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An Evaluation of Time Dependent Slope Sliding Failure of Levees within the Lower Rhine Region

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INTRODUCTION

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

Among several sorts of failure modes the global slope sliding failure is one the most crucial which may result in a sudden total failure of the structure. This slope sliding failure is critically affected by the pore water pressure distribution along the sliding surface.

Within the Lower Rhine region levees are usually designed with both a surface clay sealing and a base sealing (see figure 1). The result is that for permanent design situations (BS-P) the levee body is relatively dry also in consideration of steady state seepage conditions. But, the full pore water pressure originating from the upstream water level conditions develops underneath the base sealing reaching to downstream almost undamped.

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 face steady state seepage conditions to nearly no seepage. Hence, the factual safety level of levees are crucially depending on the actual seepage conditions which may be far away from the steady state conditions. Is this the case levees may show an overdesign and a considerable safety margin.

For levees within the Lower Rhine region the technical specifications regarding zoning and dam construction materials are so strict, that steady state seepage conditions are unlikely to occur in the dam body itself. But, high pore water pressures develop in the permeable underground layers so that heave and global slope failure are of concern and the levees does not show too much safety margin although the levee body is not subject to strong seepage.

During the planning stage of the presented case study the described topic were controversially discussed. The levee section was designed without a downstream berm although high pore water pressures occur underneath the base layer. Thus, an unsteady seepage analysis was performed in order to investigate whether the global slope failure would occur before the heave leads to a pore water pressure release in the underground. Basics for unsteady seepage modelling of levees are described, e. g., in Scheuermann (2005) or Haselsteiner (2007).

DESIGN SITUATIONS, CODES AND GUIDELINES

A levee according to DIN 19712 is an embankment along a river which is protecting against floods and which is only temporarily impounded by a flood. Assuming the occurrence of steady state conditions postulates long-lasting flood events. As far as sealing elements or low permeable soils are applied the seepage development may theoretically take years before reaching steady state conditions. For the concerned case study levee a surface sealing with a maximum permeability of $k \leq 10^{-8}$ m/s is applied. The dam body/filling shall show a permeability which is approximately maximum 100 times more permeable, so that $k \leq 10^{-6}$ m/s are stipulated. The drain again shall be maximum 100 times more permeable that the filling which results in $k \leq 10^{-4}$ m/s (STUA, 2005).

In Figure 1 the cross section of the standard levee is shown. The levee section is characterized by flat slopes, a crest road and a berm with the defense road. 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 technical guideline DWA-M 507 Part 1. The levee body is underlain by a base sealing which may be of natural origin . As shown in figure 1 a freeboard of 1.0 m is applied, the crest and berm are 5.0 m wide and the slopes show inclinations of V:H = 1:3.5.

