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Seepage in a flood protection levee – first geotechnical centrifuge test results

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Review Article

Keywords
Centrifuge modelling of a flood protection levee
Transient seepage flow
Numerical modelling of transient seepage flow

Abstract

Many flood protection levees in Europe were built more than 100 years ago. These levees often do not meet the current flood protection requirements due to increased level of safety requirements, higher damage potential in the valley plains and due to higher peak discharges or water levels expected with changing climatic conditions. A first series of centrifuge tests on two idealized cross-sections of the river Rhine flood protection levee have been carried out in the geotechnical centrifuge at Delft University of Technology in order to study the transient seepage behaviour of a horizontally layered levee consisting of layers with coarse and fine-grained material. Main features and design considerations of a specially manufactured flood simulator for the geotechnical centrifuge which allows replicating scaled flood events with predefined durations and intensities are presented. Furthermore, measured values of the pore pressure during the investigated flood event are reported and discussed in comparison to the results of finite element modelling of the levee. Finally, the potential impacts of the hydraulic boundary conditions on the seepage behaviour in the physical and numerical models of the levee are critically discussed.

1. Introduction

Many existing flood protection levees no longer satisfy current safety standards, particularly in the context of climate change. The increasing frequency and intensity of heavy precipitation, together with the rising zero-degree isotherm in alpine regions, leads to more severe flood events characterized by higher water levels (NCCS, 2021). This is illustrated by the example of the flood protection levees along the Alpine Rhine, which are currently being restored in sections. The original straightening and damming of the Rhine started in the 19th century (Zur Rhein-Hochwasserkatastrophe, 1927) and no longer fulfils current flood protection requirements (Tschanz, 2018).

A conservative engineering design approach assumes steady-state seepage flow through the levee and is often considered when investigating levee stability or developing measures against stability problems (CIRIA, 2013). However, it seems reasonable to consider transient seepage conditions, as flood events in alpine regions often only have a duration of 12 to 48 h (Aschwanden & Schädler, 1988). This could offer a potential for cost optimisation in project planning of remediation measures. Research results on a model river-embankment consisting of silty-sandy material in a geotechnical centrifuge by Giretti et al. (2022) show that the saturation front in the levee reaches the air-side of the levee only after an unrealistic long flood period of more than 100 days. Mayor (2013) reports similar results in his large-scale tests at the river Rhône levee consisting of silty-sandy material. Even an eleven-day long flood was not enough for the saturation front to reach the air-side of the levee. With further tests, however, Mayor showed that the time until the saturation front reaches the air-side of the levee depends directly on its permeability and its initial degree of saturation.

However, if transient flow conditions are to be considered in the design, a more detailed under-standing of the evolution of pore water pressure in the levee body is required to ensure safe design. Physical modelling under transient flow conditions can provide valuable data for better interpretation and potentially validation of numerical calculation methods.

A model-levee is used because the construction of a 1:1 scale model levee is complex and costly (Mayor, 2013; Toromanovic et al., 2020). A geotechnical centrifuge is used to increase the acceleration field and thus the effective stresses in the model to ensure comparable results of the model tests with the prototype (Madabhushi, 2014). A geometrically N-times smaller levee is subjected to an N-times larger g-level as the effective stresses are generally accepted to be linearly correlated with depth (Wood, 2004; Askarinejad et al., 2015).

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Another advantage of centrifuge modelling is the scaling of time for seepage processes. Assuming the intrinsic permeability does not differ between model and prototype and that water is used as the saturating fluid in the model, it can be shown that the seepage velocity v in a centrifuge model (index m) is N-times faster than in the prototype (index p) (Thusyanthan & Madabhushi, 2003):

$$v_m = N v_p \tag{1}$$

Consequently, it can be shown that the seepage process is N^2-times faster in the centrifuge model (Garnier et al., 2007) compared to the prototype, as the length of the seepage path l in the model is N-times smaller:

$$t_m = \frac{l_m}{v_m} = \frac{\frac{1}{N} l_p}{N v_p} = \frac{1}{N^2} t_p \tag{2}$$

This means that at a g-level of $N = 100$ a prototype flood with a duration of $t_p = 72$ hours is modelled within $t_m = 25.9$ seconds.

2. Method

In this study, the investigation of flow behaviour within a layered flood levee is primarily conducted through physical centrifuge modelling. Subsequently, numerical modelling was performed for one cross-section - with the same level of details and complexity similar to common engineering practice - to complement the experimental findings through a comparative assessment.

2.1 Centrifuge modelling

The experiments were conducted at 100 g in the geotechnical centrifuge at the Delft University of Technology which has a radius of 1.22 m and a basket size of 400x500x500 mm (Allersma, 1994; Zhang & Askarinejad, 2021). A flood simulator was designed and constructed for this purpose. A flood wave could be generated inflight with the help of a water tank and servo coupled valves. In addition, a constant water level was maintained at the downstream side of the levee. The flood simulator has outer dimensions of LxHxW = 615x200x120 mm, with 15 mm thick aluminium panels and a transparent side wall made of 15 mm thick acrylic glass. Figure 1 shows a model levee installed in the flood simulator in the centrifuge basket. The water tank for the flood wave (1.6 litres) is situated on top of the flood simulator.

The model levee was constructed according to the prototype of two typical cross-sections of the river Rhine levee (Figure 2). On top of the permeable Rhine gravel lies a less permeable layer of flood deposits on which the gravelly levees were built (cross-section A as given in prototype scale in Figure 2). Field investigations revealed that the less permeable layers of flood deposits below the levee do not always extend to the water side. Therefore, a second cross-section B was investigated in which this layer only extends to approximately below the crest of the levee (see Figure 2). In the following, cross-section A is referred to as levee A and cross-section B as levee B.

