Dike Reconstruction Polder Oudendijk

J. A. van Herpen
Oranjewoud B. V., Oosterhout, The Netherlands

J. de Pee
Civil Engineer, Province of Friesland, Leewarden, The Netherlands

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J.A. van Herpen
Civil Engineer, Oranjewoud B.V., Oosterhout, The Netherlands

J. de Pee
Civil Engineer, Province of Friesland, Leeuwarden, The Netherlands

SYNOPSIS: Reconstruction of dikes on soft subsoil is a common practice in the Netherlands. However in recent years some slidings of innerslopes have occured in the execution phase due to very large and rapidly placed replenishments. Therefore investigations have been made for new methods to determine the short term stability of the soil structures.
This paper describes the problems experienced during the reconstruction of a polder dike in the western part of the Netherlands, based on which several new methods are tested.

INTRODUCTION

The dike of the polder Oudendijk protects a low-lying area (polder) in the western part of the Netherlands.
The water level inside the polder is approx. 4.5 m below the constant normal outside water level.
In case of a breach the inundation depth will be 3 m. This will not only cause severe damage to economic activities in the polder (mainly farming and some small factories) but also endanger the lives of the 5000 people living in the village of Woubrugge.

Reconstruction of a section of appr. 3 km of the dike appeared to be necessary.
Due to the constant settlement of the subsoil in the course of years the crest level had become to low. In some places it was even below the normal outside water level and flooding had to be prevented by a small bund placed on the crest.
Furthermore, preliminary investigations also showed insufficient geotechnical stability of the inner-slopes (COW 1974).
At the design outside water level, which lies 0.30 m above the normal level, the safety factor, calculated using a normal slip circle analysis, was 0.79. The lowest factor found at the normal outside water level was 1.10.
Extensive seepage occured. Vast parts of the innerslope were not even practicable, Figure 1.

Fig. 1 Seepage at the innerslope

GEOTECHNICAL INVESTIGATIONS

Given the expected large replenishments of the innerslope and the soft subsoil conditions a vast geotechnical investigation was set up to achieve a firm basis for the reconstruction design.
First a visual inspection of the dike was carried out. Based on its results 12 locations (cross-sections) were selected for further investigation.
This was done in two stages: a preliminary investigation to establish the quality of the existing dike, and an additional investigation to gather sufficient information for the actual reconstruction design.
Soil borings (percussion method) and cone penetration tests were carried out to obtain information about the subsoil. To establish the phreatic line in the dike standpipes were placed.
Totally 39 soil borings were executed varying in length between 3.5 m (inner toe) and 11.5 m (crest) and 5 penetration tests (12 m). A number of 49 standpipes was placed.
In every cross-section at least 3 borings and 4 stand-pipes were considered to be necessary.
From the borings undisturbed samples were taken which were used for laboratory-tests.
From each sample the soil type, the unit weight (wet and dry), the water content and voids ratio were established.
The friction properties of the soil under consolidated conditions (angle of internal friction and cohesion) were defined based on 35 compression tests and 28 direct shear tests. To investigate the consolidation properties 32

tests were performed. The permeability was tested for 15 samples.

On the basis of the results of the site investigation a geological profile along the dike section is composed (Oranjewoud 1987), Figure 2.
At a depth of 11.20 - 12.20 m below the reference level N.A.P. sand formations formed during the late-pleistocene are present above which a thin layer of peat varying between 0.3 and 0.9 m is found.
Besides a clay layer (holocene deposits) varying in thickness between 4 and 6 m on which a 3 - 5.5 m thick layer of peat can be found. The clay layer contains relatively a high quantity sand in the lower sections and vegetation remains in the upper sections.
The upper layer, which is in fact the dike body, strongly varies in composition and thickness: besides peat, also sand, clay and debris are present. Human activities have strongly influenced this layer.