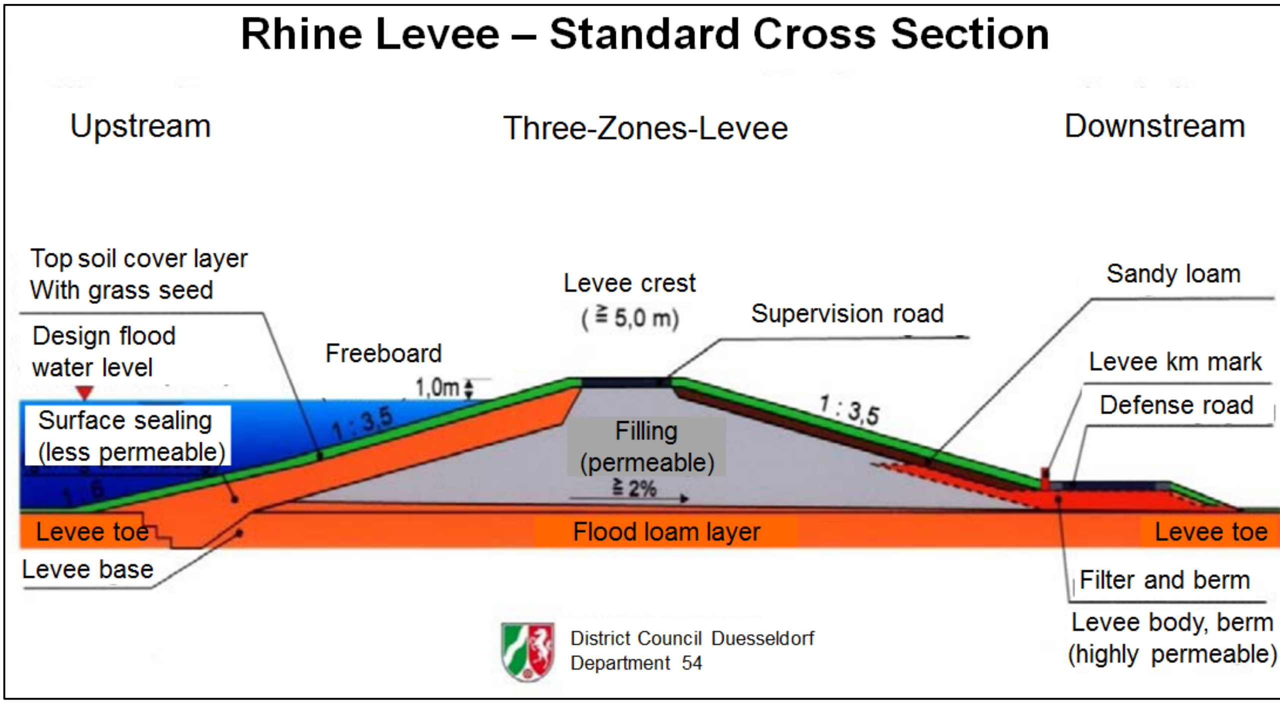

Figure 1: Levee standard cross section propagated within the district of Duesseldorf (Source: District Council Duesseldorf, Department 54) (taken and translated from Börger, 2016)

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, incorporated 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.

Usually, steady state seepage conditions are considered for the subsequent analysis of the geostatic or geo-hydraulic stability of a levee. 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 showing a permeabilities which are less than, let's say, $k_s = 10^{-6}$ 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.

SOIL PARAMETERS, FLOOD HYDROGRAPHS AND MODELING

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, 2007a, 2008) for various soils and selected sealing types (see table 1).

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)

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). The author holds the opinion that for the ordinary task of the determination of seepage conditions within embankment structures with the purpose of evaluating the stability the exact determination of the unsaturated soil characteristic as well as the definition of initial saturation conditions are not critical. The most important step is the determination and selection of an adequate permeability value concerning soils and materials in order to establish a realistic model.

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. For the presented case study 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 a nearby flow gauge (Ruhrort close to Duisburg city) (Figure 2).

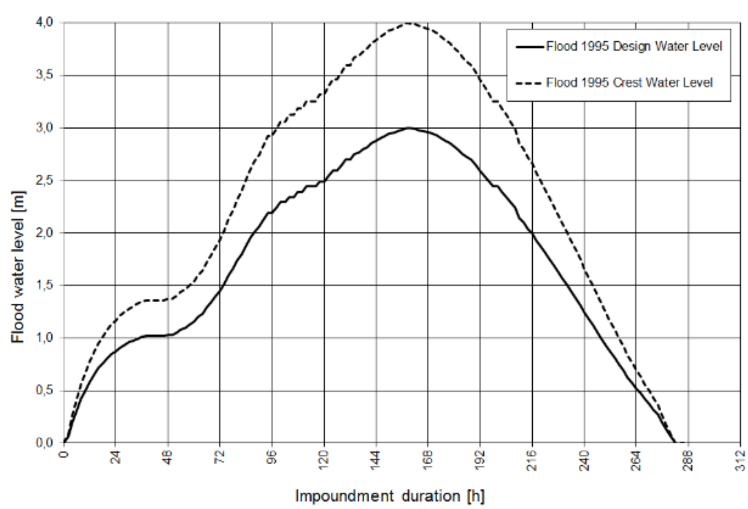

Figure 2: Considered flood hydrograph(s) for the case study 2 (left) derived from a real flood event

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 actual saturation condition of the levee are usually available. 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 which may be close to the natural field capacity. For special cases and problems also rain events can be considered before and/or while a flooding event in order to model a pre-saturation.

