Finite element analysis of levee stability for flood early warning systems

Melnikova, N.B.

Citation for published version (APA):
Chapter 6  IJkDijk case study: prediction of a dike slope failure by the Virtual Dike module

The IJkDijk slope failure experiment in Bad Nieuweschans, the Netherlands has become the ultimate validation of the Virtual Dike module and a winner of a special contest for the best failure prediction organized before the test (http://ijkdijk.rpi.edu). Several commercial corporations and scientific research organizations modelling levee systems participated in the competition; our computational model provided the best class-A prediction for the South levee macro-instability experiment, according to the decision of the jury (http://www.urbanflood.eu, (Koelewijn, 2012)). The Virtual Dike model precisely predicted the simulation mode (slope sliding) and the collapse stage in the loading sequence, while the models from other contestants underestimated stability of the dike (Koelewijn, 2012).

Finally, the dike collapsed due to slope instability, forming a deep slip surface. The experiment was designed and implemented by the IJkDijk foundation in September 2012. The goals of the project were: to enlarge scientific knowledge about physical mechanisms of levee failures and to develop efficient levee monitoring systems predicting various modes of failure well in advance. Another important challenge was to test the ability of numerical geotechnical models to predict the mode of failure and the time of collapse – our computational model was recognized as the best.

6.1 Overview of the IJkDijk tests

The essential aspects of the UrbanFlood EWS design were focused on development of robust sensor systems and efficient analysis modules which are able to detect the weak spots in levees early enough to repair or reinforce the levee. For that, a number of full-scale levee failure experiments with tests of sensor equipment in real-life failure conditions were held. The experiments were carried by the IJkDijk foundation http://www.ijkdijk.nl/en/ijkdijk and studied various mechanisms of levee failure, including slope instability, piping erosion and wave-overtopping erosion. The main objectives of the project are: (a) advancing the scientific knowledge about physical mechanisms of levee failure; (b) developing efficient sensor monitoring systems predicting various modes of failure well in advance; (c) developing and testing simulation analysis tools for correct prediction of collapse mechanisms. Obtaining informative sensor recordings is extremely important for artificial intelligence systems to get adequate time series of normal and abnormal behaviour patterns (Pyayt et al., 2011a, 2014). No less important is the ability of simulation tools to generate realistic patterns for the AI in cases when no real sensor recordings are available (for example failure modes for a stable functioning levee).

5 Parts of this chapter have been published in (Melnikova, N.B., Jordan, D., Krzhizhanovskyaya, V.V., Sloot, P.M.A. (2014) Numerical prediction of the IJkDijk levee breach experiment. Accepted for publication in Proceedings of ICE – Geotechnical Engineering, ICE Publishing)
First IJkDijk experiments took place in September 2008; the two series of full-scale tests were aimed at studying (a) macro-instability failure (Van et al., 2009) and (b) piping erosion failure (Vries et al., 2009). The analysis of the measurement results showed that movement and deformation in the levee are variables that predict a levee breach well in advance: deformations started 42 hours prior to the actual slope failure, precisely at the position where the levee collapsed (Van et al., 2009). The IJkDijk project then was continued with series of experiments in August - September 2012 in Bad Nieuweschans, province of Groningen, NL. Three well-controlled, full-scale test levees were subjected to the loading sequences which could lead to a failure by a number of different mechanisms. The South levee test had been designed to study the macro-instability failure mechanisms: possible failure modes were slope slippage or, alternatively, rupture/uplift of the levee clay cover. Eventually the levee collapsed because of the slope slippage (Koelewijn et al., 2012). For the West and East levees, possible failure mechanisms included: (a) backward seepage erosion (piping) in the sand foundation, (b) fluidization of the sand core and (c) crest overturning followed by erosion (Koelewijn et al., 2012). Ultimately both levees collapsed as a result of the failure mechanism ‘internal overturning’, where water forced its way into the body of the levee over the clay core. The levee softened and then shifted at the levee toe (at the bottom). The levee collapsed at the top and the water flowed over it.

