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Case History of an Uplift Problem

J. R. Deutekom
Delft Geotechnics, Delft, The Netherlands

R. J. Termaat
Ministry of Public Works, Delft, The Netherlands

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J. R. Deutekom
Project Specialist, Delft Geotechnics, Delft, The Netherlands

R. J. Termaat
Head of Geotechnical Research, Ministry of Public Works, Delft, The Netherlands

SYNOPSIS Deformation and related stability problems of dikes in the western parts of Holland were for many years ascribed to creep and excess pore water pressures only. However in the spring of 1988 during a high water period in the river Rhine the so called uplift mechanism was for the first time measured in practice. It was then recognised that in some cases the problems have been caused by the uplift phenomenon. For a few years the pore water pressures and the deformations were observed in a test site. With the results of the observations a procedure was formulated for design and reconstruction.

INTRODUCTION AND LOCATION OF TEST SITE

In the western parts of Holland the ground level lies usually lower than the mean river level. For that reason the land is separated from the rivers by water retaining structures called dikes or levees. These dikes exists usually of impermeable heavy clay and which are constructed on a relatively impermeable and weak subsoil of peat and clay. The pore water pressure in the sand beneath this weak layer is related to the water level of the river, in case the riverbed is in direct contact with this sand layer, as illustrated in figure 1.

This paper describes a dike section near the town Bergambacht (20 km east of Rotterdam). After reconstruction 12 years ago the road on top of that dike showed severe damage due to deformations. Maintenance was necessary each year. Also the toe of the dike moved landwards.

In the beginning of 1988 the local authorities of that area decided to investigate the origin of the problems. Therefore Delft Geotechnics carried out a small investigation programm including cone penetration tests, measurement of pore water pressures and stability analysis based on Bishops method.

The first result of the investigation was a geotechnical profile of the concerning dike near the town Bergambacht (figure 1).

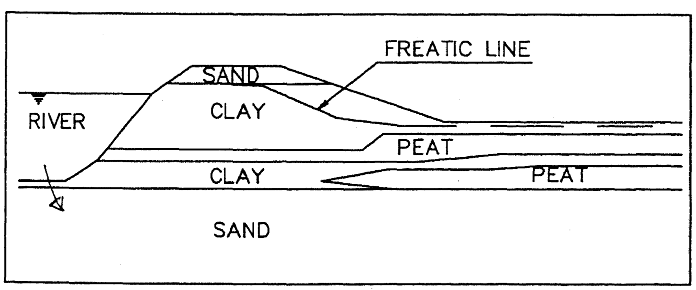

Fig. 1. Geotechnical profile of test site.

The second result was the pore water pressure distribution in the profile. In the relatively impermeable toplayers a water overpressure was measured. The overpressure was at that time ascribed to creep. The pore water pressure in the sand layer showed the expected relation with the tide of the river.

The last result was the Bishop stability analysis showing the expected low stability.

At that time with the results of the investigation it was concluded that the problems were caused by creep in combination with low stability.

OBSERVATION AND DESCRIPTION OF UPLIFT MECHANISM

After finishing the above mentioned investigation programme the measurement of pore water pressure was continued. In the spring 1988, during an extreme high water period in the river Rhine, it was suddenly observed that pore water pressure measurements in the sand reached an ultimate value while the river level was still increasing (figure 2).

At that time it was recognised that the deformation problems were not caused by creep, but by uplift of the weak toplayer at the passive zone of the dike. The measurement of the pore water pressures had reached the same value as the weight of the weak toplayer (figure 3).

During uplift conditions there will be a zone between the weak toplayer and the sand layer underneath, where no shear strength is present. Even before the uplift condition is reached the shear strength will reduce strongly. This phenomenon is of great importance for the evaluation of the stability of water retaining structures.
Uplift means a change in stress distribution. The shear stress along the sand layer is reduced and can even be zero, while the initial horizontal stress in the weak layer increases (figure 4). This causes relatively large horizontal deformations of the weak toplayer and temporary instability of the water retaining structure.

