Finite element analysis of levee stability for flood early warning systems
Melnikova, N.

Citation for published version (APA):
Finite Element Analysis of Levee Stability for Flood Early Warning Systems

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Finite Element Analysis of Levee Stability for Flood Early Warning Systems

ACADEMISCH PROEFSCHRIFT

ter verkrijging van de graad van doctor
aan de Universiteit van Amsterdam
op gezag van de Rector Magnificus
prof. dr. D.C. van den Boom
ten overstaan van een door het college voor promoties ingestelde
commissie, in het openbaar te verdedigen in de Agnietenkapel
op Woensdag 3 September, te 12:00 uur

door
Natalia Borisovna Melnikova
geboren te St. Petersburg, Rusland
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This work described here was carried in the Section Computational Science of the University of Amsterdam and in the National Research University ITMO (St. Petersburg). This work was supported by the EU FP7 project UrbanFlood, grant N 248767, by the Leading Scientist Program of the Russian Federation, contract 11.G34.31.0019 and by the "5-100-2020" Programme of the Russian Federation, Grant 074-U01

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Printed by GVO Drukkers & Vormgevers, Ede
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Chapter 1 Introduction

1.1 Motivation and scientific challenges

Floods are common natural disasters frequently taking their dramatic toll in global warming conditions and causing high economic and humanitarian losses. Hundreds and thousands kilometres of sensor-monitored flood protection barriers are built at the coastlines all over the world. Rapid development of communication technologies and innovative sensor systems has inspired scientific and engineering community for the development of smart decision support systems (DSS) for early flood protection. The power of computational science and sensor technologies helps levee maintainers in tracing early signs of failure and taking efficient steps to minimize possible losses. In case of emergency, access to a DSS lets public authorities and citizens make informed decisions on optimal ways of evacuation based on inundation forecast and on traffic jam minimisation. Alarms generated by DSS are transferred to mobile phones and to information displays in public places.

The design of Early Warning Systems (EWS) for flood protection and disaster management poses a grand challenge to scientific and engineering communities and involves the following fields of research:

- Sensor equipment design, installation and technical maintenance in flood defence barriers;
- Information and Communication Technologies in application to:
  - gathering, processing and visualizing sensor data;
  - developing Common Information Space (CIS) middleware for connecting sensor data, relevant documents, analysis tools, modelling software and advanced scientific visualization;
  - providing Internet-based interactive access to CIS for researchers, maintenance personnel and public;
- Development of computational models and simulation components for stability analysis of flood protection barriers, failure probability evaluation, prediction of flood dynamics and ways for evacuation.

The present research relates to the third group of tasks and it is focused on the development of computational models of earthen dikes for the on-line dike stability analysis. The work was carried under the frame of the UrbanFlood FP7 project (http://www.urbanflood.eu), which united the research on all three EWS design aspects mentioned above, including monitoring dikes with sensor techniques (Pyayt et al., 2014), physical study of dike failure mechanisms (Krizhizhanovskaya et al., 2011), software

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development for dike stability analysis (Melnikova et al., 2013; Pyayt et al., 2011b) and simulation of dike failure, inundation dynamics and city evacuation (Melnikova et al., 2014a; Gouldby et al., 2010; Mordvintsev et al., 2012).

A typical, established approach for dike failure prediction in flood monitoring systems implies data-driven methods (e.g., machine learning, statistical methods) and reliability analysis based on simple limit equilibrium methods and on empirical engineering failure criteria which have been worked out in geotechnical practice during past two hundreds of years and which may have been intuitively used in civil engineering for thousands of years. A detailed physical-model based approach has not been used in early warning systems before, because it’s traditionally considered as computationally heavy. However, growing computational power and rapid development of soil mechanics applications make it interesting and reasonable to integrate complex physics-based computational models into automatic and rather autonomic monitoring systems.

Below we will outline the potential benefits coming from such integration. In mathematical analysis, earthen levees are considered as deformable porous structures subjected to hydraulic and structural loads. Finite element method (FEM) in application to soil mechanics theory offers the most generic and powerful computational tool to take a deep insight in simulation of complex physical processes occurring in levees, both in normal mode and at failure. For an operating levee monitored by a EWS, such FEM levee simulator produces “virtual sensors” data to be compared with sensor recordings under real-life loads in real-time mode. Discrepancy of the signals is then treated as an alarm for the expert users of the EWS. Another beneficial and innovative idea about using a model-based approach in EWS is training data-driven models on simulated sensor data sets, when real sensor data are not available (for example, failure recordings for an absolutely healthy levee).

The complexity of failure mechanisms in dikes has brought us to a problem of finding an optimal balance between realism and adequacy of mathematical models employed in dike stability analysis, on the one hand, and fast and reliable convergence of numerical solution procedure in real-time mode, on the other hand.

Scientific questions raised in this thesis were:

- Can a computational model of an earthen dike adequately predict real-life failure under prescribed loads (obtained from sensors or pre-defined by the user)?

-Though the question is as old as soil mechanics (which started in 1773 with Charles Coulomb’s systematic application of frictional mechanics to civil engineering problems), it has not become less actual nowadays: soils are complicated non-linear structures which continue spawning a large number of mathematical models describing their constitutive behaviour. In this connection, the UrbanFlood project gave a unique opportunity for the researchers: it collected a big amount of well-organized sensor data sufficient for the profound study of physical processes in dikes and for extensive verification of computational models involved at all stages of analysis, including physics simulation modules for dike stability assessment as well as data-driven analysis in the artificial intelligence system. The sensor data were
recorded for real-life dikes, including the IJkDijk full scale failure experiments carried under well documented loading regimes (http://www.ijkdijk.nl). The sensor data have been accomplished with detailed information on soil build-up and geometrical shape of the dikes. Running ahead, we can say that our simulation model provided the best prediction of the IJkDijk slope instability experiment and won a contest for the best prediction (http://ijkdijk.rpi.edu) organized before the test by its designers (see Chapter 6).

- Can a computational model produce realistic dike failure patterns for the artificial intelligence (AI) system training, so that AI trained on such sets could afterwards make a correct and early enough detection of a real-life failure?

  - This was an innovative idea proposed and tested by us in cooperation with the AI development team of the UrbanFlood Project (Pyayt et al., 2011a). The AI analysis for structural health monitoring is nowadays a standard technique used in software for sensor monitoring systems. For real-life “healthy” levees, AI typically lacks data on failure patterns. So, for “healthy” levees it was proposed to generate failure patterns in our FEM module Virtual Dike and to train AI on these sets. In order to produce realistic failure patterns for the AI, a comprehensive simulation of complex physical processes happening in the levee at failure was absolutely necessary. After validation of the Virtual Dike module on the real-life dike failures, we have applied this innovative hybrid approach for real-time assessment of dike stability by a monitoring system: for a full-scale dike prototype, the AI system was trained on failure patterns simulated by the Virtual Dike (Chapter 7).

- Uncertainties handling: how critical are uncertainties in soil properties for the quality of dike stability assessment?

  - In this area, there have been a number of investigations on sensitivity of the stability margin to variation in soil strength parameters (such as cohesion and friction angle), we have accomplished them with analytical and numerical studies of sensitivity of hydraulic condition of a dike to variation of diffusivities (Chapter 4). On the base of sensitivity analysis, we have proposed and developed a new automatic procedure for calibration of soil diffusivities in an arbitrary heterogeneous dike equipped with pore pressure sensors (Chapter 4).

For the first time in the history of dike monitoring systems, we have integrated our finite element stability analysis module Virtual Dike into an early warning system, so that it works with live sensor data, produces real-time alarms and interacts with other components of the EWS.

Speaking about up-to-date finite element tools for dike stability analysis, it’s worth mentioning that at the industrial level, a number of well-established commercial FE packages for soil mechanics analysis have been developed during past two decades. With a wide variety and complexity of soil models available for users, commercial packages lack
flexibility for programming add-ons and they are not light-weighted enough for real-time work in a total workflow of an early warning system. Our Virtual Dike module has successfully stood a validation against commercial software (Chapter 5). The module has been easily integrated into the UrbanFlood EWS, it was efficient in real-time work with live sensor input and output, and contained a software add-on for automatic calibration of soil diffusivities procedure developed by the authors.

1.2 Early Warning Systems for Flood Protection

Many international projects are aimed at the development of flood protection systems (Krhizhanovskaya et al., 2011; Pengel et al., 2013). The European Union Framework Programme 7 (FP7) project SSG4Env is focused on the development of semantic sensor grids for environmental protection. The Flood Probe FP7 project coordinates related work on combining sensor measurement techniques. A big national Dutch project Flood Control 2015 aims to share sensor measurements datasets and to provide a user interface to explore sensor data for researchers, technical maintainers and civil population. The IJkDijk (http://www.ijkdijk.nl) is a project on experimental physical study of dike failure mechanisms. The tests are carried out on full-scale experimental dikes equipped with large sets of sensors. The project has produced extremely detailed datasets of sensor data, including pore pressures, inclinations, stresses and strains.

As it was mentioned above, the UrbanFlood project united the work on all the aspects of the EWS design, including sensors, communications and numerical modelling related to dike monitoring. The project has developed an internet based EWS service platform that can be used to link sensors via the Internet to predictive models and emergency warning systems. The data collected from the sensors are interpreted to assess the condition and likelihood of failure; different numerical models are employed to predict the failure mode and subsequent potential inundation in near real time. Through the Internet, additional computer resources required by the framework are made available on demand.

UrbanFlood validated the EWS framework and its implementation in the context of dike performance (failure) in an urban environment. For that purpose, a number of live pilot sites have been worked out to prove the methodology. The pilot sites included the following earthen levees: Livedike (Groningen, Netherlands), Boston levee (Boston, UK), “Stammerdijk” and “Ringdijk” (Amsterdam, Netherlands), Rhine dike (Germany) and three full scale levees which were built and brought to failure during the IJkDijk experiments in 2012. The test dikes have been equipped with sensor systems and the EWS service was built up from a series of dike failure and flooding specific modules which included dike breach evolution and flood-spraying models. UrbanFlood has investigated and shown the feasibility to remotely monitor dikes and floods, whether from nearby offices or from other countries and continents through secure use of web based technologies (Simm et al., 2012a; Pyayt et al., 2014). For the development of flood mitigation scenarios and the training of personnel, the framework has been connected to a simulator that computes flood responses associated with failing dikes.

A general workflow and interaction of software components in the UrbanFlood early warning system are presented in Figure 1-1.
The Sensor Monitoring module receives data streams from the sensors installed in the dike. Raw sensor data are filtered by the AI (Artificial Intelligence) Anomaly Detector that identifies abnormalities in dike behaviour or sensor malfunctions. The Reliability Analysis module calculates the probability of dike failure in case of abnormally high water levels or an upcoming storm and extreme rainfalls. If the failure probability is high, then the Breach Simulator predicts the dynamics of a possible dike failure, calculates water discharge through the breach and estimates the total time of the flood. After that, the Flood Simulator models the inundation process and Evacuation Simulator optimizes evacuation routes. Then Risk Assessment module calculates flood damage. Finally, Decision Support System provides access to different information levels, for experts and citizens. The simulation modules and visualization components are integrated into the Common Information Space (Balis et al., 2011). They are accessed from the interactive graphical environment of a multi-touch table or through a web-based application.

The Virtual Dike component runs in parallel with the Reliability Analysis module, offering direct numerical simulation to analyze dike stability under specified loadings (Melnikova et al., 2011a, 2013, 2014). The module can be run with a real-time input from water level sensors or with predicted high water levels due to upcoming storm surge or river flood. In the first case, comparison of simulated pore pressures with real data can indicate a change in soil properties or in dike operational conditions (e.g. failure of a drainage facility). In the second case, simulation can predict the structural stability of the dike and indicate the "weak" spots in the dikes that require attention of dike managers and city authorities. Simulated dynamics of dike parameters (including pore pressure, local stresses and displacements) describes the non-stationary behaviour of the dike (changing over time). We define a concept of a “virtual sensor” for the data obtained from finite
element solution in the point where a real sensor is located. Data from the virtual sensors are compared to the real-life sensor measurements.

Software architecture and tools used for the EWS implementation are described in details in (Balis et al., 2011); a general philosophy of the UrbanFlood EWS design is discussed in (Meijer et al., 2012).

1.3 Overview of the thesis

The thesis is organized as follows:

Chapter 1 contains introduction and description of the UrbanFlood early warning system workflow. Chapter 2 analyzes the collection of existing mathematical models for dikes analysis and selects the most suitable, providing the optimal balance between simulations realism and fast work in real-time mode.

Chapter 3 describes numerical solution issues: implementation of the Virtual Dike module in Comsol package, integration of the module into the UrbanFlood decision support system and parallel efficiency assessment.

The next three chapters, Chapter 4, Chapter 5 and Chapter 6, describe three validation test-sites which were studied in the present research. The test-sites are analyzed in the order of increasing complexity, starting with the Livedike (a sea dike in Groningen, the Netherlands), for which a purely hydraulic (uncoupled) model was developed and tested (Chapter 4). Besides the Livedike modelling, Chapter 4 contains analytical and numerical analyses of sensitivity of porous flow in soils to the variation of soil diffusivity and a description of an automatic procedure proposed by us for calibration of hydraulic diffusivities in an arbitrary heterogeneous dike equipped with pore pressure sensors.

In Chapter 5, a more complicated test case is considered: the Boston levee (United Kingdom) which is a real-life levee operating close to safety margin, with occasional slope failures at high tidal range, typically at springtime. The third case study (the IJkDijk slope failure in Bad Nieuweschans, the Netherlands) has become an ultimate validation of the Virtual Dike module and a winner of a special contest for the best failure prediction (Chapter 6).

Chapter 7 describes experience with training artificial intelligence system on a prototype of the Livedike. Chapter 8 and Chapter 9 bring summary and conclusions in English and Dutch, respectively; Chapter 10 gives acknowledgements; Chapter 11 lists references and the last chapter lists publications by the author.
Chapter 2 Principles of dike modelling

This chapter reviews mathematical models describing porous flow and deformations in earthen dikes, with discussion of advantages and drawbacks of each model. The models are introduced in chronological order, according to the history of their development. After careful testing of different models, we have chosen a coupled combination of plane Drucker-Prager linear elastic perfectly plastic model for modelling deformations in soil skeleton and Richards’ model for modelling filtration through porous media. This choice provided the optimal balance between realism and adequacy of the models, on the one hand, and fast and reliable convergence of numerical solution procedure in real-time mode, on the other hand.

Dike collapse mechanisms vary from macro-instability failures, like slope sliding or simple shear in a multi-layered dike, to a number of erosion failure mechanisms, such as piping, internal overtopping or wave erosion. The conventional engineering methods for levees stability analysis include: (a) probabilistic breach analysis (available for all kinds of failure) based on empirical engineering criteria (Vorogushyn et al., 2012); (b) limit equilibrium methods (LEM) only suitable for slope sliding prediction.

The first limit equilibrium method (LEM) for slope stability analysis (Fellenius/Peterson method) was proposed in (Fellenius, 1927). It considers a balance of disturbing and stabilizing forces acting on vertical soil slices located above a circular slip surface. The ratio restoring forces/disturbing forces is termed the factor of safety. Later the method underwent numerous modifications, for instance by Taylor, Bishop, Morgenstern-Price, Spencer, Janbu and others (an extensive overview of existing LEM methods and their comparison can be found in (Fredlund and Krahn, 1977; Chen and Morgenstern, 1983; Duncan, 1996)). Some of those, like Fellenius’ and Bishop’s methods, assume a circular slip surface and do not satisfy horizontal equilibrium conditions for the soil slices, while Spencer’s, Morgenstern-Price’s and Janbu’s generalized methods work with non-circular slip surfaces and satisfy all equilibrium conditions.

LEM is nowadays the most popular analysis tool in practical engineering: it is simple and robust. A serious limitation of LEM usage in the EWS design is small amount of the output data fed back from it into the EWS: basically, it generates tables with dike’s scalar factors of safety, under the prescribed load levels and loads combinations. LEM does not really allow a deep insight into the dike failure processes modelling and can not simulate dynamics of sensor measured parameters (such as pore pressure, displacements, inclinations and strains) which are of much importance for safety monitoring and AI training.

The main drawback of LEM lies in the narrowness of its application: it only predicts slope sliding failures and does not capture a great variety of different failure mechanisms mentioned above.

In opposite to LEM, the finite element method (FEM) in application to partial differential equations of continuum soil mechanics (which will be presented below) was chosen as a perfect tool for the implementation of the Virtual Dike module aimed for a detailed dike stability analysis, simulation of arbitrary and complex failure mechanisms and
feeding the AI. A FEM-based analysis is capable of modelling any failure mechanisms - naturally, on a macroscopic level.

In co-operation with HR Wallingford, we carried a cross-validation of LEM against FEM for the Boston test site (Chapter 5).

Below we review the existing mathematical models describing dike behaviour under hydraulic and mechanical loadings and select the models optimal for real-time functioning in the workflow of the EWS. The equations describe two sub-models involved in the analysis: a fluid sub-model for simulation of porous flow through the dike and a mechanical sub-model for dike deformations modelling.

### 2.1 Simulation of water flow through the dike

#### 2.1.1 Governing equations

Fluid flow through fully saturated porous media is governed by Darcy’s law (Darcy, 1856) which states that the filtration velocity \(\bar{V}\) is linearly proportional to the pore pressure gradient and to the gravity acceleration:

\[
\bar{V} = -\frac{K_s}{\mu} (\nabla p + \rho g)
\]

Here \(K_s \text{[m}^2\text{]}\) is permeability (a characteristic of soil type), \(\mu\) is dynamic viscosity of water, [Pa·s], \(\nabla\) is gradient operator, \(p\) is relative pore pressure [Pa] (which is absolute pore pressure minus atmospheric pressure), \(g\), \(\rho\) are standard gravity and water density, respectively.

The actual water flow velocity in pores equals to \(\bar{V} / f\), where \(f\) is the porosity of a medium (a ratio of pores volume to the total porous domain volume). Filtration velocity \(\bar{V}\) describes the flux in a homogenised, continuous medium.

The Darcy’s law is suitable for slow filtration velocities. For fast turbulent pore flows, a quadratic extension was proposed in (Brinkman, 1949). In practice, turbulent flows occur in gases in porous media; this is not the case of soil-water filtration and the linear form (2.1) will be used below.