The model levees were constructed at a scale of 1:100 as shown for levee A in Figure 3. The crest of the levee is 80 mm higher than the air-side terrain and has a base width of approximately 350 mm. It is important that the underlying permeable Rhine gravel is modelled as well since it may influence the seepage behaviour. Geotechnical properties of the materials used in the model are summarised in Table 1,

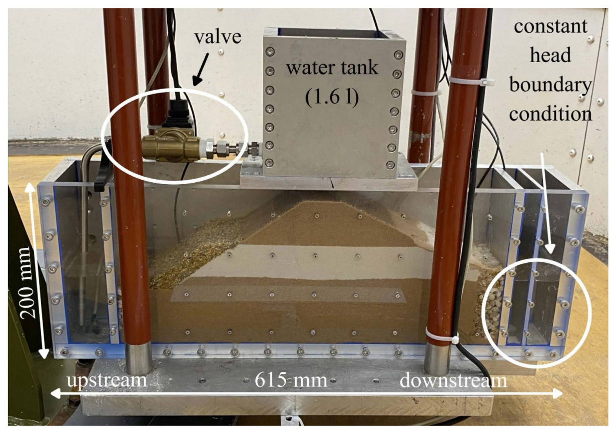

Figure 1. Model levee A inside the flood simulator.

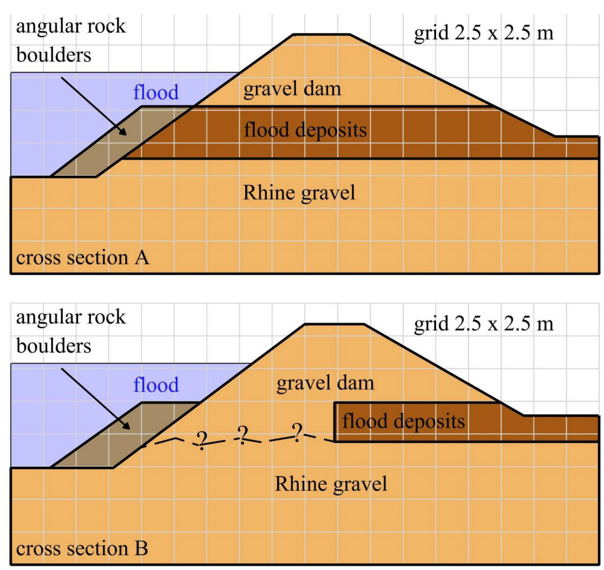

Figure 2. Typical cross sections (A & B) of the river Rhine levee in prototype scale.

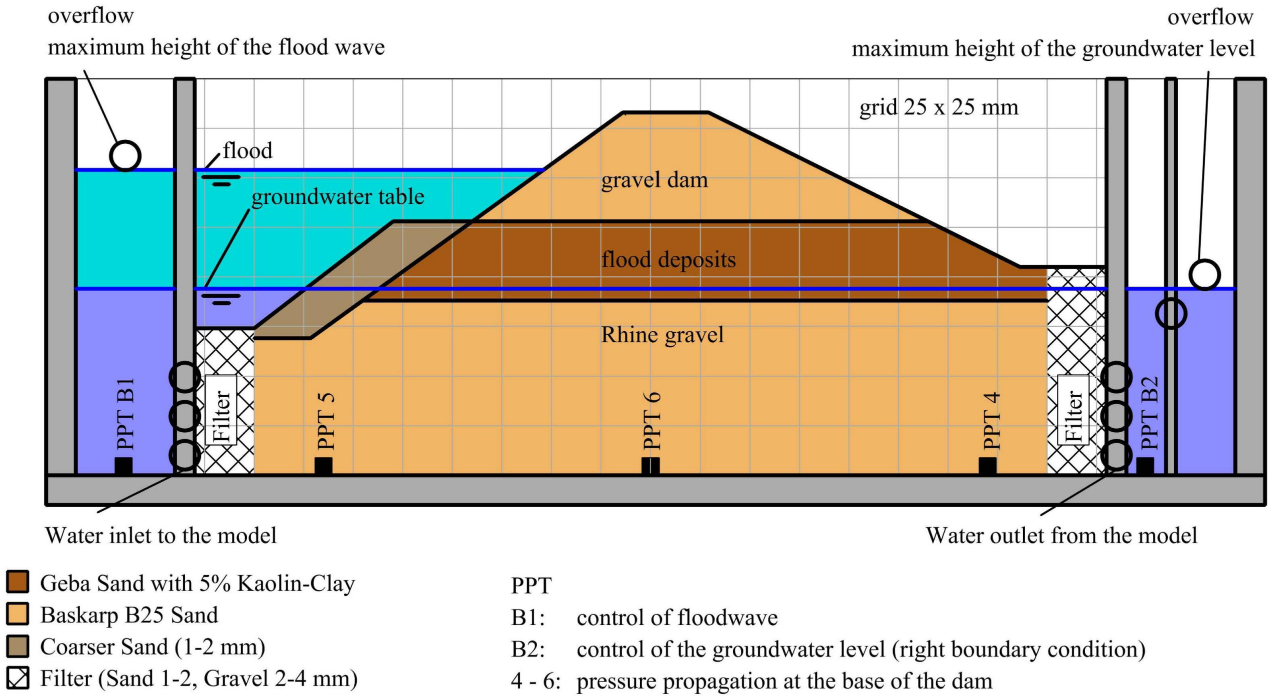

Figure 3. Geometry of levee A built into the flood simulator including positions of PPTs. Dimensions in [mm].