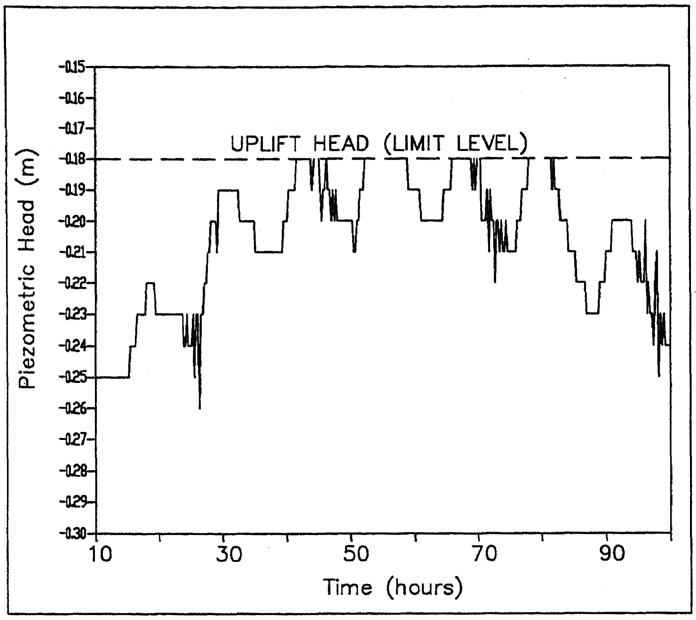

Fig. 2. Observation of the uplift mechanism.

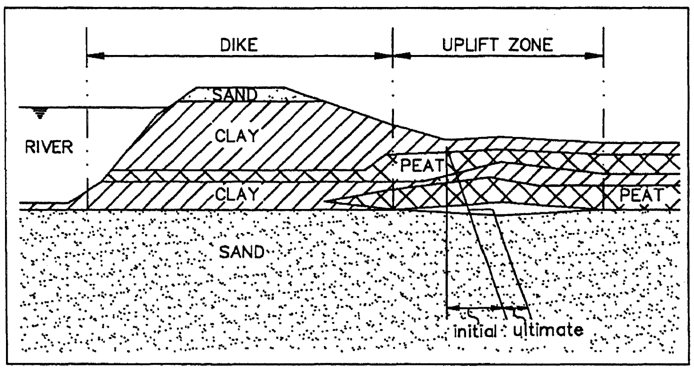

Fig. 3. The uplift mechanism with initial and ultimate pore water pressures.

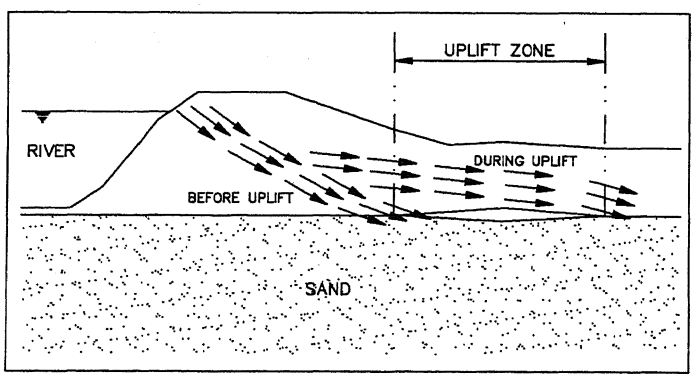

Fig. 4. Shear stresses before and during uplift conditions.

UPLIFT AND HORIZONTAL DEFORMATIONS

After the observation of the ultimate pressure at Bergambacht the Dutch Public Works Department in association with Delft Geotechnics decided to start a monitor programme, which includes monitoring of the horizontal deformations and pore water pressures for a long period (figure 5).