An important challenge was to test the ability of numerical geotechnical models to predict the failure mode and the time of collapse. Before the experiments a call for predictions for the outcome of the tests was announced to the international geotechnical community, to challenge the predictive strengths of models and their users in rather well-known conditions (http://ijkdijk.rpi.edu). The available material comprised all soil investigations including laboratory tests, the design of the test including the test procedure and changes already made to the test conditions as these appeared necessary already and instructions to submit predictions before the start of each test, in order to obtain “class A” predictions in accordance with the classification scheme presented by Lambe in the 13th Rankine Lecture (Lambe, 1973). Several commercial companies and research organizations participated in the competition. According to the committee decision (Koelewijn, 2012), we provided the best “class A” prediction for the macro-instability experiment.

For our prediction, we have used a plane strain finite element model of the levee’s cross-section described by PDE presented in Section 2.4. Concurrent models from the other contestants included: (a) finite element model created in geotechnical package Plaxis with soft soil rheological model for base clay and peat and with Mohr-Coulomb model for sand, and (b) a limit equilibrium model created in Geo-Stability software and employing Bishop’s and Van’s methods for slope stability analysis (Koelewijn, 2012).

Another competition that was held during the tests was a “class B1” prediction in the Lambe’s classification. Several companies which supplied monitoring systems for the tests gave a prediction of the levee failure within 24 hours since the start of each test and then updated this prediction every 24 hours on the basis of their own measurements. Information about the submitted class B1 predictions and evaluation of the monitoring systems performance can be found in (Vries et al., 2012).

When the South levee experiment had been completed, we performed the updated stability analysis on the basis of the reference monitoring data. The results of the analysis
are presented in section 3. Like many geotechnical analysis, this is a ‘class C1 prediction’ according to Lambe’s classification.

6.2 Test site, instrumentation and loading sequence

The IJkJDijk macro-instability experiment was carried out in September, 2012 in Bad Nieuweschans, Groningen province, the Netherlands. A 4-meter high and 50-meter long levee was constructed of sand with a 50 cm clay cover flanking the sides (Figure 6-1a). The ground level in the Nieuweschans area lies around 1 m below the mean sea level. The levee was founded on very soft interleaving layers of clay and peat which in turn lie on a stiff sand layer located about 4 m below the ground. A small auxiliary levee was built from the adjacent soil along the safe slope of the test levee (at the left side in Figure 6-1a), forming a 1 meter deep basin, which was made to illustrate sea-side conditions for the public audience. During the test, the effective height of the levee was increased by excavation of a 2-meter deep trench along the right slope. Six containers with 3 tons weight and 28 m$^3$ volume were installed on the crest of the levee, to be filled with water at certain stages of the experiment.

Construction of the levee was completed three months prior to the experiment, in June 2012, so the excess pore pressures had enough time to dissipate and the soil consolidated. The levee was equipped with a comprehensive sensor system that included piezometers (Geobeads provided by Alert Solutions), inclinometers (SAAF, StabiAlert, and Geobeads), strain and temperature meters (geo textile from TenCate, fibre optics from Koenders) and settlement gauges. The layout of the levee together with the sensor instrumentation is given in Figure 6-1a,b, for front and side views, respectively. Three textile mats from TenCate had been stretched in the sand core along the levee close to the basin slope (see red lines in Figure 6-1a, to reinforce the basin slope to ensure that only the trench slope fails during the experiment. Elevation coordinates of Geobeads sensors are listed in Table 6-2 of section 6.7.

(a)
At the start of the experiment on September 3, 2012, maximal settlement of the levee crest after construction reached 1 meter (in the cross-section A). The experiment started with slow filling the sand core and then filling the basin with water. The clay cover on the slopes provided water containment when saturating the core. The actual loading sequence is described below:

- **3.09.2012 12:30 - 16:00.** Slowly filling the sand core of the dike with water.
  - 16:00 - 4.09.2012 1:00. Continue filling the sand core.

