HYDRAULIC FAILURE BY UNDERSEEPAGE OF DYKES AND LEVEES

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The increase in frequency, magnitude and duration of floods during the past decades has become an outstanding challenge to geotechnical engineering. When dykes or levees do not have a cut-off wall fully penetrating the aquifer, underseepage may occur during high river levels. In such cases, appropriate measures against hydraulic fracture require comprehensive knowledge of failure modes of dams, dykes and levees. The installation of water pressure relief elements at the landside toe zone of dykes and dams has proven successful. The paper focuses on different relief systems based on mathematical approaches, laboratory and field tests, and on site observations. Finally, the hydraulic behaviour of relief drainage columns based on numerous experimental tests and numerical parametric studies is described.

Key words: Flood protection; Dykes; Hydraulic failure; Inner erosion; Piping; Underseepage; Relief drainage.

1. INTRODUCTION

Floods have affected millions of people worldwide in recent decades. In several regions the magnitude and frequency of flood waves have increased dramatically since long-term measurements and historical reports have existed. In Austria, for instance a 2 000 to 10 000-year flood event was back-calculated from the flood disaster in the year 2002 (Fig. 1). Such hitherto singular values cannot be taken as design values for flood protection dykes, but they underline the need for local overflow crests or spillway sections. Moreover, they clearly demonstrate that a residual risk is inevitable – despite most costly protective measures.

The risk of dykes or levees failure increases not only with the magnitude of a flood but also with its duration. For instance, the peak period of flood waves along the Austrian section of the river Danube usually lasts one to three days, whereas its tributary, the river March/Morava (Austria/Slovak border) frequently undergoes flood waves up to three or six weeks (Fig. 2). Figure 2 also illustrates the increase of magnitude and frequency of the floods since the 1990s.

Especially long-lasting flood waves exhibit in combination with a required groundwater communication below dykes a high risk potential regarding hydraulic failure. But also periodic short hydraulic loadings of flood protection dams and their subgrade can produce a failure caused by an inner erosion processes in a long-term.

Leeve underseepage analyses are commonly performed to assess the risk of excessively high pore pressure in the aquifer. These results are used for the assessment of possible failure mechanisms. After today´s practice the hydraulic failure due to underseepage may be prevented mainly by two permanent measures at landside dyke or dam toe by installing pressure relief elements or by placing of berms. Especially, relief drainages have proved very successful in Austria during the last excessive floods along the rivers Danube and Morava.

However, until now the existing design criteria for relief drainages were insufficient. Most of these approaches determine the pressure relief and the discharges only based on assumptions and experience from former projects. Therefore, various small-scale as well as large-scale model tests were performed to study the pressure relief behaviour including the quantification of discharge.

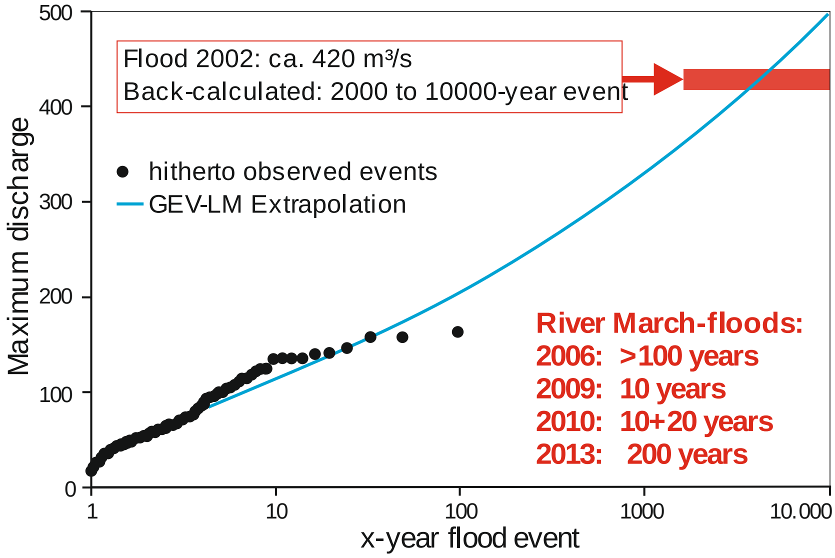

Figure 1. Statistics for the annual maximum discharge values of the river Kamp in Austria (modified / extended after Gutknecht et al. 2002). Comparison with the subsequent river March floods

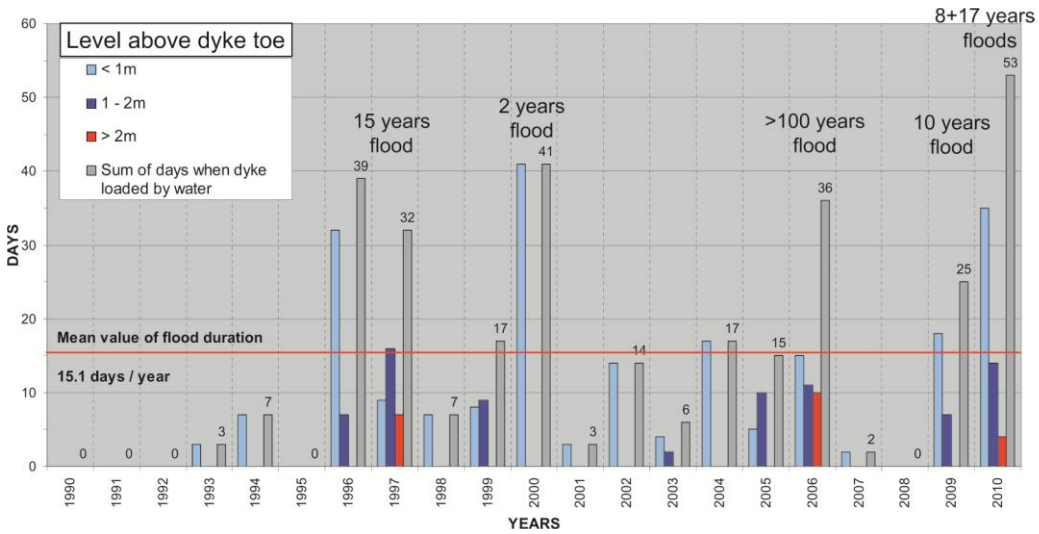

Figure 2. Duration of floods along the River March dykes (Water level at Dürnkrut – Austria/Slovakia; adapted after via Donau). Two floods within three weeks in 2010

2. FAILURE MODES OF DYKES AND LEVEES

Knowledge of possible failure modes is an essential prerequisite, both for a reliable quality assessment of existing dykes and dams and for an optimised design of new ones, frequently in connection with the application of geosynthetics. Therefore, large scale 1:1 failure tests were performed by the Institute for Ground Engineering and Soil Mechanics of the Vienna University of Technology already in the 1960s at the river Danube in Vienna. A section of the dam was flooded by creating a sheet piled area, where the water level could be raised and lowered (Fig. 3). Thus, a flood with a discharge of Q = 14.000 m^3/s was simulated.

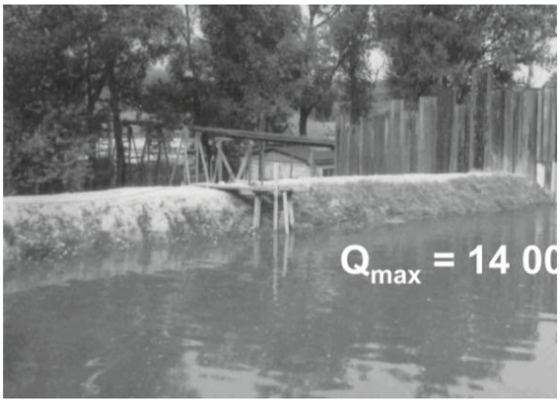

Figure 3. Large scale tests on river Danube dyke in Vienna (1967)

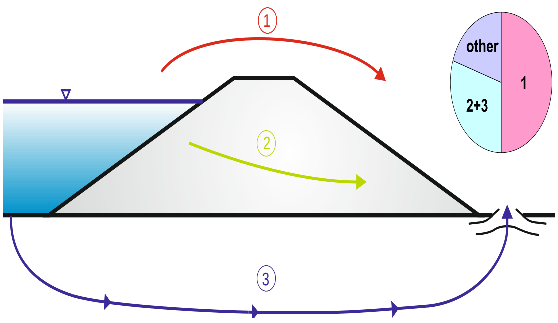

Figure 4. Main failure modes of dykes; schematic.

1 = Overflowing, overtopping → Erosion failure. 2 = Seepage through dyke → Internal erosion, Slope failure. 3 = Underseepage → Internal erosion, Upheave, Ground failure.

