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Impact of a ditch on dike stability for uplift conditions

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ABSTRACT: Slope stability for water retaining structures under design load might be endangered by the uplift phenomenon. The phenomenon can occur in subsoil layering including a permeable, sand, layer that is overlain by a low-permeable, clay or peat layer. Hydraulic contact between the permeable layer and the sea or river, will increase the hydraulic head and might lift the low-permeable, cover, layer at the toe of the dike under extreme conditions. As a consequence the available shear strength at the interface of the permeable and low-permeable layer reduces to zero. Several studies have shown this phenomenon and its impact on slope stability. This paper discusses a centrifuge test series on the uplift phenomenon with a specific focus on the presence of a ditch, which in some regions is typically present to either drain the dike body and / or the land protected by the dike. The ditch can be considered as a weak spot as the reduction in cover layer thickness near the ditch enhances local uplift. The test series contained 6 tests, 3 without a ditch and 3 with a ditch. The tests showed that the ditch has a strong localizing effect on uplift and uplifting does not or marginally extend beyond the ditch. Cracks through the cover layer allow for an hydraulic connection between the permeable layer and the ground level. Remarkably, when the hydraulic connection is established, the uplifted cover layer settles somewhat, but remains in uplifted position. This has a strong practical application, indicating the hydraulic connection does not act as an pressure relief well, in the sense that it does not prevent the cover layer from being lifted.

KEYWORDS: Safety against flooding, centrifuge testing, slope stability.

1 INTRODUCTION

Worldwide, flooding is one of most threatening natural hazards (CRED, 2025). Protection against flooding relies strongly on water retaining embankments, or dikes. The expected impact of climate change requires the need for stability assessment of existing structures under increasing loading conditions (Janga et al., 2024). As such, increasing safety against flooding requires an increase in understanding of the phenomena that threatens dike stability.

Design conditions for water retaining structures not only include the level of the free water, river or sea, acting against the dike body, but also includes the subsoil conditions (CIRIA, 2013). In deltaic regions the subsoil might contain a succession of permeable and non-permeable layers. When a permeable layer is in hydraulic contact with the river or sea at the front of the dike, a rise in the free water level causes a rise in hydraulic head in this layer. The rise in hydraulic head might reach the level at which the pore pressure at the top of the permeable layer equals to the weight of the cover layer. The cover layer might be lifted, crack and backward erosion piping could start. Moreover, the reduced friction between the cover layer and the sand layer might enhance a slip failure of the dike body, which is the topic of this paper.

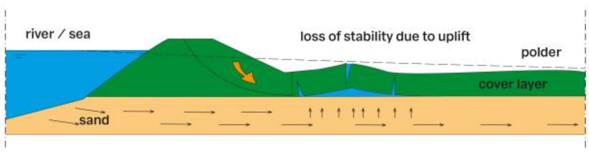

Figure 1. Sketch of the uplift phenomenon and corresponding, possible, loss of stability, from: Zwanenburg et al (2024).

The uplift phenomenon in relation to dike stability was first described by Cooling & Marshland (1953) and studied by model testing by Petfield (1978), Petfield & Schofield (1983), Allersma et al. (2002), Van et al (2005), Zwanenburg et al. (2024).

The most striking outcome of these studies was the difference in failure mechanism found for the situation with and without uplift. The situation without uplift often results in a circular, Bishop-type, failure plane. For cases that include uplift, a passive failure plane, through the cover layer, was rarely found. Instead, the cover layer was compressed horizontally and / or the uplifted part of the cover layer was pushed up further at the development of the active sliding plane.

To control the groundwater in the dike body as well as in the land protected by the dike a ditch might be present at the toe of the dike. The tests mentioned above do not account for the presence of a ditch. Depending on its size and distance from the toe, the ditch might attract and localize the uplift phenomenon. Moreover, the presence of a ditch might weaken this system or localize deformations such that, in contrast to the situation without a ditch, a passive failure mechanism will develop.

To test the impact of a ditch, a centrifuge test series is conducted in addition to the series presented by Zwanenburg et al (2024). This paper discusses the expected hydraulic head development in the sand layer in Section 2, followed by the test set up and testing programme in Section 3. Section 4 describes some of the results which are analysed in Section 5. Section 6 discusses the results followed by the conclusions in Section 7. Due to space restrictions, this paper focusses on the uplift phenomenon. A detailed description of the observed slope stability is to be published later.

