UNSTEADY SEEPAGE FLOW THROUGH LEVEES WITHIN THE LOWER RHINE REGION

R. HASELSTEINER AND B. ERSOY
Bjoernsen Consulting Engineers, Koblenz, Germany

ABSTRACT

Levees are a crucial backbone of flood protection measures along rivers. Whilst reservoir dams are always hydraulically loaded levees only have to sustain water loads during major floods.

In German design practice usually the steady state seepage conditions are applied for the check of stability and serviceability. According to the technical code for flood protection structures in Germany, DIN 19712, also the consideration of unsteady seepage conditions is allowed.

Depending on the specific zoning, the duration of a flood incident, the applied soil materials within the levees in respect to permeability, and the absolute impoundment height levees may show steady state to nearly no seepage intrusion. Hence, the factual safety level of levees is differing especially in consideration of the real seepage conditions which might occur.

Case studies are presented for both levees with conservative design basics and others for which design flood hydrographs lead to steady seepage flow conditions. Additional to the numerical Finite Element modelling (FEM) results of selected case studies the theoretical basics and input date will be explained and described, e.g., in consideration of crucial specialist literature such as Scheuermann (2005) and Haselsteiner (2007).

1. INTRODUCTION

Flood incidents during the last decades showed the vulnerability of flood protection structures in Germany. The flooding of polders was caused by overtopping. But also failures occurred due to interacting geotechnical and geo-hydraulic mechanisms. Major destructive floods occurred in 1993 and 1995 at the Rhine and 1999, 2002, 2005, 2011 and 2013 at the Elbe and Danube Rivers of which 2002 and 2013 were the outstanding incidents showing damage of approx. 10 billion € each.

In addition to flood retention in reservoirs and in the subsoil within the catchments levees represent the backbone of many flood protection concepts of the major rivers in Germany. Especially, along the Lower Rhine River the existing flood risk is extraordinary high since the population density is highest in Germany and the region is located at the lower section of the river so that potential flooding affects a relative large hinterland. Additional, the coal industry caused mining subsidence so that the levees needed to be heightened during the coal exploitation and the ongoing subsidence processes in order to guarantee flood safety of the Rhine River which kept its original elevation and course.

Due to the existing flood risk different measures were initiated by the responsible levee owners such as a due rehabilitation program and the construction of flood retention dams along the complete Rhine River in order to guarantee the stability of the levees and reduce the flood levels, respectively (see ICPR, 2012).

Usually levees are designed for steady state seepage conditions especially for levees along the large rivers in Germany. This also provides a quite high safety level, if sealings are incorporated in dam bodies or the dike body itself shows a low permeability. For high permeable dike materials also short floods may lead to considerable seepage conditions which may also results in an unfiltered exit which is considered to be the first step to backward erosion (see Fell & Foster, 2005; Fell et al, 2005).

2. FLOODS AND LEVEES OF THE LOWER RHINE

The Rhine River was stroke by floods in the years 1993 and 1995. The occurrence period for these floods were depending on the river section was T = 50 to 70 a and caused huge harm mainly within the river near cities such as Cologne and Koblenz. Major historic floods occurred in the years 1342, 1882/83, 1925/26, etc. The highest recorded water levels and discharges occurred in the years 1926 and 1883, but it cannot be excluded that the "Magdalenen Flood" in July 1342 was the worst flood which was documented in Germany since the middle age and which affected most of the large rivers in Middle Europe including Rhine, Danube, Main, Mosel, Elbe, Weser, etc.

The city of Koblenz is located at the Rhine belongs to the Middle Rhine whereas Cologne is already located within the Lower Rhine region which comprises the section between the settlements Rolandswerth close to Bad Honnef, where Rhineland-Palatinate ends and North Rhine-Westphalia begins, to Lobith in the Netherland a few kilometers downstream of the city of Emmerich which is still on German territory. The middle Rhine expands from Rhine-km 642 to 857 (IPCR, 2001). The Lower Rhine River is completely located within the state North Rhine-Westphalia.

The permission and supervision authorities for the Rhine are reflected by the district councils of Cologne in the south and Duesseldorf in the north-west. The other three of in total five district councils are responsible for tributaries of the Rhine such as the Emscher, Ruhr or Lippe Rivers.

Where mining subsidence processes occurred the levees along the Lower Rhine needed to be heightened while the elevation of the ground level decreased so that large levees formed with heights over 15 to 20 m. Hence, also the dike bodies show large extensions by installing flat slopes and wide crests in favor of a stable static behaviour.

With large levees the hinterland drainage works as well as the groundwater control works as a unavoidable consequence of the mining activities play a major role within all levee projects. Large pumping stations and the corresponding pressure pipes needs to be installed for draining natural rivers or waste water systems towards the Rhine River. These structures and other infrastructural items, such as roads, transport pipelines for gas or oil, electricity lines, etc., need to cross the levee alignment safely corresponding to the restrictions and rules in DIN 19712 and DWA-M 507 Part 1.

3. DESIGN SITUATIONS, CODES AND GUIDELINES

A levee according to DIN 19712 is an embankment dam along a river which is protecting against floods and which is only temporarily impounded by a flood water level. This is the difference to an embankment dam according to DIN 19700 which is valid for dams in general as part of, e. g., flood retention dams or other permanent reservoirs. For “dry, green” reservoirs of flood retention dams the conditions and requirements are comparable to those for levees (compare DIN 19712 and DIN 19700-12).