The simplified scheme of figure 4 illustrates the main failure modes as observed in most cases. The percentages differ regionally more or less. The dominating failure modes for typical ground conditions along rivers (near-surface low-permeability sandy to clayey silts underlain by high permeability sand or gravel) can by summarized as follows:

- slope failure due to excessive pore-water pressures, seepage or internal erosion
- overtopping or overflowing of the dyke/dam crest
- slope failure due to a rapid drop of the flood water level
- hydraulic fracture
- surface erosion and failure of the water-side slope due to wave action
- piping due to animal activities, especially from beavers and rats
- unsuitable planting of dykes (especially trees with flat roots).

Actually, it is often difficult to precisely determine the causes of a dyke failure. Several types of processes might be involved in a breach and multiple modes in a dyke failure. Statistical analyses show that overtopping and internal erosion are the most common modes of failure. While many of these failure mechanisms occur relatively fast, the erosion by underseepage develops more inconspicuously. If a groundwater communication below the dyke is possible, the aquifer or the overlaying low permeable layer can be progressively eroded during hydraulic loading. Hydraulic failure is critical because there may not be any external evidence, mostly only soil boiling can be found.

Due to this unpredictable behaviour hydraulic failure is frequently underestimated in practice and may occur in different forms (e.g. Eurocode 7; CEN 2004):

- by uplift (buoyancy),
- by heave,
- by internal erosion (Fig. 5),
- by piping (Fig. 6).

In the case of groundwater communication under a dam construction, the landside surface layer is exposed to hydrostatic stress due to underseepage. If confined conditions can develop in the aquifer, the safety against a failure of the subsoil reduces significantly. The loss of stability is usually initiated by uncontrolled hydraulic rupture (uplift) of the blanket or by concentrated transport of fine particles (erosion, suffusion, piping) from the subgrade.

Hydraulic failure may reach several tens of meters away from dykes or dams, as experience has shown (Fig. 7). This could be observed even for low flood protection embankments with a relatively small hydraulic gradient. Soil boiling may create large volcanoes (e.g. Fig. 8) that require urgent flood defence and stabilizing measures.

Figure 5. Breach of a Sava River dyke with an extensive scour on the landside (120 x 300 x 12m) due to internal erosion of the ground below the dyke. (Croatian Ministry of Agriculture, 2014)

Figure 6. Piping through a railway embankment during an already sinking flood. Train traffic was stopped

Eurocode 7 (CEN 2004) states that in situations where the pore-water pressure is hydrostatic (negligible hydraulic gradient) it is not necessary to check other than for failure by uplift. In the case of danger of material transport by internal erosion, filter criteria should be used. If the filter criteria are not satisfied, it should be verified that the critical hydraulic gradient is well below the design value of the gradient at which soil particles begin to move.

Figure 7. Piping (soil boiling) far away from the dyke and stabilizing measures to reduce the hydraulic gradient (photo: L. Nagy)

Figure 8. Boiling “volcanoes” after the flood. Retro-gressive internal erosion towards the dyke causes stability loss in the long-term (photo: L. Nagy)

Hydraulic failure may occur despite cut-off walls. If they are “imperfect” (i.e. with underseepage), groundwater communication below the dykes or levees (for environmental reasons) occurs and overpressure can develop beneath the landside blanket. Fine-grained blanket with local “windows” and low residual shear strength favours such failure modes (Fig. 9).

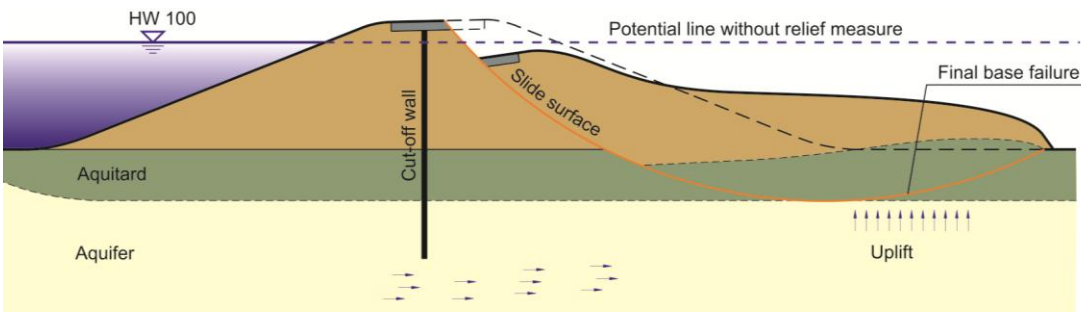

Figure 9. Hydraulic base failure despite cut-off wall, favoured by low shear strength of low permeable fine grained blanket (aquitard with “windows”)

Therefore, ground investigation should also comprise the determination of residual shear strength φr. This value is not a „constant" soil parameter, but depends on normal stress and degree of water saturation (Fig. 10).

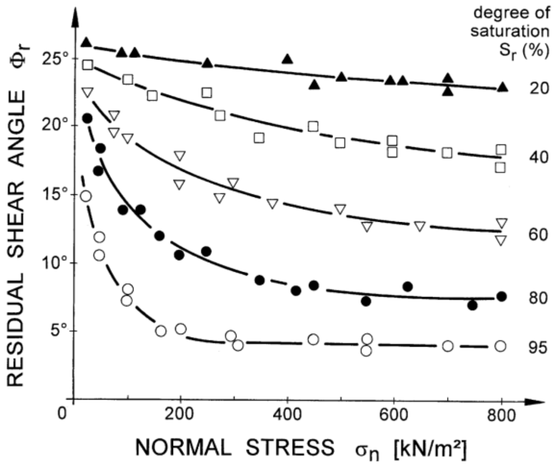

Figure 10. Residual shear angle φr of a slide prone clayey silt depending on normal stress and degree of water saturation. Results of comprehensive test series with same material (reconstituted). Direct shear tests on consolidated drained samples.

2.1 UPLIFT

The hydraulic failure by **uplift** is characteristic for a two-layer subgrade system with low permeable blanket above the highly permeable aquifer (Fig. 11). When the hydrostatic overpressure reaches the unit weight of the blanket, the soil becomes practically weightless and the uplift safety drops significantly. After exceeding the critical water pressure, the blanket ruptures mostly in the area of the landside dam toe or in adjacent hinterland. This failure process leads progressively to stability loss of the dyke due to progressive internal erosion or piping. "Volcanoes" are often formed in hinterland (Fig. 8). The uplift failure mechanism is very complex and that makes the definition of the state of failure difficult.

As uncontrolled rupture is usually preceded by a lifting of the cohesive blanket, which allows temporarily the formation of narrow cavities at the boundary layer to the aquifer. This may lead to a transport of soil particles, as it could be confirmed in experiments. However, such mechanism requires sufficient thickness as well as homogeneity of the blanket, which is not always the case. Especially, blankets with low thickness have natural or artificial cracks, which reduce the hydraulic stability of this low permeable soil layer. Due to the confined conditions in the aquifer, the water flows mostly along these cracks and inhomogeneities to the surface, thus eroding the subgrade. Therefore, the failure does not occur only as a result of uplift, but rather as combination with internal erosion and subsequent piping.

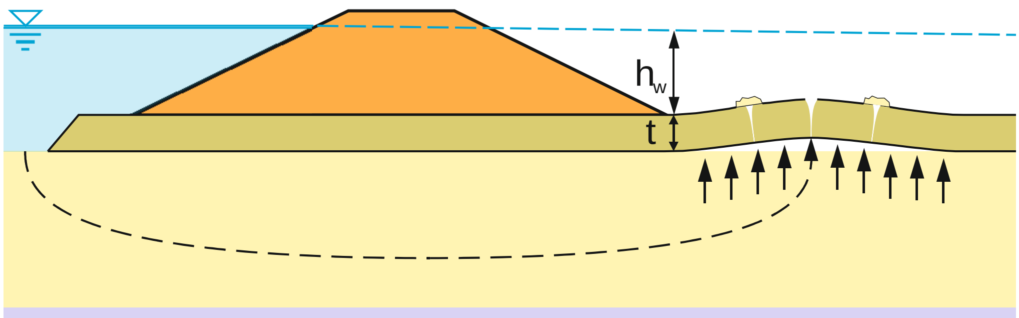

Figure 11. Uplift failure caused by an excessive pore water pressure in the foundation due to underseepage of the dyke (schematic drawing).

2.2 HEAVE

The hydraulic failure by **heave** occurs in cohesionless soils when vertical seepage forces act on the soil grains. The seepage forces are increasing until the effective stress becomes zero. At this point, the hydraulic gradient is equal to the critical hydraulic gradient i_{crit} (i_{crit} = γ'/ γ_w) and there is an erosion of fine particles in the soil leading to a formation of erosion channels, accompanied by a significant increase in permeability. This failure behavior is typical primarily in semipervious blankets, where vertical seepage can occur (Fig. 12). Also the fine-grained soils are also affected. Although due to the internal stresses, the cohesive soil has a much higher resistance against the internal erosion processes then cohesionless soils.

