Geotechnical aspects of reinforcement alternatives of the Afsluitdijk

Aspects geotechnique des possibilitees de renforcement de digue Afsluitdijk

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ABSTRACT

In the Netherlands, the Afsluitdijk connects the edge of the North Holland coast and the Friesland coast, protecting the land around the IJssellake from flooding. The dike, a national historic monument has a length of 30 km and was build from 1929 till 1933. In the years following, only limited parts of the dike have been raised.

Recently, the required safety level against flooding of the Afsluitdijk has been raised, because the dikes around the IJssellake had not the required safety level. An assessment of the current safety resulted in the statement that the Afsluitdijk does not comply with the new level. Large overtopping discharges (570 l/s/m) and a rather steep inner slope (around 1:2,5) lead to an unacceptable probability of erosion and macroinstability. The Afsluitdijk will be reinforced. The Dutch authority considers several alternatives for reinforcement, such as innerward or outerward raising, raising the fore land, building break waters or protecting the dike against erosion due to overtopping. These alternatives have been investigated as to hydraulic and geotechnical aspects, in order to optimise the design. For a negative (W+, KNMI 2006) climate change scenario the dike height has to be increased from 7,8 m to 13,9 m, when allowing for a limited overtopping of 1 l/s/m. Several options to diminish this height have been considered. Several dike failure mechanisms have been considered, resulting in recommendations for additional research. Especially the knowledge of the properties of the subsoil and the position of the freatic line are of importance.

RÉSUMÉ

L'Afsluitdijk est une digue nationale monumentale, qui relie le bord de la côte du nord de la Hollande a la côte de la Frise et protégeant les terres autour du lac IJsselmeer contre les inondations. La digue a une longueur de 30 kilomètres et a été construite entre 1929 à 1933. Plus tard, seulement quelques parties limitées de la digue ont été surélevées. Récemment, le niveau de sécurité requis contre les inondations pour l'Afsluitdijk a été augmenté, parce que les digues entourant l'IJsselmeer n'avaient pas le niveau requis de sécurité. Une évaluation de la sécurité actuelle a montré que l'Afsluitdijk n'est pas sûr. Les debits importants passant au dessus de la digue (570 l/s/m) et une pente intérieure assez raide (autour de 1:2,5) entrainent une érosion inacceptable et une macro instabilité. L'Afsluitdijk sera renforcé. Les autorités hollandaises considèrent plusieurs possibilités pour le renforcement, telles que le remblai de la face interne ou externe de la digue, ou son relèvement, la construction de brise vagues, la protection de la digue contre l'érosion due à la surverse. Ces solutions ont été analysées dans leurs aspects hydrauliques et géotechniques, afin d'en optimiser la conception. Pour un scénario de réchauffement climatique W+ a un horizon de 2100, la hauteur de la digue doit être portée de 7.8 m à 13.9 m, lorsque la surverse est limitée a 1 l/s/m. Plusieurs options pour diminuer cette hauteur ont été considérées. Plusieurs mécanismes de rupture de la digue ont été étudiés, ayant conclus a la nécessité de recherches supplémentaires. En particulier, la connaissance des propriétés du sous-sol et le niveau de la nappe phréatique sont importantes.

Keywords : Afsluitdijk, reinforcement, safety, geotechnical risks

1 LEAD OF REINFORCEMENT

Because the dikes of the hinterland around the IJssellake were not safe enough, in 2006 the required safety level of the Afsluitdijk has been increased, because the dikes of the hinterland around the IJssellake were not safe enough. The Afsluitdijk should resist a 1 in 10.000 year storm. In 2008 the Dutch government started a market survey to innovative reinforcement solutions.

The results of the study described here were provided to parties involved during this survey.

This paper concentrates on the geotechnical risks of the following reinforcement alternatives.