Table 1. Geotechnical properties of the levee materials used in the centrifuge test.

Soil type	e[1)] [-]	k_v [m/s]	φ'_{cv} [°]
Baskarp B25 Sand	0.69	$(4.6 – 5.8)\cdot10^{-5}$	32 - 34
Geba Sand with 5% Kaolin Clay	0.64	$(2.3 – 2.6)\cdot10^{-6}$	~ 31[2)]
Geba Sand with 10% Kaolin Clay	0.56	$(4.0 – 5.8)\cdot10^{-7}$	~ 31[2)]

[1)] voids ratio after compaction at w = 10%. [2)] value applies for Geba Sand without clay.

where e is the voids ratio, k_v is the corresponding coefficient of the vertical permeability and φ'_{cv} the critical state angle of friction. In prototype, the Rhine gravel and the Rhine levee have a similar permeability, so both materials are modelled with Baskarp B25 sand (Pol et al., 2021). For the less permeable layer of flood deposits a mixture of Geba Sand (Maghsoudloo et al., 2021) with 5% Kaolin Clay for levee A and 10% Kaolin Clay for levee B was used. The permeabilities were determined with constant head tests (Head & Epps, 2011). The values determined in the tests correspond to the vertical permeability k_v.

The soil materials were placed in the flood simulator in layers with a water content w of 10% and compacted uniformly. At the same time, a two-stage filter made of coarser material was installed at the up- and downstream boundary of the model. The filter is made of gravel 2-4 mm and sand 1-2 mm and is therefore significantly coarser than the Baskarp B25 sand with a grain size d_{70} of 0.26 mm (Pol et al., 2021).

After the entire flood simulator was filled, the levee was shaped by removing the excess material. At the same time, samples were taken to check the layer density.

Next, the river-side berm, which in the prototype is made of angular rock boulders as an erosion protection at the base of the levee (Figure 2), was constructed in the model. For this purpose, the berm was built up from coarse grained 1-2 mm sand, see Figure 4.

Three Pore water Pressure Transducers (PPTs) were installed at the bottom of the model (Figure 3) to record the propagation of pore water pressure in the levee. To check the boundary conditions, two PPTs were installed in the upstream inlet tank and in the downstream outlet tank, respectively.

Figure 5 shows the planned flood characteristics in the prototype scale for the tests. The water levels refer to the heights from the base of the flood simulator (Figure 3) and are given in prototype scale hereafter, with the corresponding model scale in parentheses: The riverbed accordingly lies at a height of 7.4 m (74 mm). The goal was to model a flood wave consisting of a pre-wave of 13 m (130 mm), a flood peak of 15.4 m (154 mm) and a post-wave of 12.3 m (123 mm) by opening and closing the valves, starting from an initial river level at 9.4 m (94 mm) (steady state with 2 m (20 mm) high river, see Figure 3). Water was used as the fluid for the centrifuge experiments. The model time for seepage processes is therefore reduced by a factor of 1:100^2 according to Equation 2, since the model was tested at 100 g. This means that the entire flood event with a duration of 60 h at prototype scale translates to 21.6 s in the centrifuge experiment. The experiments were recorded through the acrylic glass using a digital camera.

The test procedure is described hereafter with the corresponding prototype time steps, model time is given in parentheses: After the centrifuge reaches 100 *g* ($t = 0$), the initial water level of 9.4 m (94 mm) is kept constant for 14 d (2 min) to allow stationary conditions to be established. Subsequently, the flood wave is applied stepwise within 60 h (21.6 s) as shown in Figure 5. Finally, the model is kept under 100 *g* for another 35 d (5 min) to restore initial conditions.

After the centrifuge test, the model is checked for any signs of erosion, piping or stability problems. However, the desired flood wave could not be achieved because some of the valves could not be opened or closed properly at 100 *g* (Figure 5). Thus, it was difficult to maintain a constant water level with the water available in the tank due to the loss of water through seepage.

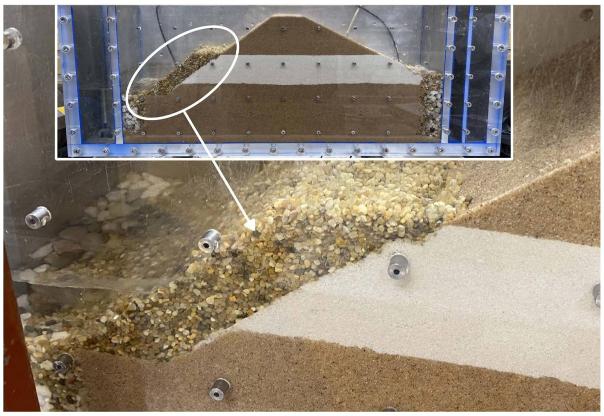

Figure 4. Upstream berm made from coarse grained 1-2 mm sand.

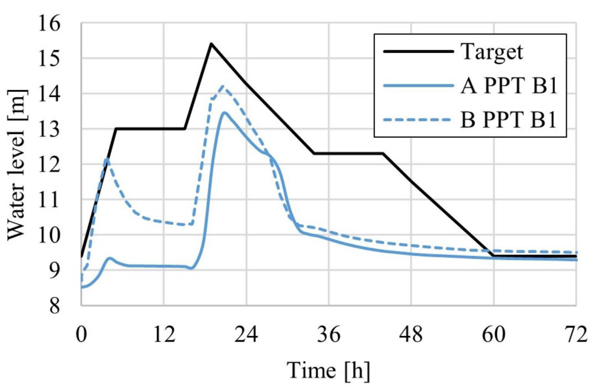

Figure 5. Target hydrograph of the flood wave in prototype scale compared to the actual measurements of PPT B1 achieved during test for levees A and B (riverbed is located at 7.4 m).