Experience has shown that the magnitude of the critical hydraulic gradient where internal erosion begins is frequently overestimated, thus underestimating the actual long-term risk. Figure 13 summarizes the critical values on the basis of field observations, geotechnical measurements, literature and long-term experience for different soils. For comparison, the conventional criterion ($i_{crit} = \gamma'/\gamma_w$), Lane's criterion, and the critical zones after Eurocode 7 (CEN 2004) or Chugaev (1965) respectively are also plotted in the diagram.

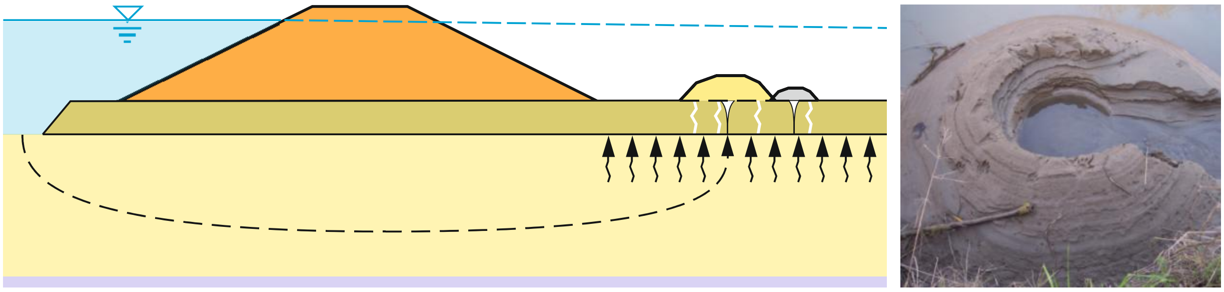

Figure 12. Hydraulic failure by heave accompanied by inner erosion of soil particles from the semipervious blanket and permeable subgrade caused by an excessive pore water pressure in the foundation due to underseepage of the dyke (schematic drawing) and a detail of a "volcano" at the dyke toe zone.

2.3 INTERNAL EROSION, SUFFUSION AND PIPING

Hydrodynamic processes by **internal erosion**, **suffusion** and **piping** always have to be considered in the close connection with the before mentioned failure mechanisms, which often represent the initial stage of a hydraulic ground failure (Fig. 15). From a long-term perspective, there is a high risk of progressive erosion, especially due to temporary hydraulic loading. After an initial local rupture of the blanket, an erosion channel forms retrogressively from the landside to the waterside during one or more floods. If this reaches the river, breaching occurs as a result of hydraulic ground failure.

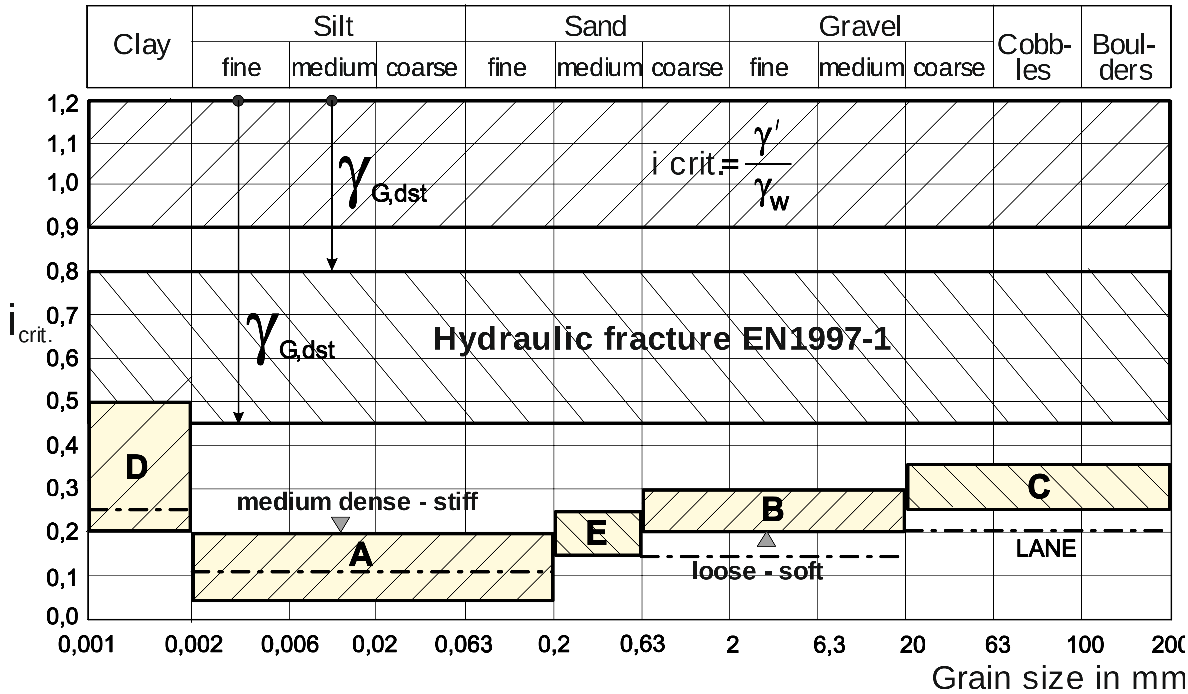

Figure 13. Critical hydraulic gradients for hydraulic fracture (internal erosion) (Brandl and Hofmann, 2006); icrit. depends not only on grain size distribution and density/stiffness but also on flow pressure; γG,dst = partial safety factor for permanently unfavourable effects.

For the assessment of internal erosion or piping, different approaches (e.g. by Chugaev, Bligh, Lane, Müller-Kirchenbauer, Weijers and Sellmeijer, Witt et al., etc.) are used to determine the critical hydraulic gradient in practice. However, due to the interaction of several hydrodynamic mechanisms, these criteria typically apply to very limited types of soils and mostly to homogenous soil conditions.

3. MAESURES AGAINST HYDRAULIC FAILURE

Hydraulic failure as an effect of underseepage may be prevented mainly by two permanent measures landside of a dyke or flood protection dam by:

- Filling of berms, thus displacing the possible starting point of internal erosion or piping farther away from the structure, and decreasing the hydraulic gradient at this point. Such berms should be designed and constructed such that they work simultaneously as access ways/roads for quick and easy dam defence in the case of severe floods.
- Installing pressure relief drainage systems in form of trenches, relief stone columns or relief wells.

Another emergency method, how to increase the stability against inner erosion is to reduce the hydraulic gradient by raising the water level at the landside in local reservoirs (Fig. 7). This method represents an emergency measure by placing sandbags around the erosion crack and is often used after recognizing local hydraulic fracture in the initial stage.

A filter stable berm compensates through its counterweight the hydrostatic pressure beneath the blanket (Fig. 14a) and prevents hydraulic failure by seepage or uplift, or by internal erosion and piping. When seepage through or beneath the dyke occurs, a free water outflow must be allowed; clogging would be counterproductive. Otherwise an excessive pore-water pressure could cause a sudden failure. Filter stable berms (filter geotextiles covered with sand, gravel, or other granular material) are often used as an emergency measure, when seepage occurs.

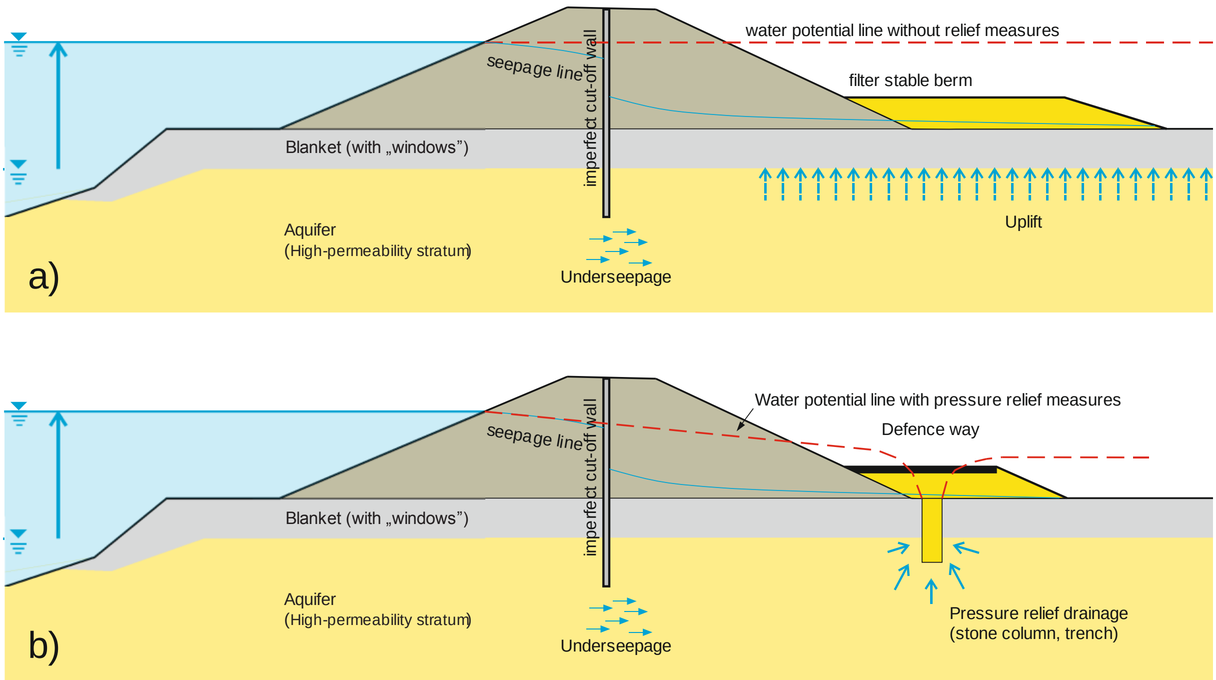

Figure 14. Permanent measures against hydraulic failure caused by underseepage of flood protection dykes: a) Filter stable berm as a counterweight; b) Relief drainage columns or trenches.