- **Filling the basin at the South side of the dike with water.**

- **4.09.2012 11:23 - 16:30.** Excavation1: excavation of the trench at the North side of the dike to a depth of 1 metre, side slopes 1:1 and a base width of 4 metres (starting at the east side). The excavation formed a berm 1.5m wide at the toe of the dike. The gradual excavation stages are shown in Figure 6-1 (a) with different colours.

- **Checking deformation rate to make sure that deformations caused by the previous excavation had stabilized.**

- **Further excavation of the trench by 1m:**
  - **5.09.2012 10:00 -11:30.** Excavation 2.1: Further excavate the trench by 0.5 m to a total depth of 1.5 meters and a bottom width of 4 meters.
  - **5.09.2012 15:00 -16:45.** Excavation 2.2: Further excavate the trench by 0.5 m to a total depth of 2.0 meters and a bottom width of 4 meters. The filter level of the pore pressure meter in the excavation was -3.15 m from reference sea level. The bottom of the ditch cracked; so the third excavation stage was cancelled and no further excavation was done.

- **Checking deformation rate: low enough**

- **6.09.2012 7:20 - 15:45.** Refilling the sand core to a level 0.25 m above the toe of the dike. Refilling the basin.
• Filling the containers in 5 steps of 0.25 m of water (initially 4 steps of 25 cm were planned):
  o 6.09.2012 17:00-17:30. Filling the containers with 0.25 m of water.
  o 6.09.2012 23:00-23:30. Filling the containers with 0.25 m of water to a level 0.5 m.
  o 7.09.2012 05:00-05:30. Filling the containers with 0.25 m of water to a level 0.75 m.
  o 7.09.2012 08:00-08:30. Filling the containers with 0.25 m of water to a level 1.00 m.
  o 7.09.2012 17:00-17:30. Filling the containers with 0.25 m of water to a level 1.25 m.

• 7.09.2012 9:30 -19:00 filling the sand core further
• 8.09.2012 0:30 -1:00 filling the containers with 0.25 m of water to a level 1.5 m.
• 8.09.2012 4:00 filling the sand core further
• 8.09.2012 8:00 pumping all the water out of the excavation.
• 8.09.2012 9:30-10:00 filling the containers 2,3,4 and 5 up to 1,75 m
• 8.09.2012 14:00 -14:27 –starting fast, forced water pumping into the sand core.
  o End of stage 11: 8.09.2012 14:27 Failure of the dike

At the last loading stage, after 30 minutes of forced pumping, at 14:27 a section of the trench-side slope located across containers 4 and 5 failed. 8 minutes prior failure, deformations became visible for the human eye, from the bulking of the slope of the trench, which at 1 min before failure concentrated in a zone across the specified containers. Within a minute, (at 14:27 pm), this trench slope broke into pieces, after which a slightly more superficial sliding plane occurred and the pore pressures in the sand core decreased quickly (Koelewijn et al., 2012). Figure 6-2 shows the top view after failure.

![Figure 6-2. IJkDijk slope instability experiment: levee failure](image-url)
6.3 Soil properties

Construction of the levee was held three months prior to the test, therefore the underlying clays had sufficient time to consolidate and gain in strength under its weight. For simulation of the levee settlement due to construction, we used drained soil properties derived from the laboratory triaxial tests. Failure of the levee was induced over a fairly short period of time and therefore would occur under essentially undrained conditions, pore pressures not having sufficient time to dissipate. Soil parameters described in sections 6.3.1 and 6.3.2 have been derived from the field and lab tests results by David Jordan (HR Wallingford) under the frame of the UrbanFlood project (Krzhizhanovskaya et al., 2012).

6.3.1 Effective strength parameters

Effective strength parameters of clay and peats in the foundation of the levee have been used for the solution of the initial consolidation problem. The parameters were derived from the results of the consolidated drained triaxial tests carried out on the peats and the clay, presented in Figure 6-3 in terms of the average principal stresses $s' = (\sigma_{v1} + \sigma_{v3})/2$ and $t' = (\sigma_{v1} - \sigma_{v3})/2$.