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. Due to the coupled analysis a mesh size of 2.0 m on average was selected for the case study.

For the modeling 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. 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 determined reliably.

STABILITY AND PORE WATER PRESSURE ASPECTS

For the presented case the global slope sliding stability was investigated hand in hand with the heave stability at the downstream levee toe. The considered levee section is shown in figure 3.

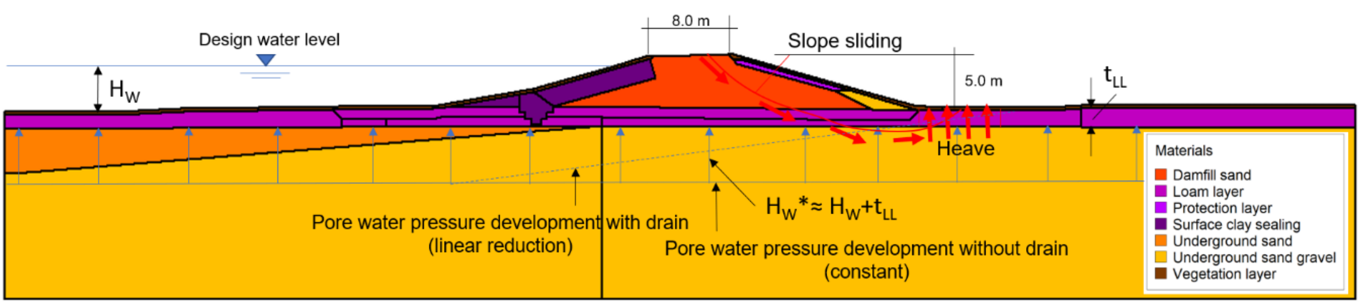

Figure 3: Principal levee section with failure mechanisms and pore water pressures

The levee is 5.0 m high and lacks a downstream berm. The crest width is 8.0 m. The slopes are inclined by V:H = 1:3.5.

As soon as the downstream loam layer shows a rupture or is drained by other means such as a drainage trench the pore water pressure will reduce as it is considered in form of a linear decrease in figure 3. If no draining effect occurs the pore water pressure will emerge in accordance to the flood level almost without reduction. In reality, there is a little reduction of 30 cm / 100 m seepage path which is negligible in the case study and its focus on the tow failure modes (see Kärcher et al., 2001).

For a better understanding the steady state seepage conditions are shown for both situations with and without drain at the downstream toe (figure 4). The pore water pressure reduces critically as soon as there is a permeable heave failure or drain structure. When heave does not occur the pore water pressure beneath the loam layer is more or less constant as aforementioned.