1. Innerward raising
2. Outerward raising
3. Overtopping protected dike
4. Protection wall (not elaborated)
5. A raised foreland in the Waddensea
6. Breakwater in the Waddensea

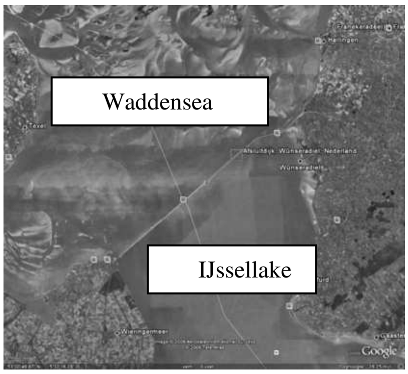

Figure 1 plane view of Afsluitdijk

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2 QUESTIONS TO BE ANSWERED

For typical design properties the following questions were treated:

- What dike height is needed and how much space is taken?
- What are the dimensions of the foreland and the breakwater for alternative 5 and 6?

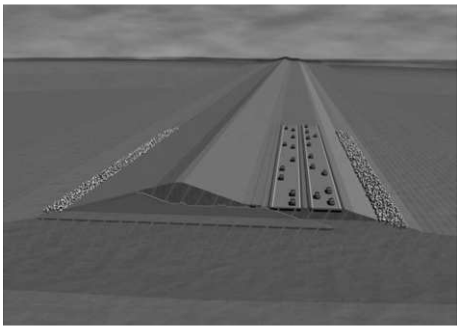

Figure 2 Innnerward reinforcement for 1 l/s/m overtopping

- What are the geotechnical risks of the reinforcement alternatives?
- What are the important aspects of construction and maintenance?
- Is it possible to maintain the current dike profile for alternatives 3, 5 and 6?
- What is the influence of the heterogeneity of the subsoil?
- As to erosion: which cover is needed for large overtopping discharges? The current dike has a stone (mainly basalt) revetment at the outer slope with a grass cover on top, on the crest and on the inner slope.

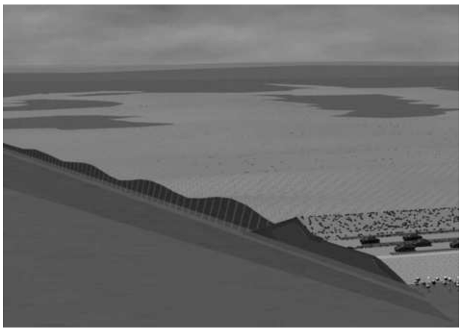

Figure 3 raised foreland before present dike

3 METHOD

Since these questions of the Afsluitdijk are complicated, a risk based approach was adopted. The approach distinguishes six general steps that are applicable for any project phase in an engineering and construction project. It starts with gathering project information. Based on this information, risks are identified and classified. Then risk remediation measures are selected and implemented. Following the implementation of remediation measures, the remaining or residual risks are evaluated. Finally, all relevant risk information is stored in a risk register and mobilized to the next phase of the project. In this section the first three steps are worked out. The last 3 steps are analysed in section 4 and 5.

The hydraulic boundary conditions were assumed to be uniform along the dike, with a significant wave height Hs = 3,5 m, Tp = 8,1 s and levels of 6,0 m + NAP and 1,46 – NAP for the Waddensea and IJssellake, respectively. In this case, for a so called robust design, a 30 cm elevation of the water level had to be applied. However, this was not done, as it was supposed that the more detailed design following will consist of a complete probabilistic analysis.

The subsoil mainly consists of sandy layers with clay inclusions, and clay layers with sand inclusions. At a few locations also organic clay and peat were found. The heterogeneous subsoil caused only limited settlements of the dike crest in the years after construction, which indicates that its bearing capacity is reasonably good. In total 15 typical subsoil conditions were distinguished. In this study rough estimates of critical conditions and parameters were made.