2.2 Comparative numerical model

A comparative numerical model of the levee A is developed at prototype scale in the program PlaxFlow2D (PLAXIS CONNECT Edition V20.04.00, 2020) with the same boundary conditions as given in the experiment. Special attention was paid to the following points:

- The flood wave is modelled according to the data measured in the centrifuge test (i.e., PPT B1 hydrograph in Figure 5).
- The permeable berm on the water side made of coarse sand is not modelled to minimise the computational effort. It is assumed that due to the higher permeability, it does not significantly influence the seepage behaviour.
- A constant head boundary condition is set on the air-side as given in the centrifuge model test.
- Figure 6 shows the FE model with the corresponding boundary conditions and the mesh. The initial water level of the river lies 2 meters above the riverbed and corresponds to the height of the constant head boundary condition on the right edge of the model (initially no hydraulic gradient). The variable water level is indicated in light blue on the left side of the levee.

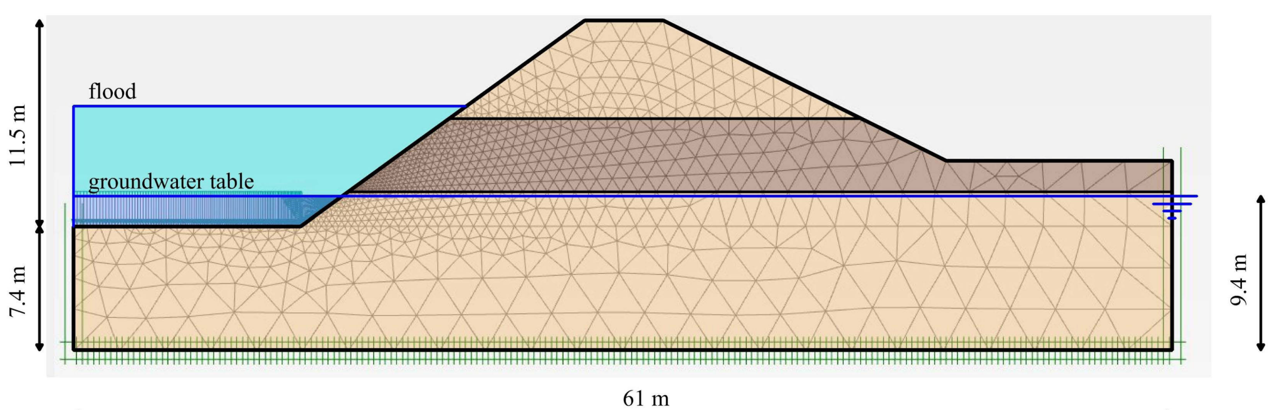

Figure 6. FE model geometry with mesh and boundary conditions for the PlaxFlow2D calculation.

The pressure conditions are compared at points that are always saturated (PPTs in Figure 3). However, it must be assumed that the modelling of the unsaturated levee area above has an influence on the results. For a comparative model, the unsaturated zones of the levee and their behaviour are described using the van Genuchten (1980) model implemented in the software. Thus, with the help of the dimensionless volumetric water content Θ (Equation 3), the relative hydraulic permeability K_r of partially saturated soils can be described (Equation 4).

$$\Theta = \frac{\left(\theta - \theta_r\right)}{\left(\theta_s - \theta_r\right)} \quad (3)$$

$$K_r\left(\Theta\right) = \Theta^{\frac{1}{2}}\left[1 - \left(1 - \Theta^{\frac{1}{m}}\right)^{m}\right]^2 \quad (4)$$

In Equation 3, θ describes the actual volumetric water content, θ_r is the residual and θ_s is the saturated volumetric water content. The parameter m is linked to the soil-water retention curve given in Equation 5 as $m = 1\text{-}1/n$:

$$\theta = \theta_r + \frac{\left(\theta_s - \theta_r\right)}{\left[1 + \left(\alpha \cdot h\right)^n\right]^{(m)}} \quad (5)$$

In Equation 5, α, n and m are model parameters describing the curvature of the water retention curve after van Genuchten (1980) and h is the pressure head (suction is positive). However, no soil-water retention curves were available for the materials used. To be able to carry out an approximate calculation and compare it to the centrifuge tests, parameter sets stored in PLAXIS were used, see Table 2. The parameter sets were selected on the basis of the particle size distribution in such a way that the numerical calculation models produced the best possible comparison to the centrifuge test results. Due to the chosen procedure, some general points must be emphasised:

Table 2. Input parameters for the transient seepage analysis.

Parameter	Geba Sand with 5% Kaolin Clay	Baskarp B25 Sand
$k_h = k_v$ [m/s]	$2.3 \cdot 10^{-6}$	$5.0 \cdot 10^{-5}$
e_{init} [-]	0.64	0.69
θ_r [-]	0.01	0.01
θ_s [-]	0.34	0.32
α [l/m]	1.7	5.21
n [-]	1.717	2.374

- The calculations represent an initial estimate. The actual water retention curves and the associated parameters α and n may differ from those used in the present model.
- A difference between drying and wetting curves can be observed in reality (Benson et al., 2014). This is not taken into account in the numerical model.
- As there is no information on the ratio of saturated horizontal permeability k_h to vertical permeability k_v and a reliable estimate is difficult, no distinction was made.