The district councils prepared levee ordinances which are obligatory for the construction, operation and supervision of levees in the districts. Whereas Cologne and Duesseldorf do have their own levee ordinances the residual districts are referring to the existing ones in North Rhine-Westphalia.

In Figure 1 the cross section of the standard levee is shown as it shall be applied where feasible in consideration of the local conditions (see case study 1, section 5). The levee section is characterized by flat slopes, a crest road and a berm with the defense road both of which show a relative large width. The levee section shows a sealing as well as a drain and therefore is a 3-zone-levee as propagated also by the code DIN 19712/2013 and by the guideline DWA-M 507 Part 1. Where required special cross sections need to be applied, e. g., when a public road is located on the crest (see case study 2, section 6) or special materials had to be applied during construction (see case study 3, section 7).

The national design code for flood protection structures such as levees, DIN 19712, is already considering European harmonization efforts on the engineering sector and, therefore, uses permanent (P), temporary (T) and accidental (A) design situations and the partial safety factor design philosophy. The ordinary flood level belongs to the permanent design situation whereas crest water level belongs to the accidental design situations. Falling water level which is critical for the geostatic stability of the upstream slope is considered as permanent situation, the malfunction of sealings or drains are accidental situations.

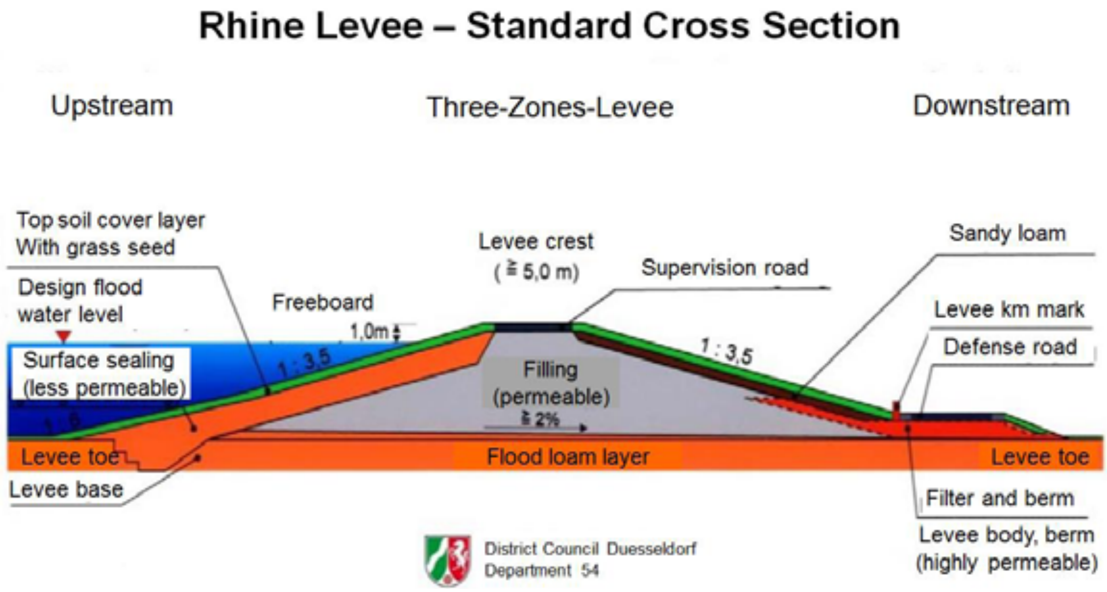

Figure 1 : Levee standard cross section propagated within the district of Duesseldorf (Source: District council Duesseldorf, Department 54) (taken and translated from Börger, 2016)

Usually, steady state seepage conditions are considered for the subsequent analysis of the geostatic or geo-hydraulic stability of a levee. This approach is usually on the safe side. An analytical evaluation of seepage through embankment dams with small heights as performed in Haselsteiner (2007) shows that the factual seepage conditions are depending on the impoundment period and the permeability of the concerned embankment and foundation materials and soils. As soon as sealings and/or low permeable materials or soils are integrated in a levee body showing a permeability which is less than, let's say, ks = 10-6 to 10-7 m/s steady state seepage conditions are very unlikely to occur during a flood event in middle Europe. The duration of a flood and the impoundment duration, respectively, are usually too short for causing a steady state seepage situation in low permeable embankments since durations of floods show days to maximum few weeks in combination with high flood water levels.

According to DIN 19712 the seepage conditions shall be modelled by using representative 2D numerical models in consideration of steady state seepage conditions. Unsteady seepage modeling is allowed for conditions when the occurrence of steady state conditions can be excluded. Unsteady seepage modeling can be used for the determination of the critical pore water pressure conditions for both the downstream and the upstream slope. For the upstream slope the falling water level as permanent situation is decisive whilst for the downstream slope the critical load case/situation is depending on the levee design/zoning and the corresponding design situation. For levees with sealings usually the malfunction of the sealing as accidental design situation shows critical stability conditions (see also Haselsteiner, 2007a, 2008).