Ground water flow equation is derived from the Darcy’s law and the mass conservation equation under the assumptions that (a) water is incompressible and (b) the time derivative of filtration velocity is negligibly small: \(\frac{\partial \bar{V}}{\partial t} = 0\) (Bear, 1972):

\[
\nabla \cdot \bar{V} = Q_s \Rightarrow \nabla \cdot [\frac{-K_s}{\mu} \nabla p] = Q_s
\]

Here right-hand side component \(Q_s\) is volume intensity of external sources and sinks.
The non-stationary extension of ground water flow equation takes into account the change of water amount stored in pores with their compaction or extraction under local pore pressure fluctuations. The formulation is based on the mass conservation equation:

\[ S \frac{\partial p}{\partial t} + \nabla \cdot \left[ -\frac{K_S}{\mu} \nabla p \right] = Q_s \]  \hspace{1cm} (2.2)

Here storage coefficient \( S \) [1/Pa] is the ratio of soil porosity \( f \) to the soil skeleton bulk modulus \( K \): \( S = f/K \); \( t \) is time.

Equations (2.1) and (2.2) are completely analogous to Fourier's law and Laplace’s equation for heat conduction in solid bodies.

Equation (2.2) describes a so called confined class of pore fluid problems, which do not contain a phreatic surface in the simulation domain. A phreatic surface is defined as a surface in a porous medium where relative pore pressure is zero. Earthen dikes typically refer to the second, unconfined class, because ground water table is naturally located within the dike. Unconfined flow modelling employs specific techniques like moving mesh (Comsol manual, 2009) or using stationary mesh with elements having artificial nonlinear permeabilities, which “turn off” elements in the vadose (partially saturated) zone (Larabi and Smedt, 1997). In the both techniques, flow in a zone of capillary fringe is neglected. In clayey soils, capillary fringe height can reach ten meters (Verruijt, 2001); under rapid hydraulic loads like rainfalls or fast rise of river level capillary fringe zones become saturated and significantly change pore pressure distribution. The effect of wetting (and drying) is taken into account with a more comprehensive Richards’ equation of variably saturated ground water flow.

Richards (1931) extended the usage of Darcy’s law on partially saturated media; with water storage and permeability depending on pore water content. Pressure-based form of Richards’ equation (Bear, 1972) is given by formula

\[ (C + \theta_e S) \frac{\partial p}{\partial t} + \nabla \cdot \left[ -\frac{K_S}{\mu} \nabla p \right] = Q_s \]  \hspace{1cm} (2.3)

Here \( C \), \( \theta_e \) are specific moisture capacity and effective water content, respectively; \( p \) is pore water pressure (negative for suctions in the partially saturated zone); \( K_S \) is permeability of saturated media; \( k_r = k_r(p) \) is relative permeability.

Specific moisture capacity and relative permeability are non-zero in vadose (partially saturated) zones; they are defined as functions of volumetric water content \( \theta = \frac{V_{\text{water}}}{V_{\text{total}}} \), where \( V_{\text{water}} \) is volume of water in porous media, \( V_{\text{total}} \) is total volume (which is soil volume + water volume + air volume).

In saturated zone \( \theta = \theta_s \approx f \), where \( \theta_s \) is saturated water content; \( f \) is porosity (pores are mostly filled with water except of small air bubbles locked inside).
Due to absorption, even at high suction pressures there is always residual water content $\theta_r$ in the soil. The amount of residual water depends on the type of soil; maximal residual water contents are observed in clays.

Effective water content $\theta_e$ is defined as follows: $\theta_e = \frac{\theta - \theta_r}{\theta_s - \theta_r}$.

A specific property of a given type of soil is water retention curve, which is expressed as a relationship between the effective water content $\theta_e$ and negative suction pressure $p$. Due to the hysteretic effect of water filling and draining the pores, different wetting and drying curves may be distinguished.

The very first water-retention curve was proposed in 1907 by Edgar Buckingham (Buckingham, 1907). A number of analytical water-retention curves were then proposed in (Brooks and Corey, 1964), (Brutsaert, 1966), (Vauclin et al., 1979), (Van Genuchten, 1980). Van Genuchten curve (Van Genuchten, 1980) is most widely adopted nowadays; it defines the dependence between suction pressure and effective water content in as follows:

$$\theta_e = \begin{cases} 
\frac{1}{(1 + (a|p/\rho g|)^n)^m}, & p < 0 \\
1, & p \geq 0 
\end{cases} \quad (2.4)$$

Here $p$ is suction pressure, $a$, $n$, $m = 1 - 1/n$ are Van Genuchten parameters specific for each type of soil.

At suction pressures close to zero, a soil is close to saturation, and water is held in the soil primarily by capillary forces. As saturated water content $\theta_s$ decreases, binding of the water becomes stronger, and at high suction pressures water is strongly bound in the smallest of pores, at contact points between grains and as films bound by adsorptive forces around particles. Sandy soils will involve mainly capillary binding, and will therefore release most of the water at relatively low suction pressures. Clayey soils, with adhesive and osmotic binding, will release water at higher suction pressures. At any given pressure, peaty soils will usually display much higher moisture contents than clayey soils, which would be expected to hold more water than sandy soils. The water holding capacity of any soil is due to the porosity and the nature of the bonding in the soil.

Moisture capacity and relative permeability are described by the following expressions (Van Genuchten, 1980):

$$C = \frac{\partial \theta}{\partial p} = \begin{cases} 
\frac{anm}{1-m} (\theta_s - \theta_r) (1 - \theta_e^{1/m})^m, & p < 0 \\
0, & p \geq 0 
\end{cases} \quad (2.5)$$
\[ k_r = \begin{cases} \theta_e \left[ 1 - (1 - \theta_e^{1/m})^m \right]^2, & p < 0 \\ 1, & p \geq 0 \end{cases} \] (2.6)

Here \( l \) is pore connectivity parameter.

### 2.1.2 Boundary conditions

Boundary conditions for (2.3) can be of two types:

(a) Pressure specified at the boundary \( S_1: p\big|_{S_1} = p_s \). Example: \( p_s = 0 \) refers to seepage to atmosphere.

(b) Flow velocity across the boundary \( S_2: V_n\big|_{S_2} = \mathbf{n} \cdot [-K_s k_r \nabla (p + \rho g y)] \), where \( \mathbf{n} \) is normal to the boundary surface. Examples: \( V_n = 0 \) at impermeable walls or \( V_n = V_n(t) \) for rainfall infiltration.

### 2.1.3 Initial conditions

For transient tasks with external loading, pore pressure dynamics is fully determined by external hydraulic loads. Initial pore pressure distribution dissipates within a finite period of time and its choice is only a technical item. A typical initial condition for fluid sub-model used in the present work was: hydrostatic pressure distribution below an estimated phreatic line connecting land side and sea side water tables, and suction pressure \( p = -5000 \ \text{[Pa]} \) above the phreatic line:

\[
\begin{cases}
p = -\rho g y, & \text{for } y \leq h_{gw}, \\
p = -5000 \ \text{[Pa]}, & \text{for } y > h_{gw},
\end{cases}
\]

where \( y \) is vertical elevation coordinate, \( h_{gw} \) is local position of the phreatic line, estimated from land side and sea side boundary conditions.

This choice of initial condition provided fast convergence of numerical solution in the fluid sub-model.

### 2.2 Modelling soil deformations

#### 2.2.1 Governing equations

Soil skeleton is considered as deformable continuum subjected to hydraulic and mechanical loads. There have been developed various constitutive soil models: Mohr-Coulomb, Drucker-Prager, hardening models: cam-clay family models, soft soil model and others. Mohr-Coulomb and Drucker-Prager models work well for stiff cohesionless soils like sands which exhibit plastic shear deformations and elastic volume deformations. Cam-clay and soft soil models have been developed for soft clays and peats, which generally...
produce significant nonlinear elastic deformations, with volume deformations becoming plastic at some load level. Plastic volume deformations are taken into account in cam-clay model by adding elliptic cap to a Mohr-Coulomb yield surface. The Cam-clay material model was developed at the University of Cambridge in the 1970s, and since then it has experienced different modifications. The modified Cam-clay model (Potts and Zdravkovic, 1999) is the most commonly used due to the smooth yield surface. The modified cam-clay model is a so-called critical state model, where the loading and unloading of the material follows different trajectories in stress space. The model also features hardening and softening of clays.

We have tested the Mohr-Coulomb, Drucker-Prager and cam-clay models for the Virtual Dike module, and finally choose the Drucker-Prager (DP) model by the following reasons:

- Drucker Prager soil model is quite simple and hence fast in computations, suitable for real-time work
- Drucker Prager function is smooth and does not cause numerical solution problems due to singularities in the flow rule, unlike the Mohr-Coulomb model which actually showed numerical convergence problems in our tests
- The cam-clay model is computationally heavy; moreover, it required model parameters which were not available from soil data on the Livedike and on the Boston levee (compression and swelling indices, initial consolidation pressure).

Below we describe the equations (2.7) suitable for classical isotropic linear elastic perfectly plastic soil models (including Drucker-Prager soil model), which assume that volume deformations of soil are always elastic, while plastic yielding occurs due to the shear deformations in soil, with sliding between material planes. According to Terzaghi’s principle (Terzaghi, 1943), stressed state of soil skeleton is characterized by the effective stress tensor \( \sigma_{\text{eff}} \) which is a sum of total stress tensor (obtained from equilibrium equation) and hydrostatic water pressure tensor \( \sigma_{\text{eff}} = \sigma + p \mathbb{I} \) (here compressive stresses are negative). The Terzaghi’s principle means that pore water provides buoyancy effect on soil skeleton. Elasto-plastic deformations of the soil skeleton are described in terms of strains and effective stresses by the general equations of plastic flow theory (Potts and Zdravkovic, 1999):

\[
\begin{align*}
\nabla \cdot \sigma + \rho \dot{g} &= 0 \\
\varepsilon &= \varepsilon_{\text{pl}} + \varepsilon_e \\
\varepsilon_e &= \frac{1+\nu}{E} \left[ \sigma_{\text{eff}} - \frac{\nu}{1+\nu} \sigma_{\text{eff}} \mathbb{I} \right] \\
\varepsilon_{\text{pl}} &= 0 \quad \text{if } F < 0, \quad \varepsilon_{\text{pl}} = q \frac{\partial P}{d \sigma_{\text{eff}}} \quad \text{if } F = 0
\end{align*}
\]
where \( \nabla = \varepsilon_x \frac{\partial}{\partial x} + \varepsilon_y \frac{\partial}{\partial y} + \varepsilon_z \frac{\partial}{\partial z} \) is gradient operator; \( \rho_s \) is soil density; \( g \) is gravity vector; \( \sigma \) and \( \sigma_{\text{eff}} \) are total and effective stress tensors, respectively (compressive stresses are negative); \( \varepsilon \) and \( \varepsilon_{\text{pl}} \) are elastic and plastic components of strain tensor, respectively; \( E \) is Young’s modulus; \( \nu \) is Poisson’s ratio; \( \mathbb{I} \) is unit tensor; \( \varepsilon \) is total strain tensor; \( q \) is plastic multiplier; \( P \) is plastic potential function, \( F \) is plastic yield function (\( F < 0 \) corresponds to elastic behaviour, \( F = 0 \) refers to plastic yield); \( K = \frac{E}{3(1-2\nu)} \) is bulk modulus, \( \sigma_{\text{eff}} = I_1 = \sigma_{\text{eff}} + \sigma_{y\text{eff}} + \sigma_{z\text{eff}} \) is the first effective stress invariant.

In general non-associated plastic flow rule, plastic potential \( P \) and plastic yield function \( F \) do not coincide. In associated plastic flow rule, they are equal: \( F = P \).

Plastic flow rule \( \dot{\varepsilon} = q \frac{\partial P}{\partial \sigma_{\text{eff}}} \) determines the ratio between plastic strain rate tensor components. The magnitude of plastic strains is governed by a scalar plastic multiplier \( q \) determined from Prager’s consistency condition, which closes the set of equations (2.7). The condition states that at yield \( \dot{F}(\sigma_{\text{eff}}) = 0 \) because \( F(\sigma_{\text{eff}}) = 0 \):

\[
\dot{F}(\sigma_{\text{eff}}) = \frac{dF}{d\sigma_{\text{eff}}} \cdot \dot{\sigma}_{\text{eff}} = 0
\]

(2.8)

The mechanical sub-model (2.7)+(2.8) is quasi-static; inertia effects are not taken into consideration. Mechanical loadings are defined as functions of a pseudo-time parameter; differentiation in (2.7), (2.8) refers to pseudo-time.

### 2.2.2 Calculation of effective stresses in vadose zones

In saturated zones, effective stresses \( \sigma_{\text{eff}} \) are calculated according to Terzaghi’s classical effective stress principle (Terzaghi, 1943):

\[
\sigma = \sigma_{\text{eff}} - p_{\mathbb{I}}
\]

where \( p_{\mathbb{I}} \) is taken with minus because compressive stresses are negative. The principle means that water resists to compressive load, unloading soli skeleton; while water does not resist shear.

Levees subjected to tidal oscillations or other fluctuations of ground water table contain variably saturated zones. It is well known that pore suctions in vadose zones stabilize slopes (see, for example, Krahn et al., 1989; Griffiths and Lu, 2005). The extension of Terzaghi’s classical effective stress principle on unsaturated soils was first proposed by Bishop (1955a) and then was extensively developed (e.g., Fredlund et al., 1978; Vanapalli et al., 1996). In these extensions, soil matric suction is relaxed by a matric
suction coefficient (which is a function of soil-water saturation) and contributes to the calculation of effective stresses from total stresses:

\[ \sigma = \sigma_{\text{eff}} - \alpha p \mathbf{I} \text{ if } p < 0 , \]

where \( \alpha \) is matric suction.

Sensitivity of slope safety margin to the pore suctions depends highly on the water table elevation, soil type and infiltration conditions (Griffiths and Lu, 2005). Case studies of unsaturated slope failures can be found in (Tsaparas et al., 2002; Cho and Lee, 2001).

In many cases, especially for loose media, pore suctions are omitted when calculated effective stress in vadose zones:

\[ \sigma = \sigma_{\text{eff}} \text{ if } p < 0 . \]

2.2.3 Boundary conditions

Possible boundary conditions for the mechanical sub-model are:

(a) displacements specified at the boundary \( S1 \): \( \mathbf{U} \big|_{S1} = \mathbf{U}_s \); 

(b) loading specified at the boundary \( S2 \): \( n \cdot \sigma \big|_{S2} = f \big|_{S2} \).

2.2.4 Initial condition

Differentiation in the mechanical sub-model is done with respect to pseudo-time, which in general case does not have to coincide with physical time. Pseudo-time is a parameter used in plastic yielding theory to describe incremental growth of all loads and corresponding increments of stresses and strains in the dike. A natural initial condition for the set of equations (2.7)+(2.8) is zero plastic strain in the domain: \( \varepsilon_{pl} = 0 \).

2.2.5 Plasticity models

Below the classical associated plasticity models (Mohr-Coulomb and Drucker-Prager) are described in details. In all constitutive models, plastic yield function \( F(\sigma) \) is defined as a function of principal stresses or stress tensor invariants.

**Mohr-Coulomb plasticity model**

In Mohr-Coulomb plasticity model, yield function is a difference between magnitude of normal effective compressing stress \( |\sigma_f| \) acting on a plane and multiplied by tangent of friction angle, and magnitude of total shear stress \( |\tau_f| \) acting in the plane:
\[ F = c + |\sigma_f| \cdot \tan \varphi - |\epsilon_f|, \]

where \( c \) is effective cohesion [Pa], \( \varphi \) is effective friction angle [grad].

In 3D, Mohr-Coulomb yield function is expressed via maximum and minimum effective principal stresses \( \sigma_1, \sigma_3 \) as follows:

\[ F = (- (\sigma_1 + \sigma_3) \cdot \sin \varphi + 2c \cdot \cos \varphi) - (\sigma_1 - \sigma_3) \]

Second effective principal stress is not active in Mohr-Coulomb plasticity model. The Mohr-Coulomb yield function has corners when plotted in principal effective stress space. For the associated plasticity modelling, the corners imply singularities in the yield function’s partial derivatives with respect to the stress components. These derivatives appear in the plastic flow rule; they are needed to define the elasto-plastic constitutive matrix, and they are not unique at the corners. This numerical problem can be avoided either by implementing special numerical procedures to treat derivatives at corners or, alternatively, by using a smooth yield function which approximates Mohr-Coulomb yield function. This yield function defines a so called Drucker-Prager plasticity model, which gives a shape of circular cone in 3D space.

**Drucker-Prager plasticity model**

Below we describe a modification of the Drucker-Prager (DP) plasticity model, specially optimized for plane strain problems by providing the best smooth approximation of the Mohr-Coulomb surface in the stress space (Chen and Mizuno, 1990). Plastic yield function is independent of Lode angle (which is the third effective stress invariant). Unlike Mohr-Coulomb model, the second effective principal stress is active in the Drucker-Prager model:

\[ F = \alpha \cdot I_1 + \sqrt{J_2} - F_{DP} \quad (2.9) \]

Here \( I_1 = \sigma_1 + \sigma_2 + \sigma_3 \) is the first effective stress invariant, \( J_2 = I_1^2 / 3 - I_2 \) is second deviatoric effective stress invariant, \( I_2 = \sigma_1 \cdot \sigma_2 + \sigma_3 \cdot \sigma_2 + \sigma_1 \sigma_3 \) is second effective stress invariant; \( \alpha \) and \( F_{DP} \) are constants: \( \alpha = \tan(\varphi) / \sqrt{9 + 12 \cdot \tan^2(\varphi)} \), \( F_{DP} = 3c / \sqrt{9 + 12 \cdot \tan^2(\varphi)} \); \( c, \varphi \) are effective cohesion and internal friction angle, respectively.

Drucker-Prager model has been used in the Virtual Dike module, as it is smooth and does not require specific treatment of numerical problems caused by cornered shape of Mohr-Coulomb function.

### 2.3 Dike stability assessment

In the limit equilibrium analysis, dike stability margin is assessed by computing the factor of safety (FoS), describing the capacity of a slope to withstand its own weight
together with applied external loadings from surcharge and groundwater. *FoS* is defined as the ratio of restoring forces $F_R$ (soil shear strength + externally applied restoring forces) to disturbing forces $F_D$ (soil self weight + externally applied disturbing forces):

$$FoS = \frac{F_R}{F_D},$$

where the sums of restoring and disturbing forces are calculated on all possible circular slip surfaces with arbitrary diameters and centre locations. The critical failure surface corresponds to that providing the minimal value of *FoS*.

Shapes of a slip surfaces in LEM are predefined and specific for each modification of the method. The classical limit equilibrium methods consider circular slip surfaces.