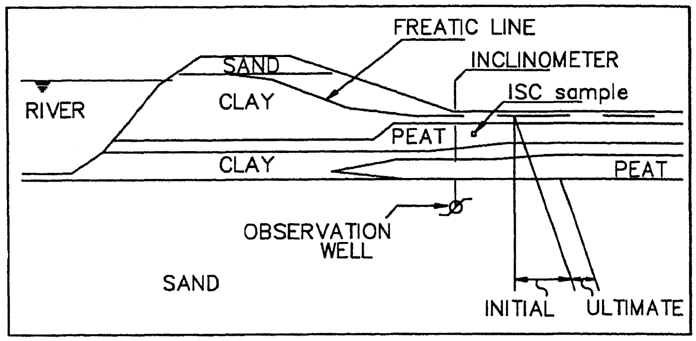

Fig. 5. Instrumentation of test site at Bergambacht.

The main result of the monitor programme was the relation between the water level of the river which is related to the pore water pressure in the sand and the horizontal deformation of the toplayer of the dike (figure 6).

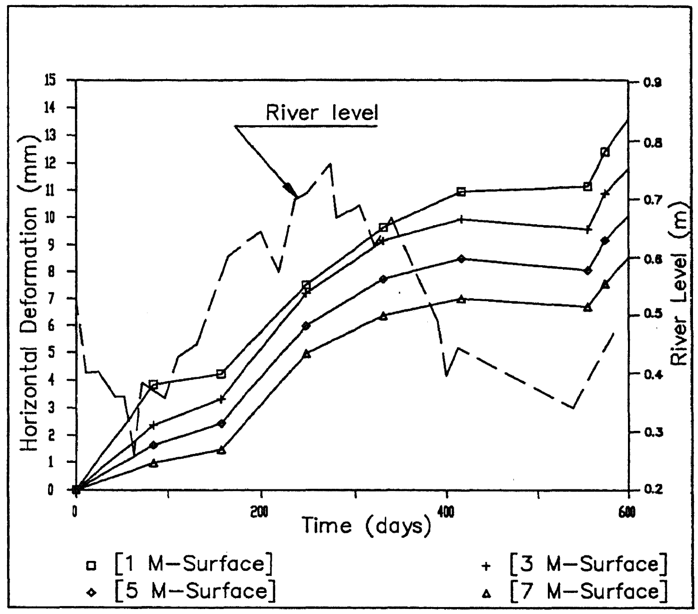

Fig. 6. Relation between tide of the river and horizontal deformation of the toplayer of the dike.

Clearly can be seen the increase of horizontal deformation velocity with increasing water level of the river and decreasing horizontal deformation velocity with decreasing water level of the river. When the water level in the river reduces, consequently the water pressure in the sand reduces too. At a level of 70% of the uplift pressure the velocity reduces to zero. The effective stress between weak toplayer and sand will probably increase and consequently the horizontal stress in the weak layer will reduce. This implies that the observed horizontal deformations of the weak toplayer behind the dike were caused by uplift and not by creep.

STABILITY AND DEFORMATION ANALYSIS

In case of uplift or nearly uplift the classic stability analysis can not be used. The slip surface will partly stay at the boundary of the weak layer and sand. Not only the stability approach is different, but the deformations have to be considered too. In the Dutch Guidelines two methods are recommended (Termaat, 1991).

The first method is based on a non circular slip surface method, i.e. Morgenstern & Price. In this method firstly the stability is checked. When the stability is sufficient then the lateral compression is determined. In this, only the horizontal part of the critical slip surface is considered. This horizontal part is analog to a beam which is loaded by compression. The load boundary conditions follow from the stability analysis, actually the interlamel force in the horizontal part. For each lamel the deformation is calculated by Hooke´s law assuming undrained conditions and neglecting the shear force and the excentricity. The deformations are calculated for additional changes in river level, because the deformations due to the normal water level can be equalised immediately after completion of the construction. The undrained conditions are used for both the stiffness and the shear strength, because the additional increase of water level span mostly a short time period.