The schematization consisted of:

- The description of the most probable subsoil conditions
- A choice of a representative geometry of the current dike profile (dike height of 7,80 m+NAP, outer slope 1:3,7, crest width 2m, inner slope 1:2,5, width of inner berm with highway = 32 m)
- The dike has a sand core and a boulder clay cover, with a sandy clay layer on top of it. At the Waddensea side the dike consists of a boulder clay core with a height of approximately 3,5 m, which was of benefit for construction, as at that time the tidal surge was stopped by it and subsequently the sandy core could be constructed behind and on top of it. The automated calculations with a 1 dimensional schematization of the soil layers did not allow for this boulder clay core. Rough estimates of soil parameters were made, as there were no laboratory tests available.
- A choice of representative sea bottom profiles (raai 2, 4 and 6 in Figure 4)
- A choice of the subsoil parameters (estimated from standards)
- Estimate of the position of phreatic line

For the determination of the minimum stability (partial safety factors), the norm (overschrijdingsfrequentie) of 1:10.000 year, a dike length of 30 km and an estimate of the schematization factor of 1,1 were used (Min. of V&W, 2007). This factor is a safety factor as to the schematization of the subsoil and pore pressures.

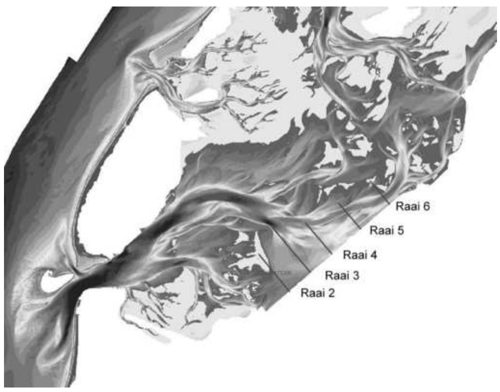

Figure 4 Plain view of the Waddensea bottom height (range 2 m - NAP to 10 m - NAP in the neighbourhood of the dike).

The description of the critical aspects per alternative of dike reinforcement consisted of:

- For the dike: failure mechanisms (see Table 1).
- In case of the foreland: erosion, avalanching, minimal width and height in order to reduce the wave attack
- For the breakwater: reduction of wave energy, limited damage is admissible

To classify the critical aspects, rough designs of reinforcement alternatives were made, considering admissible overtopping discharges of 1 l/s/m and 10 l/s/m, respectively. Several geometries were considered: inner slope of 1:2,5 or 1:3, outer slope 1:3,7, crest width: 2m or 3m. Also an outer berm, 5 or 10 m wide, at 6 m+NAP has been considered.
As the permeability of the dike cover is quite high and the dike base is in contact with the outer water, it was assumed that the phreatic line is linear between the Waddensea and IJssellake water level. In case of high overtopping discharges ($q \geq 10$ l/s/m), GeoDelft (1987) calculated that the phreatic line rises significantly up to full saturation of the dike. From recent overtopping field tests on other locations (with $q \geq 10$ l/s/m it followed that the grass cover is wetted for at least several hours in case of a storm surge of 6 hours. Therefore, it was assumed as a first approximation that the dike is fully saturated in case of large overtopping discharges.
With the above mentioned 15 subsoil conditions, geometries and choices of the phreatic line automated Bisschop macrostability calculations were perfomed. In addition, other failure mechanisms were considered. The following failure mechanisms can be distinguished:

Failure mechanism	1. Innerward raising	2. Outerward raising	3.Overtopping protected	5. Raised foreland	6. Breakwater in the Waddensea
Macrostability innerward	+	+	+	0	0
Macrostability outerward	+	+	+	0	+
Macrostability large overtopping	0	0	+	0	0
Microstability	-	-	0	-	-
Piping	-	-	0	0	0
Stability en resistance to erosion of inner slope cover	0	0	+	0	0
Stability seabottom slope Waddensea	0	+	0	0	0
liquifaction	0	+	0	0	+
Stability cover of outer slope	0	+	0	0	0

Table 1 Failure mechanisms for dike (+ is important, - is not important, 0 is of relevance).