When solving partial differential equations of soil mechanics (e.g., by FEM, FD or any other appropriate method), no assumptions on the shape of critical surface are made – slip surfaces can be arbitrary and they are obtained during simulation. Moreover, slope sliding is not the only failure mechanism simulated by soil mechanics modelling by FEM. In 1975, Zienkiewicz proposed a shear strength reduction method to evaluate the dike stability margin, for arbitrary failure mechanisms. In the shear strength reduction method, soil strength parameters are gradually scaled down until the onset of slope instability is reached (which is detected by the divergence of numerical analysis iterations). A strength reduction factor $SRF$ is then defined as the ratio of the original and scaled strength parameters.

$$SRF = \frac{c}{c_{\text{margin}}} = \frac{\tan \varphi}{\tan \varphi_{\text{margin}}},$$

where $c, \varphi$ are cohesion and friction angle of soil strata, $c_{\text{margin}}, \varphi_{\text{margin}}$ are cohesion and friction angle at the margin of instability, under given loadings.

There has been published a number of papers, comparing $SRF$ obtained by the strength reduction method and *FoS* from the limit equilibrium analysis (Totsev and Jellev, 2009), (Griffiths and Lane, 1999). The discussion about the optimal method to assess stability margin is still open. Obviously, these two factors refer to different dike models (*FoS* is computed for the original dike, while $SRF$ is computed for the dike with weakened soil strength) and they are not completely identical, as nature experiments have also proved (see our cross-validation of these two methods in Chapter 5). In physically non-linear structures, like elasto-plastic soils, response of structure to the external load does not depend linearly on the strength parameters of soil. Thus, we can not expect that scaling of soil properties with constant load in the strength reduction method would give absolutely the same result as computing ratio of disturbing and restoring forces in the original dike in the limit equilibrium analysis.
2.4 Virtual Dike model

2.4.1 Drained behaviour modelling

Equations (2.3), (2.7), (2.8) form a one-way coupled fluid-structure interaction system, describing behaviour of deformable porous media. In general case of bi-directional coupling, the term $C \frac{\partial p}{\partial t}$ in (2.3) is replaced with $\frac{\partial \epsilon}{\partial t}$, where $\epsilon$ is volume deformation of soil skeleton which is squeezing/absorbing pore water. The Virtual Dike module employs a one-way coupled fluid-structure interaction system with non-stationary form of Richard’s equation for fluid sub-model which is independent from structural deformations (eq. (2.12)). The porous flow sub-model generates pore pressure used to compute effective stresses in the mechanical sub-model, which in turn is described by eq.(2.13). This simplification was made due to the assumption that the soil skeleton does not undergo large volume deformations under mechanical loading. Having made this assumption, we loose possibility of modelling fully-coupled consolidation problems (where slow porous flow restricts solid skeleton deformations rate), but we improve numerical stability of solver and make the model more robust, as it is no longer fully coupled. Switching to one-directional coupling was conditioned by poor convergence of numerical solution for fully-coupled problems, in out tests (fully coupled systems were ill-conditioned).

$$(C + \theta \varepsilon_S) \frac{\partial \varepsilon}{\partial t} + \nabla \cdot \left[ \frac{-K}{\mu_k} k_r \nabla p \right] = 0 \quad (2.12)$$

\[
\begin{align*}
\nabla \cdot \sigma + \rho_s g &= 0 \\
\varepsilon &= \varepsilon_{pl} + \varepsilon_e \\
\varepsilon_e &= \frac{1+\nu}{E} \left[ \frac{\sigma_{eff}}{1+\nu} - \frac{\sigma_{eff}}{1+\nu} \right] \\
\dot{\varepsilon}_{pl} &= 0 \text{ if } F < 0 \\
\dot{\varepsilon}_{pl} &= q \frac{\partial F}{\partial \varepsilon_{eff}} \text{ if } F = 0 \\
\sigma &= \sigma_{eff} - p \text{ if } p \geq 0 \\
\sigma &= \sigma_{eff} \text{ if } p < 0 \\
\frac{dF}{d\sigma} \cdot \dot{\sigma}_{eff} &= 0
\end{align*}
\]

Here associated plastic flow rule is used; plastic yield function and plastic potential coincide and are equal to Drucker-Prager yield function (2.9). Pore suctions were omitted when calculating effective stresses above the phreatic line: $\sigma = \sigma_{eff}$ in vadose zones. This
assumption was possible due to low suctions in highly permeable vadose zones of the dikes studied in the research (see Chapter 4- Chapter 6 for description of the test dikes).

Equations (2.12) and (2.13) form the Virtual Dike drained soil model employing Richards’ equation for fluid flow and linear elastic perfectly plastic Drucker-Prager associated plasticity model for soil deformations simulation.

2.4.2 Undrained behaviour modelling

In undrained conditions, the rate of filtration is negligibly small compared to the rate of hydraulic and mechanical loads’ change. Filtration is so slow that it can be assumed that it does not occur during loading period. Due to impossibility of pore water drain, pore water highly resists compressive loads, making media almost incompressible. Soil-water medium works as conglomerate which behaviour is described in terms of total stresses $\sigma$. The components of total stresses are expressed as sum of effective stresses and pore pressures, according to the Terzaghi’s principle:

$$
\begin{align*}
\sigma_x &= \sigma_{x eff} - p \\
\sigma_y &= \sigma_{y eff} - p \\
\sigma_z &= \sigma_{z eff} - p \\
\tau_{xy} &= \tau_{xy eff} \\
\tau_{xz} &= \tau_{xz eff} \\
\tau_{yz} &= \tau_{yz eff}
\end{align*}
$$

For undrained behaviour, a distinction is made between steady state pore pressure and excess pore pressure:

$$
p = p_{steady} + p_{excess},
$$

where $p_{steady}$ is input data (for dikes stability analyses, $p_{steady}$ is generated on the basis of phreatic levels obtained from the drained gravity settlement problem solution).

Excess pore pressures and effective stresses are calculated from total stresses via coefficient matrices; the procedure of obtaining these matrices was obtained from (Vermeer, 1993)

Since the time derivative of $p_{steady}$ is zero, it follows that

$$
\dot{p} = \dot{p}_{excess}
$$

Applying Hooke’s law for elastic component of strains $\varepsilon$ gives us:
Considering slightly compressible water, the rate of excess pore pressure is written as:

\[ \dot{p} = -\frac{K_w}{f}(\dot{\varepsilon}_{e,x} + \dot{\varepsilon}_{e,y} + \dot{\varepsilon}_{e,z}) \]  

(2.15)

where \( K_w \) is water bulk modulus, \( f \) is porosity.

\[ \Rightarrow \text{Hooke’s law (2.14) can be written as:} \]

\[ \begin{bmatrix} \dot{\varepsilon}_{e,x} \\ \dot{\varepsilon}_{e,y} \\ \dot{\varepsilon}_{e,z} \\ \dot{\gamma}_{e,xy} \\ \dot{\gamma}_{e,xz} \\ \dot{\gamma}_{e,yz} \end{bmatrix} = \frac{1}{E_u} \begin{bmatrix} 1 & -\nu & -\nu & 0 & 0 & 0 \\ -\nu & 1 & 0 & 0 & 0 & 0 \\ -\nu & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 2 + 2\nu & 0 & 0 \\ 0 & 0 & 0 & 0 & 2 + 2\nu & 0 \\ 0 & 0 & 0 & 0 & 0 & 2 + 2\nu \end{bmatrix} \begin{bmatrix} \sigma_x + \dot{p} \\ \sigma_y + \dot{p} \\ \sigma_z + \dot{p} \\ \sigma_{xy} \\ \sigma_{xz} \\ \sigma_{yz} \end{bmatrix} \]

(2.16)

Here

\[ E_u = 2G(1 + \nu_u), \quad \nu_u = \frac{\nu + \mu(1 + \nu)}{1 + 2\mu(1 + \nu)}, \]

(2.17)

\[ \mu = \frac{1}{3f} \frac{K_w}{K}, \quad K = \frac{E}{3(1 - 2\nu)} \]
High values of $K_W$ produce undrained Poisson’s ratio $\nu_u$ close to 0.5, which leads to numerical instability. To avoid this, $\nu_u$ is assumed to equal to 0.495, which gives $K_W \approx 20K$.

Taking into account (2.15), effective stress $\sigma_{\text{eff}}$ is calculated as

$$\dot{\sigma}_{\text{eff}} = \dot{\sigma} + \dot{p}I = \dot{\sigma} - \frac{K_W}{f} \dot{\varepsilon}I$$

We simulate the undrained plastic flow on the basis of Mohr-Coulomb effective strength parameters (the procedure was proposed in (Vermeer, 1993) as undrained model A). In undrained zones, the plastic flow rule is formulated for total stresses using the effective stress parameters (namely, effective friction angle and cohesion):

$$\begin{align*}
\nabla \cdot \sigma + \rho \dot{\varepsilon} = 0 \\
\varepsilon = \varepsilon + \varepsilon_p \\
\varepsilon_p = 1 + \nu_u \frac{\sigma - \nu_u \sigma_i}{E_u} \\
\varepsilon = 0 \text{ if } F < 0 \\
\dot{\varepsilon_p} = q \frac{\partial F}{\partial \sigma} \text{ if } F = 0 \\
\frac{dF}{d\sigma} \cdot \dot{\sigma} = 0
\end{align*}$$

where $E_u, \nu_u = 0.495$ are undrained stiffness parameters, $E_u$ determined from (2.17), $F$ is Drucker-Prager yield function (2.9).

Total stresses $\sigma$ are calculated from (2.19), and effective stresses and pore pressures are derived from total stresses using relations (2.15), (2.18).

### 2.5 Conclusions

At the initial stage of this research, we have collected, tested and compared existing mathematical models for earthen dikes analysis, including models of filtration through porous media and soil mechanics. After a series of numerical tests, we selected the two-dimensional, unidirectional fluid-structure coupled model with linear elastic perfectly plastic associated flow defined by the Drucker-Prager yield function and with Richards’ model for simulation of porous flow in variably saturated media. This choice provided the optimal balance between realism and adequacy of the computational model, on the one hand, and high speed of numerical convergence in real-time, on the other hand.
Chapter 3 Implementation, integration into decision support system and performance assessment

3.1 Introduction

Equations of porous flow and deformations in earthen dikes outlined in Section 2.4 were numerically solved by the finite element method. In this chapter we briefly mention finite element method fundamentals. Next, details of implementation of the Virtual Dike module in Comsol software packaged are presented. Description of integration of the Virtual Dike module into the UrbanFlood decision support system is then followed by the parallel efficiency assessment. Performance benchmarks were carried on one computational node of a computer cloud Sara used for Virtual Dike hosting.

Finite element method is widely used for numerical solution of partial differential equations (PDE). It employs space discretization of the original unknown solution function \( U(x, y, z) \) by expressing it as a sum of pre-defined basis functions \( f_j(x, y, z) \) multiplied by unknown coefficients \( U_j \); the number of terms in the sum equals to the number of nodes in finite element mesh: \( j = N \); and coefficients \( U_j \) are nodal values of \( U(x, y, z) \) (Potts and Zdravkovic, 1999):

\[ U(x, y, z) \approx \sum_{j=1}^{N} f_j(x, y, z) \cdot U_j \]

The original PDE is then reduced to a system of algebraic equations, e.g. by applying the least squares method to the residual integral on a space domain. For physically non-linear problems like plastic flow, the resulting system of algebraic equations is non-linear, due to dependence of stiffness characteristics on the stress level. Nonlinearity introduced by the constitutive behaviour requires incremental solution procedure with linear algebraic system (LAS) at each increment:

\[ [K_G]^i \cdot [\Delta U]^i = [\Delta F]^i \]  \hspace{1cm} (3.1)

where \( i \) is number of increment, \([K_G]^i\) is incremental global stiffness matrix, \([\Delta U]^i\) is vector of nodal solution increments (of length \( N \)), \([\Delta F]^i\) is vector of nodal load increments.

---

Equation (3.1) was solved by the modified Newton-Raphson (MNR) iterational procedure (see a detailed specification of MNR procedure in (Potts and Zdravkovic, 1999)).

The finite element method was introduced to slope stability analysis in (Whitman and Bailey, 1967). Zienkiewicz (Zienkiewicz et al., 1975) has proposed to treat divergence of numerical solution as an indicator of actual slope failure, so that simulated failure occurs naturally as the shear strength of the soil becomes insufficient to resist the shear stresses. When perfect plastic yielding occurs in large zones, the stiffness matrix (while solving equations numerically) approaches a singular matrix, and iterations of the finite element solver fail to converge. Physically, it indicates the onset of global plastic yielding, so that present configuration of the dike can not remain stable under the specified load.

3.2 Virtual Dike Implementation

In the Virtual Dike module, partial differential equations are solved in the finite element package COMSOL 3.5a. Finite element mesh composed of triangle elements with second order of spatial approximation was used in simulations. Time integration was performed by an implicit backward second order method. The modified Newton-Raphson iterative scheme was used for solving nonlinear algebraic equations at each time integration step. During MNR iterations, systems of linear algebraic equations were solved by direct PARDISO solver from the BLAS library.

Automatic sensor input was implemented via MATLAB script, which reads input from a specified file, starts COMSOL simulation and stores virtual sensor output (Figure 3-1).

![Figure 3-1. Virtual Dike –sensor input/output handling](image-url)
Figure 3-2. Computational workflow in the Virtual Dike module

Fluid sub-model solution:
- Solve Richards equation using time-dependent solver with automatic time sub-stepping
- Backward time integration scheme with 2d order of approximation used for time integration
- MNR non-linear iterative procedure at each time sub-step. Relative tolerance 10^{-5}. PARDISO linear algebraic solver used within MNR iterations
- Save current solution to be used as the initial state for the next physical time step

Mechanical sub-model solution:
- Increment of hydraulic load is computed as difference between pore pressure gradient at the current time step and pressure gradient at the previous time-step
- Determine stress-strained state of the dike using parametric (incremental) static elasto-plastic solver
- Save current solution to be used as the initial state for the next physical time step

Sea water level value passed from sensors to fluid sub-model as input data

Pore pressure distribution passed to the mechanical sub-model as volume load

\[ t = t_0 \]

load initial states for the fluid- and mechanical- sub-models

\[ t = t + \Delta t \]

end
Functionality of the Comsol package supports multi-physics coupling – equations describing the time-dependent fluid sub-model and the quasi-static mechanical sub-model can be united in one system of equations, which will be integrated by a time-dependent solver. However, convergence of the time-dependent solver was poor for the highly-nonlinear mechanical sub-model: integration in time-domain required very small time-steps and finally stopped due to divergence of MNR iterations. Typically plastic deformation problems in Comsol require a steady-state parametric solver for their solution. That’s why a two-step solution scheme was used, where the porous flow sub-model was solved by a time-dependent solver; then pore pressure loads were transferred to the mechanical sub-model which in turn was solved by parametric solver. The computational workflow is described in Figure 3-2: the workflow with calls to Comsol routines is processed in a Matlab script; data exchange between the sub-models occurs in the computer RAM memory within the Matlab run-time environment. The loop in Figure 3-2 corresponds to time-stepping; stagger iterations were not used for the solution process, as the problem was considered as one-way coupled.

3.3 Integration into the UrbanFlood early warning system

Different components of the UrbanFlood EWS were running on several servers located in 4 countries: Russia, the Netherlands, Poland and the United Kingdom. The EWS has been designed in accordance with the cloud computer technology. Computer clouds use “virtualization software”, which allows multiple operating systems (=”virtual machines”) to share the same (rented) hardware. The operating systems themselves run various applications (such as Virtual Dike, Flood Simulator, AI component and others). Furthermore, the virtualisation software allows halting virtual machines (and contained running programs), move these to other computers and continue operations there (Meijer et al., 2012).

A user interface implemented on the Microsoft Surface multi-touch table provided access to all EWS components and allowed interactive simulation steering (Figure 3-3). Tests of the EWS performance showed a real-time response of system components to user manipulations.

Figure 3-3. Accessing EWS results via a multi-touch table
All simulation modules and visualization components of the UrbanFlood early warning system (Figure 1-1) have been integrated into the global communication platform - Common Information Space (CIS). CIS is an architectural framework providing services to address problems common to all early warning systems as complex software systems: integration of legacy scientific applications, workflow orchestration, allocation of computational resources and robust operation. The key components of CIS are (Balis et al., 2011):

• Integration platform (PlatIn): CIS core technologies for component integration, data exchange and workflow orchestration.

• Metadata registry (UFoReg): a generic service for hosting and querying metadata.

• Dynamic resource allocation service (DyReAlla): a service for dynamic allocation of resources to running Early Warning Systems.

A detailed description of CIS architecture can be found in (Balis et al., 2011) and at http://dice.cyfronet.pl/products/cis.

The Virtual Dike module was integrated into the CIS and deployed on a cloud computing resources of the SURFsara BiG Grid High Performance Computing and e-Science Support Centre in Amsterdam, the Netherlands https://www.surfsara.nl/. The cloud is hosted on a 128-core cluster and uses OpenNebula open source cloud computing management toolkit with KVM Virtual machine software. The web-based interface for virtual machines management provided by SARA support to users is shown in Figure 3-4.

![Virtual Dike machines](image)

Figure 3-4. Sara cloud management console with Virtual Dike machine instances

Virtual Dike simulations were run under Ubuntu Linux in a shared memory parallel mode. In the EWS workflow, the module ran in real time, receiving water level sensor signal as input data and producing “virtual sensor” signals (flow and structure parameters).

To start a new simulation, CIS launched a new instance of Linux Ubuntu virtual machine with the Virtual Dike model and wrote sensor input (sea/river level value) to the specified directory in real-time (the directory was updated every 5 minutes). The output from the Virtual Dike was stored in a specified directory on a hard drive, from where it was accessed by the CIS, compared to sensor measurements and visualized at the user front-end.
Monitoring input and output directories for new files and deleting the old files was performed by a Ruby script wrapping the *Virtual Dike* (Krzhizhanovskaya et al., 2012).

### 3.4 Performance benchmarking

Computational performance benchmarks have been carried in order to estimate the elapsed times for computationally heavy models with very fine mesh. In fact, we have tested efficiency of Comsol software package in application to the uncoupled porous flow analysis.

Besides testing parallel efficiency of the package, we have determined an optimal set of the package settings, allowing further reduction of elapse times. The tested settings include: direct solver algorithm (PARDISO versus UMFPACK), processor usage modes (owner, turnaround and throughput) and BLAS library implementations (MKL, ACML, ATLAS).

The goal of benchmarking was to find the best combinations of computational settings and to estimate feasibility and efficiency of parallel simulations. Parallel performance results have been previously published in (Melnikova et al., 2011).

For a relatively coarse mesh with 60,000 degrees of freedom (DOF) and maximum element size of 1 m, 2D porous flow simulations typically require up to 2 GB RAM in serial mode. Coupled fluid-structure problem requires up to 8 GB of memory in serial mode. The amount of occupied system memory increases when using parallel mode.