3. Results

Both levees did not exhibit any stability problems during the centrifuge tests. Furthermore, it became evident that the entire model (i.e., the centrifuge basket) tilts slightly during the experiments, which results in a water level that is inclined relative to the flood simulator. This phenomenon occurs when the centre of gravity is not in the axis of the centrifuge basket's hinge. However, due to the flood wave and the seepage, the centre of gravity is constantly changing - which explains the phenomenon (the model was installed perpendicular to the flight direction to avoid curvature of the water surface). Thus, at the beginning of the test, the water table at PPT B1 was lower at about 8.4 m than at the end of the test at about 9.3 m (Figure 5).

Analysis of images from the centrifuge tests showed that the flood wave effectively acting on the levee body reached its maximum level almost one second later than in the flood tank where the measurement of PPT B1 was carried out (Figure 3). This delay is due to the fact that water needs to flow through the gravel filter and water inlet holes (3 holes with d = 6 mm; Figure 3). One second corresponds to a delay of approximately 2.7 hours in prototype scale. This delay is taken into account in the comparison with the comparative numerical model.

3.1 Comparison of centrifuge test results for levee A and B

Immediately after the tests, the air-side slopes of both levees were examined. A difference between levee A and B was observed. Seepage paths were able to form above the continuous flood deposit layer of levee A, which led to a water outflow at the air-side embankment, resulting in sand washouts and surface erosion (see Figure 7, top). However, this phenomenon could not be observed for levee B (see Figure 7, bottom). Figure 7 also shows probable flow paths. For levee B in particular, the dark discolouration of the Baskarp B25 Sand indicates a change in the degree of saturation, although it is not possible to determine whether the dark sand is fully saturated. In contrast, no change in colour could be detected in the lighter-coloured Geba Sand Kaolin Clay mixture with the existing resolution.

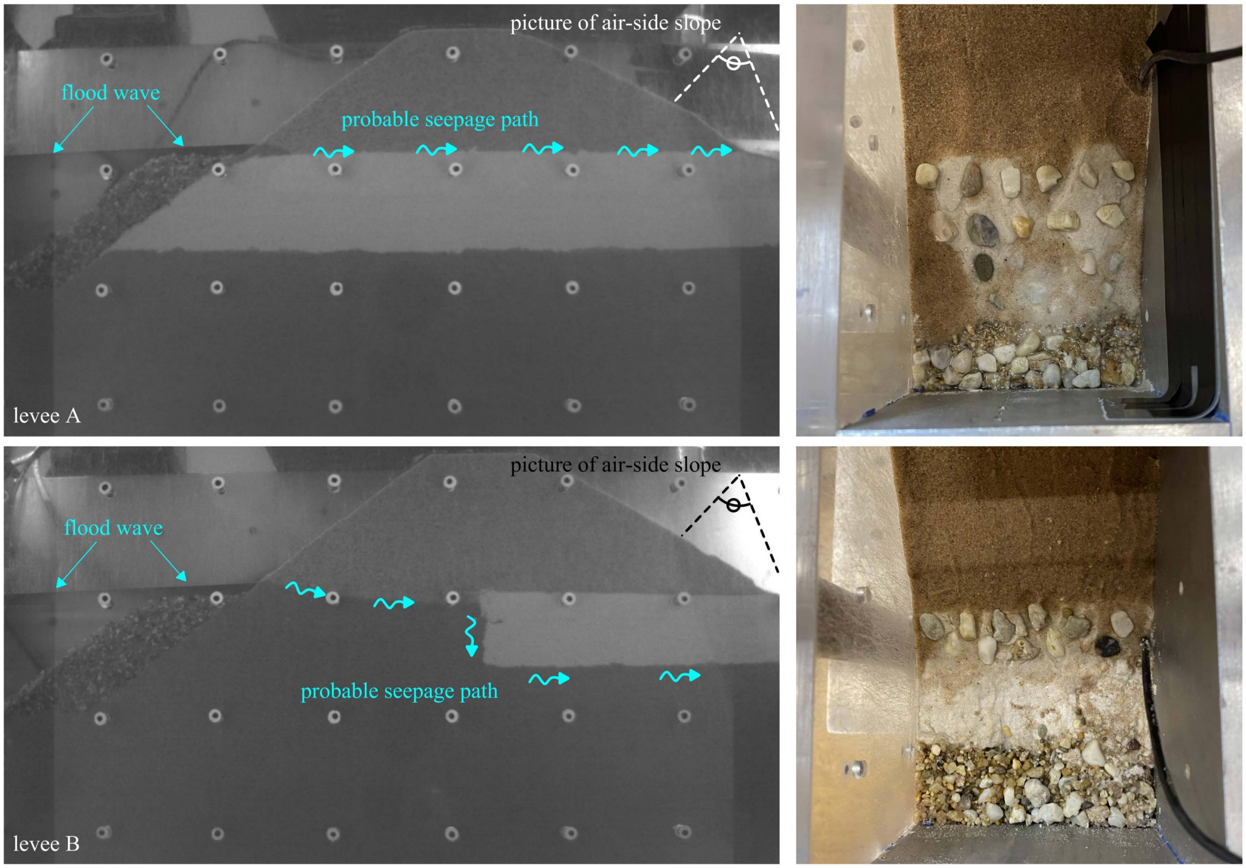

Figure 7. Left: Image of levee A (top) and B (bottom) during the centrifuge test, approximately at the flood peak. Right: Air-side slopes after the tests.