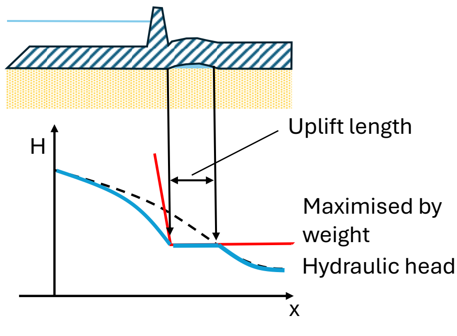

Figure 2. Development hydraulic head under uplift conditions (TAW 2004), H represents hydraulic head in sand layer, x the distance from the river.

2 HYDRAULIC HEAD UNDER UPLIFT CONDITIONS

Figure 2 sketches the development of the hydraulic head in the sand layer (TAW, 2004). The graph shows the hydraulic head, H, as a function of the distance from the river, x. If uplift would not occur, the black dotted line gives the expected hydraulic head. Design conditions typically include a river level and a corresponding high hydraulic head that reduces for increasing distance to the river (Barends 1982). The pore pressure at the top of the sand layer is maximized by the weight of the cover layer. This condition is shown in Figure 2 by the red line, where starting at the toe of the dike to some distance further away from the toe, the red line lays below the black dotted line. This represents the section in which the cover layer is lifted. The resultant hydraulic head, at the top of the sand layer, is given by the blue line. It declines below the dike, plateaus below the uplifted section of the cover layer and reduces further at increasing distance from the river. Numerical analysis and former centrifuge testing confirms this behaviour (Zwanenburg et al. 2024).

3 CENTRIFUGE TESTS

3.1 *Test set-up*

The test series was conducted additional to the series described in Zwanenburg et al (2024) and Cengiz et al (2024) and uses the same test set-up and test procedures. This section summarizes the main characteristics. The tested models, see Figure 3, were placed at a model box with dimensions, length = 870 mm, width = 200 mm and height = 450 mm.

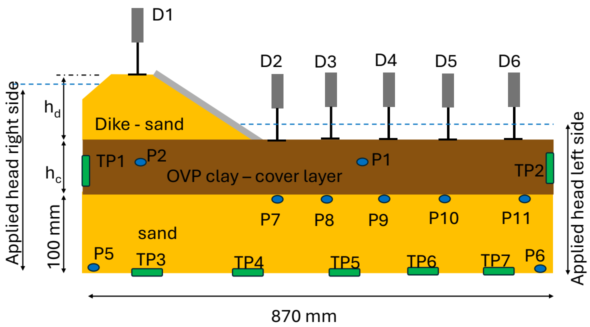

Figure 3. Sketch test set-up, not to scale, according to Cengiz et al. (2024)

The model contained a 100 mm thick coarse Baskarp B25 sand layer. At the left model boundary the hydraulic head in the sand layer could be raised, while an outflow at the right model boundary kept the hydraulic head constant.

The clay layer was built from remoulded clay from the Oostvaardersplassen, OVP, approximately 30 km east of Amsterdam. The clay was remoulded at a water content, w = 1.15×w_l, in which w_l represents the liquid limit. After remoulding the clay was consolidated in the model box at a vertical load, σ_v = 40 kPa. The target thickness of the clay after consolidation was 30 mm.

The dike body was constructed from fine Baskarp B15 sand. The target height of the dike was 90 mm and slope angle of 1(V):1.5(H). During construction the sand body was densified layer for layer by wet tamping to a relative density, R_e = 90%.

A 10 mm thick artificial, Vingerling, clay covered the slope to prevent superficial sliding and erosion of the slope. Characteristics of the clay and sand layer are given in Zwanenburg et al. (2024) and Beroya-Eitner et al (2024).

A 10 mm thin water layer was added on top of the OVP clay, to avoid the clay from drying out and prevent the development of, unknown, suction in the clay. An open outflow at the right boundary maximized the water layer thickness, while the actual thickness was measured as explained below.