Comsol supports two modes of parallel operation: distributed mode and shared memory mode (Comsol Installation and Operations Guide, 2009). Distributed mode employs MPI library for process communications; it can be used on several nodes of a Linux or Windows cluster. Shared memory mode uses common address space for inter-process communication. It can only be used on a single multi-core or multi-processor computational node. In Virtual Dike simulations, one simulation used one computational node of SARA HPC Cloud, with dual quad-core CPU. Shared-memory parallelism has been tested for the uncoupled porous flow model, in the default processor usage mode.

#### 3.4.1 Parallel performance benchmarks

Parallel performance benchmarks were performed using two alternative direct linear sparse solvers: UMFPACK (Unsymmetric MultiFrontal PACKage) and PARDISO (PArallel DDirect SOLver). Results of the tests are presented in Figure 3-5a-b.

PARDISO sparse linear solver is recommended by Comsol as providing the best parallel efficiency in comparison to the other solvers. The results confirm this only for "small" problem size with number of DOF=60,000. Formally, for larger problem sizes, UMFPACK scalability was better (see Figure 3-5a for a plot of parallel efficiency computed in percentage of a real speedup to an ideal speedup (=number of cores np)).

The parallel speedup of each solver first improves as the number of DOF grows (as computational work portion grows higher relative to the portion of information exchange). After that, we get speedup saturation or even decrease for number of cores np equal to 8. The reason of low scalability and cases of decrease in speedup for np=8 may lie [a]: in high
process synchronization costs; [b] in a relatively high portion of sequential operations included in the algorithm; [c] in a bad-balanced distribution of load among the cores for 8-core case (one of the cores was obviously occupied with system tasks, too). The solver writes down solution on the hard disk with specified periodicity. As the number of DOF grows, this sequential output work increases. Naturally this I/O option could be withdrawn from the simulation benchmarks but it was important to estimate a speedup for real working conditions. The output from the solver gets to CIS and other computational modules via the hard disk files.

![Figure 3-5. Results for 1, 2, 4, 6, 8 cores: (a) parallel efficiency; (b) computational time](image)

Computational time varies from 30 min to 7 hours (Figure 3-5b), depending on the problem size, number of cores and chosen solver. For problems of 250 000 unknowns, PARDISO was almost 50% faster. UMFPACK speed itself was significantly lower for large problem sizes (Figure 3-5b). The slow speed of UNFPACK is explained by the fact that we have a symmetric system matrix in porous flow problems while UMFPACK is designed for a more general case of unsymmetric linear systems, it does not take an advantage of symmetry, which leads to a higher number of operations and slower computational speed. This also explains most probable reason why scalability of UMFPACK for 120 000 and 250 000 unknowns was better – communications were better hidden behind computations, due to a bigger portion of computations, compared to
PARDISO. Nevertheless, absolute elapse times for PARDISO were always lower or equal to UMFPACK times, so the general conclusion was to use the PARDISO solver for the Virtual Dike module.

### 3.4.2 Processor usage modes and BLAS libraries efficiency benchmarks

Benchmark results presented below have also been obtained for porous flow problem, using one computational node of SARA. But here Comsol simulation has been coupled with MATLAB for automatic sensor input of water levels. This coupling reduced the performance and parallel scalability of the Virtual Dike module.

Comsol supports three processor usage modes: owner (recommended for processors, not occupied with other tasks), turnaround and throughput (both are recommended for multi-tasking regimes). Benchmark results showed that the owner mode is the worst (see Figure 3-6a). Turnaround mode was the best for all cases except the case of using 8 cores, which is not recommended as it increases computational time, comparing to the cases of using 4 and 6 cores.

![Comparison of processor usage modes](image1)

![Comparison of BLAS versions efficiency](image2)

Figure 3-6. Computational time: (a) comparison of processor usage modes; (b) comparison of BLAS versions efficiency
Results of BLAS libraries efficiency are presented in Figure 3-6b. ACML, Intel and MKL BLAS libraries have been tested, for 2, 4, 6 and 8 computational cores. ACML BLAS library was optimal for all cases, except the case of 8 cores. Using 6 cores with ACML library provided the lowest computational time.

3.5 Conclusions

The Virtual Dike module has become the first implementation of a finite element analysis tool for dike stability assessment working with real-time sensor input. The module also produces real-time output from “virtual sensors” and stores it in the common information space of the early warning system. The module was deployed on an 8-core node of the supercomputer cloud SARA (Amsterdam).

According to the computational efficiency benchmarks, two- and four-core parallelism was good enough, with parallel efficiency higher than 50%. For more than 4 cores, synchronisation costs were too high. PARDISO linear solver, “turnaround” CPU usage mode and ACML BLAS library were chosen as optimal settings to get lowest computational times.
Chapter 4  Livedike case study: simulation and validation of the fluid sub-model

The first validation of the Virtual Dike module was performed for the LiveDike - an earthen levee protecting a sea-port in Groningen, the Netherlands. The LiveDike was one of the research sites of the UrbanFlood project.

Besides the Livedike analysis, in this chapter we focus on the influence of variation in soil diffusivity on the pore pressure distribution. Correct calculation of pore pressures in the dike is critical for correct assessment of dike safety margin (it will be brightly illustrated on the example of another dike simulation – Boston levee validation site, Chapter 5). We did not analyze sensitivity of safety margin to soil strength parameters variation because there have been published many studies confirming that the safety margin is extremely sensitive to such variation (see, e.g., Griffiths and Lane, 1999). On the base of sensitivity analysis, we have constructed and implemented a new automatic procedure for calibration of soil diffusivities in an arbitrary heterogeneous dike based on historical pore pressure sensors recordings. The procedure has been successfully tested for the LiveDike diffusivities calibration.

LiveDike has been equipped with the Geobeads pore pressure sensors by Alert Solutions, measuring tidal fluctuations in real-time mode at the Livedike dashboard site http://livedijk-www.ict.tno.nl/, and at the Alert Solutions dashboard http://datapanel.alertsolutions.nl/asview/ (the sensor data are available for internal users only). Pore pressure readings were used to validate the fluid sub-model of the Virtual Dike module, by matching sensor data and simulated data and calibrating diffusivities of soil strata.

Realistic modelling of water flow through the dikes is necessary for correct estimation of effective stresses in the dikes and hence for predicting their stability. Calibration of diffusivities for the tidal groundwater flow is often performed by tidal methods (Smith and Hick, 2001), (Slooten et al., 2010), (Williams et. al., 1970) based on one-dimensional analytical models of semi-infinite or finite aquifers. This method is suitable for aquifers with nearly horizontal phreatic surfaces. A more accurate way that works well for high amplitude of water level variation is direct numerical simulation. In the present work, both analytical and numerical approaches have been tested and compared. Calibration of diffusivities of soil strata has been performed by matching tidal pore pressure fluctuations obtained from numerical simulation and from piezometers installed in several cross-sections of the dike. For heterogeneous soil structures, some averaged and simplified yet heterogeneous soil build-ups have been obtained, so that the response of the dike to the tidal load corresponds well to sensor measurements.

3 Parts of this chapter have been published in (Melnikova, N.B., Krzhizhanovskaya, V.V., Sloot, P.M.A. (2013). Modelling Earthen Dikes Using Real-Time Sensor Data. Journal of Hydrology 496 (2013), pp. 154-165. DOI: 10.1016/j.jhydrol.2013.05.031)
4.1 LiveDike: geometry, soil build-up, loadings and sensor data

Livedike location and the view of the levee are shown in Figure 4-1a,b. The height of the dike is 9 m, the width is about 60 m and the length is about 800 m. The dike has a highly permeable sand core covered by a 60 cm thick clay layer.

The Livedike is a heterogeneous sand dike with clayey inclusions and gentle slopes (14° and 18° for the sea side and the land side slopes, respectively). Typical value of angle of internal friction for sand lies in the range of 25°-30°; this lets to conclude that macro-instability does not threaten to the dike safety.

Figure 4-1 (a) LiveDike: (a) location of the test site near Groningen, The Netherlands; (b) photo of the dike

The LiveDike has been equipped with sensors with GPS locations shown in Figure 4-2a. Sensors are placed in four cross-sections (slices), see Figure 4-2a,b. These slices have been simulated in 2D models under tidal water loading, in order to calibrate diffusivities, simulate flow through the dike and finally analyze the structural stability of the dike.
Figure 4-2 (a) Top view at the LiveDike (Eemshaven); (b) Slices of the LiveDike; (c) 2D model of a dike slice with pore pressure sensor locations shown with red dots

A geometric model of a dike slice with sensor locations is presented in Figure 4-2c. Sensors E1-E4 and G1-G2 measure absolute pore pressure and temperature and produce data stream which is available in real-time on web dashboards. For calibration of the model, we used signals from the E3, E4 and G2 pore pressure sensors located below the phreatic surface. An input signal for simulation was the water level registered by the sensor installed outside of the dike. The sea-side toe of the dike is located at x=0 m, y=-0.7 m, while the mean sea level is at y=0 m.

The soil build-up for a longitudinal cross-section passing through the crest of the dike is presented in Figure 4-3. It contains horizontal layers of sand (1 - light orange), silty sand with small clay inclusions (5 - lemon) and 60 cm clay layer that covers the dike (2 - blue). Grey areas (4) are clayey sand. Below the sand layers lies impermeable clay layer (3 - blue). Cone penetration test (CPT) results (cone end resistance and frictional resistance) are schematically shown with black lines. More information on the CPT testing methodology can be found in (Meigh, 1987).
A sample of sensor data showing air pressure, sea level and pore pressure is presented in Figure 4-4 and Figure 4-5, for a time period that has been used for diffusivity calibration (“training period”). Sea level dynamics is presented in Figure 4-4b, with positions of local maximum and minimum marked with dashed lines. Figure 4-5 presents pore pressure measured in three slices of the dike. For calibration of diffusivities, the original pore pressure signals were smoothed (denoised) by a localized linear fit algorithm with an adaptive window (the smoothed signals are also shown in Figure 4-5). Then the levels of minimal and maximal tidal pressure were detected for the smoothed pressure signals. These levels are shown in Figure 4-5 with horizontal dashed lines. Corresponding pressure values are specified in the legends. Vertical dashed lines show the moments in time corresponding to the minimal and maximal pressure values. The corresponding time values are specified in the legends. The obtained relative pressure amplitudes and time delays between local pressure maximum and sea level maximum are presented in Table 4-1 (page 37).

E3 and E4 sensors are located at the same distance from the sea (x=50 m), but at the different levels (y=-1.5 m and y=-5.5 m from the reference level, correspondingly). E3 pressure oscillations are lower than E4 oscillations and this fact points to the presence of a vertical heterogeneity in the dike. A time delay between E4 oscillations (at x=50 m) and tidal oscillations (at x=0 m) varies in the range between 3 and 18 min, which indicates highly permeable sand in the zone 0 <x<50 m. E3 oscillations lag from tidal oscillations by 9-38 min.

Figure 4-4. LiveDike: (a) atmospheric pressure [mbar] and (b) sea level [cm] registered by sensors
Figure 4-5. LiveDike: absolute pore pressure [mbar] registered by sensors and smoothed pressure signals

E3 and G2 sensors are located at approximately the same level (-1.2 m÷-1.5 m from ref. level), but at different distances from the sea (x=50 and x=62 m, correspondingly). In the first slice, the amplitude of pore pressure dissipates quickly within 12 m of horizontal distance between E3 and G2 sensors (Table 4-1). It indicates the presence of a horizontal
heterogeneity in the sand layers, with diffusivity decreasing with the distance from the sea, up to a dense impermeable zone near G2. This impermeable zone creates high time lag between G2 oscillations and tidal oscillations: the lag equals to 49 min in the first slice.

Table 4-1. LiveDike pressure sensors measurements: relative pressure amplitudes and time delays between the tide and local pressure fluctuations

<table>
<thead>
<tr>
<th></th>
<th>Slice1</th>
<th>Slice2</th>
<th>Slice3</th>
<th>Slice1</th>
<th>Slice2</th>
<th>Slice3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sea water level sensor data</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>sea level drop: 258 cm = 253 mbar; time of local maximum: 9.01.2010 5.00</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pore pressure sensors data</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slice1</td>
<td>Slice2</td>
<td>Slice3</td>
<td>Slice1</td>
<td>Slice2</td>
<td>Slice3</td>
<td></td>
</tr>
<tr>
<td>Relative daily oscillations amplitude (fraction of tidal daily oscillations amplitude)</td>
<td>0.21</td>
<td>0.11</td>
<td>0.10</td>
<td>18</td>
<td>3</td>
<td>18</td>
</tr>
<tr>
<td>Time delay between local pressure maximum and sea level maximum, minutes</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>E4</td>
<td>0.09</td>
<td>0.10</td>
<td>0.07</td>
<td>24</td>
<td>9</td>
<td>38</td>
</tr>
<tr>
<td>E3</td>
<td>0.03</td>
<td>0.08</td>
<td>0.04</td>
<td>49</td>
<td>19</td>
<td>9</td>
</tr>
<tr>
<td>G2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In order to reproduce the actual pore pressure fluctuations, a heterogeneous 2D model of the 1st slice of the LiveDike was built. Below we present the mathematical model of porous flow, numerical and analytical studies of diffusivity influence on the pore pressure dynamics in the dike and calibration of diffusivities based on sensor data.

4.2 Governing equations, model data and computational mesh

Water flow through the dike was modelled by Richards’ equation (2.12) with the van Genuchten model (2.4)-(2.6) for water retention in partially saturated soil around the phreatic surface. The boundary conditions are shown in Figure 4-6:

magenta boundaries are walls with zero normal flux $\frac{\partial p}{\partial n} = 0$ ;

black boundaries are sea side with tidal pressure oscillations specified:

$$\begin{cases} 
  p = \rho g \cdot (h(t) - y) & \text{for } y \leq h(t), \\
  p = 0 & \text{for } y > h(t)
\end{cases} \quad (4.1)$$

where $h(t)$ is oscillating sea level [m], measured by sensors or predicted by hydrological model;

blue boundaries are land side with attenuated oscillations of ground water level:
\[
\begin{align*}
\begin{cases}
p = \rho g \cdot (h_{gw}(t) - y) & \text{for } y \leq h_{gw}(t), \\
p = 0 & \text{for } y > h(t)
\end{cases},
\end{align*}
\] (4.2)

where \( h_{gw}(t) \) denotes oscillating ground water level, representing attenuated and altered tidal signal.

In the regime of forced tidal oscillations, the initial condition in (2.12) does not affect the steady solution, due to dissipation of the initial pore pressure distribution within several tidal periods. A hydrostatic distribution below \( y=0 \) m was specified as a technical initial condition, like it was described in section 2.1.3.

In the saturated zone, where \( \theta_e = 1, C=0, k_r = 1 \), porous flow is modelled by linear and parabolic Laplace equation:

\[
\frac{\partial p}{\partial t} + \nabla \cdot [-d \nabla (p + \rho g y)] = 0,
\] (4.3)

where \( d = K_S / S \mu \) [m²/s] is soil-water diffusivity, the only soil parameter that influences pore pressure dynamics under the specified load.

Table 4-2 gives a list of soil parameters that define unsaturated behaviour and strength/stiffness characteristics of soil. These parameters have been set for the LiveDike using reference properties of sand.

Table 4-2. LiveDike – soil parameters

<table>
<thead>
<tr>
<th>Van Genuchten parameters</th>
<th>Young’s modulus ( E ), Pa</th>
<th>Poisson’s ratio ( \nu )</th>
<th>Friction angle ( \phi ), degrees</th>
<th>Cohesion ( c ), Pa</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \alpha ), [1/m]</td>
<td>( n )</td>
<td>( l )</td>
<td>( 10^{10} )</td>
<td>0.3</td>
</tr>
<tr>
<td>8</td>
<td>1.5</td>
<td>0.5</td>
<td>30</td>
<td>0</td>
</tr>
</tbody>
</table>
Water viscosity is calculated as a linear interpolation function of water temperature between the points defined in Table 4-3. Figure 4-7 shows water temperature measurements over a period of one year. Due to the variation of water viscosity, the value of soil diffusivity in summer is 1.8 times higher than in winter.

![Water temperature at water surface, Eemshaven](image)

**Figure 4-7.** Sea water temperature distribution during year cycle

**Table 4-3.** Water viscosity values

<table>
<thead>
<tr>
<th>Temperature, °C</th>
<th>Dynamic viscosity, Pa·s</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>1.004·10^-3</td>
</tr>
<tr>
<td>10</td>
<td>1.307·10^-3</td>
</tr>
<tr>
<td>0</td>
<td>1.797·10^-3</td>
</tr>
</tbody>
</table>

![Finite element mesh](image)

**Figure 4-8.** Livedike: finite element mesh with refinement zone around the phreatic line

Finite element mesh used for all computations (for both sensitivity analysis and Livedike calibration parameters) was composed from triangular elements of second order of
space approximation (Figure 4-9). The mesh contains refinement zone located around the phreatic line (which changes its shape during tidal oscillations), where pressure gradient is high. The total number of elements is about 15000.

4.3 2D numerical sensitivity analysis of pressure amplitude and time delay to the variation of soil diffusivity

Sensitivity analysis has been performed to study the influence of saturated soil-water diffusivity on tidal oscillations of pore pressure in the dike. A 2D homogeneous dike model has been considered. Geometric prototype of the model is the LiveDike slice. Boundary conditions’ zones have been described in section 4.2. At the seaside, harmonic tidal pressure oscillations are specified; at the landside, the ground water level is constant (zero meters from average sea level). A number of porous flow simulations have been performed, with saturated diffusivities varied in the range of 0.1-1000 m$^2$/s. Distribution of relative pore pressure amplitudes, normalized to tidal amplitude, is presented in Figure 4-9a, for a horizontal slice of the dike (at the level $y = -5.5$ m).

For relatively high values of diffusivity ($d = 10 \div 1000$ m$^2$/s) relative pressure amplitude distribution is linear with a very small non-linear tail close to the sea-side (left) slope. The non-linear part corresponds to the zone where the flow is essentially two-dimensional: at $x \leq 0$ m, water penetrates into the domain both through the vertical boundary and through the under-water slope of the dike (see Figure 4-10 for arrow plot of flow velocity). At $x \geq 0$ m the flow is almost one-dimensional, and relative pressure amplitude distribution qualitatively agrees with the 1D analytical solution presented in the next paragraph.

![Figure 4-9](image)

Figure 4-9. Numerical study of sensitivity of tidal oscillations to diffusivity variation, 2D analysis: (a) relative pressure amplitude distribution along the dike; (b) time delay distribution along the dike. Data shown in a horizontal slice $y = -5.5$ m (at the level of the E4 sensor, see Figure 4-2(c))
Figure 4-10. Arrow plot of velocity field (high tide). The flow is essentially two-dimensional at the sea bottom, \( x \leq 0 \) m.

