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Centrifuge Tests of Dyke Collapses on Soft Subsoil

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Abstract. This paper presents the results of an experimental investigation on the failure or collapse of dykes with a strong contrast of stiffness between the dyke itself and its foundation layers. In deltaic regions, dykes are commonly built out of stiff sandy or clayey materials and rest on soft foundations layers (e.g. soft clay or peat). Their interaction is largely unexplored and the failure mechanism unknown. The tests presented in this paper aim to highlight the difference in failure mechanisms. The tests consisted mainly of stiff dykes on soft subsoils but a few soft dykes on stiff subsoil were also investigated. The dykes were made out of Speswhite clay or Baskarp sand whereas the subsoil was made out of silicon or clay. The silicon was used to replicate a soft elastic subsoil for which it was possible to control the stiffness. The model was then subjected to increasing gravity up to 100 G-level. The results show that slope failures take place in the dyke when the foundations layers are stiff. It is believed to be the consequence of a build up of pore pressure due to the volumetric contraction of the dyke. On the other hand, soft foundation layers underwent large settlements which in turn deformed the stiff dyke without any slope failure taking place. However, diffused sheared zones were observed in the core of the dyke.

1. Introduction

The Netherlands relies on a vast network of dykes to protect its low-lands from flooding. The rapid urban expansion and the rise of sea levels are putting ever increasing economic and social pressures on these dykes. Many of them were built centuries ago with locally available materials and rest on the natural subsoils which in some areas are very soft [1, 2]. Little is known about the failure mechanism of dykes involving two different materials with a strong contrast in stiffness. This is typically the case when dykes are built on peat. However, some research has been carried out on the behaviour of embankments resting on soft soil [3, 4] but focussed on the failure of the subsoil rather the failure of the embankment itself. In order to understand the influence of the stiffness of the subsoil on the failure mechanism of dykes, a series of centrifuge tests were carried out in which dykes were made out of clay or sand and placed on stiff or soft subsoil. Centrifuge tests offer a cost and time effective way of investigating such behaviours. The design of the centrifuge tests was based on a first attempt to model the collapse of dykes in a centrifuge [7]. Full scale experiments have already been carried out in previous studies such as the IJkdijk project [5] or the field test near Bergambacht [6]. However, these tests mobilised a lot of human, technical and financial resources and are therefore limited to a single case. Centrifuge testing is an alternative with which it is possible to carry out numerous tests within
Figure 1. The beam Geo-Centrifuge at Deltares, the Netherlands

Figure 2. Model A with a stiff sandy dyke and a soft silicon subsoil

a short period of time. In the present study, nine centrifuge tests were carried out in which a different combination of materials, respectively for the dyke and the subsoil, were investigated. The models were subjected to increasing gravity up to 100 G-force. The results show single and multiple failure surfaces resulting in a slope failure or in very large deformation with diffuse shear zones.

2. Geo-centrifuge, model description and test programme

The nine centrifuge tests were carried out at Deltares, the Netherlands, with their Geo-Centrifuge (Figure 1). It is a 300 G-force beam centrifuge with an arm length of 5.5 m and a model weight capacity of 2 tonnes. It allows to test large models in various conditions. In this present study, the tests consisted in modelling the failure of dykes on soft subsoils. The material of the dyke and its subsoil were changed successively in order to investigate the different cases in which a contrast of stiffness would occur. Six tests were carried out on a clayey dyke made out of Speswhite clay and resting on a silicon block. The silicon block was used to replicate soft soils in which the deformations are large. Natural soft soils such as peat are difficult to model in centrifuge tests. Three different silicon blocks were used with respectively different stiffnesses - a soft block, a medium-stiff block and a stiff block. Three additional tests were carried on a sandy dyke made out of Baskarp sand and resting on a soft silicon block or on a remoulded natural clay called Oostvaardersplassen clay [8].

2.1. Model preparation

All models were built in a rectangular strong box (H 450 mm x H 870 mm x W 200 mm) with a glass face. They were built in a plane strain configuration. The dimensions of the dyke and the thickness of the subsoil changed between tests with a silicon subsoil (Model A) and those with the Oostvaardersplassen clay (Model B). The latter was smaller in order to reduce the preconsolidation time due to temporal constraints of the project. Preconsolidation is a necessary step in centrifuge testing in order to initialise the correct effective stress state. Figure 3 illustrates the dimensions of the models and summarises the different tests.