From the results displayed in Figure 6-3, the following parameters were derived:

- Peat: $c' = 9.7$ KPa, Phi = 27.5 degrees
- Clay $c' = 4.6$ KPa, Phi = 29.4 degrees

![Figure 6-3. Derivation of effective strength parameters for clay and peat from drained triaxial tests: relationship between principal stresses (a) clay; (b) peat](image)

6.3.2 Undrained strength parameters

Undrained strength parameters of clay and peats have been used for simulation of the slope instability experiment which was rather quick (one week), so the behaviour of clay and peats was considered undrained. The undrained shear strain was determined using the results of the Cone Penetration Tests (CPT) performed at the site before the construction. The cone resistance was converted to undrained shear strength by the relationship (Jacobs, 1996):
\[ Cu = \frac{Q_c}{N_k}, \]

where \( Q_c \) is cone resistance (in KPa) and \( N_k \) is a cone factor. Uncorrected cone resistance was used and accordingly an \( N_k=17 \) (Jacobs, 1996).

This gave the variation of undrained shear strength with depth for two of the CPT as shown in Figure 6-4(a,b)– the pink line is the strength for normally consolidated clays from the estimate relationship \( Cu = 0.22 \sigma_v' \), where \( \sigma_v' \) is the vertical effective stress after dike placement.

![CPT plots](image.png)

Figure 6-4. Cone penetration tests: undrained shear strength (KPa) versus depth (m)

The significant aspect of these plots is the higher strengths shown in the upper meter of the ground. This is thought due to desiccation rather than over consolidation. According to the CPT tests, the average shear strength \( Cu=20 \text{KPa} \) was assumed for clay and peat. The undrained shear strength value was used to determine the undrained Young’s modules for clay and peat - values of \( E50 \) (modulus at 50% of the failure strain) were obtained in the laboratory test results and plotted versus undrained shear strength (see Figure 6-5). This gave \( E=580 \cdot Cu \) for clay, \( E=80 \cdot Cu \) for peat.

![Laboratory tests plots](image.png)

Figure 6-5. Laboratory tests: Young’s modulus (MPa) at 50% of the failure strain versus undrained shear strength (KPa)
Summary of soil strength parameters is presented in Table 6-1.

<table>
<thead>
<tr>
<th>Property</th>
<th>core sand</th>
<th>base sand</th>
<th>soft clay</th>
<th>peat</th>
<th>cover clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density, kN/m³</td>
<td>18 (dry) / 20 (wet)</td>
<td>19</td>
<td>14</td>
<td>10</td>
<td>18</td>
</tr>
<tr>
<td>Young’s modulus, MPa</td>
<td>30</td>
<td>150</td>
<td>1.16 (gravity settlement) / 11.6 (reloading)</td>
<td>0.16 (gravity settlement) / 1.6 (reloading)</td>
<td>37.5</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3 (gravity settlement) / 0.49 (reloading)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Effective cohesion, KPa</td>
<td>0</td>
<td>0</td>
<td>4</td>
<td>9.7</td>
<td>75</td>
</tr>
<tr>
<td>Undrained cohesion, KPa</td>
<td>-</td>
<td>-</td>
<td>20</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Effective friction angle, grad</td>
<td>30</td>
<td>31.1</td>
<td>29.4</td>
<td>27.5</td>
<td>32</td>
</tr>
</tbody>
</table>

6.4 Finite element analysis of levee stability and prediction of failure mode

For this case, a purely mechanical model of the dike was considered (pore pressure was determined from hydrostatic distribution). Volumetric loads include gravity load from soil weight and pore pressure load. Boundary loads included containers weight at the crest and water pressure in the reservoir and in the trench. The ground water level of -1 m from NAP (Normaal Amsterdams Peil - the reference sea level in the Netherlands) was taken from the ground water level map of the Netherlands (on the same elevation with the ground level).