Figure 4-9a shows that, in a relatively short computational domain (~100 metres long) and for loose, permeable media (like gravel and coarse sand), simulated amplitude of tidal fluctuations is insensitive to the actual value of diffusivity (see lines for \( d=1000 \), \( d=100 \), \( d=10 \)). This linear distribution is only defined by the amplitude of tidal fluctuations at the sea-side and at the land-side boundaries of the domain. To the contrary, time delay is sensitive to the value of diffusivity in the whole range (Figure 4-9(b)), therefore time delay can be calibrated by tuning diffusivity value.

For diffusivities \( d \leq 1 \) m\(^2\)/s, a significant non-linearity appears in the pressure amplitude distribution: pore pressure amplitude within the dike depends on the diffusivity.

Figure 4-11 shows pressure amplitude and time delay as functions of diffusivity, in LiveDike E4 sensor location (50; -5.5). The amplitude and phase delay values of E4 sensor are shown in Figure 4-11 with dashed lines.

Figure 4-11. Numerical study of influence of soil diffusivity on: (a) relative pore pressure amplitude and (b) time delay in E4 sensor location (50m; -5.5m). Actual sensor readings shown with dashed lines.

From Figure 4-11 it is clear that matching both the amplitude and phase lag with only one parameter (diffusivity) is impossible: matching the amplitude value requires
diffusivity $d\sim 1$ m$^2$/s, while matching the time delay requires that $d\sim 100$ m$^2$/s. Formally, besides the diffusivity, one more parameter (a length of a homogeneous zone in the layered soil build-up) is necessary to match the data for one sensor. In fact, this contradiction indicates presence of heterogeneity in the LiveDike soil build-up (while the prototype dike in sensitivity analysis is homogeneous). Thus we construct a model of a dike as a set of horizontal stripes, each stripe divided into a number of homogeneous sectors with constant diffusivity. The length of a sector is the second parameter necessary for matching sensor data (Figure 4-12). Figure 4-12 presents a scheme of construction of a heterogeneous dike model to match sensor data. Sensors E$_1$, E$_2$, G$_1$ are not taken into consideration in the model as they are located above the phreatic surface and they do not produce data on pore pressure. For 6 values to match (these are pressure amplitude and time delay for 3 sensors: E$_4$, E$_3$, G$_2$), 6 parameters have been used: lengths of homogeneous zones $L_1$, $L_2$ and diffusivities $d_1$, $d_2$, $d_3$, $d_4$ (see Figure 4-12). After calibration a total length of the simulation domain equals to the sum of parameters $L_1$, $L_2$.

![Figure 4-12. Construction of a heterogeneous dike model to match sensor data](image)

**4.4 1D analytical sensitivity analysis of tidal pressure oscillations to the variation of soil diffusivity**

In this section, two analytical solutions for the problem of harmonic flow in a one-dimensional saturated homogeneous aquifer are derived and compared to the direct numerical solutions which were discussed above. A one-dimensional analytical model can be used for modelling tidal propagations through the aquifers with a low gradient of the phreatic line.

The objectives for employing one-dimensional analytical models for dike diffusivity calibration are:

- Obtaining formulas for initial guess values of diffusivity;
- Qualitative study of penetration of tidal waves through the dike

Flow in a one-dimensional saturated aquifer is described by the equation
Harmonic boundary conditions defining two different problems are considered:

- A semi-infinite aquifer with sine oscillations of water pressure at the boundary \( x=0 \):
  \[
  p(x,t) \bigg|_{x=0} = A \sin(\omega t) \\
p(x,t) \bigg|_{x \to \infty} \to 0
  \]
  \[
  (4.5)
  \]
  where \( A \) is amplitude of pressure oscillations; \( \omega \) is angular frequency;

- A finite aquifer with sine pressure oscillations at \( x=L \) and constant pressure \( p=0 \) at \( x=0 \):
  \[
  p(x,t) \bigg|_{x=L} = A \sin(\omega t) \\
p(x,t) \bigg|_{x=0} = 0
  \]
  \[
  (4.6)
  \]
  Initial conditions for (4.4) are not considered here, as we are interested in the settled, tide-forced pressure oscillations. The settled solution for the semi-infinite aquifer problem (4.4) with boundary conditions (4.5) is expressed as follows (Ferris, 1951):
  \[
  p(x,t) = Ae^{-x \sqrt{\frac{\omega}{2d}}} \sin \left( \omega(t - x \sqrt{\frac{1}{2d\omega}}) \right),
  \]
  \[
  (4.7)
  \]
  It represents a wave of pore pressure travelling in compressible soil, with an amplitude \( p_A \) [Pa] dissipating exponentially with the distance from the inlet, and a time delay \( \Delta t \) [s] growing linearly with the distance:
  \[
  p_A(x) = Ae^{-x \sqrt{\frac{\omega}{2d}}}, \quad \Delta t = x \sqrt{\frac{1}{2d\omega}}
  \]
  \[
  (4.8)
  \]
  Applying solution (4.7) to the model of the dike described in section 4.2 (for \(-30 \leq x \leq 90\)), we get distributions of relative pressure amplitude \( p_A(x)/A \) and time delay \( \Delta t \) (in a logarithmic scale), presented in Figure 4-13a,b. Diffusivity \( d \) varied in the range between 0.1 and 1000 m\(^2\)/s. Tidal frequency \( \omega=2\pi/T \), where \( T=12 \) hrs 25 min.

Figure 4-13a gives an estimate for a distance at which tidal waves penetrate into a homogeneous aquifer. For dense impermeable soils with diffusivity \( d \leq 0.1 \), pressure amplitude dissipates to a level of 4% of tidal amplitude, within the distance of 120 meters.
from the sea. For highly permeable soils with diffusivity \( \geq 10 \text{m}^2/\text{s} \), pressure amplitude distribution is linear in the whole domain, and this linear distribution has been confirmed by the 2D numerical analysis (section 4.2).

According to formula (4.8), slow seasonal water table fluctuations propagate further into an aquifer than daily fluctuations do, and this was taken into consideration for the LiveDike when specifying land side boundary conditions in the porous flow problem (section 4.5).

For the finite aquifer problem (4.4) with boundary conditions (4.6), solution representing steady harmonic oscillations and satisfying zero boundary condition \( p(x, t)|_{x=0} = 0 \) can be expressed as a sum of two complex conjugated independent partial solutions of (4.4):

\[
p = Ce^{i\omega x} \sinh(\sqrt{\frac{i\omega}{d}}x) + \overline{C}e^{-i\omega x} \sinh(\sqrt{\frac{i\omega}{d}}x), \tag{4.9}
\]

where \( i = \sqrt{-1} \); \( C = \text{Re}(C) + i \cdot \text{Im}(C) \) is a complex constant to be determined from the harmonic boundary condition:

\[
p(x, t)|_{x=L} = A \sin(\omega t) \leftrightarrow\]

\[
\leftrightarrow C(\cos(\omega t) + i \cdot \sin(\omega t)) \sinh(\sqrt{\frac{i\omega}{d}L}) + \overline{C}(\cos(\omega t) - i \cdot \sin(\omega t)) \sinh(\sqrt{-\frac{i\omega}{d}L}) = A \sin(\omega t) \tag{4.10}
\]

![Figure 4-13. Analytical study of sensitivity of tidal oscillations to diffusivity variation, 1D model of a semi-infinite saturated aquifer: (a) relative pressure amplitude distribution along the domain; (b) time lag, minutes, along the domain – in a logarithmic scale](image)
From (4.10) follows that:

\[
C = \frac{A}{2i \sinh(i \frac{\omega}{d} L)},
\]

(4.11)

(4.9)+(4.11) \Rightarrow

\[
p(x,t) = 2 \text{Re} \left( \frac{A \cdot \sinh(i \frac{\omega}{d} x)}{2i \sinh(i \frac{\omega}{d} L)} \cdot (\cos(\omega t) + i \cdot \sin(\omega t)) \right),
\]

(4.12)

Taking into account that

\[
\sinh(i \frac{\omega}{d} L) = \cos(\frac{\omega}{4d} L) \sinh(\frac{\omega}{4d} L) + i \cdot \sin(\frac{\omega}{4d} L) \cosh(\frac{\omega}{4d} L),
\]

(4.13)

(4.12) can then be written as:

\[
p(x,d,L,t) = p_A(x) \cdot \sin(\omega(t - \Delta t)),
\]

\[
p_A(x,d,L) = A \frac{\cosh(\frac{\omega}{d} L) - \cos(\frac{\omega}{d} L)}{\cosh(\frac{\omega}{d} L) - \cos(\frac{\omega}{d} L)},
\]

(4.14)

\[
\Delta t(x,d,L) = \frac{1}{\omega} \arctg \left( \frac{\exp r_1}{\exp r_2} \right) \text{ if } \exp r_2 > 0,
\]

\[
\frac{1}{\omega} (\pi + \arctg \left( \frac{\exp r_1}{\exp r_2} \right)) \text{ otherwise}
\]

Here

\[
\exp r_1 = -\sinh(\frac{\omega}{4d} (x + L)) \cdot \sin(\frac{\omega}{4d} (x - L)) + \sinh(\frac{\omega}{4d} (x - L)) \cdot \sin(\frac{\omega}{4d} (x + L)),
\]

(4.15)

\[
\exp r_2 = \cosh(\frac{\omega}{4d} (x + L)) \cdot \cos(\frac{\omega}{4d} (x - L)) - \cosh(\frac{\omega}{4d} (x - L)) \cdot \cos(\frac{\omega}{4d} (x + L))
\]

Expression (4.14) describes distributions of relative pore pressure amplitude and oscillations’ time lag along the bounded aquifer (see Figure 4-14 for their graphical representation).
Figure 4-14. Analytical study of sensitivity of tidal oscillations to diffusivity variation, 1D model of a bounded saturated aquifer: (a) relative pressure amplitude distribution; (b) time lag, minutes– in a logarithmic scale

For dense soils with diffusivity $d \leq 1 \text{ m}^2/\text{s}$, the analytical model predicts non-linear profiles of pressure distribution, however the absolute values of pore pressure do not agree with the 2D numerical simulation. For example, for $d=1 \text{ m}^2/\text{s}$, analytical relative pressure amplitude in point $x=50 \text{ m}$ $P_A = 0.511$, while in the 2D numerical solution simulated amplitude $P_A = 0.2$.

Possible sources of mismatch between the two models are: two-dimensional flow behaviour at the sea-side and diffusion of water above the phreatic line, which is considered in the 2D numerical model only.

Calibration of the LiveDike soil parameters based on the sensitivity analysis is described in section 4.5.

### 4.5 Automatic procedure for diffusivities calibration

Calibration has been performed for the first slice of the dike. As it was mentioned in section 4.3, we have to find the values of 6 parameters: lengths of homogeneous zones $L_1$, $L_2$ and diffusivities $d_1$, $d_2$, $d_3$, $d_4$ (see Figure 4-12). Below we describe the procedure of diffusivity calibration using measured data from 3 sensors: E3, E4 and G2 (Figure 4-12). The algorithm is generic and can be used for any number of sensors in a dike slice.

Initial estimate values of $L$ and $d$ parameters are obtained by superposition of analytical solutions derived from the solution (4.14) for various periodic boundary conditions:

- In the $1^\text{st}$ zone ($d=d_1$, $0<x_1<L_1$): $p_1(x_1,t) = p_{11}(x_1,t) + p_{12}(x_1,t)$, where $p_{11}(x_1,t)$ is a solution of (4.4) with the boundary conditions:

$$p_{11}(x_1,t)ig|_{x_1=0} = A \sin(\omega t), \quad p_{11}(x_1,t)ig|_{x_1=L_1} = 0,$$

$$p_{12}(x_1,t) = \frac{1}{4} \left[ p(0,t) - p(L_1,t) \right].$$

(4.16)
and $p_{12}(x_1, t)$ is a solution of (4.4) with the boundary conditions:

$$p_{12}(x_1, t)igg|_{x_1=0} = 0, p_{12}(x_1, t)igg|_{x_1=L_1} = A_{interface1} \sin(\omega t + \varphi_{interface1}),$$

(4.17)

Here $A$ is tidal amplitude, $\omega$ is tidal frequency, $A_{interface1}$, $\varphi_{interface1}$ are local amplitude and phase delay on the interface of zones #1 and #2 (not known a priori, to be determined from a continuity condition (4.19);

- In the 2\textsuperscript{d} zone ($d=d_2$, $0<x_2<L_2$): $p_2(x_2, t)$ is a solution of (4.4) with the boundary conditions:

$$p_2(x_2, t)igg|_{x_2=0} = A_{interface1} \sin(\omega t + \varphi_{interface1}), p_2(x_2, t)igg|_{x_2=L_2} = 0;$$

(4.18)

- Continuity condition for the interface between 1\textsuperscript{st} and 2d zones states that the value of flow velocity does not change at the interface:

$$\frac{\partial}{\partial x_1} p_1(x_1, t)\bigg|_{x_1=L_1} = \frac{\partial}{\partial x_2} p_2(x_2, t)\bigg|_{x_2=0},$$

(4.19)

From equation (4.19), we obtain two independent conditions: one for oscillation amplitude $A_{interface1}$ and one for oscillation phase $\varphi_{interface1}$.

- In the 3\textsuperscript{d} zone ($d=d_3$, $0<x_1<L_1$): $p_3(x_1, t) = p_{31}(x_1, t) + p_{32}(x_1, t)$, where $p_{31}(x_1, t)$ is a solution of (4.4) with the boundary conditions:

$$p_{31}(x_1, t)\bigg|_{x_1=0} = A_{interface1} \sin(\omega t), p_{31}(x_1, t)\bigg|_{x_1=L_1} = 0$$

(4.20)

$p_{32}(x_1, t)$ is a solution of (4.4) with the boundary conditions:

$$p_{32}(x_1, t)\bigg|_{x_1=0} = 0, p_{32}(x_1, t)\bigg|_{x_1=L_1} = A_{interface2} \sin(\omega t + \varphi_{interface2}),$$

(4.21)

where $A_{interface2}$, $\varphi_{interface2}$ are unknown local amplitude and phase delay on the interface of zones #3 and #4;

- In the 4\textsuperscript{th} zone ($d=d_4$, $0<x_2<L_2$): $p_4(x_2, t)$ is a solution of (4.4) with the boundary conditions:

$$p_4(x_2, t)\bigg|_{x_2=0} = A_{interface2} \sin(\omega t + \varphi_{interface2}), p_4(x_2, t)\bigg|_{x_2=L_2} = 0$$

(4.22)

- Continuity condition for the interface between 3\textsuperscript{d} and 4\textsuperscript{th} zones is:
\( \frac{\partial}{\partial x_1} p_3(x_1, t) \bigg|_{x_1=L_1} = \frac{\partial}{\partial x_2} p_4(x_2, t) \bigg|_{x_2=0} \) (4.23)

Similar to (4.19), (4.23) gives 2 scalar conditions: one for oscillation amplitude and one for oscillation phase.

Equations (4.19), (4.23) together with 6 conditions (4.16), (4.17), (4.18), (4.20), (4.21), (4.22) equating amplitudes and time lags in virtual sensors with those in real sensors E3, E4, G2 form a system of 10 scalar equations to determine the initial guess values for the parameters \( L_1, L_2, d_1, d_2, d_3, d_4, A_{\text{interface1}}, \phi_{\text{interface1}}, A_{\text{interface2}}, \phi_{\text{interface2}} \).

After obtaining initial guess values, more accurate values of \( d_1, d_2, d_3, d_4 \) are found by running a series of numerical simulations as described in section 4.3, comparing the results with real sensor data and tuning the parameters.

### 4.6 Livedike diffusivities calibration

A training period of 48 hours has been used for diffusivities calibration. The following parameters values have been derived (Table 4-4):

<table>
<thead>
<tr>
<th>(Horizontal diffusivity)·(water viscosity), Pa·m(^2)</th>
<th>Zone length</th>
</tr>
</thead>
<tbody>
<tr>
<td>( d_1 \cdot \mu )</td>
<td>( d_2 \cdot \mu )</td>
</tr>
<tr>
<td>0.1( \cdot )10(^{-4})</td>
<td>0.01( \cdot )10(^{-3})</td>
</tr>
</tbody>
</table>

Simulation results for a "training" period of two days are shown in Figure 4-15(a), for the E4 pressure sensor.

For long-term behaviour, slow attenuated fluctuations of the ground water level \( h_{gw}(t) \) at the land side of the dike should be represented in the boundary condition. Month simulations for January 2010 and August 2009 periods have been performed. The attenuated signal \( h_{gw}(t) \) has been obtained by averaging the tidal signal \( h(t) \) with a one-day sliding window and multiplying it by a dissipation coefficient \( q \): \( h_{gw}(t) = q \cdot h(t)_{\text{averaged}} \) (see Figure 4-16 for January ground water table plot). The value of \( q \) varied depending on the season (\( q=0.15 \) for January and \( q=0.25 \) for August). Looking at Figure 4-16, we can see that the averaged tidal signal represents slow oscillations with the period varying between 2 and 3 days. Variation of the dissipation coefficient \( q \) with the season qualitatively agrees with the analytical solution (12) for propagation of slow fluctuations in homogeneous aquifer: according to (12), \( q = e^{-\frac{x}{\sqrt{Td}}} \Rightarrow \)
\[ q_{\text{august}} = e^{-\frac{L_1 + L_2}{Td} \sqrt{\frac{D}{T}}} = 0.28, \] where homogeneous aquifer diffusivity \( d = 0.1 \text{ m}^2/\text{s} \), for slow oscillations with period \( T = 48 \text{ hrs} \), aquifer length \( L_1 + L_2 = 95 \text{ m} \);

\[ q_{\text{january}} = e^{-\frac{L_1 + L_2}{Td} \sqrt{\frac{D}{T}}} = 0.18, \] for the aquifer with diffusivity \( d = 0.1/1.8 \text{ m}^2/\text{s} \) (which is summer diffusivity scaled by \( \mu_{\text{january}} / \mu_{\text{august}} \)), \( T = 48 \text{ hrs} \), \( L_1 + L_2 = 95 \text{ m} \).

Pore pressure fields for the high and low tides are shown in Figure 4-17. The unsaturated zone is shown with the white colour, so that the phreatic line is depicted as a boundary between white and coloured zones.

Fragments of the effective saturation field at the seaside are shown in Figure 4-18 for the high and low tides. During the low tide, water level inside the dike decreases and the sand located above the water table gets dry (yellow colour in Figure 4-18a). During the low tide the upper layer of clay, located above the water table, stays wet due to high water capacity of clay. In the high tide phase, water table rises and that can be seen in Figure 4-18b (blue saturated zone has enlarged, relatively to the Figure 4-18a).