The second method, which is used for calculation the effects of lateral compression, is a finite element approach with a linear elasto-plastic stress strain relation. The advantage of this method is that the deformation and stability are coupled (Vermeer, 1990).

In both methods the stiffness is an important parameter. Special attention was paid to this parameter as described in the next paragraph.

PERMANENT INCREASE OF HORIZONTAL STRESS AND STIFFNESS AFTER UPLIFT

After each uplift or semi uplift period the stiffness of the weak toplayer (the compression beam) will increase. In the field the horizontal stiffness was measured by Pressuremeter tests. In the laboratory the increase of stiffness was simulated in an Independant Stress Controll Cell (ISC-Cell).

Four cubic samples were taken from the weak toplayer behind the dike. In the ISC-Cell the three directions of the samples were kept under the same conditions as in the field during uplift conditions. This means that the vertical direction of the sample was kept under plain stress conditions simulating a constant vertical stress. The horizontal direction parallel to the dike axes was kept under plain strain conditions simulating a two-dimensional situation. In the other horizontal direction the uplift mechanism was simulated by increasing the horizontal stress under undrained conditions from the stress before uplift to the stress during uplift. After the undrained increase of horizontal stress the horizontal deformation was fixed and from that moment drainage was allowed until the end of consolidation and relaxation. The test programme had three cycles.

The horizontal stress index observed in the four ISC tests, after 1, 2 and 3 loading cycles shown a permanent increase of horizontal stress index ratio after each uplift period. Figure 7 shows a typical test result. From this figure the increase of the horizontal stress index can be derived. In the same figure a square hyperbolic fitting is given together with the 90% reliability interval.

This has resulted in an avarage formula for the horizontal stress index Kn after N uplift periods, dependant on the initial horizontal stress index Ko and the number of uplift periods N:

$$Kn = Ko * \left(1 + \frac{N}{1.96 * N + 4.93} \right) \qquad (1)$$

For design purposes the 5% lower bound formula has to be used:

$$Kn = Ko * \left(1 + \frac{N}{3.18 * N + 8.39} \right) \qquad (2)$$

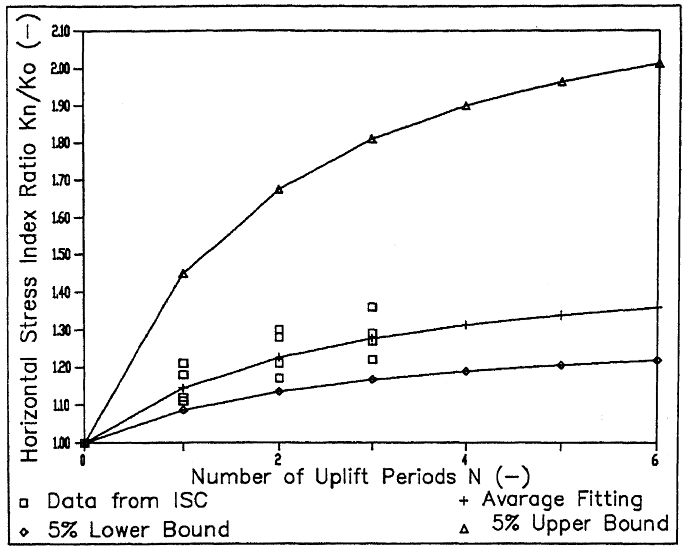

Fig. 7. Typical test result and square hyperbolic curve fitting of stress index ratio and number of uplift periods.

Field observations with the Pressuremeter shown an index ratio of Kn/Ko of 1.23 between the uplift zone and the undisturbed landward zone. This means that in the field 2 uplift periods have occurred according to the avarage ISC formula .

The results also shown a permanent increase of horizontal stiffness. The increase of stiffness can be expressed by the stiffness ratio Gn/Go. The observed ratio´s in the four ISC tests after 1, 2 and 3 loading cycles are plotted in figure 8 together with a square hyperbolic fitting and a 90% reliability interval.