PPT B1 shows that the flood wave for levee B is approximately one meter higher than for levee A, see Figure 5. The pre-wave is also much more pronounced for Levee B. In order to be able to directly compare the two levee cross-sections, the PPT measurements are normalised in relation to the maximum value of PPT B1 in Figure 8. The initial letter stands for the corresponding levee cross-section (A PPT B1 for the measurement of PPT B1 at levee cross-section A). The time is given at prototype scale. The following observations from the comparison of the two centrifuge model tests should be pointed out:

- There is a significant damping of the flood wave upon entering the levee. PPT 5 reaches approximately 85% of the peak of PPT B1 with a distinct delay in time.
- The damping of the flood wave in relation to the maximum pressure head at PPT B1 is practically identical in the two cross-sections. For example, PPT6 in the centre of the levee shows for both levees a pressure head of approximately 3/4 of the maximum flood wave.
- The development of the pore water pressure over time, however, appears to be different in the two cross-sections. The pressure peaks are reached faster in cross-section B compared to cross-section A. However, it must be emphasised that not only the cross-section is different, but also the saturation of the levee fill during the main flood wave is probably greater in cross-section B due to the larger pre-wave.

3.2 Comparison of centrifuge test results for levee A to a comparative numerical model

Results obtained from the centrifuge test for levee A are compared to the results of a comparative numerical model (section 2.2). As previously outlined, it is necessary to add a time increment of 2.7 hours to the measurement of A PPT B1 in order to model the flood wave acting on the levee (A PPT B1 + 2.7 h is designated as ‘flood’ in Figure 9). By comparing the pressure changes from the centrifuge test with those from the comparative numerical model (FEM in Figure 9), the following observations can be made: The infiltration behaviour of the numerical model differs from the physical model. For the latter, it is noticeable that there is a significant damping of the flood wave upon entering the levee (see PPT5) and the propagation of the flood wave within the levee is significantly slower on the water side than towards the air-side. This becomes clear by comparing the shifting of the pressure peaks over time.

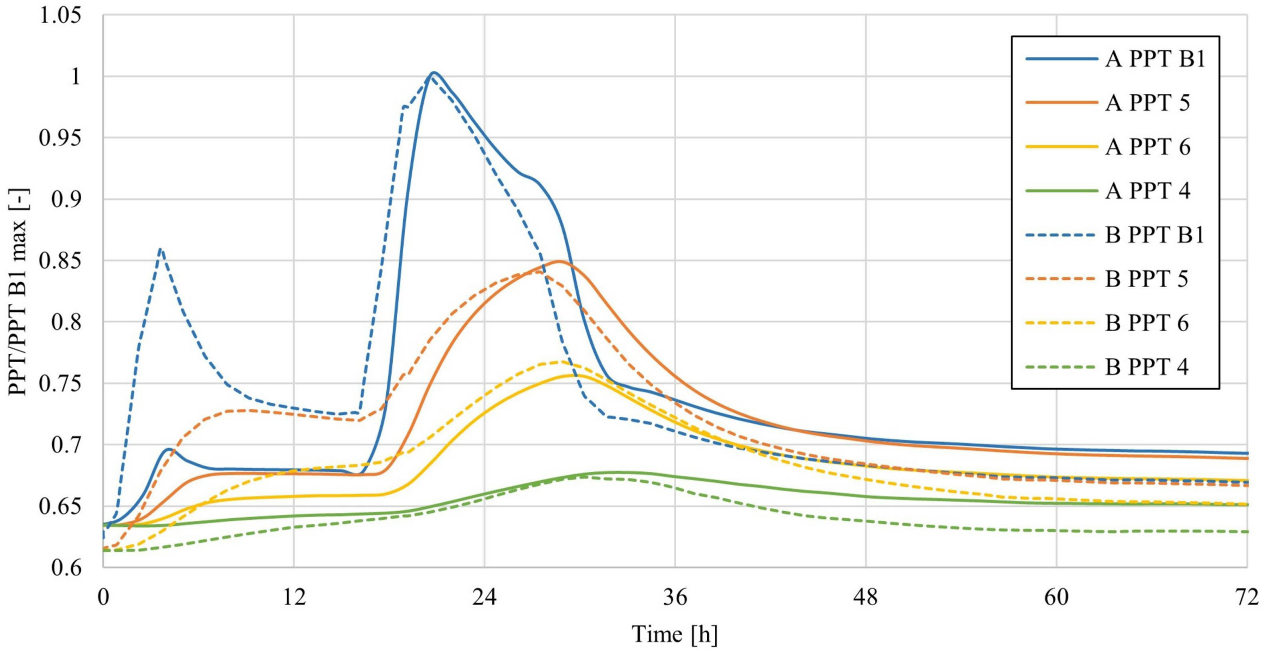

Figure 8. Comparison of normalised PPT measurements of levee A and levee B (time in prototype scale).