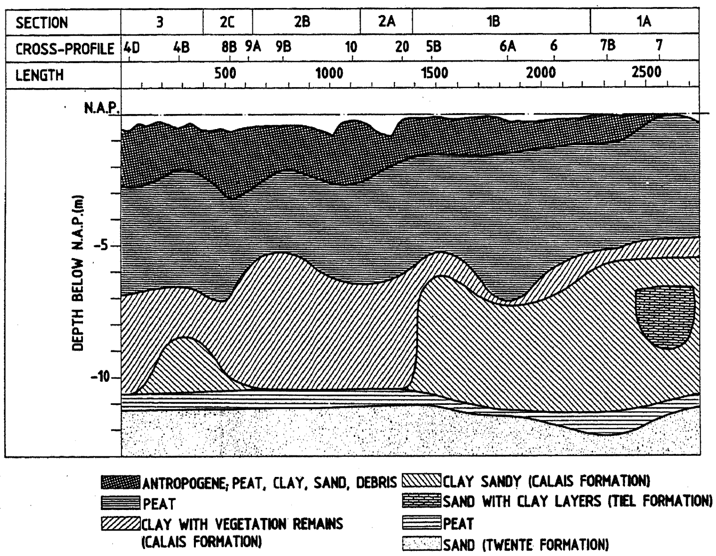


Fig. 2 Geological profile of the dike of polder Oudendijk

To give an indication of the soil properties in Table 1 the results of the laboratory tests carried out on the samples taken from the additional site investigaton are shown.

TABLE 1 Results from laboratory tests (mean values)

Soil type	Volume weight (kN/m³)	Angle of internal friction (dgr)	Cohesion (kN/m³)	Compression modulus	Permeability (m/s)
Antropogene	12,6	20,7	6,5	8,8	$1,3.10^{-7}$
Peat	10,2	20,8	4,7	4,2	$1,5.10^{-6}$
Clay, veg. remains	13,9	25,5	2,6	9,1	$2.9.10^{-9}$
Clay, sandy	16,8	25,5	1,9	22,3	$7.7.10^{-10}$

Not only longitudinal variation in the geological stratification is found but also perpendicular to the axis of the dike.
An example is given in Figure 3.

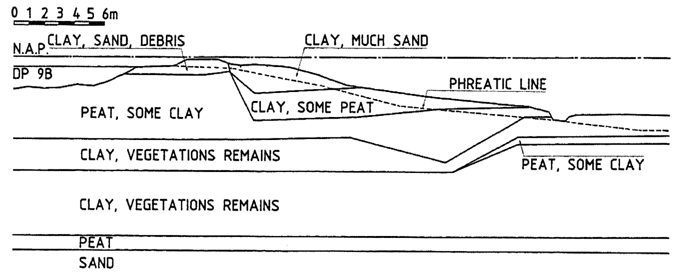


Fig. 3 Bottom schematization cross-section 9B

The geological investigation carried out was for Dutch circumstances very extensive, even for the type of dike considered in this paper, where the bottom profile varies within short ranges.
However, as was experienced later on, it appeared to be insufficient.
Local variations did occur which were not detected.

RECONSTRUCTION DESIGN

Design criteria

The design of the dike should meet to the following criteria:

- the dike should fulfill the criteria for a period of at least 20 years;
- the crest level should lie at least 0.50 m above the normal outside water level;
- the geo-technical stability has to be sufficient;
- seepage has to be reduced;
- if possible the highly-valued vegetation, that occurs on a 350 m long section of the innerslope should be preserved.

The stability of the dike can be improved and the seepage can be reduced by replenishments of the innerslope as well as by lowering the phreatic-line in the dike body.
The latter can be obtained for example by an impermeable screen at the crest of the dike or a drainage system at the innerslope.
A significient reduction of the phreatic line however causes large additional settlements which on their turn lead to an extra crest-elevation.
Since the crest has to be heightened anyway the method of innerslope-strengthening is chosen. The replenishments were carried out with sandy clay that is more permeable than the subsoil on which it is placed, is sufficient resistant to erosion and allows a good grass-growth.

Crest-heigth

Settlement calculations were necessary to establish the crest height in such a way that in twenty years it would not become lower then the design level.

The calculation method applied was according to Terzaghi's formula:

$$z = \frac{h}{C} \ln \left(\frac{p_2}{p_1}\right) \qquad (1)$$

In which: z = settlement (m)
h = thickness of the soil-layer (m)
C = coefficient of compressibility
p_1 = effective stress before loading (kN/m²)
p_2 = effective stress after loading (kN/m²)

For the peat layers use has been made of the adaption by Fokkens (Soudyn 1972):

$$z = \frac{h}{C + 1{,}5 \ln \left(\frac{p_2}{p_1}\right)} * \ln \left(\frac{p_2}{p_1}\right) \qquad (2)$$

The existing cycle-track on the crest had to be replaced. The settlement calculations were carried out for several types of road construction varying in weight. The construction chosen was a top-layer of 0.08 m of asphaltic-concrete on a sub layer of 0.40 m of foam-slags.
Table 2 shows the results of the calculations for several cross-sections:

TABLE 2 Results of settlement-calculations
(* = N.A.P. = the ordnance-datum)

Cross-section (no.)	Former crest-level (m - N.A.P.)*	New crest-level (m - N.A.P.)*	Expected settlement in 20 years (m)	Percentage of final settlement
8b	-0,60	+0,60	0,70	93
9a	-0,49	+0,75	0,85	96
9b	-0,42	+0,30	0,40	87
10	-0,21	+0,40	0,50	92
4b	-0,51	+0,20	0,30	98
4d	-0,61	+0,40	0,50	99

To increase settlements a temporary extra crest-elevation of 0.50 m was executed which after two years was excavated. This results in a decrease of rest-settlements.

