The influence of erosion processes on the stability of permanent flood protection systems

Influence des procès d'érosion sur la stabilité de systèmes protectifs permanent contre les crues

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ABSTRACT

The erosion in layered soil, especially due to horizontal flow under consideration of the stress level has been researched by the Institute for Soil Mechanics and Geotechnical Engineering at the University of the Bundeswehr Munich for several years. Up to now, two model test stands have been developed. The first one simulates the conditions within dams and dykes where the seepage water can cause subsurface erosion failure. This kind of failure occurs for instance within layered soils as well as at the dam base. The second model test stand takes into account axis-symmetric conditions, which are important for soil improvement methods such as gravel columns. Tests for horizontal hydraulic conditions with parameters varying such as the material for cohesive soil, the grain size distribution of the filter material, the stress state and others, have been performed. To start the inner erosion, which in context of dam or dyke stability is also known as piping, the water flow velocity has to reach a critical value, so that first the process of suffusion will start, which finally turns into erosion. This whole process produces the collapse of the dam. A calculation method to determine the critical hydraulic gradient has been set up on the base of all tests performed at the geotechnical laboratory of the Institute for Soil Mechanics and Geotechnical Engineering at the University of the Bundeswehr Munich. The applications of these new aspects on hydraulic contact erosion within flood protection engineering are highlighted.

RÉSUMÉ

L'institut de la mécanique des sols et de géotechnique de l'université de la Bundeswehr à Munich a suivi des recherches au sujet de l'érosion dans les sols jointés, spécialement en raison de l'écoulement horizontal compte tenu du niveau de tension durant quelques années déjà. Jusqu'à présent deux installations expérimentales ont été réalisées. L'une d'elles simule les conditions régnant dans des remblais et digues où l'eau d'infiltration produit l'érosion souterraine. Ce genre de défaillance apparait par exemple dans des sols jointés ainsi qu' à la base de remblais. L'autre installation expérimentale respecte les conditions axisymétriques importantes pour les améliorations du sol tels que les colonnes de gravier. Des essais pour des conditions hydrauliques horizontales ont été effectués en modifiant des paramètres comme le matériel du sol cohérent, la répartition granulométrique du filtre, le niveau de tension et autres. Pour produire l'érosion intérieure, connue dans le cas de la stabilité de remblais comme renard, la vélocité de l'écoulement de l'eau doit atteindre uns valeur critique de sorte que le processus de suffusion commence. L'érosion produit l'effondrement du remblai. Une méthode de calcul pour déterminer le gradient hydraulique critique a été inventée basée sur tous les essais effectués au laboratoire géotechnique de l'institut de la mécanique des sols et de géotechnique de l'université de la Bundeswehr à Munich. Les applications de ces nouveaux aspects sur l'érosion de contact hydraulique dans l'ingénierie de la protection contre les crues sont singularisées.

Keywords : contact erosion, flood protection systems, critical hydraulic gradient

1 INTRODUCTION

A more ever actual becoming question is flood protection by dams and dykes in maritime areas. The undercurrent of the embankment can occur particularly at dykes (Cedergren, 1967). The flow velocity depends on the hydraulic gradient. If the water level rises at the upstream face of the dyke, the flow velocity in the subsoil will increase. Rearrangements of soil particles appear as a consequence of the streaming forces. The typical areas with the possible characters of transport are shown in Figure 1. Illustration a) shows the contact erosion caused by lowering the water level, b) the contact erosion due to vertical flow, c) the hydraulic breakthrough, d) as a result of the hydraulic breakthrough the piping in the water-bearing stratum, e) the contact erosion in the cohesive stratum caused by parallel flow and f) the inner suffusion. If these erosion processes occur at the contact zone (illustration e)) and reach a critical value, this phenomenon is called hydraulic contact erosion. So-called piping phenomena due to this hydraulic contact erosion can occur which in consequence can lead to the total loss of stability of the dam or dyke and may result in a breach of a dyke or in a flood calamity.