Simulations were performed using a non-linear incremental parametric solver, with hydraulic and structural loads defined as functions of pseudo-time \( \tau \). Simulation stages in pseudo-time were defined so as to represent the actual loading stages of the experiment described in Section 6.2. The total number of simulation stages was less than the actual number of experimental stages due to some simplifications: multiple experimental steps of filling the containers were combined into one simulation stage; steps of filling the sand core with water were combined, too. An automatic pseudo-time stepping procedure was used to optimize the speed of non-linear solution. A slow incremental growth of the loads provided a slow variation of the stiffness matrix in pseudo-time. Some of the results of the macro-stability analysis have been presented in (Simm et al., 2012b).

6.5 Results of the FE simulation

Figure 6-6 shows displacements fields at different loading stages. After the trench excavation, maximal simulated displacements concentrated at the trench slope due to soil expansion (Figure 6-6a). Formation of a deep sliding surface started with filling the containers on top of the levee with water (Figure 6-6b). At the collapse stage, the sand core of the levee was pumped up with water - floatation of the levee is shown in Figure 6-6c.
Figure 6-6d presents horizontal displacements, with their asymmetry preserved from the container loading stage; the sliding surface shape is similar to that shown in Figure 6-6b.

(a) Total displacements (m) after excavation
(b) Total displacements (m) after container filling
(c) Total displacements (m) at the collapse stage
(d) Horizontal displacements (m) after container filling

Figure 6-6. Displacements in the levee during the test, numerical simulation: (a) full excavation stage; (b) full container load stage (displacements scaling factor 5); (c), (d) – total and horizontal displacements at the collapse stage

Simulated failure of the levee happened at the last loading stage, like it was in the actual experiment. Strength reduction factor obtained from the finite element simulations was $SRF = 0.95$.

(a) Effective plastic strains after excavation
(b) Effective plastic strains at the collapse stage

Figure 6-7. Effective plastic strains: (a) full excavation stage; (b) collapse stage

Figure 6-7a gives effective plastic strain distributions after the trench excavation, with strains concentrated at the corner of the trench slope toe. In real-life experiment, the excavation finished with cracking of the bottom trench (Koelewijn et al., 2012). Distribution of the effective plastic strains at the last stage is shown in Figure 6-7b. Maximal intensity of simulated plastic strains is observed in a spot at the toe of the trench slope. Divergence of numerical solution was caused by intensive plastic strains developed at the trench slope toe. These results afforded to draw a conclusion that the collapse would
start from the trench slope failure and then will be developed in the form of a deep sliding surface observed in Figure 6-7b,d and Figure 6-7b. The shape of the slip surface agrees with the real experiment (see Figure 6-2).

6.6 Comparison of simulated data and sensor data

![Recorded side tilt vs. and simulated displacement in AG2](image)

Figure 6-8. Measured tilt and simulated total displacement in the AG2 sensor

Qualitative comparison of tilts [degrees] measured by Geobeads sensors with simulated displacements [metres] is presented in Figure 6-8, for the virtual sensor located at (-15.5;-1.5) - under the trench-side toe. The comparison shows that the computational model adequately predicts real sensor dynamics, including response of virtual sensors to excavation, container loading and core water pumping. This proves that virtual sensors time series can be used for feeding artificial intelligence systems with abnormal behaviour curves, for slope instability cases training. A detailed analysis of sensor tilt recordings is given in section 6.7.2.

6.7 Sensor data analysis

6.7.1 Water level and pore pressure measurements

The experiment started on the 3rd of September with slow filling the sand core and then filling the basin with water. Water levels measured in the basin and in the trench are presented in Figure 6-9(a). Basin water level increased slightly with pumping of water into the sand core. These increases were interspersed with small reductions in level during the breaks in pumping, due to diffusion of water into the adjacent soil. Water level in the trench slowly increased after the excavation was completed due to seepage from the surrounding ground. On 8 September, 9:00, the water was rapidly pumped out of the excavation at 9 a.m.