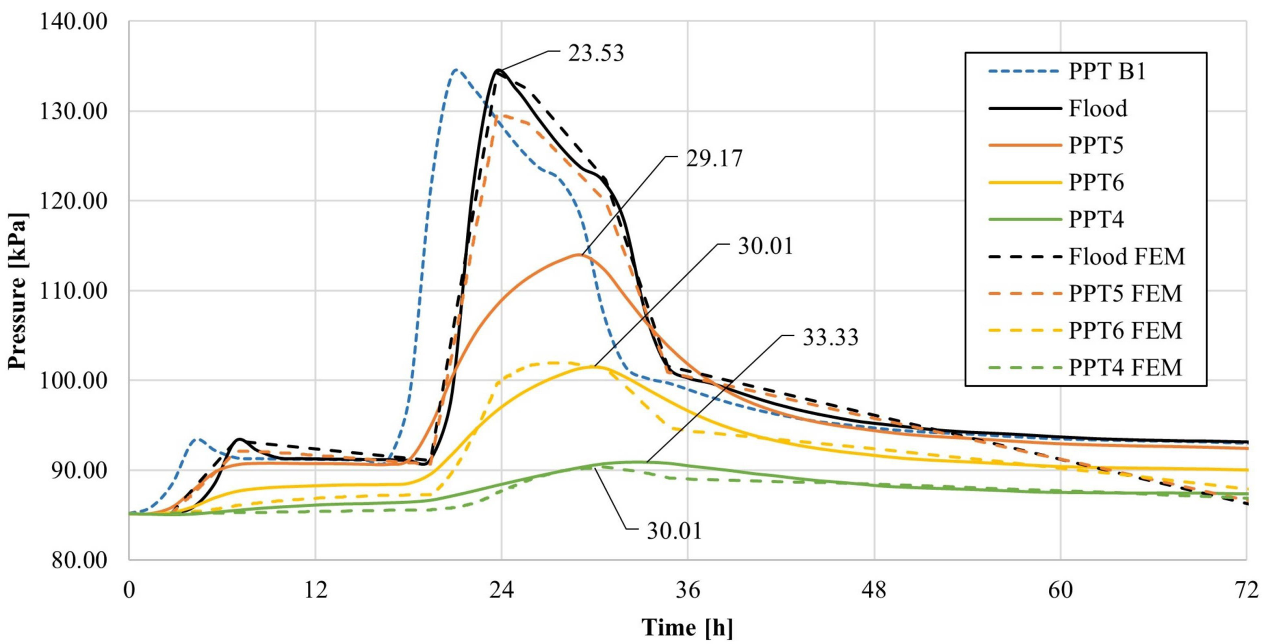

Figure 9. Comparison of PPT measurements of levee A and the FEM simulation (time in prototype scale).

The delay between the flood wave and PPT5 is about 5.6 hours, whereas it is only about 3.3 hours between PPT6 and PPT4. The numerical simulation, on the other hand, shows almost no damping for PPT5.

Nonetheless, PPT6 and PPT4 show relatively good agreement at least in the damping of the pressure magnitude. In the centrifuge test it took about 10 hours to reach the maximum pore pressure at the air-side of the levee (peak of PPT4 compared to peak of 'flood'), whereas in the numerical model there are only about 7 hours in between. Furthermore, all pore water pressures almost reach the initial value again at the end of the numerical simulation.

4. Discussion and conclusions

A comparison of the behaviour of two different cross-sections in the centrifuge model (Figure 8) shows that the damping magnitude of the flood wave does not differ significantly with different soil stratification. This result is worth mentioning because different flow paths are formed in the two models. Based on the test data of pore water pressure increase over time, it cannot be clearly determined whether the faster propagation of the flood wave through levee B is primarily due to the fact that the saturation of the levee fill material and thus the permeability is higher, or whether it is due to the different soil stratification.

Comparing the test results of the physical model test to those of the comparative FEM simulation (Figure 9), a difference in time of rise and magnitude of the pore water pressure due to high water levels could be observed. The largest deviations could be found on the water side of the levee. The numerical model overestimates the speed at which the flood wave infiltrates the levee and underestimates the damping of the pore water pressures on the water side. The difficulties in predicting the pore water pressure distribution and the development of the saturation front in levees were already pointed out by Vaughan (1994) in his Rankine lecture, since most models do not take stress dependent permeability in terms of overburden pressure and heterogeneity of the material into account. In addition, there are considerable uncertainties in soil permeability measurements (anisotropy etc.). However, the behaviour observed in the physical model compares well to the numerical analysis considering the large scaling factor for time of $t_p = 100^2\ t_m$ (Equation 2).

Regarding the question of whether transient seepage conditions for the design or assessment of flood protection levees should be considered and what benefit the results of centrifuge model tests in this context have, the following aspects should be pointed out:

- Centrifuge model tests and comparative numerical simulations lead to a better understanding of the levee behaviour. The well controllable test conditions in the centrifuge models serve to assess the quality of numerical simulations.
- The transfer of the present findings to the prototype scale poses a number of challenges to project engineers. For example, major uncertainties lie not only in the investigation and variability of relevant geotechnical parameters (water retention curve, permeability and anisotropy, friction angle, etc.) but also in the description of the initial state in terms of levee saturation, to which great importance must be given when analysing transient seepage conditions (Mayor, 2013). However, the initial degree of levee saturation is generally unknown in practice.

A further topic to be addressed is the influence of the hydraulic boundary conditions, as it is reasonable to assume, that the type of the air-side boundary condition not only has an influence on the levee stability, but also on the overall seepage behaviour. This is particularly relevant as the centrifuge tests are intended to represent the conditions within the prototype levee in order to more realistically portray the behaviour under flood conditions. The tests presented in this paper are based on hydraulic boundary conditions on the air-side of the levee corresponding to a constant hydrostatic water pressure (Figure 10 a).

If the levee was constructed on a widespread fine-grained layer, as it is the situation in parts of the Alpine Rhine, some excess pore water pressures may develop beneath the bottom of the levee on the air-side during a flood. As an extreme case, there could be a vertical impermeable boundary such as a construction pit closure in the levee footing area, see Figure 10 b. The pore water pressure distribution beneath the bottom of the levee on the air-side therefore depends directly on the permeability and the distance of the air-side boundary condition (this applies to physical and numerical models as well as to prototypes) and must be carefully chosen.