The alternatives 5, raised foreland and 6, breakwater were designed as follows: for admissible discharges of 1 l/s/m and 10 l/s/m, respectively, the required reduction of the wave energy was calculated. The present dike profile was maintained and the breakwater was located at a distance of 500 m in front of the dike. By means of XBeach (Deltares, 2008) the required dimensions of the sandy foreland were determined, allowing for erosion and avalanching during the design storm.

4 RESULTS

4.1 Innerward raising.

The dike height is very sensitive to the amount of overtopping. For example a height of 13,86 m is needed for 1 l/s/m (see figure 2) and 11,66 m for 10 l/s/m, which is more than 2 m less, resulting in a gain in space of 2 x 3,7 = 7,4 m.
Innerward raising is only feasible if:
- the overtopping discharge = 10 l/s/m or less
- the inner slope of 1:3 or smaller

In case of steeper slopes or more overtopping, quite elaborate additional research is needed.
The dike stability is very sensitive to the subsoil properties, and the boulder clay core should be taken into account in more detailed calculations.
The stability is very sensitive to the position of the phreatic line, which means that infiltration is a very important process.

A 10 m wide outer berm gives a significanty smaller crest height and is of benefit for inspection. There is a good match to previous reinforcements, which consisted also of innerward raising. Attention is needed for differential settlements of the inner berm with the highway A7. Dike heightening with a core of sand is preferred, as the sand allows for a better drainage, and thus a lower phreatic line.

4.2 Outerward raising.

As shown in Table 1 outerward raising results in a high risk of liquifaction at the Waddenseaside, especially at raai 6 (see Figure 4). An outer slope of 1:3,7 is possibly too steep.
In addition, the positive effect of the boulder clay core on the outer slope stability will be less, as the core is not located at the outer side any more.
Construction has to be done outside the storm period, which means that this takes much longer. In addition, there will be some disturbance of protected nature in the Waddensea.
When a suitable geometry is chosen, this alternative is technically feasible. An example is shown in figure 5. As to the influence of the subsoil and the phreatic line, similar conclusions hold as for the innerward raising.

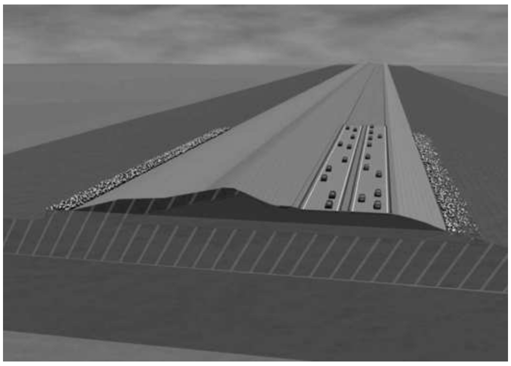

Figure 5. Outerward reinforcement for 1 l/s/m overtopping and a 10 m wide outer berm.

4.3 *Overtopping protected dike*

In this case it was investigated whether the current dike geometry and cover can resist large overtopping discharges.
As to erosion it is possible to protect the dike with a suitable cover for discharges as large as 100 l/s/m. There is little experience with the discharge of 566 l/s/m at design conditions.
However, a limitation of amount of infiltration is necessary, as a high phreatic line is unsafe. For the present profile it was found that:
- An inner slope of 1:2,5 is expected to be unstable and a slope of van 1:3 or less is needed.
- More research is needed to determine the inner slope stability and the stability of the cover on that slope. Infiltration tests will be done, in order to determine the stability of the grass cover on the inner slope for overtopping discharges of 10 to 30 l/s/m.

Combinations of overtopping protection with a raised foreland of breakwater are possible, see sections 4.4 and 4.5.

4.4 A raised foreland in the Waddensea

The required reduction in wave attack can be achieved with a foreland height of at least 4,25 m + NAP, wich is made of sea sand without a cover. The required width of the upper part is 125 m and the width of the kwelder (wetland) is ca. 400 m (see figures 3 and 6).