Variation of water level in the containers on the crest is shown in Figure 6-9(b). Seven stepwise stages of containers filling are displayed.
Figure 6-9. Sensor measurements: (a) basin and trench water levels; (b) water level in containers; (c) pore pressure

Figure 6-9(c) presents variation in pore pressure recorded by the Geobeads sensors. Upon initial filling of the sand core piezometers AG1, AG5 appear the most sensitive (as with subsequent fillings of the sand core). During the trench excavation stage, most of pore
pressure sensors responded by a rapid reduction of pore water pressure. This was primarily because of the stress relief at the position of the sensors from removal of soil within the trench. Sensors AG1 and AG5 showed relatively slow decrease in pressure levels, as they are located further from the trench.

Whilst the containers were filled with water, pore pressures were relatively static, or showed a slow drop. This is probably due to the relatively minor stress increase at the position of the sensors from loading of the narrow containers, though there may have been some consolidation seepage into the trench during this time. Sensors AG1, AG5 (located in the sand core, closer to the containers) show a much smoother variation in pore pressure over time with little response to the trench excavation but with a gradual increase in pressure during filling of the core. The other sensors (located in clay, peat and base sand) registered weaker response to the filling of the core. Draining the trench appears only to have caused the AG6 sensor (located within the thick layer of organic clay) to drop slightly.

Table 6-2. Elevation coordinates of Geobeads sensors

<table>
<thead>
<tr>
<th>Cross-section A</th>
<th>Sensor Depth [m NAP]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sensor GB-AG-1 (in the sand core)</td>
<td>-1.52</td>
</tr>
<tr>
<td>Sensor GB-AG-2</td>
<td>-3.00</td>
</tr>
<tr>
<td>Sensor GB-AG-3</td>
<td>-4.30</td>
</tr>
<tr>
<td>Sensor GB-AG-4</td>
<td>-5.62</td>
</tr>
<tr>
<td>Cross-section B</td>
<td></td>
</tr>
<tr>
<td>Sensor GB-AG-5 (in the sand core)</td>
<td>-1.70</td>
</tr>
<tr>
<td>Sensor GB-AG-6</td>
<td>-2.97</td>
</tr>
<tr>
<td>Sensor GB-AG-7</td>
<td>-4.30</td>
</tr>
<tr>
<td>Sensor GB-AG-8</td>
<td>-6.02</td>
</tr>
</tbody>
</table>

6.7.2 Inclinations

As noted above, the Alert Solution Geobeads sensors recorded both pore water pressures and inclination (in terms of degrees). With an alternative installation, these tilts could be integrated over the length of the instrument to provide a horizontal displacement profile (as with the SAA Inclinometers). Development of tilt within particular sensors is an indication of ground movement, though the actual displacement cannot be calculated. Even so, rapid changes in tilt might be an indication of significant movements taking place within the ground. Accordingly, these data have been examined.

Geobeads sensors tilts recorded during the test are presented in Figure 6-10a-b, (a) – front angles, (b) – side angles. Two sensors (AG4 and AG6) were excluded from the analysis due to probable malfunctioning. Among front angle measurements, deep AG3 and AG7 sensors produced the very early signs of local slippage development: monotonic growth of tilts started already on the 4th of September. The AG3 and AG7 sensors were located at a depth of 3.3 m below ground level, presumably in close proximity to the slip surface that was developing.

Side angle developments started from the 4th of September in AG3 and AG2 sensors, with the most intensive response recorded by the AG3 sensor; AG2 signal (the sensor is just above AG3 in the same cross-section) was synchronous with AG3 but much more muted. AG7-AG8 sensors located in the other cross-section developed smaller side tilts starting from about September, 5th. Sensors AG1, AG5 located slightly above the
ground surface in the levee sand core did not indicate intensive side angle change until the global collapse of the levee which started with forced pumping at 14:00 (see a fragment of sensor readings in Figure 6-10d).

Figure 6-10. Sensor tilts recorded during the test: (a) – front angle; (b) – side angle; (c) – front angle, fragment, at failure; (d) – side angle, fragment, at failure
Like pore pressure sensors, tilt sensors responded more intensively to the stages when filling of the sand core occurred rather than to the stages of filling the containers. Presumably, high pore pressures played more important role in the failure process than the mechanical pressure acting on the top of the levee.