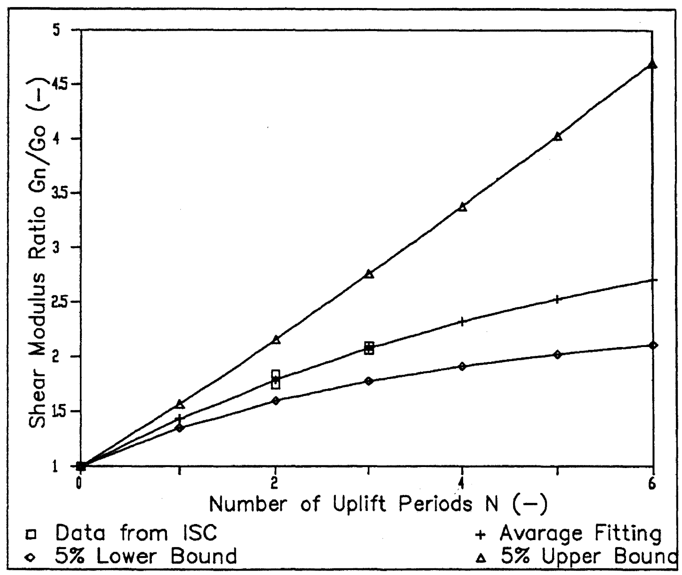

Fig. 8. Test results and square hyperbolic curve fitting of shear stiffness ratio and number of uplift periods.

This has resulted in an avarage formula for the horizontal stiffness Gn after N uplift periods dependant on the initial stiffness Go and the number of uplift periods N:

$$Gn = Go * \left(1 + \frac{N}{0.24 * N + 2.06} \right) \qquad (3)$$

For design purposes the 5% lower bound formula has to be used:

$$Gn = Go * \left(1 + \frac{N}{0.51 * N + 2.33} \right) \qquad (4)$$

With Pressuremeter tests a stiffness ratio of Gn/Go of 1.79 was measured. This means that in the field 2 uplift periods have occurred according to the avarage formula.

IMPLEMENTATION OF RESULTS IN PRACTICE

With the results of the field observations and the laboratory testing a plan for the reconstruction of the river dike was made, taking into account the houses at the toe of the dike. A soil improvement was realised within the inner side of the dike in combination with a drainage system (figure 9). This improvement makes sure that there will be a limit stress increase and consequently a limit deformation of the weak toplayer. After the reconstruction the monitoring of horizontal deformations still continue. Until now no further deformations were observed (figure 9).

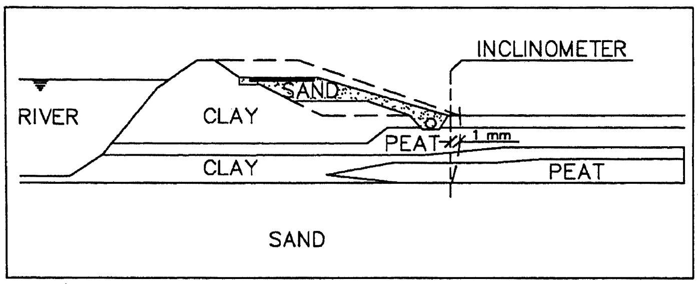

Fig. 9. Cross section of reconstruction and measured deformations.

CONCLUSIONS

After the field observations, the laboratory testing and the lessons from the reconstruction three conclusions were formulated:

1) Be aware of uplift.
2) Calculation of both stability and deformation are required under uplift conditions.
3) The increase of stiffness after each uplift or semi uplift period plays an important role in the history of the dike and must be taken into account in case of reconstruction.

REFERENCES

Deutekom, J.R. (1991), "Case History of an Uplift Problem", Delft Geotechnics, Delft.

Termaat, R.J. (1991), "Dams on Soft Soils: Lessons from the Dutch Delta Projects", International Commission on Large Dams, Vienna.

Vermeer, P.A. (1990), "Users Manual Plaxis", Balkema Publishers, Rotterdam.