The instrumentation contained the following, for the position of the numbered items see Figure 3:

- Pore pressure in the clay layer, P1 and P2
- Pore pressure in the dike body, P3 and P4
- Pore pressure at the inflow opening, P5
- Pore pressure at the outflow opening, P6
- Pore pressure at the clay – sand interface, P7 – P11
- Total pressure at the left and right model boundary in the clay, TP1 and TP 2
- Total pressure at the bottom of the model, TP3 to TP7
- Vertical displacement of the crest, D1
- Vertical displacement of the cover layer D2 to D5
- Changes in water level D6, for this purpose a floating footing was applied to the D6.

The model box contained a transparent window allowing Two L-VIT 2500 Color cameras, 7,920 ×6,004 pixels to make photos at 0.5 Hz during spinning up and 1 Hz during the actual test.

3.2 *Testing programme*

The testing programme contained 6 tests, see Table 1. The first two tests were reproductions of the tests from the previous series and were meant to test repeatability. The third test had the same configuration, but uplift was not applied. This test was meant to study the effect of uplift on dike stability. The tests 4, 5 and 6 did contain a ditch configuration. Each of the ditches had a depth of 15 mm on model scale, just reaching the middle of the cover layer. The slopes at both sides were 1(V):2(H). The following variations in ditch configuration was applied, with L_1 and L_2 according to Figure 4:

- Test 4, narrow ditch near toe, L_1 = 20 mm, L_2 = 0 mm
- Test 5, wide ditch far from toe, L_1 = 170 mm, L_2 = 60 mm
- Test 6, wide ditch near toe, L_1 = 30 mm, L_2 = 60 mm

In practice ditches have a wide variety in dimensions. The dimensions selected here are considered realistic for typical field cases.

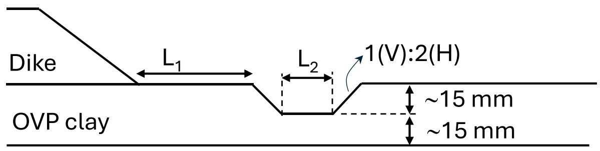

Figure 4. Sketch ditch configuration, L_1 represents the distance between ditch and toe of the dike, L_2 is the width of the ditch at the bottom.

The tests contained the following phases:

1. Pre-consolidation of the clay for 1 g conditions at 40 kPa
2. Preparation of the model, construction dike body and ditch when present, installation of measurement equipment etc.
3. Spinning to 80 g in 2 hours for Test 1 to 5 and 3 hours and 7 minutes for Test 6.
4. Rising hydraulic head at the left model boundary until uplift of the cover layer is clearly visible
5. Rapid increase g level until slope failure
6. Spinning down

Table 1. Testing programme

Test nr	Description
1	Replication previous test series, no ditch
2	Replication previous test series, no ditch
3	Test without uplift*
4	Ditch configuration 1, narrow ditch close to toe
5	Ditch configuration 2, wide ditch far from toe
6	Ditch configuration 3, wide ditch close to toe

* this test will not be discussed in this paper

4 TEST RESULTS

Uplift was reached in all tests in which uplift was intended. Test 3 was intended to be run without uplift and is therefore not discussed in this paper. An impression of uplift is given by Figure 5.