4. SOIL PARAMETERS, FLOOD HYDROGRAPHS AND MODELING

4.1 General

For the modeling of unsteady seepage conditions additional information and data are required in comparison to steady state seepage modeling. The seepage flow through unsaturated soils is important as well as the setting of the initial saturation conditions for all soils in consideration of the environmental effects such as precipitation and drying. These drying and wetting effects within the unsaturated zone can be usually modeled by simple Van-Genuchten soil equation as explained in Haselsteiner (2007) for various soils and selected sealing types.

Flood hydrographs can be derived from measured floods, which are adjusted in consideration of design requirements regarding absolute water levels and durations, or from flow modeling. In order to be able to use the flood hydrographs for the purpose of seepage modeling water level data have to be prepared and evaluated for the specific levee section. Design flood hydrographs are frequently calibrated by real flood incidents. For design purposes the original hydrograph is transformed in concern of time and water level in consideration of safety aspects and uncertainties related to hydrology.

For the modelling of unsteady seepage conditions some modeling techniques and tricks should be applied which are shortly explained below.

4.2 Soil Parameters

For steady state seepage considerations usually the saturated conductivity, the pore content and the anisotropy factor are required for modeling. Sometimes also the flow in the unsaturated zone shall be considered, too, so that the unsaturated zone needs also to be modeled by allocating a degree of saturation to permeability by, e. g., using a coupled set of Van Genuchten and Mualem parameters as shown in Table 1. How detailed, scientific and elaborated the modeling approach

Table 1 : Geo-hydraulic soil parameters for selected, typical levee soils (taken from Haselsteiner, 2007)

			Drain gravel	Filling gravel	Subsoil gravels	Sand	Surface sealing	Flood loam
			Gravel, narrow graded	Gravels, sandy, silty		Sand, gravelly, silty	Silt, sandy, clayey	
	DIN 4020		G, st	G, s, u		S, g, u	U, s, t	
	DIN 18196		GE	GI oder GW		SE oder SU	UM	
Porosity	n	[-]	0.20 (0.15 - 0.32)	0.25 (0.15 - 0.32)	0.30 (0.25 - 0.35)	0.35 (0.30 - 0.38)	0.35 (0.28 - 0.37)	0.45 (0.39 - 0.56)
Natural moisture content / field capacity	$\theta_{r,FK}$	[-]	0.01 (< 0.03)	0.05 (0.03 - 0.06)	0.08 (0.05 - 0.15)	0.175 (0.15 - 0.28)	0.25 (0.25 - 0.40)	0.30 (0.25 - 0.40)
Residual mooisture content / Permanent wilting point	θ_r	[-]	0.00	0.00	0.00	0.05 (0.03 - 0.16)	0.05 (0.03 - 0.06)	0.05 (0.03 - 0.06)
Air pore content (0,1 - 0,5 $\theta_{r,FK}$)	θ_a	[-]	0.005	0.025	0.040	0.035	0.025	0.040
Saturated moisture content	θ_s	[-]	0.195	0.225	0.26	0.315	0.325	0.30
Saturated conductivity	k_s	[m/s]	$2\cdot10^{-2}$ ($1\cdot10^{-0}$ - $1\cdot10^{-3}$)	$5\cdot10^{-4}$ ($1\cdot10^{-2}$ - $5\cdot10^{-4}$)	10^{-3} ($1\cdot10^{-2}$ - $5\cdot10^{-4}$)	$2\cdot10^{-5}$ ($1\cdot10^{-3}$ - $5\cdot10^{-7}$)	10^{-7} (10^{-7} - 10^{-8})	10^{-6} (10^{-5} - 10^{-6})
Anisotropy factor	k_h/k_v	[-]	1 (2 - 30)	2 (2 - 30)	5 (2 - 30)	2 (2 - 30)	2 (2 - 30)	10 (2 - 30)
Capillary height	h_k	[m]	0.03 (0.03 - 0.05)	0.05 (< 0.20)	0.10 (< 0.20)	0.30 (0.20 - 0.40)	4.00 (1.00 - 5.00)	2.00 (1.00 - 5.00)
van Genuchten Parameter – Wetting	α_w	[1/cm]	0.200 (0.005 - 0.035)	0.050 (0.005 - 0.035)	0.070 (0.005 - 0.035)	0.060 (0.005 - 0.035)	0.050 (0.005 - 0.035)	0.060 (0.005 - 0.035)
van Genuchten Parameter – Wetting	n_w	[-]	4.0 (1.5 - 10)	5.0 (1.5 - 10)	5.0 (1.5 - 10)	2.5 (1.5 - 10)	2.0 (1.5 - 10)	2.0 (1.5 - 10)
van Genuchten Parameter – Drying	α_d	[1/cm]	0.150	0.040	0.060	0.030	0.010	0.020
van Genuchten Parameter – Drying	n_d	[-]	4.0	2.5	2.5	2.5	2.0	2.0
Mualem Parameter	L	[-]	0.75 (0.26 - 1.03)	0.80 (0.26 - 1.03)	0.80 (0.26 - 1.03)	0.60 (0.26 - 1.03)	0.50 (0.26 - 1.03)	0.50 (0.26 - 1.03)

may be approached, the driving parameter is always the saturated conductivity. This parameter shows also the highest sensitivity and variability so that, frequently, a range of permeability values is considered in the course of a sensitivity analysis. The sensitivity analyses performed by the authors in many projects always documented that the value of the permeability is evident and sensitive. All other parameters show less variability and also less sensitivity on the results.