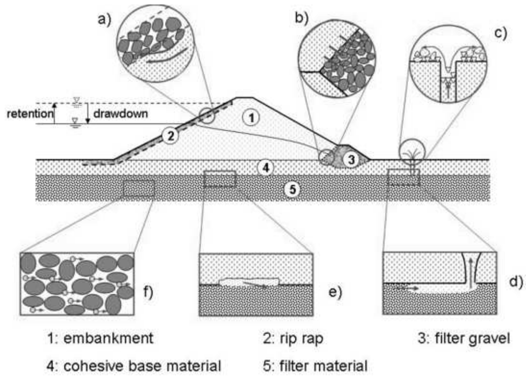

Figure 1. Location of possible places for erosion

The critical hydraulic gradient i_{crit} which causes the hydraulic contact erosion is affected by different parameters, e.g. the shear strength of the cohesive material at the contact zone and the pore volume of the filter material.

The phenomenon of hydraulic contact erosion has been investigated at the Institute for Soil Mechanics and Geotechnical Engineering at the University of the Bundeswehr

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Munich for several years. The main focus of the investigations lies on the verification of the stress dependency on the dimension of the critical hydraulic gradient i_{crit}. For these investigations element tests varying the external load from 10 to 120 kPa were performed in purpose-built model test stands.

2 TESTS FOR CONTACT EROSION

2.1 *Test equipment and soil material*

Figure 2 shows an already built in test set-up. The strati-graphic sequence was chosen so that the cohesive soil is situated above the filter layer. Here a 4-8 mm filter gravel (Figure 2,) and silt-sand-mixture (Figure 2,) were used as base material. It was possible to apply a vertical load on the stratigraphic sequence through a compressed air pipe and via a water pad. The hydraulic head was measured with piezometer at the intake and outlet, what allows to determine the real hydraulic gradient.

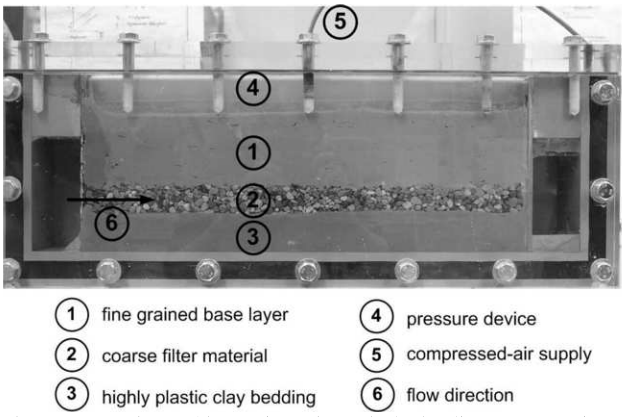

Figure 2. Experimental box to investigate the hydraulic contact erosion

Only cohesive soils were applied as base material in these introduced studies, because mainly cohesive soils were and still are used as sealing element in dam and dyke constructions. Figure 3 shows the grain size distributions. The tested soils are silt-fine sand mixtures, but also kaolin-fine sand and quartz-limestone powder-mixtures that were artificially produced in the laboratory. None of the materials showed a dispersive (Mitchell & Soga, 2005) behaviour. In order to eliminate the influence of salt in the seepage water, only partially desalted water was used for the tests. Müllner (1991) already referred to the influence of salt on the erosion process.

Three different filter materials were used, since in addition to the influence of the extra load also the influence of the filter grain on the dimension of the sustainable hydraulic gradient should be analysed. Figure 4 shows their grain size distribution.