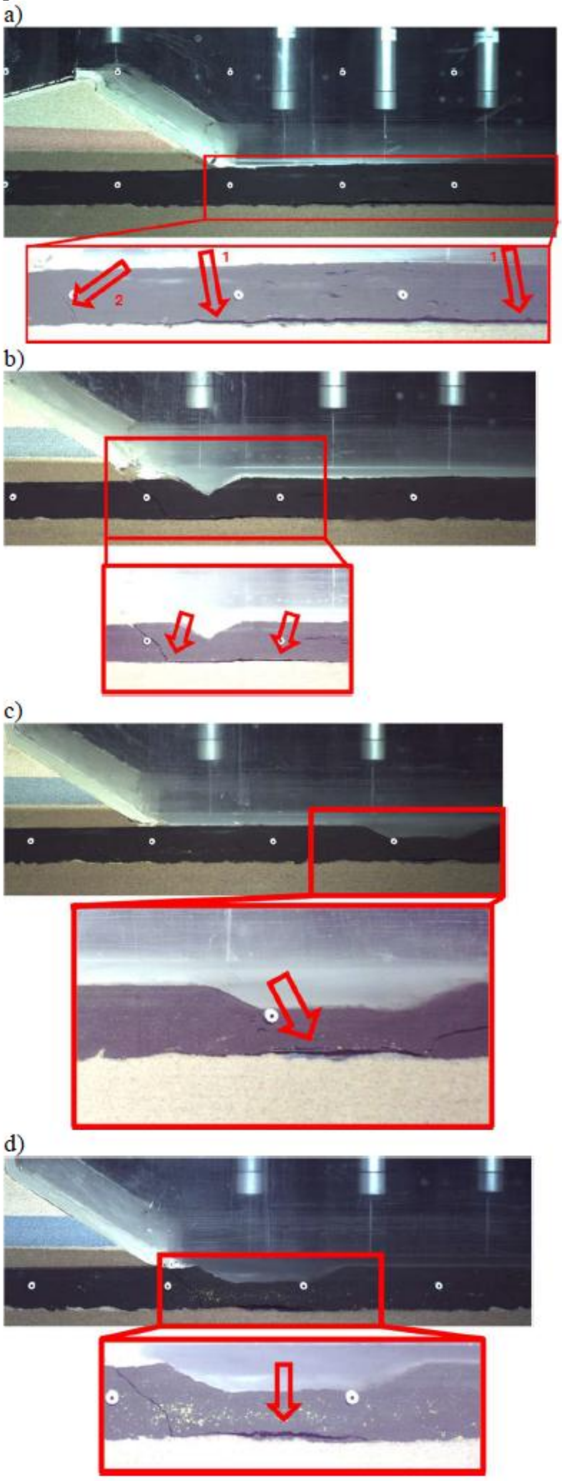

Figure 5. Uplift reached in a) Test 2, no ditch, arrows 1 point towards opening between cover layer and sand, arrow 2 to diagonal crack, b) Test 4, narrow ditch near toe, c) Test 5, wide ditch far from toe, d) Test 6, wide ditch near toe

Figure 5*a* shows the uplift of the cover layer in Test 2. The clay layer is clearly detached from the sand and an opening is visible, indicated by arrows 1. This observation corresponds to the uplift for test 1 and the uplift found in the previous centrifuge testing series (Zwanenburg, 2024).

Arrow 2, in Figure 5*a*, points towards a crack through the cover layer running from the bottom of the cover layer to the toe of the dike and runs from the front to the back wall of the model box. Also this crack, referred to as the diagonal crack, is consistent with the test results of the previous test series, Zwanenburg et al. (2024).

Figure 5*b* to *d* show the uplift observed in the tests 4 to 6. The photos show that the presence of the ditch had a clear impact on the uplift-length, as defined in Figure 2. The uplifted zone was concentrated along the ditch. Only for the combination of the narrow ditch near the dike the uplift extended beyond the dike, but the distance along which the cover layer was uplifted was still clearly smaller than found in the tests without a ditch.

The diagonal crack also developed in the tests 4 and 6, with the ditch near the dike. It is remarkable that this crack did not run between the bottom of the cover layer and the bottom of the ditch, but ran to the toe of the dike. In Test 5, with the ditch far from the toe of the dike, no diagonal crack was found.

5 HYDRAULIC HEAD AND UPLIFT FACTOR

To analyse uplift conditions, the hydraulic head at the sand–clay interface, pore pressure transducers P7 to P11, was examined. The uplift ratio, n, follows from the upward and downward forces acting at the interface. Under the assumption that shear forces in the cover layer can be neglected, the 1D equilibrium in vertical forces reduces to $n = \sigma_v / \sigma_w$, with $n = 1$ representing uplift conditions and $n > 1$ representing no uplift.

The pore pressure at the sand-clay interface, σ_w, is calculated from the measured water pressure, σ, corrected for:

- Measurement position, $\sigma_{position}$
- Levelling factor, σ_{level}
- Calibration factor, f

$$\sigma_w = \frac{\sigma}{f} + \sigma_{position} + \sigma_{level} \tag{1}$$

Six pore pressure transducers were used: P5, P7, P8, P9, P10 and P11. None were located exactly at the sand–clay interface. Therefore, the pore pressure at the interface was calculated based on the vertical distance, Δy, in which P5 was 95 mm below and others 20 mm below the interface. The position correction is:

$$\sigma_{position} = \Delta y \cdot g \cdot N \tag{2}$$

When reaching 80g the hydraulic head in sand layer is initially constant in x-direction in in the sand layer. This was used to calibrate the pore pressure readings, resulting in calibration factor of 1.12 for P7 and 0.908 for P9 and 1.0 for the others.