The following observations may be considered to improve the results from further experiments:

- More PPTs should be installed on the upstream side of the levee to better quantify the flood wave propagation into the levee close to the riverbed. This is where the largest differences between centrifuge tests and numerical simulation have been observed. An additional PPT should also be installed at the level of the riverbed above the gravel filter to record the actual flood wave acting on the levee.
- Attention should be paid to the boundary condition on the upstream side. To enhance control of water flow, it is

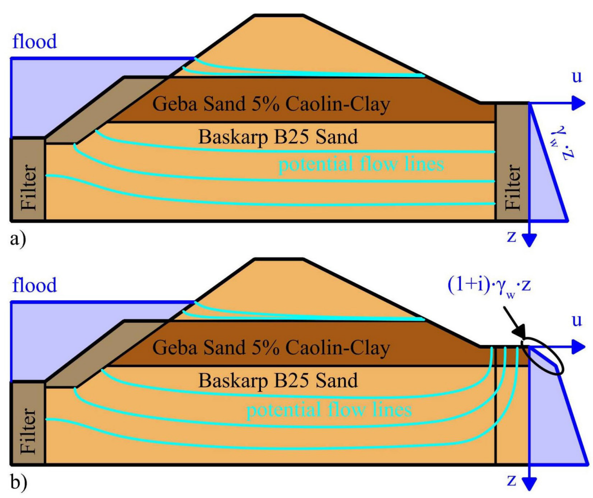

Figure 10. Variation of the boundary conditions on the air-side of the levee and their potential influence: a) constant head boundary as modelled in the first test series; b) impermeable boundary – excess pore water pressure assumed below the interlayer as a result.

recommended to maintain full permeability between the inlet water tank and the model compartment by drilling additional holes. It is also recommended to install an additional impermeable foil between the gravel filter and the levee base (i.e. Rhine gravel in the prototype, see Figure 3), given that flood loading in the prototype is predominantly characterised by inflow from the river bed and not by horizontal inflow to the base of the levee.

- The initial degree of saturation of the levee above the seepage line is unknown - but this significantly influences the infiltration time (Mayor, 2013). The installation of tensiometers to measure suction could therefore be advantageous (Askarinejad et al., 2018; Giretti et al., 2022).
- The intended flood wave, which is inspired by real flood events, could not be achieved due to the complexity of the test control. If the flood wave characteristics are to be simplified for further tests, possible saturation changes of the levee before the flood should be considered.
- Although the comparison of numerical simulation results and centrifuge model tests shows good agreement (with one exception mentioned above), additional laboratory tests should be carried out to investigate soil behaviour in more detail. In this context, two aspects of soil behaviour are of particular interest: i) Anisotropic permeability: isotropic permeability was assumed in the numerical simulations ($k_h = k_v$), whereas artificially compacted soils often exhibit anisotropic permeability ($k_h/k_v > 1$). ii) Estimation of soil water retention curves (SWRCs): it is recommended that SWRCs are determined with additional laboratory tests for further simulations. So far, SWRCs have been estimated on the basis of grain size distributions.
- It should be considered whether the test may be carried out with viscous fluid instead of water to reduce the time scaling factor (Wood, 2004). This could help to extend the simulated flood duration to achieve better experimental control and data collection.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Gregor Portmann: conceptualization, data curation, formal analysis, methodology, investigation, visualization, writing – original draft. André Arnold: conceptualization, funding acquisition, methodology, project administration, resources, supervision, validation, writing – review & editing. Yuen Zhang: methodology, investigation, writing – review & editing. Amin Askarinejad: conceptualization, methodology, supervision, project administration, resources, validation, writing – review & editing.

Data availability

The datasets generated and analysed in the course of the current study are available from the corresponding author upon request.

Declaration of use of generative artificial intelligence

This work was prepared with the assistance of generative artificial intelligence (GenAI) DeepL (https://www.deepl.com) to provide assistance in translating text elements from German into English. The entire process of using this tool was supervised, reviewed and when necessary edited by the authors. The authors assume full responsibility for the content of the publication that involved the aid of GenAI.

List of symbols and abbreviations

d	diameter
d_{70}	70% of the soil grains are finer than the given value
e	voids ratio
e_{init}	initial voids ratio after compaction
g	gravitational acceleration, 9.81 m/s^2
h	pressure head (suction is positive)
k	coefficient of permeability, k_v: for vertical and k_h: for horizontal permeability
l	seepage path length, l_m: in the model, l_p: in the prototype respectively
m	model parameters describing the curvature of the water retention curve after van Genuchten (1980)
n	model parameters describing the curvature of the water retention curve after van Genuchten (1980)
t	time, t_m: in model scale, t_p: in prototype scale respectively
v	seepage velocity, v_m: in the model, v_p: in the prototype respectively
w	water content
FEM	Finite Element Method
H	height of the flood simulator
K_r	relative hydraulic permeability of partially saturated soils

L	length of the flood simulator
N	centrifuge acceleration level (N times earth gravity)
PPT	Porewater-Pressure-Transducers
SWRC	Soil Water Retention Curve
W	width of the flood simulator
α	model parameters describing the curvature of the water retention curve after van Genuchten (1980)
φ'_{cv}	critical state angel of friction
θ	volumetric water content
θ_r	residual volumetric water content
θ_s	saturated volumetric water content
Θ	dimensionless water content

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