The clayey dykes were made out of Speswhite clay which is a fined grained kaolin with a low permeability ($K_{sat} = 10^{-9}$ m/s). It was prepared in a stiff tub and consolidated with a vertical pressure of 50, 100 or 150 kPa. It was then extracted from the tub with a vacuum crane, cut into a block and placed in the strong box on the silicon block (Fig. 4A-B). The faces of the strong box were smeared with Vaseline in order to reduce the friction between the model and the strong box. The contact between the dyke and the subsoil was carefully prepared in order
Figure 3. Centrifuge model with description of the material used in each tests

to obtain a perfect contact between them. Once the block was in place, the Speswhite clay was trimmed into a dyke (Fig. 4C-D). Speckles were then blown on the face of the dyke in order to facilitate Particle Image Velocimetry (PIV) analysis. The front and rear panel of the strong box were mounted and the container was sealed until the test was performed the next day.

The sandy dykes were made out of Baskarp sand which is uniformly graded silica sand with a dominant grain size of 0.1 mm. The sandy dyke was directly built in the strong box. The silicon block was first placed in the strong box and the sides of the strong box mounted. Then, the sand was placed using the drizzle method [9] in which the sand was pluviated in de-aired water (Figure 5A). This method allowed to control the density and homogeneity of the sand as well as providing repeatability of the tests. The dykes were built to achieve a relative density of 90% ± 5%. The different layers of sand were coloured in order to facilitate the PIV analysis (Figure 5B). The sand was then left to drain by gravity. The unsaturated conditions allowed some suction tension to appear. The sand was then trimmed to the desired dimensions (Figure 5C-D). The strong box was then covered and the test took place the next day.

The silicon subsoil was prepared in the form of blocks and to achieve a specific stiffness. Then, a block was chosen according to its stiffness and placed in the strong box. The dyke was then built on it. The mechanical behaviour of silicon is complex (e.g. viscosity). However, the influence of the time dependent behaviour of silicon was limited due to the short testing time (100 minutes). It was supposed that the silicon was incompressible with a Poisson ratio ($\nu$) of 0.5 and has an elastic behaviour. The secant elastic modulus ($E$) was determined by means of element testing and were respectively for the soft, medium-stiff and stiff blocks 114 kPa, 565 kPa and 1,377 kPa.

Two tests with sandy dykes were carried out with a subsoil made out of a natural clay called Oostvaardersplassen clay. It is a soft silty clay which was remoulded from a natural clay in a tub and subjected to a preconsolidation pressure of 30 kPa (Figure 6A). It was then trimmed to the desired dimension (Figure 6B) and placed in the strong box with a vacuum crane (Figure 6C). The block was then trimmed into a dyke. Speckles were then blown on the subsoil to facilitate the PIV analysis (Figure 6D). The dyke was then built following the aforementioned procedure.
The strong box was sealed and stored until the next day when the tests took place.

2.2. Test programme
Figure 3 shows a table with the different tests. The first series of tests with Model A focussed on the behaviour of dykes made out of Speswhite clay (Tests 1-5 and 9) or Baskarp sand (Test 6) and resting on silicon subsoil. The second series of tests with Model B focussed on dykes made out of Baskarp sand and a subsoil made out of a natural clay called Oostvaardersplassen clay (Test 7 and 8).

The model was subjected to a linear increase in gravity acceleration from 1 to 100 G-force in 100 minutes. Following the scaling laws [10], an increase the stresses which in effect results in a difference in the size of the prototype being tested. With other words, the tests covered a wide range of prototypes and allowed the identification of a critical prototype size. Additionally and for operational reasons, a vacuum is created in the centrifuge chamber at 20 G-force. No other loads were applied to the model.