In many cases berms merely move the hydraulic problem farther away from the dyke or dam, and retrogressive internal erosion may finally reach it in the long term (after several floods). Boiling and internal erosion have been observed up to 20 to 50 m away from dykes and dams (Fig. 7), even though they were only 3 to 6 m high. Moreover, wide berms are frequently not possible under confined space conditions; therefore relief drainages are preferred under these circumstances.

Pressure relief systems are linear or punctual drainage elements with high permeability and variable embedment length into the aquifer (Fig. 14b). They are an integral part of the dyke at the landside embankment toe and connect the terrain surface with the permeable soil layer. This hydraulic connection allows a controlled pressure relief during floods, while water can freely discharge the drainage at the landside dam toe. To prevent the drainage element from clogging due to fine particles transport caused by suffusion of the aquifer, they must be wrapped into a filter stable geotextile.

Filter protection of berms is generally provided by the use of non-cohesive granular material (natural soil) that fulfils adequate design criteria for filter materials. Filter geotextiles have been used increasingly since the early 1970s. Common filter criteria for soils are from Terzaghi and Sherard, and for geotextiles from Holtz et al. (1997), Giroud (2003, 2010), and Heibaum et al. (2006). All criteria have particular limitations, whereby non-cohesive and cohesive soils have to be distinguished. While two criteria are sufficient for granular filters (the permeability criterion and the retention criterion), four criteria are required for geotextile filters (Giroud 2010): the porosity criterion and the roughness criterion also have to be considered.

The roughness criterion is important especially for the installation (temporary state), whereas the hydraulic criteria influence the long-term behaviour of relief drainages. However, an increase of the strength parameters leads to a reduction of the nonwoven geotextile permeability. Therefore, a compromise between both criteria has to be found when selecting suitable geotextiles.

3.1 RELIEF TRENCHES

A pressure relief trench is a longitudinal drainage of coarse gravel fill material wrapped in a filter stable geotextile at the landside dyke toe. Wider trenches may consist of granular fill material with horizontally differing grain curves as separating two- or three-stage filters (also in the base). They usually do not penetrate into the permeable aquifer, but rest mostly on top of it. However, a hydraulic connection must always be ensured so that the pressure relief is possible. At the same time, the groundwater must always freely discharge the relief drainage and is mostly conducted within the trench or flows into the adjacent hinterland.

The trench geometry is based on the dimensions of excavating tools. According to experience, the minimum drainage width is about 60 cm. The embedment depth is influenced by the soil characteristics of the subgrade as well as by the groundwater level. Mostly, a hydraulic excavator with a backhoe carries out the excavation. However, trenches excavated in very soft soil collapse immediately before geotextiles and the fill material can be placed. The installation of trussed retaining panels would be too expensive. These problems could be overcome by installing of relief granular columns coated with a filter geotextile.

3.2 RELIEF STONE COLUMNS (“GRAVEL PILES”)

Jacketed (coated) stone or gravel columns have been installed in Austria since 1992. At first they were used mainly for drainage purposes, for instance as drainage walls to improve the stability of old flood protection earth dams, dykes and levees, resp. This method has significant construction advantages over conventional drainage trenches in loose or soft soil.

Pressure relief stone columns are cylindrical pressure relief elements at the landside dyke or dam toe, which penetrate through the low permeable blanket and embed into the high permeable ground layer (aquifer). They are made of high permeable fill material (usually rounded grains, clean: 16/32 mm; permeability factor: $k \geq 1 \times 10^{-2}$ m/s) that is wrapped with a nonwoven geotextile. This geotextile filter has to fulfill the separation and the filter function.

The common diameter of these “gravel piles” is about 60 cm to 70 cm, but it can be easily adapted to the required relief effect. The installation is carried out with a designed center-to-center distance by means of vibroflotation or auger drilling method (Fig. 15).

The conventional top-feed process of the vibro technique is not suitable for jacketed granular columns. In this case, the sophisticated vibroflotation technique with bottom-feed vibrators is required. The main advantage of this method is that the vibrator remains in the ground during installation, making the technique ideal for unstable ground and high groundwater levels. The granular material is discharged from skips into the chamber at the top of the vibrator and placed at depth. To avoid geotextile damage, the vibrator is sometimes first lowered without the geotextile sleeve into the ground to displace soil.

The classical auger technique was modified for the purpose of geotextile coated columns installation. In this case, the bore hole is cased during the whole excavation to the final depth. This allows the installation of a geotextile sleeve into the casing and filling it with gravel fill material. Finally, the casing is withdrawn. The advantage of this method is the possibility of visual assessment of soil parameters as well as of the aquifer horizon that has a significant influence on the column’s drainage length.

Figure 15. Installation methods of relief drainage columns wrapped in nonwoven geotextile. (left: vibroflotation method; middle: rotary drilling method with casing; right: “infinite” relief column system at the dyke toe)

Both techniques allow a rather flexible adaptation of the element length to the local soil profile. Thus, each stone column forms a part of the relief system along the dam/dyke section and is mostly connected to a longitudinal drainage on top of the column heads. This trench collects the water from relief columns and brings it to pump stations.

Figure 16. shows a cross-section through a new flood protection dam after removal of the old one, which had been destroyed by a severe flood. The coated gravel columns usually exhibit a spacing between 2.0 and 8.0 m, depending on local factors (geotechnical and ecological parameters, infrastructure, risk potential, etc.).

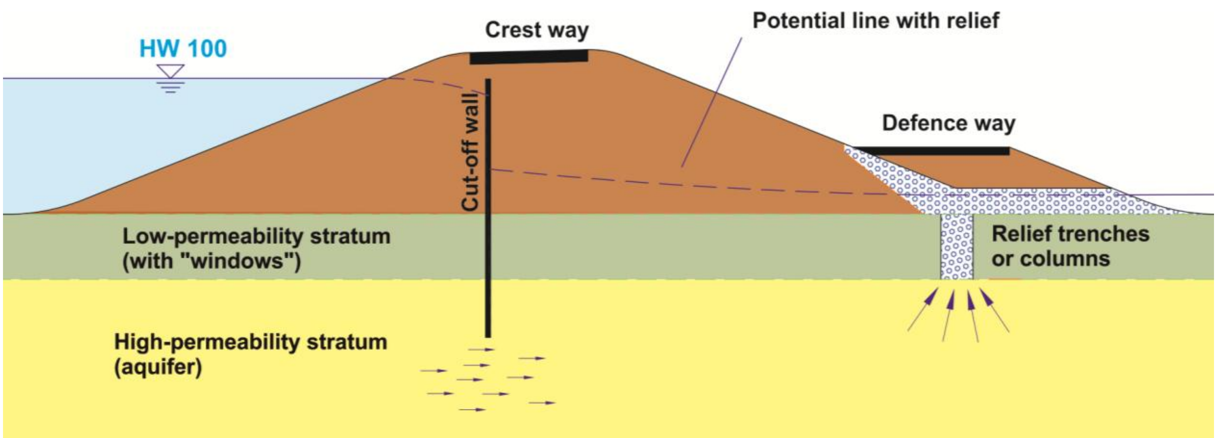

Figure 16. Standard cross-section of a new 75 km long flood protection dam in Austria. Widely reuse of old fill material from removed dykes. Cut-off wall works also as a barrier against beavers.

4. ANALYTICAL APPROACH FOR THE ASSESSMENT OF PRESSURE DISTRIBUTION DUE TO DYKE UNDERSEEPAGE

An uncontrolled underseepage due to seasonal floods represents a high risk for dykes and levees in terms of excessively pore pressure beneath the blanket. This overpressure can lead to hydraulic failure of the subgrade at the dyke toe or in adjacent hinterland. In order to assess the pressure distribution beneath the impervious/semipervious blanket and to determine appropriate measures, a levee underseepage analysis is required. This can be performed by numerical simulations. However, the accuracy of numerical results depends strongly on idealizations and the assumptions made. A simple method of assessing the pressure distribution can be done by analytical approaches. They provide exact solution for simplified flow systems with defined boundary conditions. Based on these results first estimations of the hydraulic behavior of the subsoil due to groundwater flow are possible.