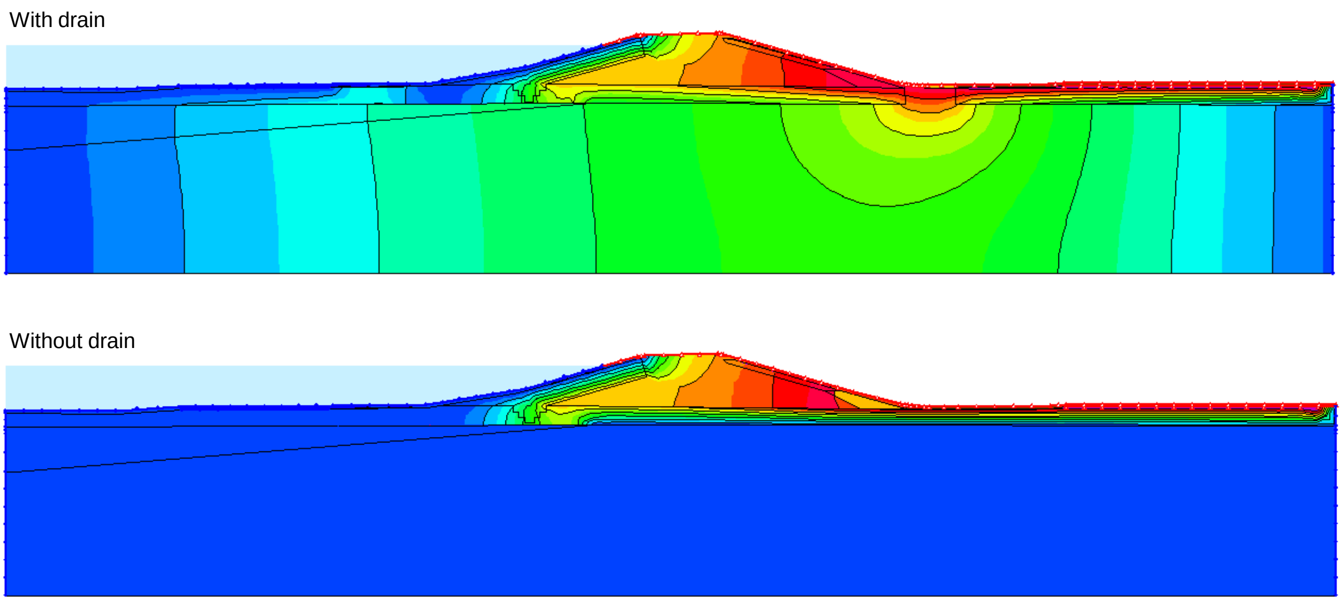

Figure 4: Pore water pressure distribution with and without downstream drain effect

CASE STUDY – LEVEE CROSS SECTION WITHOUT BERM

The case study levee shows a cross section without a downstream berm (figures 1, 3, and 4). The seep model is illustrated in figure 5. 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 low permeability. The levee is founded on the levee base with a thickness of 1.0 to 1.5 m which consists of flood loam.

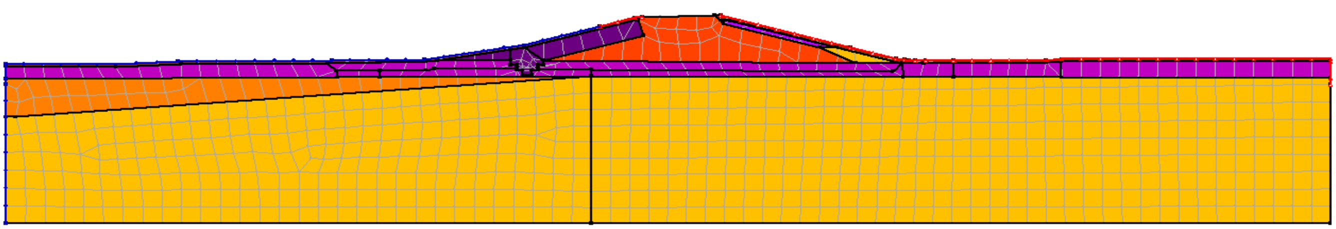

Figure 5: Model area with levee geometry, mesh and regions

The processed results of the slope and heave stability analyses are shown in figure 6. The flood hydrograph is integrated in the graph and is reaching a maximum water level of 4.0 m at 77.5 h. The flood impoundment starts at 70 hours.

At a water level of 2.5 m the degree of utilization of the heave is reached after approx. four days after impoundment started. The initial degree of utili ation for sliding of the downstream slope is m 0.2 in dry conditions with low groundwater level. Two days after heave gets critical so six days after impoundment started the slope sliding failure is reaching critical conditions at a water level of 3.5 m. The maximum load is reached one and a half days after the theoretical slope failure hand in hand with the maximum water level. The reason for this simultaneous effect is that the dam body itself is still dry and the destabilizing pore water pressures beneath the loam layer which are corresponding with the upstream flood water level.

The graphs were prepared for permanent design situation considering the flood water level (BS-P). The partial safety factors were defined according to BS-P. The levee reveals highly unstable conditions at water levels larger than 3.5 m, reflecting three days a situation which does not comply to the design specifications.