Stability

Under stability is meant the resistance against sliding of large parts of the dike body along a straight or a curved plane in which due to overloading no equilibrium of forces may be present (TAW 1985).

The method commonly used is based on a circular sliding surface where the potential failure mass is divided into slices.
The safety factor is defined by the ratio of the moment of shear along the failure plane and the moment of weight of the failure mass.
The objective is then to find the centre and radius of the circle with the lowest safety factor.

In this project use has been made of the simplified Bishop Method of Slices in which it is assumed that the resultant of the vertical forces acting on the sides of any slice is zero.

The shear force, needed to determine the moment of shear at a certain point of the slide-plane, is determined by:

$$\tau_{max} = \frac{c' + \sigma'_v * \tan \phi'}{1 + \frac{\tan \phi' * \tan \alpha^*}{F}} \qquad (3)$$

In which: τ_{max} = the maximum shear force (N/m²)
σ'_v = the effective pressure acting on the slide plane (N/m²)
ϕ' = the angle of internal friction (dgr)
c' = the cohesion (N/m²)
F = safety factor
α^* = α for $\alpha > -\frac{1}{4}\pi + \frac{1}{2}\phi'$
α^* = $-\frac{1}{4}\pi + \frac{1}{2}\phi$ for $\alpha \leq -\frac{1}{4}\pi + \frac{1}{2}\phi'$
α = the angle between the horizontal and the tangent of the slide-circle

The above mentioned conditions for α prevent the shear force from obtaining extreme values in the passive zone (α = negative in the passive zone).

The design dike profile is determined using the Bishop-method in which the shear stress is calculated under drained circumstances. The criteria is that the minimum safety factor is 1.3.
The calculations are carried out for 11 cross-sections. Figure 4 shows an example for one section.

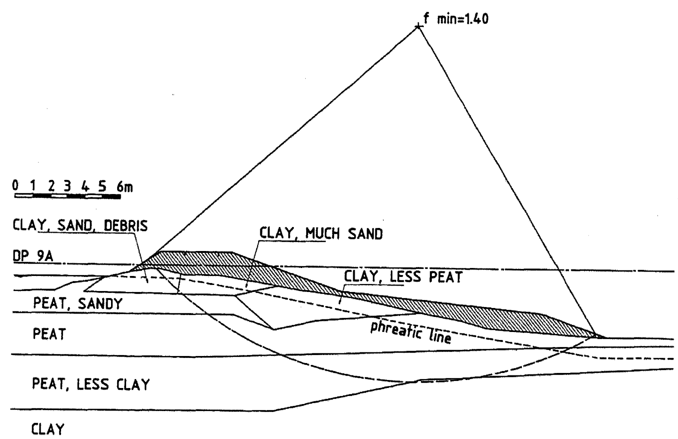


Fig. 4 Necessary replenishment and minimum slide-circle for cross-section 9a

Over a section of 1.3 km it was necessary to excavate a part of the toe of the dike and replace it by sand. Thus not only the stability was improved but also the seepage moved from the innerslope to the toe.

A distinction has to be made between the short and long term stability, or the design and execution stability. In case a rather impermeable subsoil (clay, peat) is present the placing of extensive innerslope replenishments may cause very high excess pore water pressures which will only slowly reduce and result in a proportional increase of the effective pressures.
This means that in these cases the short term stability is generally decisive (De Pee, 1986).

To establish the execution-stability a common method in the Netherlands was to use the geotechnical parameters (ϕ', c') under drained conditions. The short term effect is achieved by applying an extra hydraulic pressure for the less permeable subsoil layers similar to the weight of the replinishment.
This means that the effective vertical stress used to determine the shear stress is equal to the effective vertical stress present just before the loading.

Although no criteria are present for the execution phase it was attempted to obtain a minimum safety factor of 1.2.

These calculations showed that the total replenishment could be carried out in one stage. However it was decided to strenghten first the innerslope and to heighten the crest afterwards.

THE SLIDINGS

Nevertheless in the last phase of the work two slidings of the innerslope took place both appr. 24 hours after replenishing, Figure 5.
Immediately after detection security measurements were taken consisting of bringing extra soil, also from the crest, to the toe of the dike, placing a ship in front of the danger-zone and installing a drainage system to discharge the water in the dike body as quickly as possible.