The corresponding graphs for the saturation – suction and saturation – relative permeability relations are included in Haselsteiner (2007) or other publications of the authors such as Haselsteiner (2007a, 2008, 2011).

As aforementioned the author holds the opinion that for the ordinary task of the determination of seepage conditions within embankment dam structures with the purpose evaluating the stability the exact determination of the unsaturated soil characteristic is not critical but the determination and selection of an adequate permeability value concerning soils and materials. Hence, the presented parameters and characteristics of the five characteristic soils above may enable the engineer to model also various other soils on a solid basis just by adaptation of the kS-value.

For case study 3 a special set of parameters were applied to model the unsaturated conditions of black shale residual waste material which is characterized by a permeability of kS = 10-5 m/s and a saturated pore content of 0.18.

4.3 Flood hydrographs

As aforementioned a water level hydrograph is required as the upstream boundary condition and sometimes also for downstream for the simulation of the flood level. The absolute flood level and the duration of the flood event are evident. Usually the river bed itself shows a hydraulic capacity which corresponds to a 2- to 10-years flood so that the flood duration of the complete flood event itself is usually much longer than the impoundment period of a levee.

In Haselsteiner (2007) a number of 94 floods were evaluated which occurred on Bavarian Rivers during the period between 1988 and 2005 in order to characterize flood incidents including smaller floods before and after a main event. From these data a set of seven theoretical design flood hydrographs were developed which could be used for the design of a level in consideration of the specific local conditions such as catchment area and height of the levee.

As also aforementioned design flood hydrographs are frequently derived from gauge measurements and are processed in consideration of the design requirements. Important parameters are usually the peak of the flood and the duration which are determined by the rising and falling velocity of the water level for which Haselsteiner (2007) shown that it is likely that the falling velocity is half of the rising velocity. For example, for Bavarian rivers a rising water level velocity of 1-5 cm/h is characteristic for many large rivers such as the Danube, Iller, Lech, Inn, Isar, etc.

For the case studies 1 and 2 the Rhine flood discharge hydrograph was derived from a real recorded flood in 1995. The data was transferred to a water level hydrograph by applying the discharge – water level characteristic from the flow gauge at Ruhrort close to Duisburg city (Figure 2, left). The duration of the flood was not changed but the absolute peak water level was adjusted to the design water level of the dike which is 1.0 m below the dike crest (design flood level – design situation P) and to the crest level (design crest flood level – design situation A). For the case study 3 the hydrograph was derived from a rainfall-runoff model of the client (Figure 2, left).

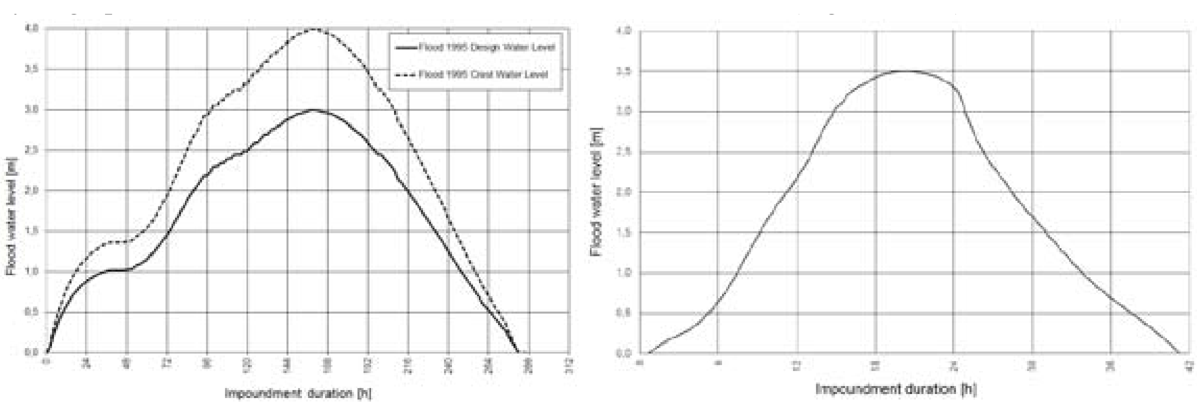

Figure 2 : Considered flood hydrographs for the case studies 1 and 2 (left) derived from a real flood event and for case study 3 (right) from rainfall-runoff model

The left hydrograph characterizes a Rhine flood and the levee impoundment lasts for almost 288 hours which is 12 days whilst the flood curve on the right which is characteristic for a smaller tributary of the Rhine lasts only 42 hours. The complete flood in 1995 within the considered region lasted approximately 24 days which is double of the defined impoundment time.

4.4 Modeling aspects

For the unsteady seepage modeling the initial conditions need to be defined. This is usually quite a scientific work since only limited data concerning the saturation condition of the levee are usually available. For a new construction project no data is available and the information has to be derived from laboratory tests or from literature. Latter seems to be sufficient in the opinion of the authors for the listed soils above. For clays it might be different. The initial conditions are characterized by the groundwater level which should be located somewhere within the subsoil/foundation, and the unsaturated moisture content of the soils and materials of the levee body and the subsoil above the groundwater.