![Figure 4-15. Relative pore pressure oscillations in sensor 1E4 with calibrated soil properties: (a) Comparison of real sensor data (blue) with simulation results (magenta) on training dataset; (b) The same, for a longer period of 12 days](image)

![Figure 4-16. LiveDike calibration: sea level and attenuated ground water level at the land side boundary](image)
Figure 4-17. Pore pressure field: (a) low tide; (b) high tide

Figure 4-18. Effective saturation field: (a) low tide; (b) high tide

4.7 Conclusions

One of the scientific questions posed in this research was estimation of uncertainties influence on the dike stability assessment; it was important to find out how variations in soil properties alter dike safety margin.

We have focused on the influence of variation in soil diffusivity on the pore pressure distribution. Correct calculation of pore pressures in the dike is critical for the appropriate assessment of dike safety margin. Sensitivity analysis has shown that for coarse media (gravel, coarse sand), distribution of pore pressure amplitudes within a dike is close to linear and is defined by boundary conditions at the seaside and landside boundaries, while diffusivity value does not affect this distribution. For dense soils (fine sands, clays), pressure amplitudes distribution is highly non-linear and to a large extent depends on the diffusivity value.

On the base of these conclusions, we have constructed and implemented a new automatic procedure for calibration of soil diffusivities in an arbitrary heterogeneous dike based on historical pore pressure sensors recordings. The procedure has been successfully tested for the LiveDike diffusivities calibration: simulation results with calibrated soil parameters match experimental data, not only on the "training set" but also for a much longer period of time. The calibration procedure employs analytical solution obtained by us for the problem of tidal propagation in a one-dimensional finite homogeneous aquifer.
Chapter 5 Boston dike case study: simulation and validation of a coupled flow-structure interaction model under tidal loads

The first scientific question raised in this research was check of a principal ability of an earthen dike computational model to adequately simulate complex physical processes occurring at failure and correctly predict failure under prescribed loads. The Boston levee validation test site has become the next step towards the positive answer to this question and a platform for cross-validation of three computational models for dike stability assessment: the Virtual Dike model, a finite element model built in the commercial software package Plaxis and a limit equilibrium analysis model based on the Bishop’s method.

5.1 Test site description and ground conditions

The earthen levee at Boston, a town on the east coast of England at high risk of flooding, is known for a history of frequent toe slippages on the river side. This mechanism is presumably caused by high pore water pressures remaining in the dike when the river water recedes. The dike forms the right bank of the River Haven, which has a tidal range of about 6 m. The crest level of the dike is at 6m above the mean sea level; the deepest part of the river bed is at 2m from the mean sea level. The site is predominately grassed with several trees (Figure 5-1a). It has suffered from instability at the toe along the majority of its length (Figure 5-1b).

Figure 5-1 (a) Boston levee location at the right bank of the river Haven; (b) View of embankment with signs of toe slippage

Parts of this chapter have been published in (Melnikova, N.B., Jordan, D., Krzhizhanovskaya, V.V., Sloot, P.M.A. (2014). Slope instability of the earthen dike in Boston, UK: numerical simulation and sensor data analysis. Submitted to Journal of Computational Science, Elsevier)
The area was investigated in 2010 as part of the Boston Barrier Phase 1 Ground Investigation, this included a single borehole within the study dike. Further boreholes and Cone Penetration Tests were carried out as part of the installation of the sensors for the UrbanFlood Project. From the investigation, the variation in ground conditions across the site may be summarized as shown in Figure 5-2; beneath made ground and a thin layer of fine sand, lies some 5m of soft to firm alluvial clays. These in turn overlie sands and stiff boulder clay. The obtained soil parameters are presented in Table 5-1. Sensor locations are specified in Figure 5-2 relative to the Ordnance Datum (OD), which is the reference sea level in Great Britain (defined as the mean sea level at Newlyn in Cornwall between 1915 and 1921).

![Figure 5-2. Scheme of the cross-section A: soil build-up and sensors elevations, metres from Ordnance Datum](image)

**Table 5-1. Summary of soil parameters for the Boston levee**

<table>
<thead>
<tr>
<th>Property</th>
<th>Made ground</th>
<th>Fine sand</th>
<th>Soft brown clay</th>
<th>Dark brown sand</th>
<th>Firm grey clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hydraulic conductivity, m/day</td>
<td>10</td>
<td>1</td>
<td>0.01</td>
<td>10</td>
<td>0.01</td>
</tr>
<tr>
<td>Van Genuchten parameter $\alpha$, 1/m</td>
<td>2</td>
<td>2</td>
<td>0.5</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>Van Genuchten parameter $n$</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>Saturated water fraction</td>
<td>0.4</td>
<td>0.4</td>
<td>0.43</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Residual water fraction</td>
<td>0.045</td>
<td>0.045</td>
<td>0.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Density dry kN/m$^3$</td>
<td>19.5</td>
<td>17.5</td>
<td></td>
<td>18</td>
<td>22</td>
</tr>
<tr>
<td>Density wet kN/m$^3$</td>
<td></td>
<td></td>
<td></td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td>Effective Young’s modulus, MPa</td>
<td>18</td>
<td>28</td>
<td>2</td>
<td>18</td>
<td>2</td>
</tr>
<tr>
<td>Effective Poisson’s ratio</td>
<td>0.35</td>
<td>0.3</td>
<td>0.35</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Effective cohesion, kPa</td>
<td>5</td>
<td>0</td>
<td>2</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>Friction angle, grad</td>
<td>30</td>
<td>28</td>
<td>25</td>
<td>27.6</td>
<td>23.5</td>
</tr>
</tbody>
</table>

*Table cells containing “-” refer to the properties not used in corresponding soil strata. For example, Van Genuchten parameters for dark brown sand and firm grey clay were not used in simulation because these soil strata stay saturated during tide oscillations.

5.2 Instrumentation and sensor data analysis

The dike has been equipped at each of two cross-sections with six GeoBeads MEMS (micro-electro-mechanical) sensors registering pore pressure and media temperature. Differences in temperature measurement curves strongly indicate water flow through the soil - any drop in sensor temperature might be an indication of the development of piping. The Geobeads sensors have been installed in boreholes in the two planar transversal cross-sections (depicted as A and B in Figure 5-3). Positions of the GeoBeads sensors in the cross-section A are shown in Figure 5-2; vertical elevations are specified relative to the Ordnance Datum. Sensors AC1-AC4 are of increasing depth at the crest of the bund whilst sensors AS1-AS2 are located at about mid-slope height.

Figure 5-3. Boston levee site, with the cross-section locations
Figure 5-4. (a) river level dynamics; (b, c) sensor readings: (b) pore pressure; (c) medium temperature
The instrumentation and control building for the Grand Sluice on Haven River provided an ideal location for situating the computer equipment and providing power to the sensors. The use of mains power eliminated the need to replace batteries and reduced the level of maintenance required on the sensor equipment, ensuring uninterrupted data signals. The control building is located at 129 m distance upstream from cross-section A and is shown in the top of Figure 5-3 near the bridge with the sluice.

Sensor readings in the Boston levee had been previously analyzed in (Simm et al., 2012a), for a period of one year since the installation of sensors in May 2011. In the present research we focus on the response of the dike to tidal loading (putting aside slow seasonal processes), so we analyzed sensor data and dike stability for a one-month period in January 2012. The readings in cross-section A are presented in Figure 5-4; these are: water level dynamics in the river Figure 5-4a; pore pressure readings (Figure 5-4b) and water and air temperature readings (Figure 5-4c).

The results show a good response to tidal variations particularly in AS1, the upper sensor on the slope. Plateau-like segments of the AS1 curve correspond to the low-tide phases: the large soft brown clay layer stays saturated even at a low-tide phase (when the river bed is almost dry) - hence the pressure distribution and hydraulic load in the clay layer do not change during the low-tide phase. There is no significant time-lag between the tide and piezometric levels. The tidal range in mid-river is about 6m whilst the AS1 pressure head range is about 1m (100 mbar). This limited magnitude of response reflects the position of the piezometers within the slope relative to the maximum tidal variation in the centre of the river channel. It is considered that the piezometers are measuring an undrained elastic response in the soil-water continuum due to the loading caused by variation in water levels in the adjacent river channel. The assumption on the undrained state of the clay stratum is confirmed by comparing mean pressure heads measured by sensors in the soft brown clay stratum with pressure heads in the river and in the underlying dark brown sand layer (the comparison is discussed below).

According to sensor readings, AC1 is located in a dry zone above the ground water level, which is why it is not shown in Figure 5-4b. AC2 and AC3 sensors located in the soft clay layer far from the river-side slope are almost insensitive to the tide. Mean value of pore pressure oscillations (relative to atmospheric pressure) in sensors AC2 and AS1 is about 90 mbar. Both sensors are located at the same elevation of 1.64 metres above OD – so the mean ground water table in the dike is at 2.54 m above OD. According to the soil build-up scheme (Figure 5-2), the phreatic line goes through the thin layer of fine sand.

AC4 naturally produces high response to the tidal loading due to high conductivity of the dark brown sand layer where the sensor is placed. Mean pore pressure in AC4 is 450 mbar and its elevation is 3.96 m below OD, which gives mean value of total head +0.54 m above OD. This value agrees with the mean river level (Figure 5-4a), which is 0.6 for the considered period of time, proving that the dark brown sand layer is hydraulically connected to the river. As it was mentioned above, in the upper layers of soft brown clay, fine sand and made ground, the mean total head is at 2.54 m above OD which is two meter higher than in the underlying sand layer. This discrepancy confirms the assumption that the clay layer and the overlying soil layers (fine sand, made ground) are hydraulically isolated both from the foundation sand layer and from the river due to low permeability of clay. The
massive clay layer retains large amount of water coming with precipitation which is most intensive during winter time.

Soil and air temperature curves measured by sensors are shown in Figure 5-4c. At winter, the soil temperature is naturally higher than the air temperature, due to the high heat capacity of soils. Sensor AS1 produces minimal temperature values as it is closer to the land surface than the other sensors. AC1-AC2 and AS1-AS2 temperatures gradually decrease during the month, together with the dynamics of the mean air temperature value (and presumably of the water temperature in the river). Sensors AC3-AC5 located in deep soil strata below the ground water level produce nearly constant values of temperature around 10-12°C.

In the case of piping erosion in the dike, temperature sensors can provide warning information: decrease of local temperature value from expected soil temperature to the water temperature indicates that piping is occurring in the dike (Pyayt et al., 2014). However piping has never been visually observed or sensor detected at this site. As it was mentioned above, the dike is prone to occasional river-side toe slippages, such as shown in Figure 5-1b. There is no data available from the maintenance records about the precise moments in time when these local slippages occurred.

A detailed analysis of Boston site sensor data has been reported in (Pyayt et al., 2013a), where a data-driven approach with a neural network were applied for modelling a transfer function between the sensors within the Artificial Intelligence software module.

### 5.3 Mathematical models and numerical implementation

For the analysis of the Boston levee stability, two approaches have been employed: finite element modelling and limit equilibrium method. Below we separately describe mathematical models and numerical solution procedures for the two approaches.

#### 5.3.1 Finite element model

In FEM analysis, a one-way coupled fluid-to-structure interaction model for the planar cross-section of the dike has been considered.

Water flow through the porous media is described by Richards’ equation (2.12) with the Van Genuchten model (2.4)-(2.6) for water retention in vadose zones. Specific moisture capacity $C$ and relative permeability $k_r$ in the unsaturated zone are defined as functions of the effective water content (Van Genuchten, 1980).

The mechanical sub-model describes stress-strain state of the dike under hydraulic load, gravity and volumetric pore pressure load obtained from flow simulation. Linear elastic perfectly plastic strains of the soil skeleton are described by the general equations of plastic flow theory: (2.13) for drained behaviour, (2.19) - for undrained behaviour.

In the present work, we omitted pore suctions when calculating effective stresses above the phreatic line, assuming effective stresses to be equal to total stresses in the vadose zone. This assumption was based on the piezometers readings (see section 5.2 for details), which showed that the clayey part of the Boston levee volume stays fully saturated.
during tidal cycles, while a vadose zone exists on top of the dike in the loose, coarse-grained stratum built from the mixture of soil and debris and thus producing quite low suctions (“made ground” stratum in Figure 5-2):

\[ \sigma = \sigma_{eff} - pI \] in saturated zones, \[ \sigma = \sigma_{eff} \] in vadose zones.

Plastic flow has been modelled with a modification of the Drucker-Prager plasticity model, optimized for plane strain problems by providing the best smooth approximation of the Mohr-Coulomb surface in the stress space (Chen and Mizuno, 1990):

Equations (2.12)+(2.13) form a one-way coupled flow-structure interaction systems, where the porous flow sub-model generates a volume load (computed as pore pressure gradient) for the mechanical sub-model.

Boundary conditions for fluid and mechanical sub-models are schematically shown in Figure 5-5a,b.

Hydraulic boundary conditions are listed below:

- Black line (Figure 5-5a) - the river side, pressure boundary condition:
  \[
  \begin{align*}
  p &= \rho g \cdot (h(t) - y) & \text{for } y \leq h(t), \\
  p &= 0 & \text{for } y > h(t)
  \end{align*}
  \]
  where \( h(t) \) is river level, metres;

- Cyan line (Figure 5-5a) - the land side, pressure boundary condition:
  \[
  \begin{align*}
  p &= \rho g \cdot (h_L(t) - y) & \text{for } y \leq h_L(t), \\
  p &= 0 & \text{for } y > h_L(t)
  \end{align*}
  \]
  where \( h_L(t) \) is ground water level at the land side;

- Magenta line shows impervious walls.

Ground water table at the land side was specified using mean values of AC2 and AC4 pore pressure readings discussed in the previous section. For the dark brown sand and firm grey clay layers, \( h_L = 0.5 \) m; for the soft brown clay layer and overlying layers, \( h_L = 2.5 \) m.

Figure 5-5. Boundary conditions; (a) fluid sub-model; (b) mechanical sub-model
Mechanical boundary conditions have been specified as follows:

- Cyan line (Figure 5-5b) - free surface with total normal stress specified:
  \[ \mathbf{n} \cdot \mathbf{s} = -\rho g \cdot (h(t) - y) \quad \text{for} \quad y \leq h(t); \quad \mathbf{n} \cdot \mathbf{s} = 0 \quad \text{for} \quad y > h(t); \]

- Green line - roller condition: normal displacements and shear stresses are zero
  \[ U_x = 0, \quad \sigma_{xy} = 0; \]

- Blue line - fixed condition: zero displacements \( U = 0 \).

The overall solution procedure includes two loading stages:

1) Gravity settlement problem solution with stationary hydraulic boundary conditions (see section 5.4 for details). Gravity load and buoyancy load were applied incrementally to an initially stress-free domain. Mechanical behaviour of clay layers (brown soft clay and firm grey clay) was simulated as “drained” at this stage. Stresses obtained at this stage were used in the next stage to define pre-stresses in the domain.

2) Tidal mode simulation. Initial condition: pore pressures and stresses are obtained from the previous completed stage; displacements are zero. At each physical time step, a filtration problem was solved with a time-dependent FE solver; than the obtained pore pressures were passed to the mechanical sub-model as volume load. Increments of hydraulic loads were computed and gradually applied in the incremental parametric solver. Mechanical behaviour of clay layers was simulated as undrained at this stage; the undrained analysis was performed on the basis of the effective strength parameters (the procedure was described in section 2.4.2).

For each tide phase, strength reduction factors \( SRF \) were calculated according to the procedure described in paragraph 2.3.

A two-dimensional finite element mesh was composed of 6328 second-order triangular elements (Figure 5-6), this mesh was used for both fluid and mechanical analysis. High density of mesh is caused by using highly nonlinear Van Genuchten rheological model for variably saturated soil.

Figure 5-6. Boston levee model - finite element mesh

The functionality of the Comsol package supports multi-physics coupling – the equations describing the time-dependent fluid sub-model and the quasi-static mechanical
sub-model can be united in one system, which will be integrated by a time-dependent solver. However, convergence of the time-dependent solver was poor for the highly-nonlinear mechanical sub-model: integration in time-domain required very small time-steps and finally stopped at divergence of non-linear iterations. Typically plastic deformation problems in Comsol require a parametric solver for their solution. That’s why a two-step solution scheme described in Figure 3-2 was used.

5.3.2 Limit equilibrium model

The Geo-Stability software package has been used for the limit equilibrium analysis. This uses the well known Bishop method of slices (Bishop, 1955b) to calculate the factor of safety of the dike’s slope.

By assuming varying groundwater profiles within the dike for varying external water conditions, the module was able to provide a matrix of factors of safety for varying water levels (see Table 5-2 in the next section). This matrix of results relates external water levels and pore pressures (as measured in the sensors) with factor of safety. Additional values of factor of safety are then determined by an interpolation routine that finds intermediate values between those within the look-up table. Development of such stability matrices or look-up tables that could be interrogated using actual sensor values significantly simplified the process of using sensor information compared to the approach originally envisaged. Hence, the look-up table approach has been adopted within the study.

5.4 Finite element simulation results

First, fluid sub-model of the Virtual Dike FEM model has been calibrated using pore pressure sensors recordings. The river level dynamics within the training period considered for calibration is presented in Figure 5-7a. Hydraulic conductivity of the dark brown sand layer has been adjusted so that the amplitude and time lag of simulated pressure oscillations agree with those registered by the AC4 sensor (Figure 5-7b). The calibrated value of conductivity is given in Table 5-1. Calibration of the soft brown clay conductivity was not performed due to undrained condition of the layer. The calibrated Virtual Dike FEM model then was used for stability analysis.

The first stage of the stability analysis was gravity settlement simulation. Clay behaviour was simulated as drained at this stage. The boundary conditions were stationary: in dark brown sand and firm grey clay layers, both river level and land side ground water level were equal to \( h_L = 0.5 \text{ m} \) (value based on the AC4 pressure sensor readings); in soft brown clay layer and the overlying layers \( h_L = 2.5 \text{ m} \) (based on the AC2 and AC3 pressure measurements). Initial condition was specified as follows: vadose zones (made ground, fine sand): \( p = -5 \text{ kPa} \); saturated zones (soft brown clay with the underlying layers): hydrostatic pore pressure distribution according to water tables specified in the boundary conditions. During simulation period of 10 days pore pressures reached steady-state condition (Figure 5-8a). Gravity settlement of the dike under these hydraulic loads was 0.355 m (Figure 5-8b).
Figure 5-7. Dark brown sand layer conductivity calibration: (a) river level dynamics during training period; (b) comparison of real and simulated signals in AC4 sensor

(a) Pore pressure, Pa
(b) Total displacements, m

Figure 5-8. Gravity settlement simulation results; (a) pore pressure; (b) total displacements

Figure 5-9. Tidal mode simulation (a) pore pressure; (b) effective saturation; (c) effective plastic strains (displ. scaling factor 50); (d) total displacements: (displ. scaling factor 50)
Resulting distributions of pore pressure and stresses were then used as starting point for the subsequent pre-stressed tidal mode analysis. In the tidal mode, clay behaviour was simulated as undrained. River-side boundary conditions in the fluid and mechanical sub-models were non-stationary (see section 5.3.1 for their specification).