Fig. 5 Sliding of the innerslope of the dike

In order to detect the cause of the slidings additional soil borings, laboratory tests and stability calculations were carried out.
With the help of visual interpretations, geological measurements and the borings the extent and shape of the slidings were estimated. The slide planes seemed to be rather circular. Also some horizontal displacements in the direction of the polder took place.

Two major causes for the slidings were derived:
1. compared to the geological profile extreme differences are detected, especially in the upper bottom layers. These mainly originate from human activities, such as former repairs of old slidings, old drainage tubes, etc.;
2. the calculation method did not correspond adequatly to reality.

In order to elaborate the second cause, an alternative technique to establish the stability of soil structures in the execution phase was tested.
For this a team was formed consisting of members of the responsible water authority, the consultants, the Delft University of Technology and the Dutch Ministry of Public Works.
In this technique, named the c_u-method, which is already applied in many other countries, use is made of the undrained shear strength of the soil.

It is assumed that the poorly permeable subsoil-layers behave under undrained circumstances as frictionless material (ϕ = o) with an apperant cohesion c_u.
The value for c_u for normally consolidated soil is estimated with the help of the following derivation (Vermeer, 1983):

$$c_u = \tfrac{1}{2}\,(\sigma_v' + \sigma_h')_o * \sin\phi' + c' * \cos\phi' \qquad (4)$$

In which: $(\sigma_v' + \sigma_h')_o$ = the sum of the vertical and horizontal effective stresses present just before loading

The horizontal effective stress can be estimated by:

$$\sigma_h' = (1 - \sin\phi')\sigma_v' \qquad (5)$$

The c_u-value also can be found using (Termaat 1985):
- the Skempton Formula:

  $c_u = (0.11 + 0.0037 * Ip)\sigma_{vo}'$
  in which: Ip = the plasticity index
- vane tests
- direct shear tests

Additional stability calculations are carried out using two methods based on this technique. The results are compared to those obtained from the previous described method in which drained conditions and $\sigma_v' = \sigma_{vo}'$ are used.
The first method is a normal slide-circle analysis according to Bishop in which ϕ = o and c = c_u are taken.
In the second method the equilibrium of stresses in each element of the soil related to the highest possible loading is established. Owing to the static undetermined nature of the soil then also the deformation properties have to be taken into account.
Use has been made from the finite elements computerprogramme Plaxis that is based on an elasto-plastic behaviour of the soil. Failure is determined by a so-called flow-criterion derived from the Mohr-Coulomb stress-shear relation:

$$f = \tau^* - \sigma^* * \sin\phi' - c' * \cos\phi' \qquad (6)$$

$$\tau^* = \{\tfrac{1}{4}\,(\sigma_{xx}' - \sigma_{yy}')^2 + \sigma_{xy}'^{\,2}\}^{\frac{1}{2}} \qquad (7)$$

$$\sigma^* = \tfrac{1}{2}\,(\sigma_{xx}' + \sigma_{yy}') \qquad (8)$$

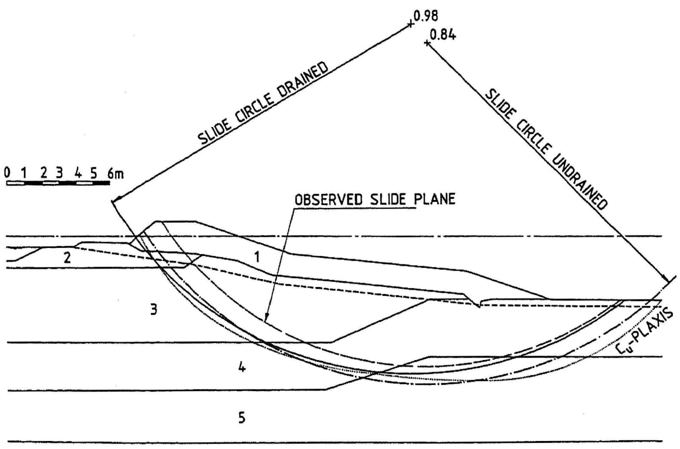
In which: f = flow-criterion
if f = o: plastic soil behaviour
if f < o: elastic soil behaviour
σ_{xx}', σ_{yy}' = normal effective stresses
σ_{xy}' = shear stress

The results of the stability calculations are given in Table 3.

TABLE 3 Results of stability calculations of failures

Calculation method	Safety factor	
	Sliding no. 1	Sliding no. 2
Slide circle drained ($\sigma'_v = \sigma'_{vo}$)	0,90	0,98
Slide circle undrained	0,81	0,84
Plaxis undrained	0,81	0,84

Figure 6 shows the slip-circles for the second sliding observed at the spot as well as found by means of calculations.