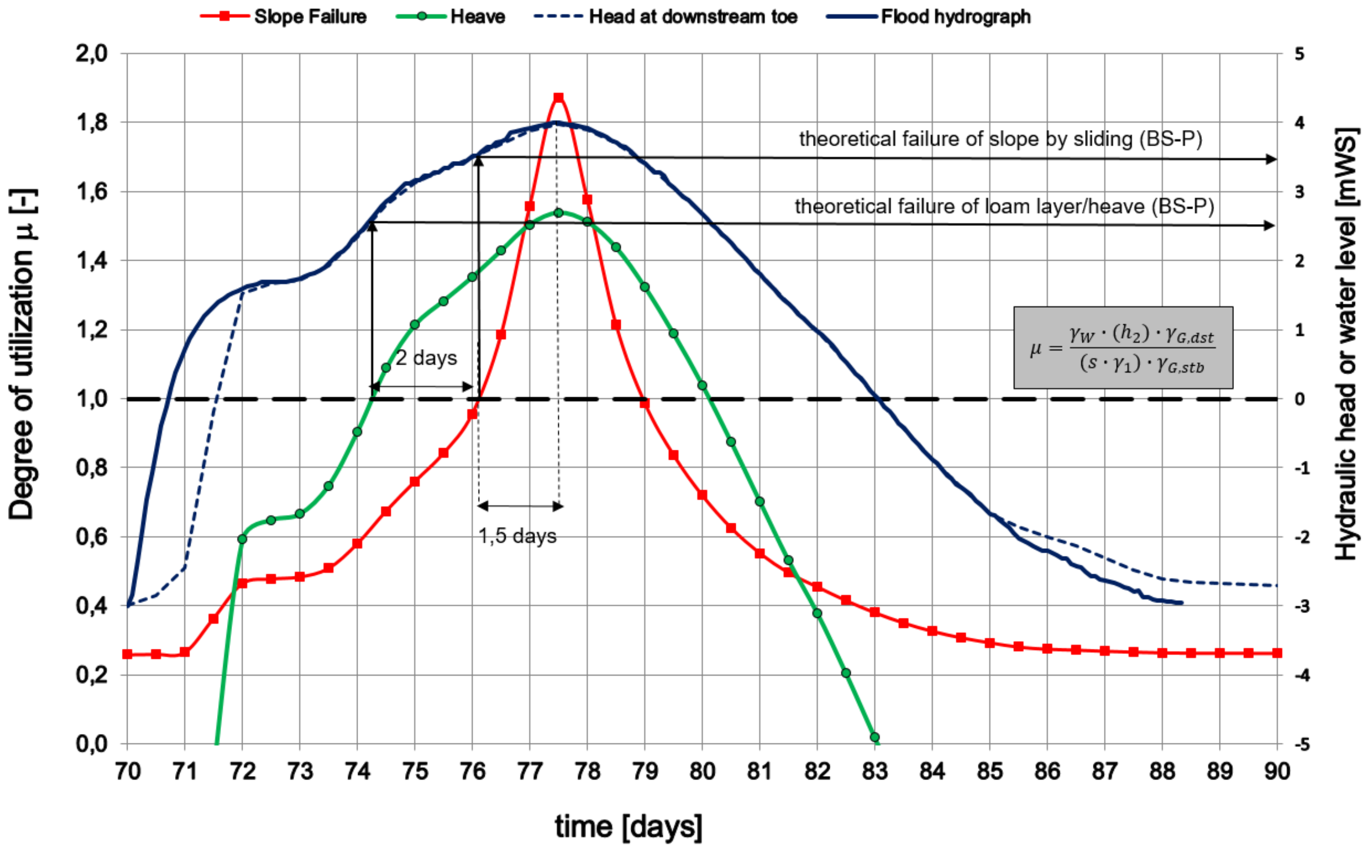

Figure 6: Results of modelling – comparison of degrees of utilization for slope failure and heave vs. time

Several more analyses and parameter studies were performed with the same model in order to gain a better understanding of levee's response to changing loads and shear strength parameters. Generally, the results show that critical heave conditions are arising before the critical slope sliding conditions. But, things may change if the loam layer shows strong cohesion and its position/elevation is lower compared to the upstream terrain. As soon as the loam layer withstands its loads and no local failure in form of a heave occurs global slope sliding is a matter of concern.