Figure 5-9a gives the pore pressure distributions at high tide and low tide phases. In the clay layer, it is only a local zone at the river slope that reacts to tidal oscillations. Maximal response to the tide is naturally observed in the dark brown sand layer. Effective saturation distribution changes very little with the tide and looks similar for the low tide and high tide phases (Figure 5-9c) - the thick layer of soft clay located between the levels of -2 m and +4 m does not really change saturation during relatively fast diurnal oscillations, hence the hydraulic load in clay does not change much from high tide to low tide.

Effective plastic strain distributions in the deformed domain are presented in Figure 5-9b for high tide and low tide phases. Effective plastic strain characterizes intensity of plastic strains and is calculated from the components of plastic strain rate tensor by formula:

\[
e_p = \int_0^t \dot{\varepsilon}_p \, dt, \quad \dot{\varepsilon}_p = \frac{\sqrt{2}}{3} \sqrt{(\dot{\varepsilon}_{pz} - \dot{\varepsilon}_{py})^2 + (\dot{\varepsilon}_{px} - \dot{\varepsilon}_{pz})^2 + (\dot{\varepsilon}_{py} - \dot{\varepsilon}_{pz})^2 + 6\dot{\varepsilon}_{pxy}^2 + 6\dot{\varepsilon}_{pz}^2 + 6\dot{\varepsilon}_{pyz}^2}
\]

Simulations have converged, indicating that the dike is stable under the tidal load. However, formation of a shallow slip surface located entirely in the soft clay layer can be seen at the low tide (Figure 5-9c).

Total displacements distributions are shown in Figure 5-9d. At high tide, total displacements are mostly produced by vertical flotation of the dike; at low tide, the slip surface formation is clearly identified at the river slope.

### 5.5 Limit equilibrium modelling results

Safety factors obtained in the Geo-Stability LEM analysis are presented in Table 5-2, in the form of a look-up table with variable river levels (RL) and ground water levels (GWL) within the dike. Columns AC1-AS2 show pore pressure values calculated at sensor locations. In the limit equilibrium analysis, the phreatic line shape was determined as a straight line connecting river and land side water levels. This simplification caused a significant difference in the distribution of pore pressures and soil weights in the dike, compared with the FEM modelling results, as the clay layer is less saturated in LEM for the most of the GWL conditions.

Figure 5-9c from the previous paragraph shows that the FEM is predicting at low tide an approximately circular failure surface, thus proving that use of the LEM is appropriate, though because of the simplifications required modelling for the latter different results were obtained from the two methods – see section 0.

The lowest value of the Factor of Safety FoS =1.04 has been obtained for the combination of the low tide condition RL = 0m with the highest ground water levels (GWL = 4m and GWL = 6m). This fact confirms the conclusion made from the FEM simulation.
results presented in the previous section: the less stable mode corresponds to the combination of maximal saturation of the dike with the low-tide water levels.

Slip surfaces (circles) obtained by Bishop’s method for the high tide phase (RL=4 m, GWL=0 m) and for the low tide phase (RL=-1.1 m, GWL=0 m) are shown in Figure 5-10b,d. The slip surfaces are shallow, with the high-tide slip circle having much larger radius than the low-tide circle. The shape of the low-tide slip surface agrees well with the real-life slippage observations.

Table 5-2. Stability factor values for various hydraulic conditions

<table>
<thead>
<tr>
<th>River level (RL)</th>
<th>Assumed ground water level (GWL)</th>
<th>Simulated relative pore pressure, in sensor locations, cross-section A (Atmos=0kPa)</th>
<th>Safety factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>[m]</td>
<td>[m]</td>
<td>[kPa]</td>
<td>[-]</td>
</tr>
<tr>
<td>-1.1</td>
<td>0</td>
<td>Atmos 8 39 63 Atmos</td>
<td>7.94 1.55</td>
</tr>
<tr>
<td>0</td>
<td>0</td>
<td>Atmos 8 39 63 Atmos</td>
<td>7.94 1.52</td>
</tr>
<tr>
<td>0</td>
<td>2</td>
<td>Atmos 3 28 59 83 2.94 27.94 1.28</td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>4</td>
<td>8 23 48 79 103 22 47 1.04</td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>6</td>
<td>28 43 68 99 123 22 47 1.04</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>Atmos 8 39 63 Atmos</td>
<td>7.94 1.67</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>Atmos 3 28 59 83 2.94 27.94 1.55</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>4</td>
<td>8 23 48 79 103 22 47 1.08</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>6</td>
<td>28 43 68 99 123 22 47 1.08</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>0</td>
<td>Atmos 8 39 63 Atmos</td>
<td>7.94 2.11</td>
</tr>
<tr>
<td>4</td>
<td>2</td>
<td>Atmos 3 28 59 83 2.94 27.94 2.11</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>8 23 48 79 103 22.94 47.94 1.88</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>6</td>
<td>28 43 68 99 123 22.94 47.94 1.60</td>
<td></td>
</tr>
</tbody>
</table>

5.6 Models cross-validation: comparison of FEM and LEM results

Within the UrbanFlood project, three independent models of the Boston levee were designed (Krzhizhanovskaya et al., 2012). These were: our Virtual Dike finite-element model, the limit equilibrium model from HR Wallingford and a finite-element model developed in Plaxis software by Siemens. Comparison of the results obtained by different tools for the low-tide and high-tide phases is presented in Table 5-3. Ground water level (GWL) was set at 2.5 m (based on sensor measurements). River level (RL) during high tide was +4 m above mean sea level; at low tide, RL was -1.1 m. For the cross-validation of LEM and FEM models, factors of safety for GWL=2.5 m have been obtained by interpolating between values in Table 5-2.
The values of strength reduction factors SRF obtained in Virtual Dike and in Plaxis by strength reduction method agree very well (the difference is 6% and 5% for high tide and low tide, respectively). The values of FoS obtained in Geo-Stability LEM program are much higher (by 22% at low tide and by 100% at high tide).

Table 5-3. Safety factors calculated by Virtual Dike, Plaxis and Geo-Stability

<table>
<thead>
<tr>
<th></th>
<th>High tide (RL=4 m, GWL=2.5 m)</th>
<th>Low tide (RL=-1.1 m, GWL=2.5 m)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Virtual Dike (FEM) SRF</strong></td>
<td>1.04</td>
<td>1.03</td>
</tr>
<tr>
<td><strong>Plaxis (FEM) SRF</strong></td>
<td>1.10</td>
<td>1.08</td>
</tr>
<tr>
<td><strong>Geo-Stability (LEM) FoS</strong></td>
<td>2.05</td>
<td>1.26</td>
</tr>
</tbody>
</table>

Values of SRF obtained by FEM do not significantly differ for the high tide (RL=4 m) and for the low tide (RL=-1.1 m). This is due to the fact that a thick layer of soft clay located between the levels of -2 m and +4 m OD at the river slope has low permeability and does not significantly change saturation during relatively fast diurnal oscillations (see Figure 5-9b). We believe that in reality the hydraulic load changes quite insignificantly with the tide due to the large amount of clay in the dike (this is confirmed by sensor readings in Figure 5-4b) and this effect has been reproduced in the more realistic FEM simulation. In the case of continuous rainfall infiltration, the stability factors will most likely decrease due to saturation of the top made ground layer and the increase in the weight of the dike, which can result in slope slippage.

In Geo-Stability LEM analysis, hydraulic loads differ significantly for the high tide and low tide, as the ground water table was assumed to be a straight line connecting river and landside water levels. Due to the same reason, the difference between high and low tide critical slip surfaces in Geo-Stability (Figure 5-10b,d) is much higher than in the Virtual Dike (Figure 5-10a,c).

In all models, the critical slip surface is shallow. For the high tide, it ends slightly above the river side toe of the dike, while for the low tide, the radius of the surface increases and slip surface ends below the river side toe. The radiuses of slip surfaces increase from high tide to low tide, both in Virtual Dike and in Geo-Stability programs. However, in the Virtual Dike FEM model the critical surfaces are located entirely within the soft clay layer whilst in the Geo-Stability LEM model they cross the fine sand and made-ground layers.

Distributions of total displacements obtained in Plaxis and in Virtual Dike qualitatively agree (see Figure 5-11). However the values of maximal displacements differ. The difference is due to the added value of the dynamical (time-dependent) simulations in the porous flow sub-model, compared to the static calculation in Plaxis. The other possible reason of this difference is use of different constitutive models of plasticity: classic Mohr-Coulomb plasticity model used in Plaxis via 2D Drucker-Prager approximation model in Virtual Dike.
Figure 5-10. Slip surfaces; high tide: (a) FEM (strength parameters scaled by $SRF=1.04$), (b) LEM; low tide: (c) FEM (strength parameters scaled by $SRF=1.03$), (d) LEM

Figure 5-11. Comparison of total displacements; high tide: (a) Plaxis, (b) Virtual Dike; low tide: (c) Plaxis, (d) Virtual Dike
5.7 Conclusions

The first scientific question raised in this research was check of a principal ability of an earthen dike computational model to adequately simulate complex physical processes occurring at failure and correctly predict failure under prescribed loads. The Boston levee validation test site has become the next step towards the positive answer to this question.

The Boston dike analysis included cross-validation of the Virtual Dike module against two other models: a FEM model built in Plaxis software and a LEM model analyzed Geo-Stability software package. Two FEM models (Plaxis and Virtual Dike) have produced very close results, particularly, close values of strength reduction factors well (the difference is 6% and 5% for high tide and low tide, respectively). LEM analysis was a bit less precise: a typical LEM assumption on the hydrostatic distribution of pore pressures in the dike has become critical for the clayey Boston levee – the capillary fringe is very high in the dike and water storing effects must be taken into consideration (like it was done in Virtual Dike and Plaxis models).

Piezometers readings show that the massive soft brown clay layer stays fully saturated during tidal cycles and its condition is undrained. According to the stability analysis carried by finite element method and by Bishop’s limit equilibrium method, slope failure occurs with the development of an approximately circular slip surface located in the soft brown clay layer. Both methods, LEM and FEM, confirm that the least stable hydraulic condition is the combination of the minimum river levels at low tide with the maximum saturation of soil layers. The factors of safety calculated by Bishop’s limit equilibrium method are significantly higher than strength reduction factors calculated by FEM (by 22-100 %). In our case, the discrepancy between LEM and FEM results is predominantly due to the differences in calculation of hydraulic loads in the dike from tidal oscillations.

Virtual Dike results indicate that in real-life winter and spring conditions, the dike is almost at its limit state, at the margin of safety (strength reduction factor values are 1.03 and 1.04 for the low-tide and high-tide phases, respectively). These results agree well with the real-life observations, showing occasional slope failures at high tidal range.
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., Krzhizhanovskaya, 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 overtopping’, 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 IJkDijk 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.
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.** Excavation 1: 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)
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: \( c' = 9.7 \text{KPa}, \phi = 27.5 \text{ degrees} \)
- Clay \( c' = 4.6 \text{KPa}, \phi = 29.4 \text{ degrees} \)

(a) clay

(b) peat

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

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.

\begin{align*}
\text{CPT 35} & \quad Cu, \text{ KPa} & \text{CPT 45} & \quad Cu, \text{ KPa} \\
\text{depth, m} & \quad 0 & \quad 10 & \quad 20 & \quad 30 & \quad 40 & \quad 50 & \quad 0 & \quad 10 & \quad 20 & \quad 30 & \quad 40 & \quad 50
\end{align*}

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 \( E_{50} \) (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.

\begin{align*}
\text{Clay} & \quad Cu, \text{ KPa} & \quad E_{50}, \text{ MPa} \\
\gamma = 0.549 \text{b} & \quad \gamma = 0.079 \text{b}
\end{align*}

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

Table 6-1. Soil parameters

<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 set.)</td>
<td>0.16 (gravity set.)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>11.6 (reloading)</td>
<td>1.6  (reloading)</td>
<td></td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3 (gravity set.)</td>
<td>0.49 (reloading)</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 6-7a 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>
<thead>
<tr>
<th>Table 6-2. Elevation coordinates of Geobeads sensors</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>GeoBeads sensor</strong></td>
</tr>
<tr>
<td>Cross-section A</td>
</tr>
<tr>
<td>GB-AG-1 (in the sand core)</td>
</tr>
<tr>
<td>GB-AG-2</td>
</tr>
<tr>
<td>GB-AG-3</td>
</tr>
<tr>
<td>GB-AG-4</td>
</tr>
<tr>
<td>Cross-section B</td>
</tr>
<tr>
<td>GB-AG-5 (in the sand core)</td>
</tr>
<tr>
<td>GB-AG-6</td>
</tr>
<tr>
<td>GB-AG-7</td>
</tr>
<tr>
<td>GB-AG-8</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.

![Geotextile sensors - strains](image)

**Figure 6-11. Longitudinal strains, micrometers per meter**

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.
Chapter 7  Artificial intelligence system training\textsuperscript{6}

One of the main objectives of this research was development of a numerical dike model able to reproduce complex behaviour of real dikes, particularly under failure conditions. Validation cases analyses presented in chapters 4-6 have proved that the module realistically simulates sensor dynamics in normal modes (for Livedike, Boston dike) and at failure (the IJkJDijk experiment). Reaching this high level of realism and accuracy allowed us to implement an innovative hybrid approach of combining FE modelling with artificial intelligence (AI) analysis, so that simulated virtual sensors data was used as training sets for the AI system. This hybrid method has been proposed and implemented for the UrbanFlood EWS, in cooperation with Alexander Pyayt, a researcher and developer of the AI module (Pyayt et al., 2011a) and successfully tested on a full-scale Livedike prototype model, for a simulated strong storm with very high water level. The artificial intelligence module detected the onset of dike instability after being trained on the data from the Virtual Dike finite element simulation.

7.1 Comparison of numerical approaches to dike failure prediction: data-driven analysis, model-based prediction, hybrid method

A data-driven approach to dike failure prediction is bounded by various numerical methods of processing raw sensor data registered by dike monitoring systems. Data processing can be real-time or pre-computed. The data-driven methods include machine learning methods (neural networks), statistical methods (central moments, linear correlation, clustering), soft computing and others (see (Solomatine and Ostfeld, 2008), (Jaksa et al., 2008) for an overview of data-driven methods). Implementations of this approach for various cases can be found in (Baars, 2005), (Noortwijk et al., 1999), (Khan et al., 2010).

A model-based approach (in geotechnics, it’s typically represented by limit equilibrium methods and by finite element analysis) requires information about physical properties of a monitored object. A model-based approach based on finite element analysis of dike stability has been thoroughly described all over this thesis. The constructed finite element models are independent of on-line measurements, while sensor measurements (e.g., water level and pore pressure) define external loadings and boundary conditions for the computational model. During construction of the model, historical sensor data are used for calibration of soil parameters and overall validation of the model. In a model-based approach, high risk of failure is detected if factors of safety computed are close to 1 (or lower than 1).

Data-driven methods are highly dependable on availability of sensor data, both for normal conditions and for abnormal (failure) modes. If such data are not available for a

\textsuperscript{6} Parts of this chapter have been published in (Pyayt et al., 2011a)
particular dike, a model-based approach can simulate sensor response to use as input for the AI training, including pattern recognition and classification tasks.

The idea of combining FE with AI brought us to an innovative hybrid approach, which had a successful numerical validation on a prototype model of the LiveDike.

### 7.2 Virtual Dike Simulation Results

For testing the proposed hybrid AI+FEM model approach, we designed a prototype dike similar to the LiveDike, a sea dike protecting in Groningen (see Chapter 4). The geometric configuration and boundary conditions of the prototype model are similar to those of the LiveDike, while material properties have been artificially weakened to obtain a more pronounced plastic zone under the condition of a simulated flood.

A prototype model is composed of homogeneous sand. A two-dimensional plane stress structural problem is solved on second-order triangular finite elements. The problem size is 29788 degrees of freedom. Boundary conditions are: (a) roller constraint on the dike’s base (allowing x-movement only) and roller constraints on the vertical cuts in the sand (allowing y-movements); (b) water pressure acting both on the seaside and landside. The landside is subjected to water pressure because of the channel behind the dike.

In order to generate a training set for the AI component, an abnormal behaviour of the dike has been simulated. Flood condition has been modelled by linearly increasing the water level from a Mean Sea Level (MSL) to the artificially extremely high level of +6.6 m above the MSL. The top of the dike is at 9 m.

Time dynamics of stresses in the dike are non-linear due to the plastic deformation. This nonlinearity is illustrated in Figure 7-1a, which shows principal stresses acting in cross-section plane for six "virtual sensors" located at the land-side slope of the dike (Figure 7-1b).

Plastic yield function defined by formula (2.9) has naturally been considered as a scalar stability criterion: the function indicates occurrence of plastic yielding in a material point, and plastic yielding is a local instability. When large areas of the dike are captured by plastic yielding, a global instability and failure of the dike occurs.

Stability criterion dynamics during the flooding is shown in Figure 7-1c. The criterion values become negative when plastic deformations occur. Similar to curves shown in Figure 7-1a, a change of trend in stability criterion dynamics occurs with intensive plastic deformations development.

A distribution of stability criterion in the dike is presented in Figure 7-2 for two flood phases (namely, for water levels 0 m and 6.6 m above NAP). Pore pressures in the dike grow with the height of water level, which reduces the magnitude of effective normal stresses, compressing the sand. Zones of plastic yielding grow as water level increases. Plastic zones are shown as dark red areas in Fig. 9. When water level reaches 6.6 m above the reference level, the stiffness matrix becomes singular. This means that a significant part of the dike has yielded to plastic deformation, which may indicate the onset of a failure process.
Figure 7-1. Simulated dynamics for six "virtual" sensors measuring principal stresses and simulated dynamics of stability criterion: (a) signals of six virtual sensors; (b) locations of the virtual sensors; (c) stability criterion dynamics

Figure 7-2. Stability criterion distribution, for different load steps: (a) zero water level; (b) critical state, water is at 6.6 m above reference level. Plastic zones are shown with dark red

7.3 Detection of Artificially Generated Anomaly by the Artificial Intelligence Component

The first principal strain and X deformation measured by the virtual sensor located at point (X=55 m, Y=-4.2 m) were used as input parameters for the one-side classifier by Neural Clouds (Lang et al., 2008). Figure 7-3 shows input variables and time dynamics of the confidence value. Vertical red line in Figure 7-3 shows the moment when confidence value went down from the values close to 1 (normal behaviour) to zero (detected anomaly).
Figure 7-3 demonstrates the ability of Neural Clouds to detect anomaly: confidence value went down to zero at the moment when the stability criterion changed the slope angle. The dike failure can occur when the stability criterion becomes zero and lower. In Fig. 12 it happens around the time step 1100, which means that the AI detected the onset of forthcoming dike instability over 600 time steps earlier (Pyayt et al., 2011a).