Usually, the author defines the unsaturated soil characteristics in consideration of the values presented in Table 1 and applies a "wet" moisture content value or steady seepage conditions for the initial conditions. After this setting a dry period of 100 to 180 days is modelled only considering the mean groundwater level so that the soils have the possibility to drain and dry until they reach their natural water content or field capacity. For special cases and problems also rain events can be considered before and/or while a flooding event. In Haselsteiner (2007) also this effect was investigated with the result that precipitation events can usually be neglected in the context of ordinary design works.

The size of a levee model needs to be adapted in consideration of the local conditions. Usually the extension of the model to 10 times of the height of the levee towards up- and downstream direction (x-axis) and minimum two times of the height to the levees (y-axis) to consider foreland, hinterland and underground conditions is sufficient.

The meshing should be done in consideration of the required accuracy of the resulting values and the computing time. For unsteady seepage modeling the computing time is increasing by multiple in comparison to steady state conditions. A global mesh size of 0.5 to 1.0 m should not be exceeded for small embankment structures with a height of less than 5 to 10 m. For preliminary investigations coarser meshes should be applied in order to reduce computing time.

For the modelling the SEEP/W module of the GEOSTUDIO software package was used. The program enables the engineer modeling steady and unsteady seepage and integrating specific soil functions. Pre- and post-processing is made easy so that also unsteady seepage modeling can be done cost efficient. With the 2018 version the coupling of seepage and slope stability analysis is possible at every time step so that also the critical slope stability situation can be safely determined. This is a step forward in comparison to other approaches when the design engineer still needed to select the decisive seepage conditions manually.

5. CASE STUDY 1 – STANDARD RHINE LEVEE

The standard Rhine levee (compare Figure 1) was modeled as shown in Figure 3. The Rhine River is located on the left. Generally, the underground consists of high permeable sand-gravels, the levee fill shows fine sands with a permeability of kS = 10-6 m/s which is quite low in comparison to ordinary embankment fill materials. The levee is founded on the levee base with a thickness of 1.0 m which consists of flood loam.

The levee shows a height of 4.0 m, flat slopes, a wide crest, a berm and a surface sealing with a thickness of 2.0 m. The natural surface sealing and the levee base have permeability values of kS = 10-8 m/s and kS =10-7 m/s, respectively. The low permeable levee body cannot reveal its effect since the sealing and the base already hinder the seepage flow.

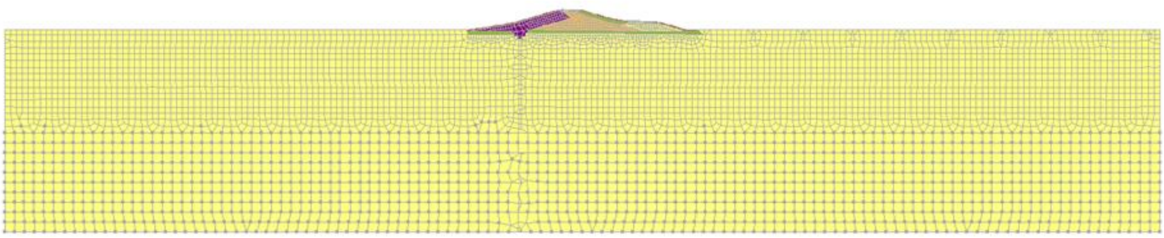

Figure 3 : Model area with levee geometry, mesh and regions for case study 1

In Figure 4 the pore water pressure distribution is shown for high and low water levels at flood times 144 h and 288 h (Figure 2, left). The state at 144 h is the design state for the stability of the downstream slope. The complete levee body is staying dry since the surface clay sealing and the loam levee base show a low permeability and no water is seeping through within the considered time of only several days. The situation at 288 h reflects the design situation for the stability of the upstream slope. The slope drains with the falling water levels through the permeable vegetation layer. Hence, both slopes are dry which is positive for the shear strength and the geostatic stability of the slopes.

The results show clearly that steady state conditions are not reached by far for the considered flood hydrograph and the standard levee section. The results also show that the consideration of steady state conditions as recommended in DIN 19712 reflect quite an unrealistic design situation which is unlikely to occur in practice. Hence, the conservative steady state seepage approach provides quite a considerable safety margin by overestimation of the real pore pressure conditions.

For the design crest level hydrograph (design situation A) the pore water pressure distribution is shown in Figure 5. When reaching the crest water level the pore water pressure distribution shows almost identical conditions as for the design flood hydrograph since the surface sealing and levee base are controlling the seepage development and the little difference in time and water pressure created by the crest impoundment does not cause a remarkable change concerning the seepage conditions.