The levee body shows unsaturated conditions which could be also considered as unsaturated strength values within the analysis. For the first approach this additional withholding resistance was neglected. As one of the countermeasures the placement of a berm was considered as shown in figure 7. But this countermeasure could not show the required effect. The consideration of the unsaturated cohesion in the loam layer and the levee body could contribute to model a more realistic behavior.

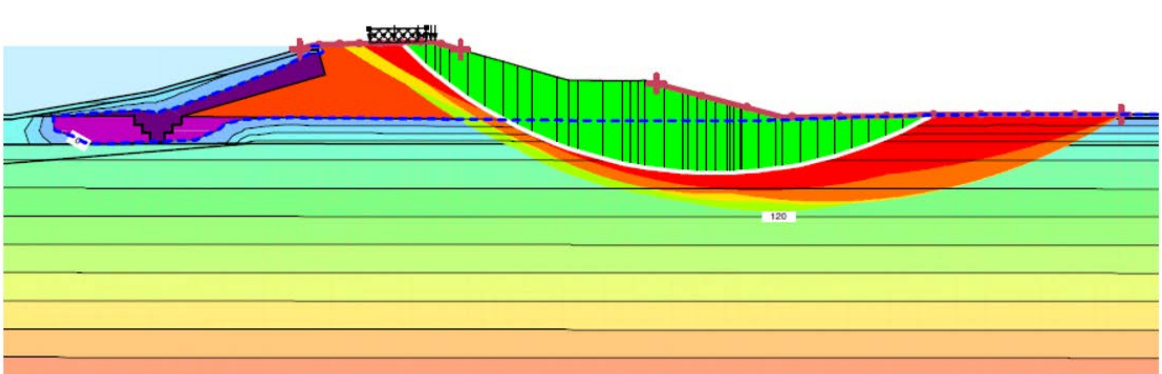

Figure 7: Slope sliding circles for levee with berm – critical maximum degree of saturation at 77.5 h with m ≥ 1.5

CONCLUSION

The coupled unsteady seepage and stability analyses revealed that heave is theoretically occurring earlier than the slope sliding. Therefore, a pore water pressure release may help to reduce the actual pore water pressures. The high critical pore water pressures underneath the loam layer/base sealing which are responsible for the global slope sliding failure may not develop. Thus, the local failure of the loam layer by heave prevent a global slope failure.

Of course, the theoretical analysis contains some inaccuracies and rough assumptions concerning the real drainage effect of a heave process as well as the actual shear strength of the affected soils and materials. In addition the heave failure may also initiate backward erosion (Fell et al., 2005) which again may have some negative influence on the failure mode. And, also the sliding deformation may also effect the pore water pressure conditions in the subsoil.

As a countermeasure a berm with a height of approx. 2 to 3 m and a width of 5 m can be added to stabilize the downstream slope. Nevertheless, due to the unhindered water pressure development underneath the loam layer heave still may occur downstream and cannot be avoided by structural load but only by draining works. Additionally, the placement of a berm may not completely be sufficient to theoretically stability the slope due to the high pore water pressures and the low effective shear forces along the sliding surface underneath the loam layer.

For the time being, two engineering views/opinions are facing each other. One side claims that a controlled pore water pressure release by the placement of a drain is the only way to get rid of the heave failure; the other side assumes that the loam layer itself is adding safety to the overall levee structure and in case of heave failure gets critical it will drain the subsoil anyway. In this context the local failure by heave is acceptable.

The author holds the opinion that the available approaches do not reflect actual conditions and that the complete failure process at the downstream toe should be investigated thoroughly by large scale tests. Pore water pressures, saturation, and deformation of the concerned levee parts should be monitored in order to obtain a better understanding of the actual processes. The stress-deformation-pore water pressure behavior of both discussed failure types should be considered not independently but it is expected that both are interacting. Since no adequate approach or solution is available for the described practical problems research is required in order to define the actual safety of concerned levees.

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