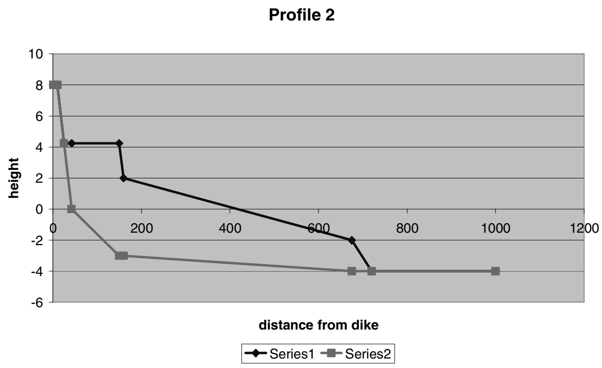

Figure 6 A raised foreland in the Waddensea (series 1 is the raised foreland, series 2 is the current profile).

Construction of this foreland is only possible for raai 2 to 4 (see figure 4), as the gully Doove balg causes enhanced currents and a need of much more construction material.
The kwelder is of relevance for the development of nature, in fact this is desirable in the Waddensea.
Apart from the foreland, the dike needs some limited reinforcement too.

4.5 Breakwater in the Waddensea

The required height of the breakwater is large, in order to obtain the intended reduction of wave attack (i.e. 7,7 m + NAP and 8,5 m + NAP for 10 l/s/m and 1 l/s/m overtopping, respectively, see figure 7).

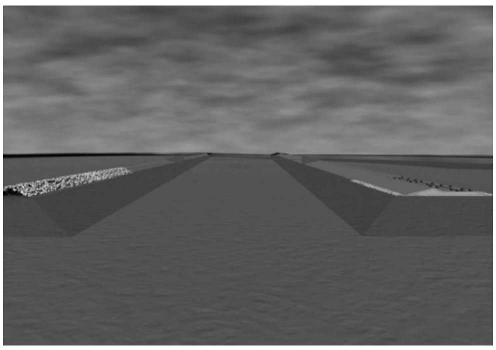

Figure 7. A breakwater at 500 m distance from the dike with an overtopping discharge of 10 l/s/m for the current dike.

The Waddensea subsoil is good enough to allow for construction of a breakwater. There is a risk of liquifaction, there will be changes in currents and local erosion may occur.
Behind the breakwater over a distance of 500 m from the dike there will be a substantial increase of wind waves.
The impact on the Waddensea is considerable, also due to changes in currents. Some reinforcement of the current dike profile is needed.

CONCLUSIONS

All alternatives seem technically feasible, using a suitable geometry. A considerable reinforcement is necessary.
The design assumptions are realistic, but the steeper slopes are undesirable (for example an inner slope of 1:2,5).
The overtopping protected dike (current profile) is not easy to realize, so much attention is needed.

For larger amounts of overtopping, the location of the phreatic line is not known. This has to be solved, as it has a large effect on the stability.
The strength of the dike is very sensitive to the subsoil conditions (change in stability factor of order of 1 is possible).
A maximum overtopping discharge of 10 to 30 l/s/m seems to be feasible for the current dike.

5 RECOMMENDATIONS

More detailed local hydraulic boundary conditions have to be determined.
Proper soil investigation is needed, in particular at the outer slope, including laboratory experiments.
The placement of standpipes and porewater pressure transducers is desirable, in order to determine the response of the dike and subsoil to tides changes and storm conditions.
Measurement and modelling of infiltration during overtopping is needed. In addition, the resistance against erosion of the gras cover can be determined.
Probabilistic analysis is required in order to avoid the 30 cm supplement on the design water level and is useful for the determination of the schematization factor. This factor is a safety factor as to the schematization of the subsoil and pore pressures.
The strength of the current basalt revetment on the outer slope has to be checked for design conditions, especially when the outer slope needs no adjustment, which is the case for the innerward raising.
On the basis of the findings in this study, the global cost of the respective alternatives can be determined.

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