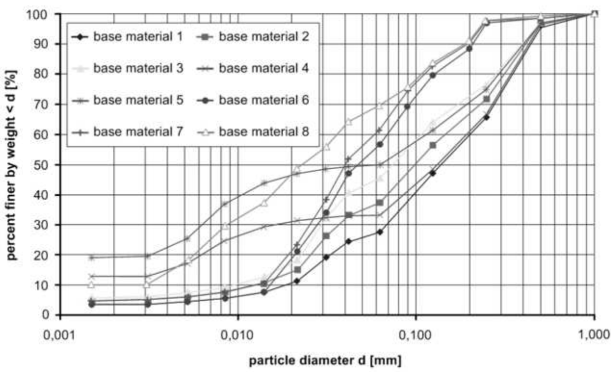

Figure 3. Grain size distribution of the tested base materials

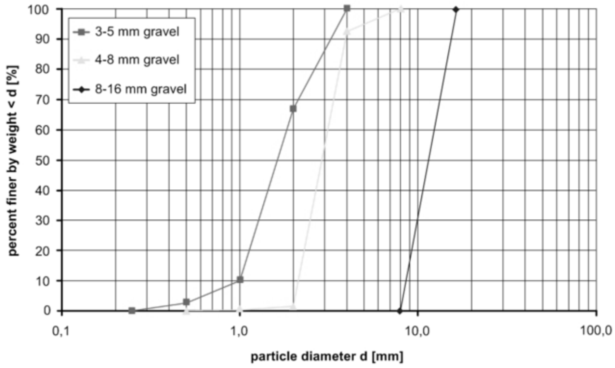

Figure 4. Grain size distribution of the tested filter materials

First of all the filter material was installed and then the almost waterlogged base material was built in. The vertical load stayed for at least 24 hours on the system before the percolation was started. Every 24 hours the hydraulic gradient i that determines the percolation was increased until the erosion of the cohesive material occurred. The emerged material was collected and its mass was determined. The test was stopped when the mass of the eroded material had a critical range. Then erosion in the contact area occurred, what could also be detected visually. The critical hydraulic gradient i_{crit} corresponds to the gradient that causes the critical mass and is defined by the flowed through length Δl and the difference of the hydraulic head Δh when erosion occurs.

2.2 *Test results and observations*

So far a total of 22 tests were performed. One can find a correlation between the extra load p and the critical hydraulic gradient i_{crit} within these testings. Figure 5 exemplifies the test results for both base material 2 and 4 (Figure 3) with a 4-8 mm filter gravel. A higher critical hydraulic gradient could be detected with a higher vertical stress p. One can suppose on the basis of the test results that the influence at very high extra loads will be not that important (asymptote towards the extra load p).

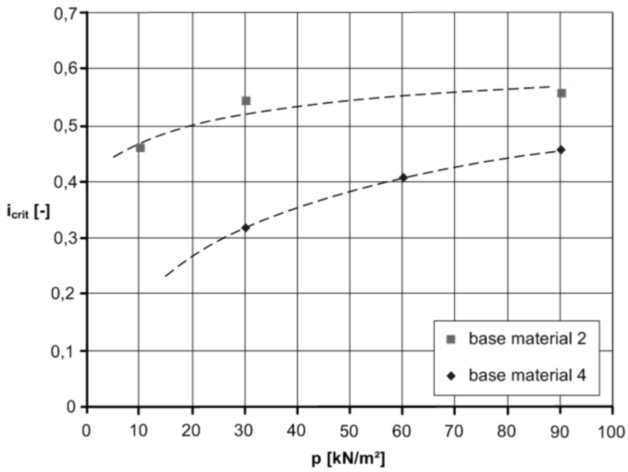

Figure 5. Test results for base material 2 and 4 with the same filter material

Several tests with different filter materials were performed to verify the influence of the filter material on the dimension of the sustainable hydraulic gradient. Figure 6 shows the comparison of the tests on base material 2 with a 3-5 mm filter material and a 4-8 mm filter material. The finer filter material leads to a higher critical hydraulic gradient. This can be explained by the stress distribution at the border between filter and base material. One can assume that on base of the filter material an arch in the base material is shaped which is stable up to a certain load. The bigger the distance between the contact faces and the greater the span width of the arch, the lower will be the sustainable loads.