The first analytical approaches for the determination of the pressure potential due to a steady-state flow in a homogeneous aquifer were published by Bennett (1946) and later adapted by U.S. Army Corps of Engineers (1956, 2000). Today, the blanket theory is commonly used in the practice. In the following, analytical approaches for a steady-state flow model with a dam on semipervious blanket and homogenous aquifer will be exemplified based on Benett (1946), Mehaan (2012), Szabo (2017).

4.1 DAM MODEL ON IMPERVIOUS BLANKET WITH A HOMOGENOUS AQUIFER

Underseepage of dams on impervious blankets (e.g. low permeable, silty-clayey soil top layer) can be mathematically described by one-dimensional flow in a porous media with an impervious horizontal boundary at the top and the bottom of the flow field. The inflow and outflow boundary conditions are also important for the estimation of the pressure distribution.

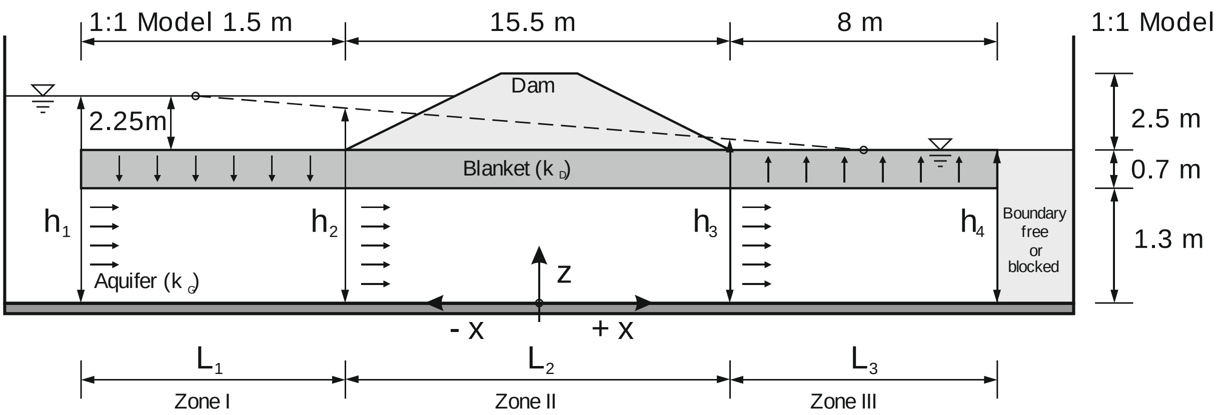

Figure 17. Idealized cross-section of dam on a semipervious/impervious blanket above a homogenous aquifer (flow field) with free inflow boundary and free or blocked outflow boundary (Szabo, 2017)

The steady-state flow in a confined aquifer is defined by following differential equation:

$$\frac{\partial}{\partial x}\left(k\ m\frac{\partial h}{\partial x}\right)=0 \qquad (1)$$

The general solution of the Eq. 1 results for an aquifer with a constant thickness *m* and permeability *k* into:

$$h(x)=C_1\ x+C_2 \qquad (2)$$

The mathematical solution results into a linear pressure distribution between the two water potentials (h_1 and h_4; see Fig. 17) at the inflow and outflow boundary of the flow field with a length L_i, where a free groundwater flow is defined.

$$h(x)=h_1+\frac{h_4-h_1}{L_1+L_2+L_3}\left(x+L_1+\frac{L_2}{2}\right) \qquad (3)$$

In the case of blocked water flow at the outflow boundary the groundwater flow will stagnate. Considering such conditions, the water potential in the whole flow field corresponds with the water pressure at the inflow boundary (pressure potential h_1). This flow behavior can be described by following equation:

$$h(x)=h_1 \qquad (4)$$

4.2 DAM MODEL ON SEMIPERVIOUS BLANKET WITH A HOMOGENOUS AQUIFER AND FREE INFLOW AND BLOCKED OUTFLOW BOUNDARY CONDITION

Semipervious blankets above a pervious foundation (aquifer) can be found relatively often in the nature. They are characterized as soil layers with low permeability. If the water pressure acting on the soil layer is high enough, seepage in vertical direction may occur.

In order to describe the water pressure distribution for a dam model with homogenous aquifer on a semipervious blanket, the flow field must be divided into different zones according the fragment theory. If the dam is assumed to be impermeable, the aquifer zone beneath the dam corresponds to a confined aquifer between two impervious boundaries (see chapter 0). In contrast, leaky-aquifer conditions prevail in foreland as well as in hinterland of the dam. In such case, the water flows in horizontal direction through the aquifer and in vertical direction through the blanket. The mathematical solution for the whole model is derived from the combination of the elementary solutions for each fragment (zone) considering the element boundary conditions.

Based on the fragment method, the pressure distribution is determined for the foreland zone considering the seepage through the blanket. The general equation of the steady-state flow is:

$$\frac{\partial^2 h}{\partial x^2} - \frac{h-h_1}{\lambda^2} = 0 \tag{5}$$

The parameter λ is the leakage factor, which defined the water flow in vertical direction through the blanket.

$$\lambda = \sqrt{\frac{k_G\ m\ t}{k_D}} \tag{6}$$

The general elementary equation for the leaky-aquifer in the zone I is:

$$h(x) = h_i + C_1\, e^{\frac{x+L_1+\frac{L_2}{2}}{\lambda}} + C_2\, e^{-\frac{x+L_1+\frac{L_2}{2}}{\lambda}} \tag{7}$$

Thus, the equation for the determination of the pressure distribution in the zone I can be determined under consideration of the boundary condition (water potential h_1 and h_2; see Fig. 17) and the flow through the semipervious blanket to:

$$h_I(x) = h_1 + (h_2 - h_1)\,\frac{\sinh\left(\frac{2x+2L_1+L_2}{2\lambda}\right)}{\sinh\left(\frac{L_1}{\lambda}\right)} \tag{8}$$

In the zone II (see Fig. 17), confined aquifer conditions prevails in the flow field beneath the dam. The pressure distribution can be calculated according to the general Eq. 2 considering the water potential h_2 und h_3 at the fragment boundaries.

$$h_{II}(x) = \frac{h_2+h_3}{2} + \frac{(h_3-h_2)}{L_2}\,x \tag{9}$$

In case of a blocked outflow in hinterland, no-flow condition has to be considered at the landslide model boundary of the fragment III. This results into a stagnation of the flow. Therefore, the flow behaviour as well as the pressure distribution depends only on the leakage through the semipermeable blanket in the vertical direction. The pressure distribution in the zone III (hinterland) can by calculate with following equation:

$$h_{III}(x) = h_4 + (h_3 - h_4)\,\frac{\cosh\left(\frac{-2x+2L_3+L_2}{2\lambda}\right)}{\cosh\left(\frac{L_3}{\lambda}\right)} \tag{10}$$

The groundwater flow rate over the model width B can be calculated as a first derivation of the Eq. 10 with following solution:

$$Q_{III} = T\,\frac{(h_3-h_4)}{\lambda}\tanh\left(\frac{L_3}{\lambda}\right) = k\,m\,\frac{(h_3-h_4)}{\lambda}\tanh\left(\frac{L_3}{\lambda}\right) \tag{11}$$

Assuming that the flow rate *Q* must be constant at the fragment boundaries for continuity reasons, the unknown piezometer heads h_2 and h_3 can be solved by equating the flow rates for the adjoining zones $Q_I = Q_{II}$ or $Q_{II} = Q_{III}$.

$$h_2 = \frac{\lambda\, h_1 + h_1\, L_2 \tanh\left(\frac{L_3}{\lambda}\right) + \lambda\, h_4 \tanh\left(\frac{L_1}{\lambda}\right)\tanh\left(\frac{L_3}{\lambda}\right)}{\lambda + L_2 \tanh\left(\frac{L_3}{\lambda}\right) + \lambda \tanh\left(\frac{L_1}{\lambda}\right)\tanh\left(\frac{L_3}{\lambda}\right)} \tag{12}$$

$$h_3 = \frac{\lambda\, h_1 + h_4\, L_2 \tanh\left(\frac{L_3}{\lambda}\right) + \lambda\, h_4 \tanh\left(\frac{L_1}{\lambda}\right)\tanh\left(\frac{L_3}{\lambda}\right)}{\lambda + L_2 \tanh\left(\frac{L_3}{\lambda}\right) + \lambda \tanh\left(\frac{L_1}{\lambda}\right)\tanh\left(\frac{L_3}{\lambda}\right)} \tag{13}$$

This exemplified analytical solution of underseepage beneath a dam on a semipervious blanket describes simplified method for the determination of the pressure distribution in a homogeneous leaky-aquifer with steady-state groundwater flow with blocked outflow at the hinterland boundary. Such flow conditions are characteristic for the performed physical model tests, which are described in the following chapter.