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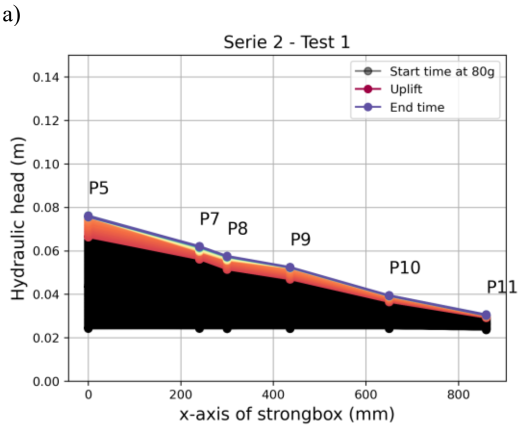

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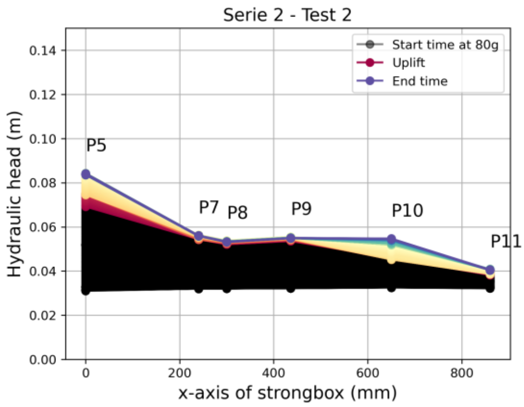

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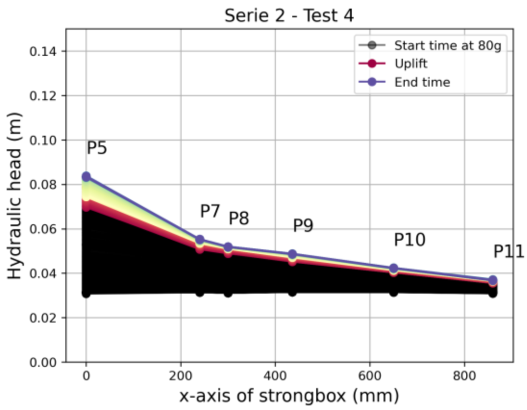

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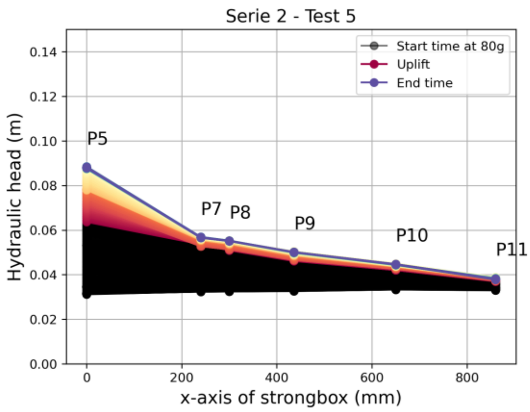

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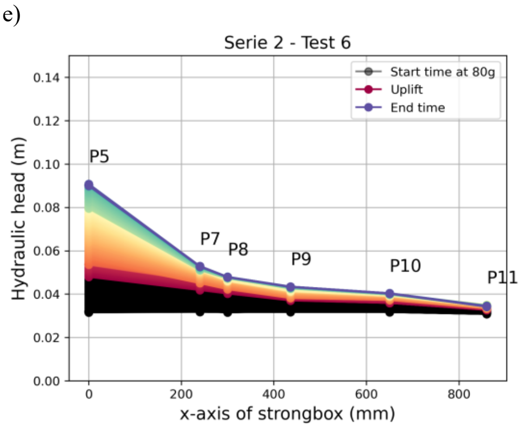

Figure 6. Development hydraulic head, a) Test 1, b) Test 2, c) Test 4, d) Test 5 and e) Test 6

Figure 6 shows the hydraulic head development. For the locations of P5 to P11, see Figure 3. Initially, at the start of testing phase 4, the hydraulic head was constant in the sand layer, as explained above. Then, at the left model boundary, the hydraulic head was raised and kept constant at the right model boundary. The end of Testing phase 4 is represented by the blue line.