2.3. Instrumentation
The models were equipped with different arrays of instruments. The deformation of the dyke and the subsoil was tracked with PIV. Four cameras were mounted on the container - 3 on the side view and 1 on the plan view. In order to use PIV, the models were sprinkled with speckles.
or coloured in layers. Additionally, a LVDT with a range of 50 mm and accuracy of ± 0.3% was mounted to measure the displacement at the crown of the dyke. Pore pressure transducers with a maximum range of 300 kPa ± 0.1% were also placed in the soil in order to track both positive and negative pore pressures. However, some difficulties arose during testing. Some sensors lost contacts with the soil and others seem to have been affected by the vacuum in the centrifuge chamber. The vacuum was necessary to decrease the friction between the centrifuge and the ambient air and to allow it to spin up to a 100 G-force.

3. Results of centrifuge tests

3.1. Test 1: Speswhite clay ($\sigma'_{v,0} = 50$ kPa) dyke on - stiff silicon

The first tests consisted of a soft Speswhite clay dyke ($\sigma'_{v,0} = 50$ kPa) on a stiff silicon subsoil ($E = 1377$ kPa). The initial water content ($w$) of the clay was 63% and was higher than expected. The undrained strength ($s_u$) was measured with a pocket penetrometer and had a value of 3 kPa. The dyke was very soft. Two pore pressure transducers were placed in the dyke and showed negative initial pore pressures of -4 kPa and -8 kPa. It implies that some suction strength enhancement can be expected.

During the test, the dyke exhibited large deformations. At only 4 G-force, it experienced a brittle slope failure as shown in Figure 7.1. The mobilised mass flowed on the silicon block. The large deformation of the dyke out ranged the LVDT measurements and no readings were possible. As the acceleration carried on, the dyke continued its large deformation and a second slide took place. This mechanism carried on until the end of the tests by which point the entire dyke was flattened. The pore pressure measurements showed a rapid development of positive pore pressures which suggests that the failures took place in undrained conditions (conservation of the pore water mass within the soil). The build up of these pressures was caused by the volumetric compaction of the dyke.

3.2. Test 2: Speswhite clay ($\sigma'_{v,0} = 150$ kPa) - stiff silicon

The second tests consisted of an overconsolidated Speswhite clay dyke ($\sigma'_{v,0} = 150$ kPa) and a stiff silicon subsoil ($E = 1377$ kPa). The initial water content ($w$) was 53% and its undrained strength ($s_u$) was 12 kPa which was significantly higher than for Test 1. The initial pore pressures measured by the two transducers were -46 kPa and -57 kPa.

As the gravity was increased in the centrifuge, the weight of the dyke was increased and the stiff silicon subsoil settled which caused a differential displacement of the dyke. Additionally, some plastic volumetric deformation took place within the dyke itself, albeit limited. The PIV analysis showed a displacement of 17 mm at the crown of the dyke at 99 G-force. At a 100 G-force, the dyke underwent large deformation including a circular failure surface which resulted in a very brittle slope failure. Both the PIV analysis and the LVDT readings showed that the failure was fast developing. Figure 7.2 shows the dyke after the failure and in which the failure surface is clearly visible. It is believed that the increased strength and stiffness, in comparison to Test 1, limited the volumetric deformation and additional loading was required to develop sufficient positive pore pressure to cause the failure. Furthermore, the initial pore pressures were significantly more negative. The overconsolidation of the Speswhite clay also caused additional brittleness to the material.

3.3. Test 3: Speswhite clay ($\sigma'_{v,0} = 150$ kPa) - soft silicon

The third test consisted in the same overconsolidated Speswhite clay ($\sigma'_{v,0} = 150$ kPa) as in Test 2 but with a soft silicon base ($E = 114$ kPa). The initial water content ($w$) was 52% and the undrained shear strength ($s_u$) was 10 kPa. The initial pore pressures measured by the two transducers were +3 kPa and -37 kPa.
| Layer | Model | Failure
|-------|-------|--------|
| 2 cm | Model A | failure at 100 kgf
| 5 cm | Model B | failure at 100 kgf
| 10 cm | Model A | failure at 40 kgf

**Figure 7.** Results of the centrifuge tests
As gravity was increased in the centrifuge, the mass of the dyke increased and the soft silicon base deformed substantially and formed a dip in which the dyke sat without any slope failure taking place. The dyke exhibited some shearing towards the end of the simulation. It is believed to be caused by excessive deformation of the subsoil and the influence of the stiff boundary conditions (strong box). Figure 7.3 shows the final deformation of the dyke and its subsoil in which the shear bands are marked in red dotted lines. Despite the absence of a slope failure, the dyke has undergone such a large deformation that, at prototype scale, flooding of the low-lands is to be expected.