Figure 18 shows the pressure curves for different permeability factor from k = 1 x 10^{-4} m/s to k = 1 x 10^{-9} m/s of the semipermeable blanket in the 1:1 model (chapter 5.1.2). In case of high leakage factor, the hydrostatic pressure in the aquifer is relieved due to the vertical flow through the blanket. If the permeability of the blanket is low, confined aquifer condition can be found (see chapter 4.1).

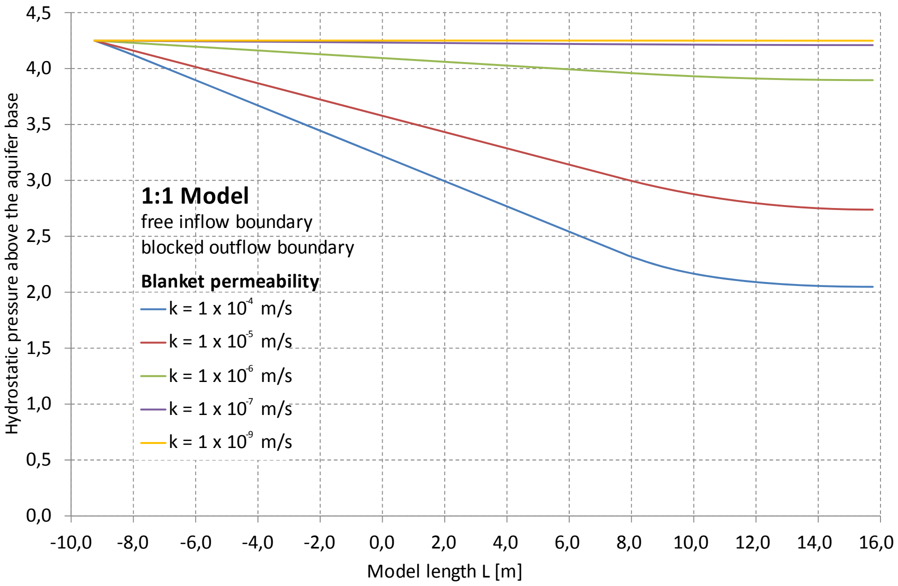

Figure 18. Pressure curves depending on leakage factor of the semipervious blanket in a flow field with blocked outflow boundary

5. MODELLING OF RELIEF DRAINAGE BEHAVIOUR

Until now the design of relief measures (drainage columns or trenches) was based on rather insufficient basic principles, strong simplifications and idealizations. For the quantification of the water outflow from relief columns as well as for a pressure assessment beneath the blanket only assumptions based on numerical models are in use. These approaches allow indeed comparative calculations of the quantity of seepage through and under the dyke.

Consequently, model tests on dykes including the subgrade (blanket and aquifer) are the best solution to assess the relief behaviour and to quantify the water outflow from relief elements during hydraulic loading. Experimental tests performed under laboratory conditions allow a higher degree of reliability than mere numerical simulations. But based on the results from physical modelling an exact calibration of numerical models can be performed.

5.1 EXPERIMENTAL MODEL TESTS

In the first phase of experimental underseepage studies small-scale (1:10) model tests were carried out at the Vienna University of Technology, Institute of Geotechnics (Fig. 19). The tests results were used for the design of an experimental facility, which served for 1:1 underseepage model tests (Fig. 22).

5.1.1 Small-scale modelling

The small-scale dam model represents within a glass channel a vertical cross-section of a dyke including the subgrade in a scale 1:10. Thus, the channel walls form an impermeable boundary condition for the investigation of relief elements behaviour due to underseepage.

The 25 cm high dam model is founded on a two-layer strata. Beneath the silty-clayey blanket ($k = 10^{-9}$ m/s) with a thickness of about 10 cm follows a homogenous aquifer layer of coarse sand with a thickness of about 18 cm. In addition, the dam has an imperfect cut-off wall (i.e. with underseepage), which slightly embeds into the aquifer.

The pressure distribution beneath the blanket due to underseepage and pressure relief was measured by means of piezometers pipes in the longitudinal and transversal model axis. During the tests different types (trench, stone columns and wells) and geometries of relief elements (width, diameter, embedment length, etc.) were studied. In addition to pressure measurements also discharge from relief elements was quantified.

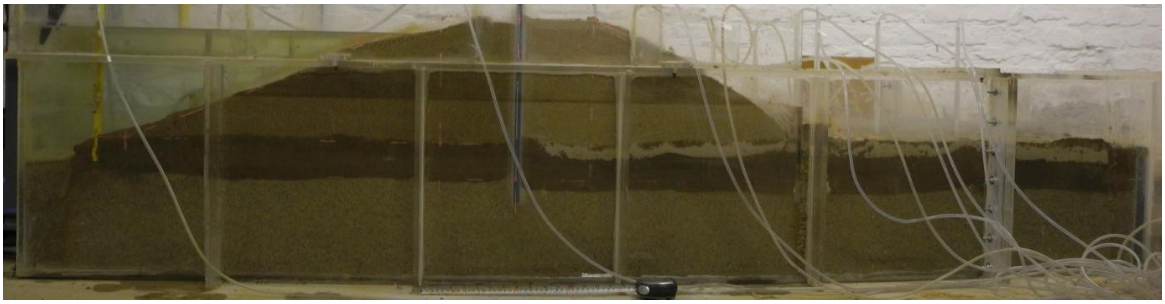

Figure 19. 1:10 small-scale dam model on two-layer subgrade for pressure relief behaviour study

Figures 20 and 21 show the pressure distribution lines beneath the blanket for test series with single and multiple relief elements depending on its permeability and embedment length into the aquifer. Both – the relief gravel column as well as the relief well – lead to a significant reduction of the hydrostatic pressure on the waterside. If no relief measures would be installed, due to the blocked outflow boundary the hydrostatic conditions on the landside would became equal to the hydrostatic head on the waterside. It can be also seen that the element permeability significantly influences the pressure reduction. A similar behaviour can also be observed in terms of embedment length. However, the effect of embedment depth on the pressure relief, as supplementary investigations with the calibrated numerical model show, is also strongly influenced by parameters of the aquifer.

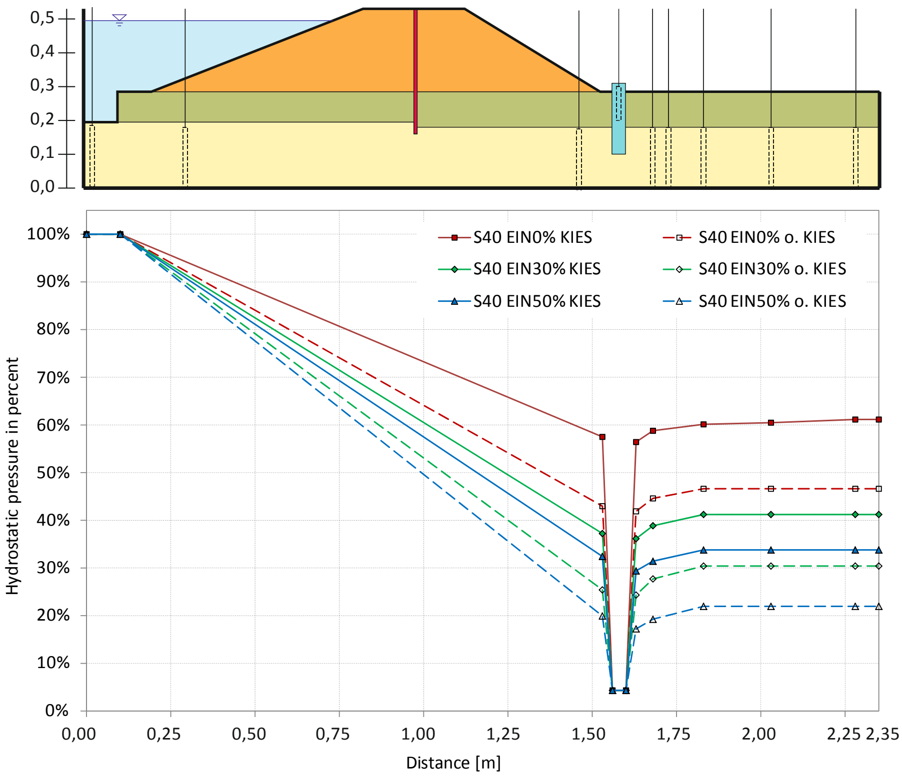

Figure 20. Pressure lines along the longitudinal axis of the 1:10 model with a single relief column depending on column length and permeability. (S40 = single column system with Ø 40 mm; EIN = column embedment in percent; KIES /o. KIES = with / without gravel; HW = water level)