Layer no.	Soil type	γ_n (kN/m³)	Ø (dgr)	c (kN/m²)
1	Sand, clay, debris	15,5	25	0
2	Various	10,2	18	5,8
3	Peat	10,2	19	4,5
4	Clay, veg. remains	11,9	23	3
5	Clay	13,3	21	2,5

Fig. 6 Slide-circles for the second instability

Contradictory to the slide-plane methods the critical plane in Plaxis is found as result of the calculations. Figure 7 shows the displacement increments at failure. It has to be noted that not an exact circular slide plane is found.

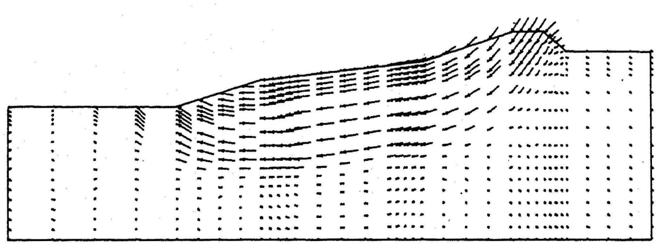


Fig. 7 Displacement increments for the second sliding

The calculated failure circles are rather comparable, Figure 6.
Also the safety factor determined with the c_u-method.

The safety factor established with the method in which drained soil parameters and hydrodynamical pressures are used gives much higher values. Especially at the passive section of the slide-plane (the toe of the dike) this method takes into account to favourable values for the shear strength. It is found that large groundwater pressures occur far beyond the dike profile which cannot be explained from a normal elastic theory.
In Figure 7 the results of measurements from waterpressuremeters in an other cross-section of the dike are given. It proved that the raise of the hydraulic pressure in the meters 1 and 2 passably corresponds with the weight of the loading; meter 3 however shows extensive pressures beyond the toe of the dike. The latter is taken into account by the c_u-method.

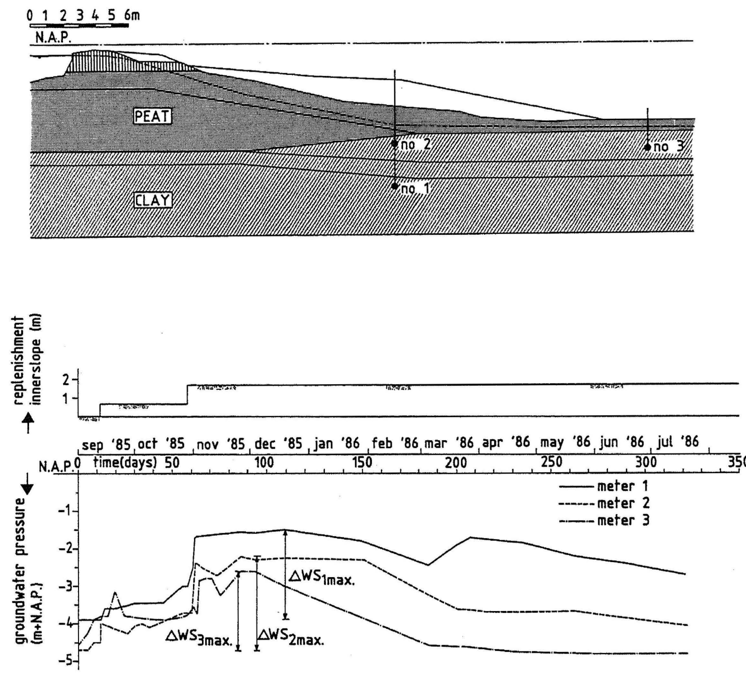


Fig. 8 Waterpressure measurements in cross-section 17a

CONCLUSIONS

Since variations in the subsoil can occur within short distances not only along the dike-axis but also perpendicular to this it is for this kind of dikes very important that geotechnical investigations are carried out in such a manner that discontinuities can be detected adequatly.

The short term stability (execution phase) is decisive. Using a slip circle analysis and taking into account drained soil properties and hydrodynamic pressures corresponding to the weight of the replenishment ($\sigma'_v = \sigma'_{vo}$) results in high safety factors.
The method that applies to the apparent cohesion of the soil gives much lower values and has also a better theoretical background. Also the large pore water pressures occuring beyond the dike-toe are taken into account.

A slide-circle calculation according to the c_u-method can be carried out very easily and in case of circular slide planes provides the same results as the more advanced finite element method.

ACKNOWLEDGEMENTS

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