3.4. Test 4: Speswhite clay ($\sigma'_v,0 = 100$ kPa) - soft silicon
The fourth test was the same as Test 3 but in which the Speswhite clay was consolidated at a lower load ($\sigma_v,0 = 100$ kPa). The silicon subsoil was soft ($E = 114$ kPa) as for Test 3. The initial water content ($w$) was 55% and the undrained strength ($s_u$) 7 kPa. The initial pore pressures measured by the two transducers were -21 kPa and -24 kPa. The results show a similar behaviour as for Test 3. The dyke and its subsoil underwent large deformations without any slope failure taking place. Figure 8 shows the contour plot of the shear strains obtained from the PIV analysis. It clearly shows that, despite the absence of a slope failure, the dyke did develop multiple shear surfaces. Figure 7.4 shows the deformation at a 100 G-force. The deformation of the subsoil formed a dip in which the dyke sat. The final configuration of the dyke is similar to the one observed in Test 3.

3.5. Test 5: Speswhite clay ($\sigma'_v,0 = 100$ kPa) - medium-stiff silicon
The fifth test consisted of the same dyke as for Test 4 (Speswhite clay, $\sigma_v,0 = 100$ kPa) but with a medium-stiff silicon base ($E = 565$ kPa). The initial water content ($w$) was 55% and the undrained shear strength ($s_u$) was 8 kPa. The initial readings of the two pore pressure transducers were -28 kPa and -30 kPa. The results show that the silicon base was deformed by the weight of the dyke. However, the dyke itself underwent larger deformation. At 48 G, two circular failure surfaces developed simultaneously in the dyke and slide in a rotational manner. Figure 9 shows a contour plot of the shear strains and in which the two failure surfaces are clearly visible. Unlike Test 4, the shear deformation was concentrated in these two failure surfaces. Figure 7.5 shows the final configuration of the model in which the two failure surfaces are clearly visible.

3.6. Test 6: Baskarp sand ($D_r = 90\%$) - soft silicon
The sixth test consisted of a sandy dyke made out of Baskarp sand and a soft silicon base ($E = 114$ kPa). The sand was compacted to achieve a relative density $D_r$ of 90% which gave it
a stiff and brittle mechanical behaviour.

The results show that during the increase in gravity, the de-aired water present in the dyke was drained out by gravity. This was due to the increase in weight of the water and facilitated by the increase in permeability. Suction tension was hence increased and an enhancement of the strength and brittleness of the sand is to be expected. Figure 7.6 shows the final deformation of the dyke in which the groundwater table is visible. During the tests, the weight of the sand increased which increased the load on the soft silicon base. The settlement was in the range of the one with a clayey dyke. This settlement obliged the dyke to deform with it. The dyke developed a series circular failure surfaces allowing it to rotate like a knee-joint and can be seen in Figure 7.6. Unlike clay, dense sand does not compact significantly upon isotropic compression but dilates when sheared. Consequently, it was difficult to saturate the pores and develop positive pore pressure. No slope failure was noticed.

3.7. Test 7 and 8: Baskarp sand (Dr = 90%) - Oostvaardersplassen clay
The seventh and eighth tests were duplicates. They consisted of the same sandy dyke as for Test 6 but resting on a remoulded natural soft clay called Oostvaardersplassen clay. The subsoil had an initial water content (w) of 149% and an undrained shear strength (su) of 7 kPa. Two pore pressure transducers were placed in the clay and the initial pore pressures were zero. During the test, the water in the dyke drained out as for Test 6. However, the behaviour of the clayey subsoil differed from the silicon subsoil. The clayey subsoil underwent a continuous plastic deformation with an overall contraction. The entire subsoil is believed to have yielding possibly due to liquefaction (zero effective stress). However, in both Test 7 and 8, shear surfaces developed in the clayey subsoil and propagated to the dyke. The large deformation of the subsoil obliged the sandy dyke to deform and develop shear surface to allow it to deform. In Test 6, a straight shear band developed to the surface whereas in Test 8 circular shear bands developed as in Test 6. These shear bands of Test 7 and 8 can be seen respectively in Figure 7.7 and 7.8.