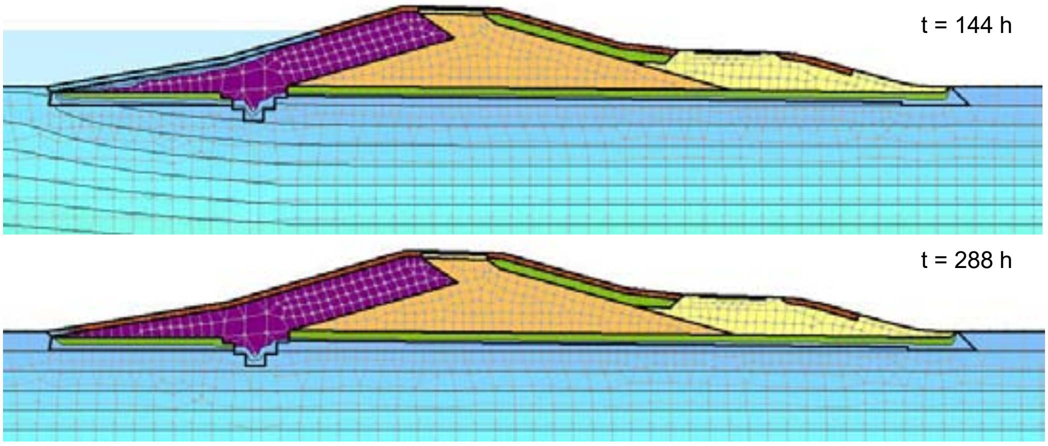

Figure 4 : Pore water pressure distribution at 144 h and 288 h for case study 1 (design flood hydrograph and design situation P)

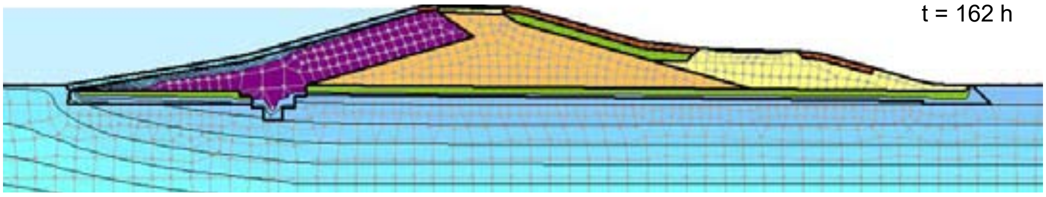

Figure 5 : Pore water pressure distribution at 162 h for case study 1 (crest flood hydrograph and design situa-tion A)

6. CASE STUDY 2 – SPECIAL RHINE LEVEE WITH PUBLIC ROAD

This case study represents an existing, old levee which is hosting a regional road. The height is almost 4.0 m, the crest width is approximately 10 m. The levee consists of silty sands and sandy silts. The levee is founded on a loam layer.

The silty sands and sandy silts show an average, characteristic permeability of $k_S = 10^{-7}$ m/s which is also valid for the flood loam. The slopes are steeper than for the standard levee section in Figure 1. The geotechnical investigation revealed that also a levee base exists, but it could not be guaranteed that the base does not show leakages (see Figure 6).

In Figure 7 the pore water pressure conditions for t = 152 h and t = 288 h is illustrated. Thanks to the low permeability values of the levee fill and the flood loam the seepage does only infiltrate in a limited extend to the levee body. The downstream slope is not affected at all. The upstream slope drains with the falling water table also since the water does not infiltrate too much during the rising water level. The seepage conditions are favorable in regard with the stability considerations.

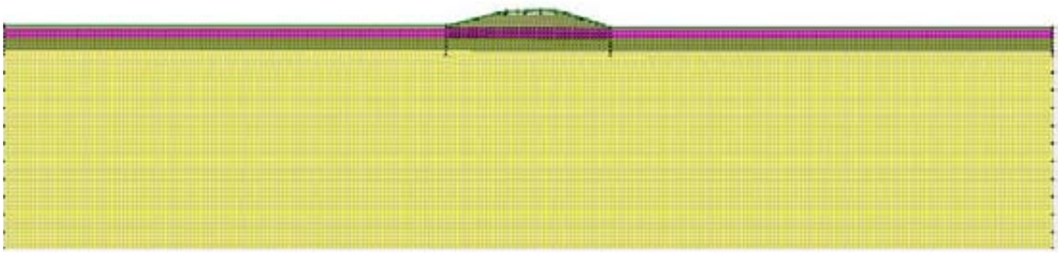

Figure 6 : Model area with levee geometry, mesh and regions for case study 2

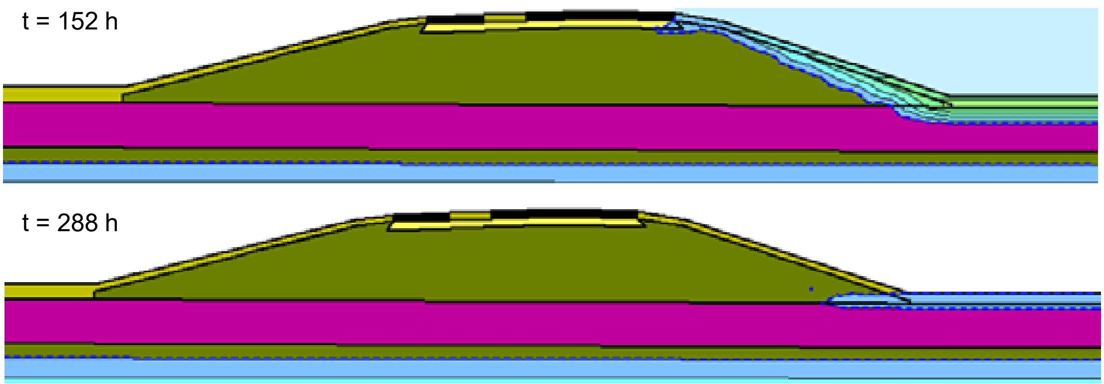

Figure 7 : Pore water pressure distribution at 152 h and 288 h for case study 2 (crest flood hydrograph and de-sign situation A)