Test 2, Figure 6b, showed the plateau along the uplifted zone, between P7 and P10, as predicted by Figure 2. It is unclear why Test 1 showed a different result. Observed anomalies in the preparation phase might possibly explain, resulting in repeating Test 1 by Test 2. The absence of a plateau in the tests 4, 5 and 6 is explained by the presence of a ditch and the short uplift length.

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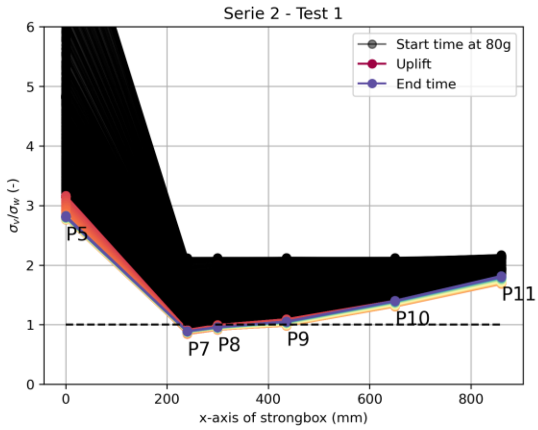

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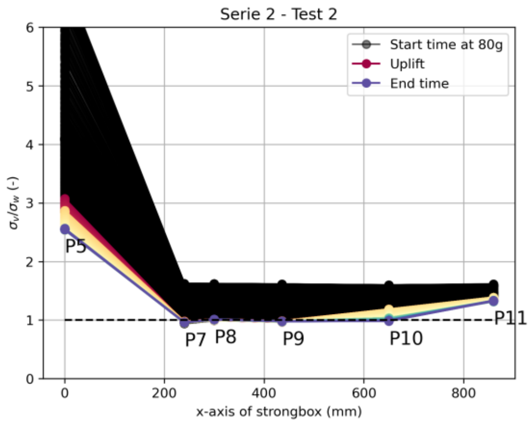

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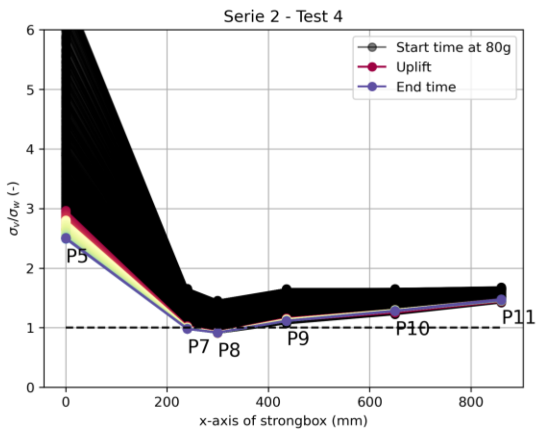

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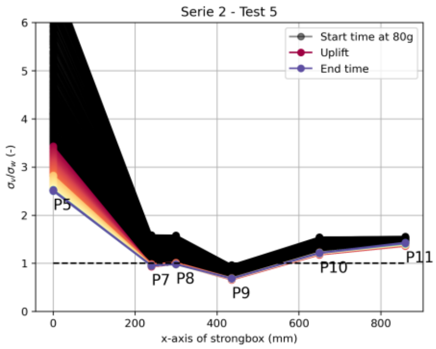

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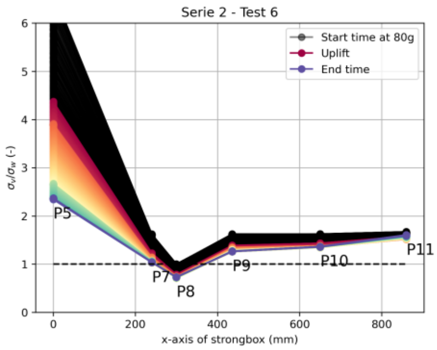