3.8. Test 9: Speswhite clay (σv,0 = 100 kPa) - stiff silicon
The ninth test was the continuation of Test 4 and 5 in which the dyke is made out of an lightly overconsolidated Speswhite clay (σv,0 = 100 kPa) and rested respectively on a soft and medium-stiff silicon base. In Test 9, it rested on a stiff silicon subsoil (E = 1377 kPa). The initial water content (w) was 56% and the undrained shear strength (su) was 6 kPa. The readings of the two pore pressure transducers showed pressures of -26 and -28 kPa.

The results show very little deformation until 40 G-force after which some large deformation took place. A curved failure surface then propagated from the base to the crown of the dyke and part of the dyke slid as a block. At 80 G-force, a second failure took place but with a circular failure surface. The deformation of the stiff silicon subsoil was smaller than for Test 2. This may be due to the hardening of the silicon in between the tests.

4. Discussion of results and conclusion
The results show that the stiffness of the subsoil plays an important role in the collapse mechanism of the dyke. Stiff subsoils allowed the dyke to develop a failure surface which rose to a slide. Soft subsoils deformed excessively which formed a dip in which the dyke sat. The imposed deformation of the subsoil to the dyke led to diffused sheared zones or 'knee-joint' shear bands giving the dike more freedom in movement. Figure 10 shows the LVDT measurement of the displacements at the crown. It clearly shows that the soft subsoil leads to a progressive and continuous deformation of the subsoil and hence the dyke sitting on it. On the other hand, the results show a stiffer response with the stiff silicon subsoil. The brittle failure of the dyke can be seen with the appearance of a plateau. It is interesting to note that the preconsolidation pressure of the Speswhite clay influenced its undrained strength and hence when the failure
Figure 10. Displacement of the crown during loading for the clayey dykes

will took place. Test 5 with a medium-stiff silicon subsoil and an intermediate preconsolidation pressure of the Speswhite clay confirms this thought. The displacement rate controlled by the subsoil is in between the stiff and the soft case and the failure takes place at similar G-force as Test 9 which has a stiffer subsoil.

It is unclear if the failures took place in undrained (without any groundwater flows) or drained (with groundwater flows) conditions and if the pores were saturated or unsaturated. It is believed that the clayey dykes were largely undrained due to low permeability. However, the pores were initially unsaturated but, as the gravity increased, the pore space decreased and positive pore pressure was able to develop. The readings of the pore pressures confirmed this behaviour. Therefore, it is believed that the failures in the clayey dykes were undrained failures. On the other hand, it is believed that the sandy dykes remained unsaturated throughout the entire tests. Its high permeability was increased as the gravity was increased. Therefore, it is believed that the failures in the sandy dykes are drained failures. For all tests, the initial pore pressure were negative implying that the dyke is unsaturated. The presence of menisci, an inter-particle bonding force, provides an enhancement of the hydro-mechanical properties of the material.

The stiffness of the subsoil is demonstrated with the silicon blocks. The stiff block led to brittle failures as the dyke was free to move on the subsoil. In contrast, soft subsoil settled forming a dip in which the dyke sat. The PIV analyses showed the emergence of diffused sheared zones in the core of the dyke. The dip in which the dyke sat prevented any mass from moving. The two cases with a the Oostvaardersplassen clay subsoil showed that the development of shear surface in the subsoil propagate to the dyke.

The current study highlighted the influence of the subsoil, partial saturation and material properties in the collapse mechanism of dyke. The next phase of the study will cover cases of loose sandy dykes on stiff subsoils. It will be followed by a detailed investigation of the unsaturated conditions and the effect of a drop of air pressure in the centrifuge chamber. A three-phase numerical investigation with the Material Point Method is currently under process.
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