7 CASE STUDY 3 – SPECIAL LEVEE BLACK SHALE MATERIAL ALONG A TRIBUTARY OF THE RHINE

Case study 3 shows a levee along a tributary of the Rhine. The levee design is extraordinary and shows a sand core covered by a layer consisting of black shale residual waste material which is waste material originating from coal industry and was frequently used for embankment fills in the past due to its favorable shear strength behavior showing a high friction angle and solid cohesion. But, the drawback of the material in concern of levee design is the relative high permeability which is assumed to be $k_s = 10^{-5}$ m/s.

The levee crest is quite slim and the slopes steep. The height of the levee is 3.5 m. The levee is founded on a relatively permeable sand-gravel subsoil with a permeability of $k_S = 10^{-3}$ m/s. The model is a symmetric model and shows only half of the river which reflects an artificial channel with a limited width (Figure 8). Due to a missing levee base the levee body is loaded by flows through the upstream slope and the foundation.

The preliminary investigation of the unsteady seepage behaviour of this levee should show whether an unfiltered exit occurs on the downstream slope of the levee. If so, the hydrodynamic soil deformation processes in form of suffusion and erosion should be assessed in detail.

Although the flood duration is relatively short the line of seepage exits the downstream slope at 13 h of the impoundment event which corresponds to a water level of 2.5 m. At the crest water level the seepage exit flow and height is strongest at 21 h of impoundment event (see Figure 2, right) before the line of seepage again sinks under the levee toe surface at 33 h at an upstream water level at approx. 1.3 m. The maximum seepage conditions show almost the same conditions as steady state conditions since the levee is quite permeable and, thus, the impoundment period is relative long.