![Figure 7-3. Detection of anomaly using the Neural Clouds (NC) approach: (a) - training data set (first principal strain and X-axis deformation) and calculated confidence values. Blue lines indicate the training period (time steps 1-460), black lines indicate the testing period (times steps 461 and further); (b) Stability criterion calculated by the Virtual Dike finite element model (Pyayt et al., 2011a)](image)

7.4 Conclusions

An innovative hybrid approach for the assessment of monitored dikes’ stability has been proposed and implemented in the present research, in cooperation with the AI system development team of the UrbanFlood project (Pyayt et al., 2011). The approach combines the finite element modelling with the artificial intelligence methods for real-time signal processing and anomaly detection. This combined method has been developed for the UrbanFlood early warning system and tested on a numerical model of a large-scale sea dike, under strong storm conditions, with very high water level.

In the Virtual Dike module, plastic deformation of a sand dike subjected to flood loading has been simulated. Stability of the dike has been analysed and a critical water level detected as a margin of dike stability.

After training, the artificial intelligence module successfully detected the onset of dike instability. The AI module showed a very sharp drop of the dike safety confidence
value from 1 (normal behaviour) to zero (anomaly). After this anomaly detection, it took another 640 time steps (about 10 hours) to develop real dike instability as evaluated by the stability criterion calculated in the Virtual Dike model.

A natural direction for further research on hybrid modelling would be training AI on a FE simulation of a real experimental dike collapse (for example, the IJkDijk South Levee slope failure simulation), with subsequent validation of the AI failure prediction against the actual observations.
Chapter 8 Conclusions and directions for further research

8.1 Conclusions

In this thesis, a problem of employing finite element analysis of earthen dikes stability in a functional workflow of an early warning system for flood protection was studied. As it was mentioned in Chapter 1, a detailed physical-model based approach has not been used in early warning systems before, because it has been traditionally considered as computationally heavy. For the first time in dike monitoring practice, we have integrated a finite element module called Virtual Dike into the UrbanFlood EWS and studied the benefits of such integration, including outcome from Virtual Dike solo work and from its interaction with the artificial intelligence (AI) module. Virtual Dike has become the first finite element module for dike modelling that works with live sensor data, produces real-time alarms and interacts with other components of the EWS, according to the workflow described in Chapter 1.

All scientific objectives posed in the research (Chapter 1) have been accomplished. Below we discuss separately the progress on each objective.

At the early stage of this research, we have collected, tested and compared existing mathematical models for earthen dikes analysis, including mathematical models of filtration through porous media and soil mechanics (Chapter 2). After a series of numerical tests, we selected the two-dimensional model with linear elastic perfectly plastic associated flow rule defined by the Drucker-Prager yield function. This choice provided the optimal balance between realism and adequacy of the computational model, on the one hand, and high speed of numerical convergence in real-time, on the other hand.

Finite element method in application to soil mechanics theory has been chosen for the Virtual Dike implementation, as the most powerful computational technique suitable for simulation of arbitrary failure mechanisms in dikes on a macroscopic level. The module has been implemented in Comsol finite element package and ported on a computational core of supercomputing cloud SARA of the University of Amsterdam. Efficiency benchmarks performed on SARA let us choose optimal settings for the package in order to minimize computational times (Chapter 3).

The first scientific question raised in this thesis was check of a principal ability of an earthen dike computational model to adequately simulate complex physical processes occurring at failure and to predict failure under prescribed loads. Given a huge amount of sensor data recorded for the experimental dikes under documented loading regimes, we had a unique opportunity for validation of our computational model. Validation analyses of the module included one absolutely “healthy” dike in Groningen (the Livedike), the Boston levee prone to occasional slope failures at high tidal range at spring, and one full-scale experimental dike intentionally brought to slope failure during the IJkDIjk-2012 experimental series (Chapter 4-Chapter 6).
The Boston levee analysis (Chapter 5) included cross-validation of the *Virtual Dike* against two other models: a FEM model built in finite element software package Plaxis (commercial software for soil mechanics modelling) and a LEM model based on the established Bishop’s method. Two FEM models (Plaxis and *Virtual Dike*) have produced very close results, particularly, close values of strength reduction factors. LEM analysis was a bit less precise: a typical LEM assumption on the hydrostatic distribution of pore pressures in the dike became critical for the correct assessment of the clayey Boston levee stability: a capillary fringe in the dike is very high and water storing effects must be taken into consideration (and they were accounted in FEM models).

The IJkDijk slope failure experiment in Bad Nieuweschans, the Netherlands, (Chapter 6) 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. The *Virtual Dike* model precisely predicted the simulation mode (slope sliding) and the collapse stage in the loading sequence. 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 training. This innovative hybrid method of combining artificial intelligence analysis with FEM-based modelling was proposed and implemented by us in co-operation with our colleague form the UrbanFlood project (Pyayt et al., 2011). The objective for using the hybrid approach lies in a rather typical situation — lack of failure sensor data for “healthy” levees.

The hybrid method was successfully tested on a numerical model of a Livedike prototype with weakened soil strength, under strong storm condition with very high water level (Chapter 7). The FEM module generated time series for the first principal strain and horizontal displacement, which were fed into the AI module as training sets. After training, the artificial intelligence module successfully detected the onset of dike instability. The AI module showed a very sharp drop of the dike safety confidence value from normal behaviour to anomaly. After the anomaly detection, it took about 10 hours to develop real dike instability as evaluated by the stability criterion calculated in the *Virtual Dike* model.

Hybrid approach testing (Chapter 7) allowed us to accomplish the second scientific objective and gave a positive answer to the question of feasibility of the hybrid approach.

The third scientific question posed in the research was estimation of uncertainties influence on the dike stability assessment; it was important to find out how variations in soil properties alter dike safety margin.

We have focused on the influence of variation in soil diffusivity on the pore pressure distribution. As we have shown in Chapter 5, correct calculation of pore pressures in the
dike is critical for the appropriate assessment of dike safety margin. Sensitivity analysis has shown that for coarse media (gravel, coarse sand), distribution of pore pressure amplitudes within a model dike is close to linear and it is defined by water levels at the boundaries (water levels are obtained directly from sensor readings), while diffusivity value does not affect this distribution. For dense soils (fine sands, clays), pressure amplitudes distribution is highly non-linear and to a large extent depends on the diffusivity value. Time lag between local oscillations and tide is entirely determined by diffusivity, both for coarse and fine media. On the base of these conclusions, we have constructed and implemented a new automatic procedure for calibration of soil diffusivities in an arbitrary heterogeneous dike based on historical pore pressure sensors recordings (Chapter 4). The procedure has been successfully tested for the LiveDike diffusivities calibration: simulation results with calibrated soil parameters match experimental data, not only on the "training set" but also for a much longer period of time. The calibration procedure employs the analytical solution obtained by us for the problem of tidal propagation in a one-dimensional bounded aquifer.

8.2 Directions for further research

Our future plans include an extensive study of our hybrid approach to dike health monitoring: training AI on a FE simulation of a real experimental dike collapse (for example, the IJkDijk 2012 South Levee failure), with subsequent validation of the AI failure prediction against the actual observations.

An extremely interesting direction for the future research is combining macro- and micro-scale models. In connection to dike stability analysis, this approach offers a deep insight into mathematical modelling of erosion processes and related failure mechanisms (e.g., piping, wave erosion). For piping modelling, it looks reasonable to couple our porous flow - soil deformations model with the pore-scale erosion model; the latter would assess variation of macroscopic soil parameters (strength, stiffness, density) with the particles transfer. For wave erosion simulation, a CFD (computational fluid dynamics) model would represent the macro-level, with water-soil erosion at the pore-scale level.

The problem of designing a dike stability analysis module for early warning systems is worth developing an own program code. We have programmed the module using MATLAB scripts and Comsol FE package; however, implementing entirely original program code would allow us to:

(a) Reduce memory demand and increase computational speed – and hence, consider fully-coupled and three-dimensional dike models in real-time mode;
(b) Use tracing and profiling tools for Virtual Dike efficiency enhancement;
(c) Increase parallel efficiency of the module (in the present implementation, it was reasonable to use not more than 2-4 cores; for more than 4 cores, synchronisation costs were too high (Chapter 3))

Another interesting issue relates to the aspects of long exploitation of the Virtual Dike module within the EWS. In the UrbanFlood project, we have fed the Virtual Dike module with sensor data recorded during weeks or months. If the module functions for...
years or even decades, perhaps there will be a need in special treatment of simulation errors accumulation, as well as in organizing file handling procedures to clean disk storage from multiply input and output data files generated in the working process.
Chapter 9 Nederlandse Samenvatting

In deze dissertatie wordt het probleem bestudeerd van het toepassen van eindige-elementanalyse (EEA) van dijkstabiliteit in een functionele workflow van een waarschuwingsstelsel voor overstromingen (‘early warning system’, voorts EWS). Voor de eerste keer in de praktijk van dijkmonitoring hebben wij de eindige-elementenmodule Virtual Dike (‘virtuele dijk’) geïntegreerd in het UrbanFlood EWS systeem en de voordelen van zo’n integratie bestudeerd, waaronder de uitkomst van het Virtual Dike solowerk en de uitkomst van diens interactie met de kunstmatige intelligentie (AI) module. Virtual Dike is de eerste eindige-elementenmodule geworden voor dijkmonitoring die werkt met rechtstreekse sensordata, die real-time alarmen bewerkstelligt en die interacteren met andere EWS componenten, volgens de EWS workflow.

Het eerste wetenschappelijke vraagstuk van deze dissertatie was het principiële vermogen van een rekenkundig dijkmodel om adequaat de complexe fysieke processen te simuleren die optreden bij bezwijken en om bezwijken correct te voorspellen bij voorgeschenen belastingen. Gezien de enorme hoeveelheid sensordata die waren geregistreerd voor de gedocumenteerde belastinggevallen, hadden wij een unieke mogelijkheid voor validatie van ons rekenmodel. Er zijn drie succesvolle validatieanalyses gedaan van ons model voor echte dijken: een stabiele dijk in Groningen (de Livedike), de rivierdijk in Boston die gevoelig is voor incidenteel bezwijken van de helling bij een grote getijdevariatie tijdens springtij, en een experimentele dijk op ware grootte die met opzet tot bezwijken werd gebracht tijdens de reeks IJKDijk-2012 experimenten. Het IJKDijk-experiment van hellingsbezwijking in Bad Nieuweschans in Nederland is de ultieme validatie van de Virtual Dike module geworden en tevens een winnaar van de speciale wedstrijd voor de beste bezwijkingspredictie die kort voor de test in het leven werd geroepen (http://ijkdijk.rpi.edu). Verscheidene commerciële bedrijven en wetenschappelijke organisaties die dijksystemen modelleren namen deel aan de competitie; ons rekenmodel gaf de beste klasse A-voorspelling voor het macro-instabiliteitsexperiment in de zuidelijke dijk, volgens de jury (http://www.urbanflood.eu; Koelewijn, 2012). Het Virtual Dike model voorspelde exact de simulatiemodus (hellingsverzakking) en de ineenstortingsfase in de belastingreeks. Vergelijking a posteriori van kantelingen gemeten met Geobeads-sensoren en gesimuleerde verplaatsing heeft uitgewezen dat het rekenmodel de sensordynamiek op realistische wijze reproduceert, inclusief de responsie van sensoren ten gevolge van uitgraving, containerbelasting en pompen.

Succesvolle validatie van onze module op alle drie de testlocaties, in het bijzonder op de zuidelijke dijk van het IJKDijk bezwijkingsexperiment, hebben laten zien dat ons rekenmodel in staat is om virtuele-sensorpatronen van abnormaal en normaal gedrag voor het kunstmatige-intelligentiesysteem te genereren. Deze innovatieve, hybride methode om kunstmatige-intelligentieanalyse met EEA-modeleren te combineren is door ons voorgesteld en geïmplementeerd met onze collega van het UrbanFlood-project. Het doel van de hybride benadering ligt in een typische situatie van gebrek aan sensordata van de bezwiking voor gezonde dijken. De hybride methode is succesvol getest op een numeriek model van een Livedike prototype met verzwakte bodemsterkte onder sterke stormcondities met een zeer hoog waterniveau. In de Virtual Dike module is plastische vervorming van een zanddijk onder overstromingsbelasting gesimuleerd. De dijkstabiliteit is geanalyseerd en een kritisch waterniveau werd gevonden. De EEA module genereerde tijdreeksen voor de
eerste hoofdspanning en de horizontale verplaatsing, deze werden doorgevoerd naar de AI module als trainingsets. Na de training detecteerde de kunstmatige-intelligentiemodule (AI) de start van dijkinstabiliteit met succes. De AI module liet een zeer sterke daling zien van de betrouwbaarheidswaarde van de dijkveiligheid van normaal gedrag naar anomalie. Na de detectie van de anomalie duurde het tien uur voordat de dijkinstabiliteit zich daadwerkelijk ontwikkelde, zoals geëvalueerd met het stabiliteitscriterium berekend in de Virtual Dike module.

Testen met de hybride benadering stelde ons in staat om de tweede wetenschappelijke doelstelling te verwezenlijken en gaf ons een positief antwoord op de wetenschappelijk vraag die letterlijk luidde “Kan een rekenmodel van een dijk realistische dijkbezwijkingspatronen produceren voor de training van een kunstmatige-intelligentiesysteem (AI), zodat de AI vervolgens een correcte en tijdige detectie kan maken van een bezwijking van een echte dijk?”

De derde wetenschappelijke vraag gesteld in het onderzoek was de schatting van onzekerheden die van invloed zijn op de evaluatie van dijkstabiliteit; het was belangrijk om te achterhalen hoe variaties in de bodemeigenschappen de dijkveiligheidsmarge veranderen. We hebben de invloed van variatie in bodemdiffusie op poriedrukverdeling onderzocht. Gevoeligheidsonderzoek heeft laten zien dat voor grove grond (grind, grof zand), de verdeling van poriedrukamplitudes in een dijk bijna lineair is en vastligt met de randvoorwaarden op de eindpunten, terwijl de diffusiewaarde deze verdeling niet beïnvloedt. Voor fijne grondsoorten (fijn zand, klei) is de drukamplitudeverdeling sterk non-lineair en hangt deze grotendeels af van de diffusiewaarde. Op basis van deze conclusies hebben we een nieuwe automatische procedure opgezet en geïmplementeerd voor kalibratie van bodemdiffusies in een arbitraire heterogene dijk, gebaseerd op historische sensoropnamen van de poriedruk. De procedure is met succes getest voor de Livedike kalibratie van diffusies: resultaten van simulaties met gekalibreerde bodemparameters komen overeen met experimentele data, niet alleen voor de trainingset maar ook voor een veel langere tijdsperiode. De kalibratieprocedure gebruikt een analytische oplossing die door ons is afgeleid voor het vraagstuk van getijvoortplanting in een eendimensionale, eindige, homogene watervoerende laag.

Zodoende zijn alle wetenschappelijke doelstellingen gerealiseerd. Voor de eerste keer in de praktijk van het ontwerpen van monitoringsystemen is een benadering met gedetailleerde fysische simulaties voor dijkstabiliteitsanalyse succesvol geïntegreerd in de workflow van een waarschuwingssysteem (EWS).
Chapter 10  Acknowledgements

First of all, I would like to express my deepest gratitude to my promoter, Peter Sloot - a bright scientist and an enthusiastic optimist, for giving me the opportunity to extend my horizons and for introducing me scientific life at large. I am very grateful to my Russian supervisor, Valeria Krzhizhanovskaya, for inviting me to participate in the international research project, guiding me through the research field and helping me use writing and presenting to make a strong case. I would like to thank the Ph.D. committee members: Alexander Boukhanovski, Marian Bubak, Cees de Laat, Robert Meijer and Nikolay Shabrov, for their efforts in assessing this thesis and for their helpful comments.

I express my deep gratitude to David Jordan (HR Wallingford), a co-author of my paper on the IJkDijk breach experiment, for very fruitful cooperation in the UrbanFlood project. During preparation for the IJkDijk South levee prediction contest, David has clearly demonstrated English principles of fair play by sharing his soil assessment results with us. Our prediction of the IJkDijk slope failure (which has become a winner of the contest) employed soil parameters derived by David from the field and lab tests results provided to the contestents.

I would like to thank my colleagues who were “in the same boat” with me working on their theses in the UrbanFlood project and in ITMO: Alexander Pyayt and Christiaan Erdbrink, for their help in my daily work and for sharing efforts in planning and organizing the defence ceremony. I separately thank Alexander Pyayt for our productive co-operative work on developing and testing the hybrid approach employing artificial intelligence analysis in junction with finite element modelling. Together we have carried the research described in Chapter 7 and published a conference paper on this topic. 

I sincerely thank my colleagues within the UrbanFlood project who helped me at different stages of my work: Andre Koelewijn (Deltares) – for supplying me with lots of valuable data on the monitored sites and for his expert-level consultations; Marek Kasztelnik, Tomasz Bartyński (Cyfronet) – for their work on wrapping the Virtual Dike module at the stage of integration into the common information space; Jeroen Broekhuijsen – for providing connection to live sensor data for the whole EWS components, including Virtual Dike as well; Gleb Shirshov – for his assistance in porting the module to computer cloud Sara.

I’d like to thank Bernhard Lang and Denis Shevchenko (Siemens) for their cooperation on a cross-validation analysis between Virtual Dike and Plaxis, performed for the Boston levee site. Many thanks go to Denis Shevchenko, for his efforts in the cross-validation study; some of his results from the open-access UrbanFlood deliverables (Krzhizhanovskaya et al., 2012) were briefly presented in Chapter 5.

I express my gratitude to Alexander Boukhanovski (eScience Research Institute of NRU ITMO) for giving me the opportunity to work in one of the most advanced supercomputer labs of Russia and for a strong financial support of my work. His brilliant erudition in the field of applied mathematics and intelligent computational technologies is my great admiration.
And I would also like to thank my chief at the Polytechnic University (Saint-Petersburg), Nikolay Shabrov, for his attention to my research work, for his empathy and comprehension when scheduling my work in Polytech during past four years.

I thank my friends (Elena, Lubov’, Alexander) for their constant emotional support and encourage.

My biggest thanks go to my family, particularly to my son Vladimir, whose love, openness and thirst for knowledge ever inspire me so much.

Let me also mention that history and culture of the Netherlands, with its heroic experience of conquering land from the sea and beautiful paintings of Dutch landscapes, have been my great admiration and a source of real inspiration for me.
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