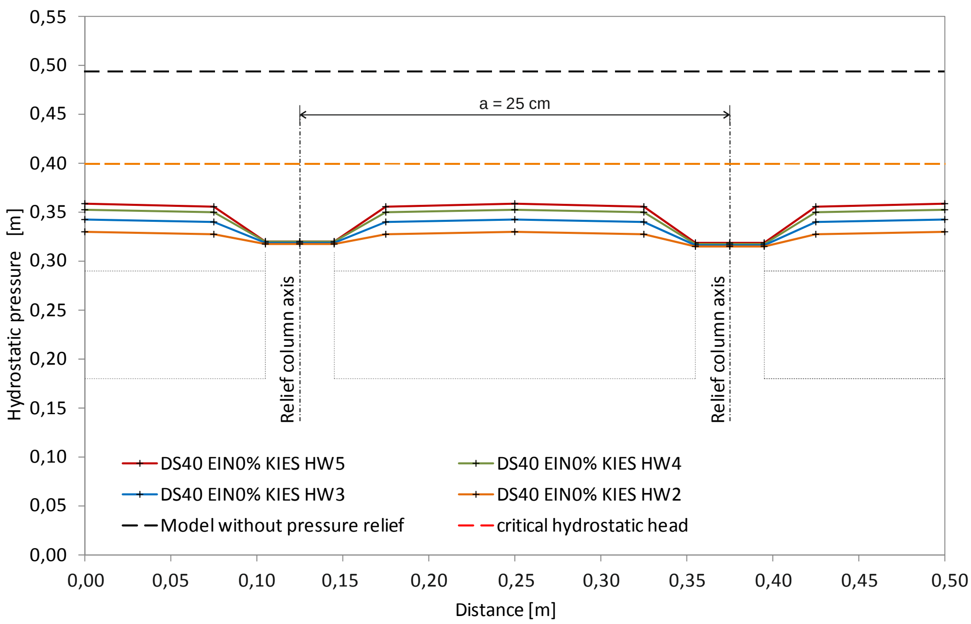

Figure 21. Pressure lines along the transversal axis of the small-scale model with a multiple relief column system. (DS40 = multiple column system with Ø 40 mm; EIN = column embedment in percent; KIES = with gravel; HW = water level)

5.1.2 Large-scale modelling

Based on the small-scale modelling, a large scale test facility (1:1) was built to study pressure relief drainages. The ground plan area of the reinforced concrete box was 25 x 4 m with a constant wall height of 5 m (Fig. 22).

The 1:1 dam model represents a 4 m wide and 25 m long cross-sectional model of rehabilitated dyke at the river March together with a two-layer subgrade (Fig. 23). The homogenous dam had a height of 2.5 m and the slope ratio of 1:2.5. For the dam sealing a silty-clayey core was installed, which penetrated the clayey blanket (thickness of about 0.7 m) and embedded only few centimetres into the 1.3 m thick aquifer layer of sandy gravel.

For the pressure relief observation due to the controlled underseepage, a measuring system was used, which allowed a continuous recording of the pressure potentials beneath the blanket as well as a time-synchronous recording of the water discharge from the relief gravel column.

Figure 22. The large scale test facility (length 25 m, height 5 m und width 4 m) for investigating the underseepage of dykes and levees in a 1:1 scale

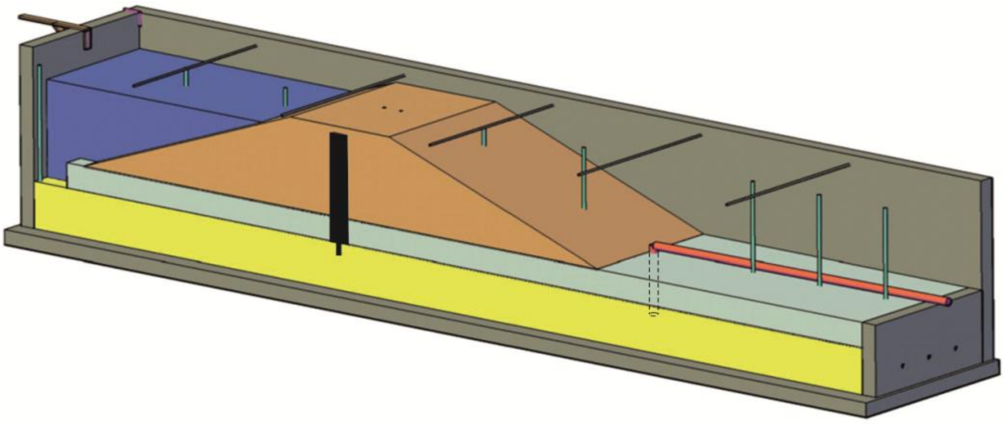

Figure 23. Schematic drawing of the 1:1 dam model on a two-layer foundation.

In numerous series of measurements with varied column parameters, the relief behavior of drainage columns could be confirmed despite certain anomalies. An example of the pressure relief behaviour due to a single relief gravel column (Ø 600 mm) without embedment into the aquifer shows figure 24. The pressure reduction at the beginning of the measuring profile was partly caused by possible local clogging of the aquifer in combination with the relieving effect through the relief column. This resulted in a reduction of 50% in the hinterland. The maximum discharge for the highest water level was about 0.95 l/s per relief column with 4 m spacing.

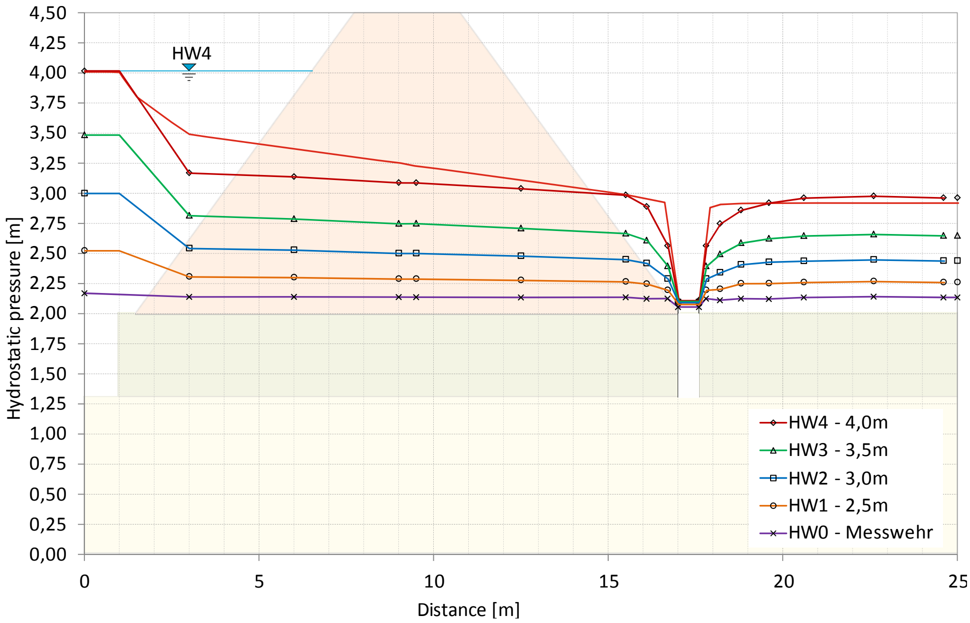

Figure 24. Pressure distribution lines due to a single relief drainage column of 600 mm diameter and without embedment into the aquifer in a 1:1 model. (HW = water level)

5.2 NUMERICAL MODELLING

The numerical modelling of underseepage is an important contribution to the description of the pressure relief behaviour and is often used for the design of such relief systems. In order to examine the general application of numerical results, a model calibration with the GGU Software was carried out based on the model tests. For this purpose, the small-scale model test results were used, because the 1:10 model had a higher degree of homogeneity as the 1:1 dam model. The calibration was first carried out for a fully penetrating relief trench by variation of the aquifer permeability and subsequently extended to a three-dimensional model with a single relief column (Fig. 25). After the calibration, numerous comparative simulations were performed for the small-scale model as well as the large-scale model. They verified the application of the calibrated three-dimensional numerical model for further studies of different parameters beyond the limits of the physical modelling.