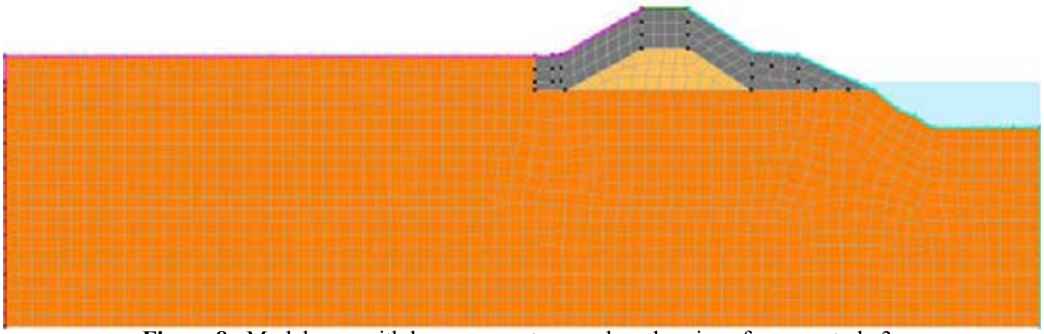

Figure 8 : Model area with levee geometry, mesh and regions for case study 3

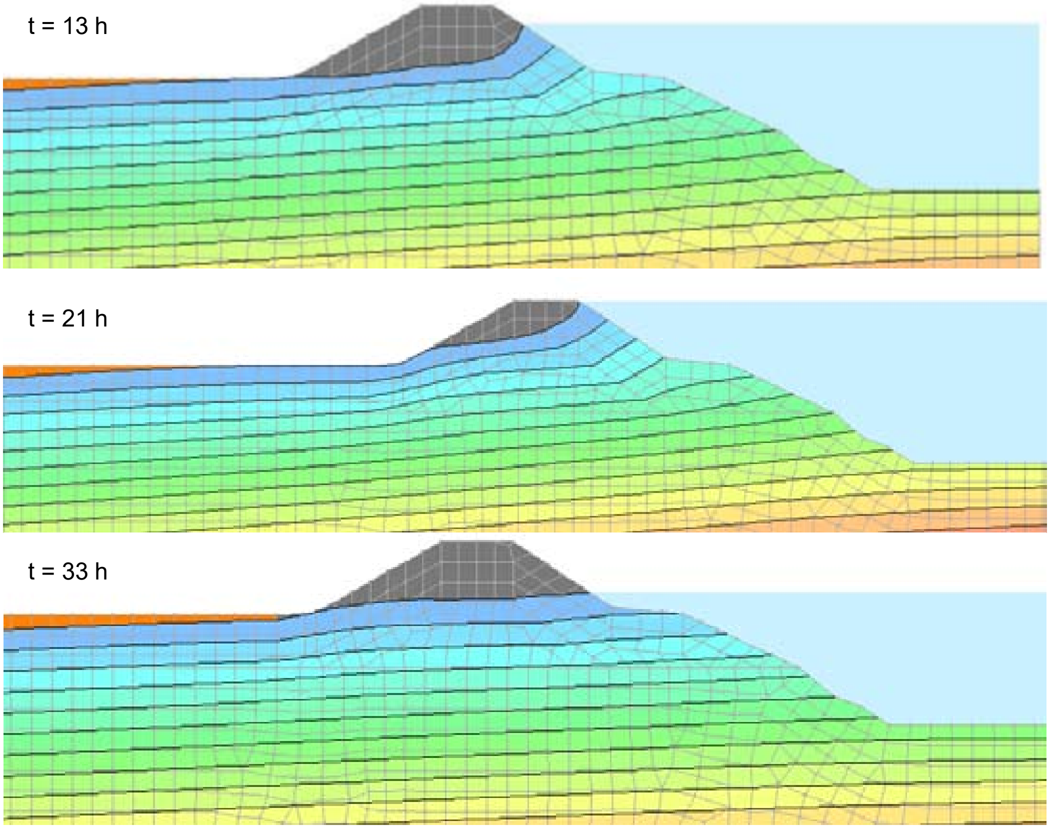

Figure 9 : Pore water pressure distribution at 13 h, 21 h and 33 h of with unfiltered seepage exit at the downstream levee toe for case study 3 (crest flood hydrograph and design situation A)

8. CONCLUSION

Unsteady seepage modeling reflects an important tool and method to determine more realistic pore water pressure conditions for levees with zones and materials of low permeability. The modeling results for case studies 1 and 2 show that steady state seepage conditions are not achieved by far. For case study 3 the permeability of the materials is quite high so that also the considered short flood hydrograph leads to almost steady state seepage conditions with also an unfiltered seepage exit on the downstream slope.

Hence, depending on the levee system the assumption of steady state seepage conditions as recommended by DIN 19712 provide a considerable safety margin. This safety margin may not be available anymore as soon as high permeable materials are controlling the characteristics of the levee body.

Although, the unsteady seepage modeling requires more knowledge, data and information and also engineering efforts for pre- and post-processing as well as more computing time the results may show more realistic seepage conditions as a basis for a reliable stability evaluation. Realistic seepage conditions on the one hand may grant a better understanding of the levee performance and, on the other hand, if a considerable safety margin is detected allow a less conservative design and cost savings which should comply with the design requirements and safety levels of national and international codes and guidelines.

REFERENCES

Börger, M. 2016. Hochwasserschutz am Niederrhein – Eine Zwischenbilanz. 46. Internationales Wasserbau-Symposium Aachen (IWASA), "Mobil oder Nicht-Mobil? Konventioneller und Innovativer Hochwasser-schutz in Praxis und Forschung", Aachen

DIN 19700. 2004. Stauanlagen. Deutsches Institut für Normung e.V. (DIN), Berlin

DIN 19700-12. 2004. Stauanlagen – Hochwasserrückhaltebecken. Deutsches Institut für Normung e.V. (DIN), Berlin

DIN 19712. 2013. Hochwasserschutzanlagen an Fließgewässern. Deutsches Institut für Normung (DIN), Ber-lin

DWA-M 507-1. 2011. Deiche an Fließgewässern - Teil 1: Planung, Bau und Betrieb. Merkblatt 507-1, Deutsche Vereinigung für Wasserwirtschaft, Abwasser und Abfall e.V. (DWA), Hennef

Fell, R.; Foster, M.; Wan, C.F. (2005a): A framework for assessing the likelihood of internal erosion and pip-ing of embankment dams and their foundations. Contribution to the Workshop of internal erosion and piping of dams and foundations. Aussoise, France, April 05

Fell, R.; Macgregor, P.; Stapledon, D.; Bell, G. (2005b): Geotechnical Engineering of Dams. A. A. Balkema, Taylor & Francis Group, London

Haselsteiner, R. 2007. Hochwasserschutzdeiche an Fließgewässern und ihre Durchsickerung. Phd-Thesis. Lehrstuhl und Versuchsanstalt für Wasserbau und Wasserwirtschaft, Mitteilungsheft Nr. 111, Technische Universität München.

Haselsteiner, R. (2007a): Die Durchströmung von Deichen und Dämmen. 14. Deutsches Talsperrensymposi-um, 7th ICOLD European Club Dam Symposium, Beiträge zur Tagung am 17. bis 19. September 2007 in Freising, Berichte des Lehrstuhls und der Versuchsanstalt für Wasserbau und Wasserwirtschaft, Nr. 115, S. 143 – 149

Haselsteiner, R. 2008. Die Durchströmung von Hochwasserschutzdeichen mit künstlichen Innendichtungen. Wasserwirtschaft 98, Heft 12/2008, S. 25-30

Haselsteiner, R. 2011. Flood Protection and Groundwater Recharge in the Batinah Region in Oman. Interna-tional Conference on Drought Management Strategies in Arid and Semi-Arid Regions, 11.-14.12.2011, Muscat (Oman)

ICPR. 2001. Atlas 2001 – Atlas der Überschwemmungsgefährdung und möglichen Schäden bei Extremhoch-wasser am Rhein. International Commission for the Protection of the Rhine (ICPR), Koblenz

ICPR. 2012. Aktionsplan Hochwasser 1995-2010: Handlungsziele, Umsetzung und Ergebnisse, Kurzbilanz. International Commission for the Protection of the Rhine (ICPR), Koblenz

Scheuermann, A. 2005. Instationäre Durchfeuchtung quasi homogener Erddeiche. Institut für Bodenmechanik und Felsmechanik, Universität Karlsruhe, Heft 164, 2005