With the readings of the piezometers being complicated by the stress reversal of trench excavation, tilt measurements appear to offer a simpler method of detecting the onset of failure. While the sensors can only record tilts at the discrete positions at which they are installed, extensive inclinometer installations may offer rather more warning of the development of failure surfaces.

The early signs of progressing local instabilities in the levee were registered by tilt sensors more than 4 days before the global collapse – a time which is sufficient for the levee maintenance service to take necessary steps to reinforce the slope.

Changing of the loading stages can be traced in the tilt curves as variation in the velocities of tilt development (in the second derivatives of the signals). Besides monitoring the absolute values of tilts, monitoring the second derivatives of smoothed tilt signals can be recommended for the functional content of artificial intelligence allowing detection of qualitative changes in the loading regime possibly leading to damage.

6.7.3 Strains

Samples of longitudinal strain measurements recorded by the Fibre Optic cables installed within the TenCate geo textile fabric along the levee are presented in Figure 6-11.

![Figure 6-11. Longitudinal strains, micrometers per meter](image)

A general tendency is compression growing until the actual failure. But unlike tilts, strains did not grow monotonically – when filling of the sand core stopped, the strains relieved (see Figure 6-11, periods right before container filling and later on September 7th, between two subsequent fillings of the sand core). The difference between monotonic tilts and non-monotonic linear strains is natural and caused by specifics of plasticity mechanisms in soils as granular materials: irreversible plastic deformations are associated
with shear strains (tilts), while linear strains remain elastic and reversible. Strain relief in cables in pauses between the loading stages could happen due changes in hydraulic load with seepage of water from the levee core into the drainage trench. The longitudinal strains presented in Figure 6-11 are not represented in the 2D plane strain simulation model (they are zero in the plane strain problems) and hence can not be compared to simulation data.

Like the pore pressure readings, fibre optics strain measurements are complicated by strain reversals. Therefore inclination sensors can be recommended as optimal for monitoring levee slope stability: growing shear strains measured by them entirely correspond to development of plastic deformations bringing a levee to macro-instability failure.

### 6.8 Conclusions

The IJkDijk slope failure experiment in Bad Nieuweschans, the Netherlands, has become the ultimate validation of the Virtual Dike module and a winner of a special contest for the best failure prediction organized shortly before the test (http://ijkdijk.rpi.edu). Several commercial corporations and scientific research organizations modelling levee systems participated in the competition; our computational model provided the best class-A prediction for the South levee macro-instability experiment, according to the decision of the jury. The Virtual Dike model precisely predicted the simulation mode (slope sliding) and the collapse stage in the loading sequence, while the models from other contestants underestimated stability of the dike.

A posterior comparison of tilts measured by Geobeads sensors with simulated displacements has shown that the computational model realistically reproduces sensor dynamics, including response of sensors to excavation, container loading and core water pumping.

Successful validation of our module on all three test sites, and especially on the IJkDijk South levee failure experiment, has shown that our computational model is capable of generating virtual sensor patterns of the abnormal and normal behaviour for the artificial intelligence system (the development of this idea is presented in Chapter 7).

Another important conclusion drawn from the IJkDijk slope instability experiment analysis referred to the optimal type of sensors able to detect dike failure as early and as clearly as possible. Tilt measurements appear to offer the simplest method of detecting the onset of slope failure. The early signs of progressing local instabilities in the levee were registered by tilt sensors more than 4 days before the global collapse – a time which is sufficient for the levee maintenance service to take necessary steps to reinforce the slope. Measurements of pore pressure and fibre optics strain measurements are complicated by non-monotonic dynamics with occasional strain reversals (amid growing tilts and global decrease of slope stability). For early slope instability detection, tilt sensors have proven to be the best. The above conclusion on the optimal type of sensors refers to the slope instability monitoring. For erosion detection, pore pressure and temperature sensors can be of much higher importance.