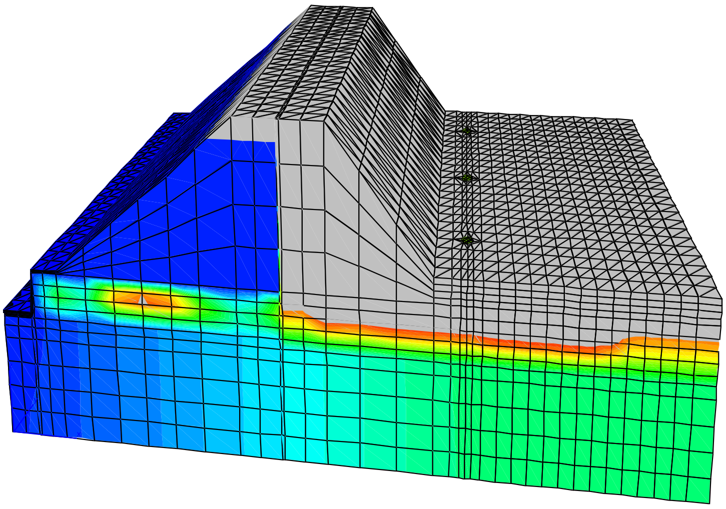

Figure 25. Three-dimensional calibrated dam model with “infinite” relief drainage column system

Figure 26 shows the pressure distribution beneath the blanket over the whole flow field of the calibrated numerical model due to a single relief drainage column. The hydrostatic pressure at the relief element spot falls to the minimum and then increases to a constant value in hinterland that is primarily influenced by the outflow boundary condition. This was defined as a blocked boundary and correspondents with the physical model, where the outflow was prevented by the impermeable channel walls.

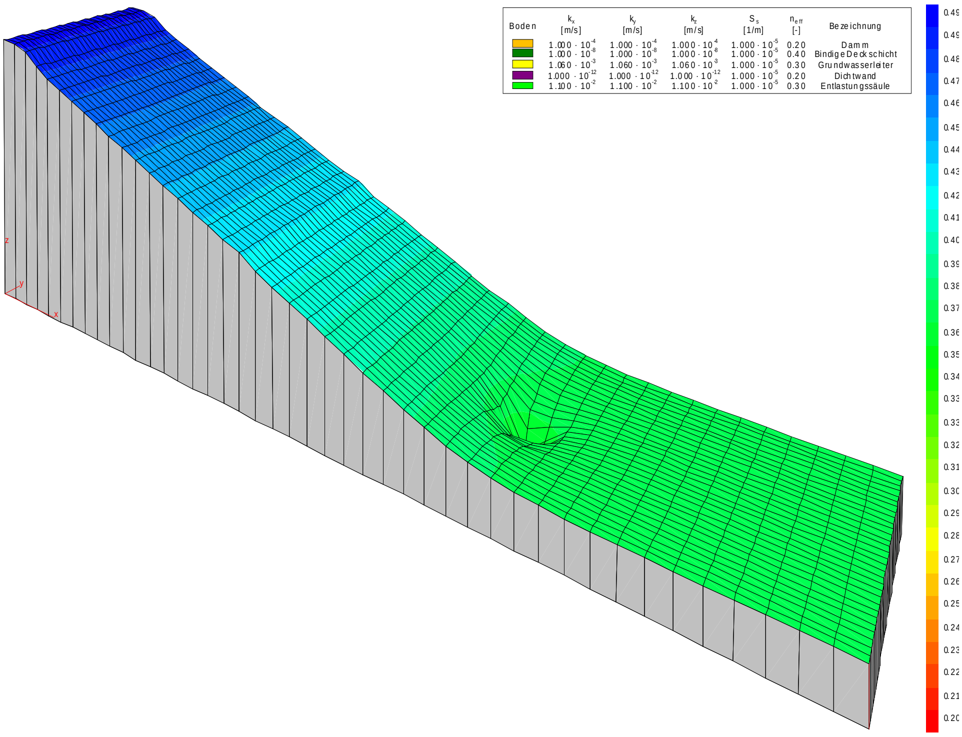

Figure 26. Pressure distribution in the aquifer due to a single relief drainage column

In addition, the parameters of the relief column (diameter, permeability, embedment length, spacing of a multiple column system), the influence of the model geometry (distance between the river and the dyke, thickness of the subgrade, etc.) as well as different permeability factors of the aquifer were studied (Fig. 27). Based on these calculation results, recommendations were derived for the practical use of relief columns. It was found that the permeability of the relief drainage fill material significantly affects the relief behaviour. The effect of the embedment length is also strongly influenced by the column permeability in combination with the thickness of the aquifer. For the standard relief drainage column (Ø 600 mm and k ≥ 1 x 10^{-2} m/s) and an aquifer thickness of about 10 m only little differences in the pressure reduction depending on the embedment length could be found.

Figure 28/left shows the midway pressure for an infinite relief drainage column system in a homogenous flow field with defined geometry and blocked outflow boundary as a function of column spacing and aquifer permeability. In the diagram, also the discharge (Fig. 28/right) associated with the achieved pressure reduction can be achieved.

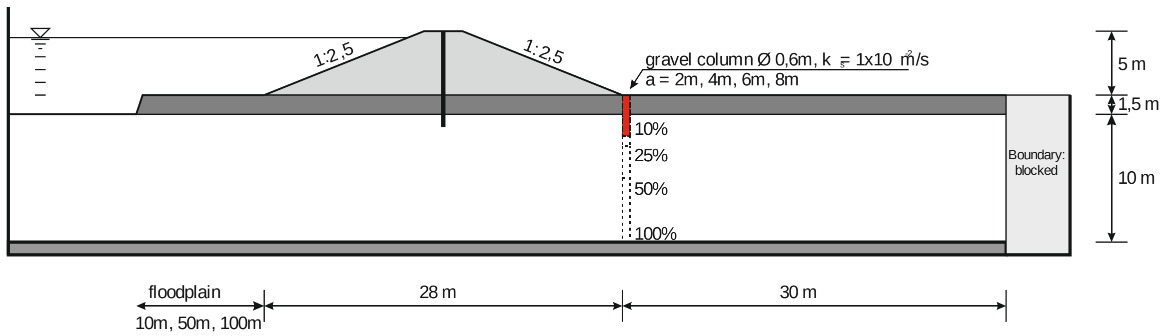

Figure 27. Cross-section of a dyke on two-layer subgrade - calculation profile for parametric studies of the relief behaviour due to an infinite relief column system (a = spacing of relief columns)

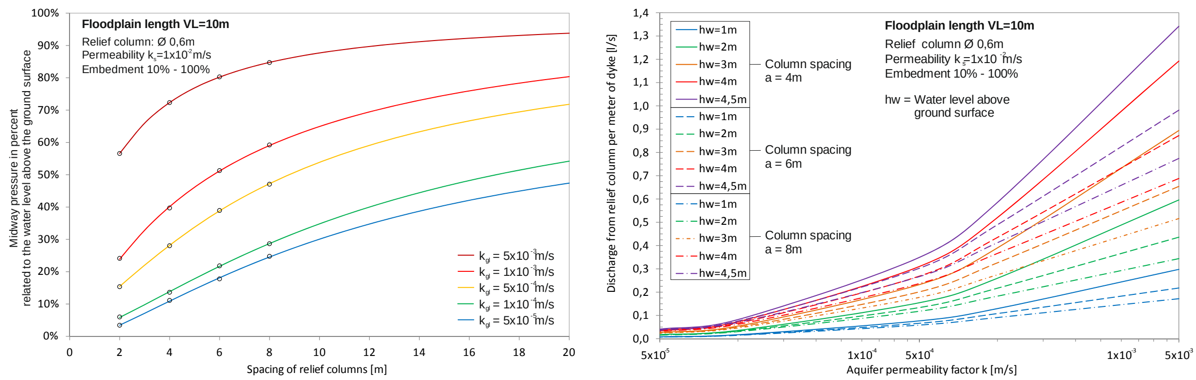

Figure 28. Results of parametric studies depending on column spacing and aquifer permeability (definition of model parameters see figure 27): Diagram on the left: midway pressure for a relief gravel column system. Diagram on the right: calculated discharge from relief drainage columns.

6. CONCLUSIONS

Underspeepage of dams, dykes or levees may lead – especially in long-term – to erosion of the ground due to (subsequent) floods. The pressures that develop landside of the dyke during floods may cause heaving or uncontrolled rupture of the near-surface blanket resulting in a concentration of seepage flow, often accompanied by piping, and potential dyke failure. Such a hydraulic failure develops mostly rather inconspicuously; therefore it is often underestimated in practice. Erosion criteria can be used to describe the critical state for different soil types found during ground investigation. For the control of underseepage pressures in the aquifer and for hydraulic failure prevention stabilizing measures at the landside dyke or levee toe zone are essential. Filter stable berms, relief drainage columns or trenches, sometimes also relief wells have proven very successful.

If pressure relief measures are required, the selection of a suitable relief system has to consider the geotechnical, hydraulic and local conditions. For the design of the selected relief system (drainage columns, trenches, wells) numerical methods are commonly used in practice. A technically and economically optimized design is only possible, if hydraulic boundaries of the flow field (e.g. long-term data from groundwater observation) and the permeability of the aquifer (field and laboratory tests) can be determined as precisely as possible, as could be experienced in the performed experimental and numerical studies. Finally, monitoring of existing projects contributes significantly to continuous calibration and sophistication of numerical modelling and practical design.

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