Figure 7. The uplift ratio σ_v / σ_w for a) Test 1, b) Test 2, c) Test 4, d) Test 5, e) Test 6

To compute the uplift ratio, the downward forces, σ_v, acting on the sand-clay interface need to be quantified. These forces result from the weight of: the clay layer, the water above the clay and the dike, which is only relevant for P5. Figure 7 shows the uplift ratio for the different tests. On the left uplift ratio is large due to the weight of the dike. Beyond the toe of the dike, at the right side of approximately x = 250 mm, σ_v is constant due to the constant cover layer thickness. In Figure 7 the initial conditions are represented by the top line. When the hydraulic head rises, the uplift ratio declines. The end of testing phase 4 is given by the blue line. For Test 1 and 2, the uplift factor drops to σ_v/σ_w = 1, see Figure 7a and b. However, the tests with a wide ditch did show that the ratio clearly dropped below 1. This was found at the location of the ditch at the positions zoomed in by Fig. 5*b*, *c* and *d*. An explanation is found by stress distribution and shear stress in the clay layer that is not accounted for in the 1D assumptions in deriving the uplift factor.

6 DISCUSSION

Figure 8 plots the vertical displacements, measured by D2 to D5 during Testing phase 4, the phase in which uplift is realized at 80 g. Information on the change in water depth on the cover layer and pore pressure at P5, the inflow at the left boundary is added to the graphs. The presented displacements, water depth increments and time are related to the start of Testing phase 4. It should be noted that 1 mm of displacement represents 80 mm displacement on prototype scale. The hydraulic head was raised in a stepwise manner, the time between the steps was reduced in Test 4, 5 and 6 compared to Test 2, as the waiting periods did not seem to have added value.

Test 2, Figure 8a, shows uplift at the start of the second step in increasing the hydraulic head. The measurements correspond to Figure 2, Figure 6 and Figure 7. Upon increasing the hydraulic head at the left model boundary, uplift is first found near the toe of the dike. Then the length along which uplift is found increases, indicated by D2, that responds first followed by D3, D4 and D5. D3 reaches the highest peak, which can be explained by the distance to the toe of the dike that might have a restraining impact on the deformations at D2.

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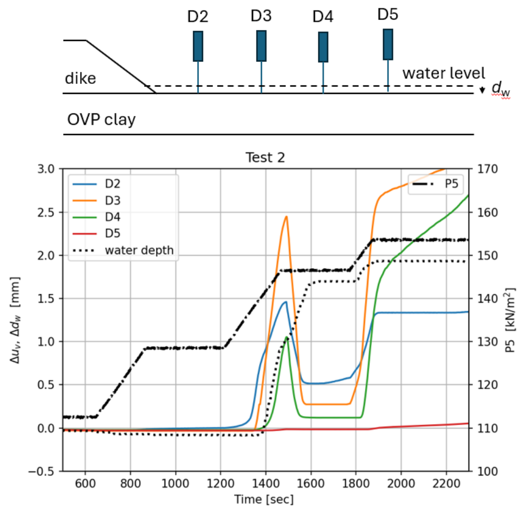

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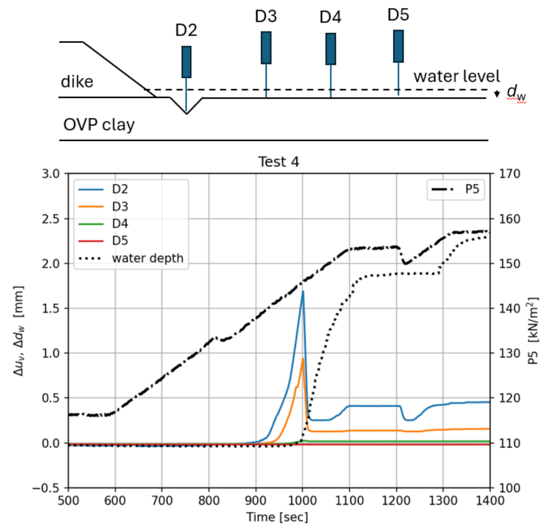

