obtain the same safety factor. Similarly, a deeper sand layer means a larger cantilever length and a larger moment in the wall.

In Table 3, the results are given for a sand layer that is 0.500 m deeper. They confirm that a lower safety factor and a higher moment are obtained.

4.2 Volume weights

The volume weight of the soil plays an important role in the design of the diaphragm wall. Lower volume weights in the soft layers result in lower effective stresses and a lower passive resistance. Failure of the upper slip surface will occur faster, which results in a smaller safety factor and larger moments.

In Table 3, two cases are investigated where a reduced volume weight (reduction of 1.5 kN/m³ over 2 m) is applied at the top and at the bottom of the soft layers. A lower factor of safety and larger moment are obtained for both cases. While the influence on the safety factor is relatively small for both cases, it is shown that a reduction of the volume weight in the top of the soft layers has a larger effect on the moment, as it has a larger lever with respect to the clamping point in the sand layers.

4.3 Ground surface level

The effect of the surface level in the polder is similar as for the volume weight of the soil. A lower surface level results in a smaller passive resistance of the upper layers. This results in a lower safety factor and a larger moment. This is confirmed quantitatively for a polder surface level that is 0.18 m lower (Table 3).

As observed in the slip surface in Figure 5, the ground surface level up to a large distance behind the dike determines the stability. It is therefore important to correctly determine the ground surface levels up to remarkably large distances from the dike. In the design of the diaphragm wall in the KIS project, slip surfaces up to 80 m behind the dike are found.

4.4 Summary

Table 3 gives an overview of the sensitivity analysis.

Table 3. Results of the sensitivity analysis

Case	Safety factor	Forces
Reference case	1.362	1554 kNm/m
Deeper sand layer	-3%	+5%
Reduced volume weight (top)	-3%	+15%
Reduced volume weight (bottom)	-2%	+6%
Lower surface level (polder)	-2%	+9%

5 CONCLUSION

In the present paper, the results of a FE based design of a dike reinforced with a diaphragm wall are presented. The complex shape of the slip surface illustrates the need for a FE based design.

The structural forces are determined by the relative displacements in the soil which in turn depend on the strength parameters of the soil. Failure in the upper soft layers results in large displacements and, hence, in large structural forces, particularly for the unanchored diaphragm wall.

Analysis of the failure mechanism allows to identify some of the key parameters of the design. A sensitivity analysis shows that particularly the forces are sensitive for variations of these key parameters.

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