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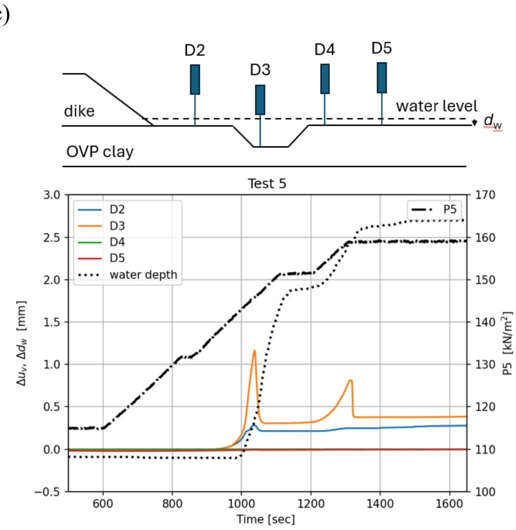

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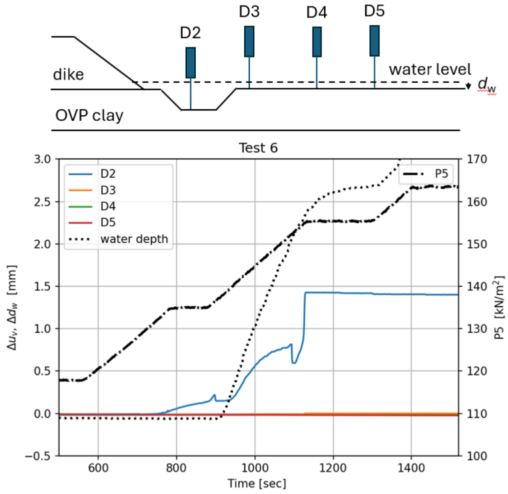

Figure 8. Vertical displacements, Δu_v, change in free water level, Δd_w, and pressure at P5 for a) Test 2, b) Test 4, c) test 5 and d) Test 6, $t = 0$ s represents the start of testing phase 4.

When the diagonal crack develops, see Figure 5*a*, an hydraulic connection is created and water runs to the top of the clay layer. This has an additional downward weight and causes a downward displacement of the cover layer, which however remains in uplifted position. Then, the hydraulic head is raised further, causing a new rise of the cover layer. The water level has reached the opening for free outflow and cannot raise any further.

For the Tests 4, 5 and 6, Figure 8*b*, *c* and *d*, an equivalent pattern is found. No significant vertical displacements are found at the right side of the ditch, D4 and D5 in Test 4 and 5 and D3, D4 and D5 in Test 6. The tests show a rapid uplift of the bottom of the Ditch, D2 in Test 4 and Test 6 and D3 in Test 5. the rapid uplift is found until, due to cracking, an hydraulic connection is realized and the free water level starts to rise. Next, the vertical displacements drop, but uplift remains. Remarkably, Test 5 showed 2 peaks in vertical displacement of the ditch bottom.

The observation that uplift remains, after the development of an hydraulic connection has clear consequences for the safety assessment of dikes. Although, some water from the sand layer can run off freely, the hydraulic connection does not act as a pressure relieve well. The hydraulic head remains intact, which corresponds to the measurement data in Figure 6 and the cover layer remains uplifted. For slip circle analysis this means that the zero effective stress conditions and corresponding low shear stress, at the interface along the uplifted zone remains, despite the presence of hydraulic shortcuts.

7 CONCLUSIONS

An additional series of centrifuge tests was run to study the impact of the presence of a ditch at the toe of the dike on slope stability under uplift conditions. The found failure mechanisms and failure loads of the additional series complied well with the results of the preceding series.

The tests showed that the ditch had an impact on the length of the cover layer that is prone to uplift. The uplifted zone concentrated around the ditch and no uplift was found at the polder-side of the ditch.

Upon uplift cracks in the cover layer were developed. However, despite the hydraulic connection created by the cracks, the cover layer remained in uplifted position. Moreover, the pore pressures, measured at some depth, 20 mm, below the interface between sand and cover layer are not influenced by the free outflow of water to ground level.

This has an impact on the slope stability under uplifted conditions. Expected cracks and corresponding hydraulic connections do not impact the uplift, and the unfavourable uplifted conditions remain.

8 ACKNOWLEDGEMENTS

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