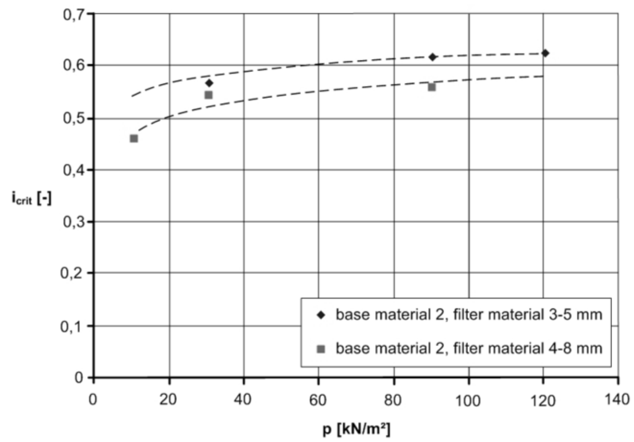

Figure 6. Comparison of the test results with the same base material but different filter material

Figure 7 shows the typical failure mechanism at the critical hydraulic gradient of the cohesive soil at parallel flow. At all tests the erosion occurred suddenly and were stopped after erosion appeared, as shown in Figure 7 on the right. One can see in Figure 7 that almost the whole contact zone is affected by contact erosion, if the test wasn't stopped after the first appearance of erosion.

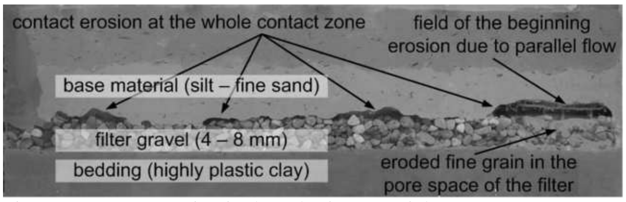

Figure 7. Contact erosion in the cohesive material

3 THEORETICAL MODEL

A mathematical model which allows the calculation of the critical flow velocity v_{crit} and thus the critical hydraulic gradient i_{crit} was developed on the base of physical considerations. Firstly both extrema of the consistency of cohesive material – solid state and fluidity – were considered. A transition zone was defined, since cohesive material at the layer border to the filter material doesn't exist neither in solid state nor in fluidity, i.e. both model reflections were combined. Zanke (1982) found a similar approach considering the transport of sediments on river beds.

Cohesive soil with firm consistency can be described as flowed pipe whose wall properties are described by the cohesive material (see model representation in Figure 8).

On the base of fluid mechanics one can now describe the flow velocity depending on the shear stress τ, the viscosity ν and the density ρ of the fluid. In Prandtl (1944) one can find the solution for the characterisation of the flow velocity v with the implementation of Nikuradse's (1932) test results. If one supposes that the critical flow velocity $v_{solid,crit}$ is existent when the shear strength of the cohesive material τ_{fs} is exceeded, caused by the shear stress of the moving fluid, one can equate both variables. Now it is possible to write Prandtl's equation as follows:

$$v_{solid,crit} = \sqrt{\frac{\tau_{fs}}{\rho_W}} \cdot \left(5,52 \cdot \log\left(\frac{y}{\nu} \cdot \sqrt{\frac{\tau_{fs}}{\rho_W}}\right) + 5,84\right) \quad (1)$$

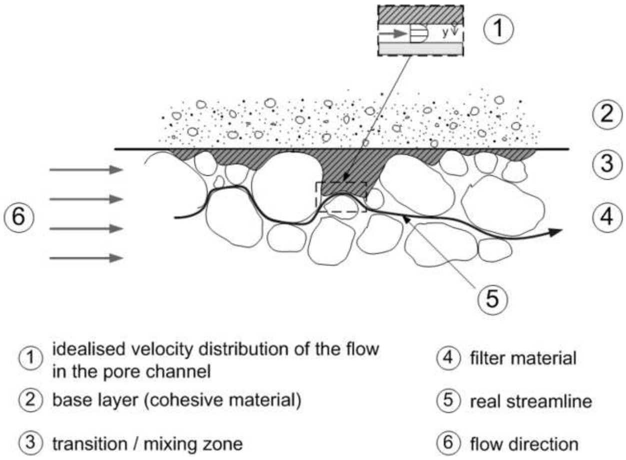

Figure 8. Pore channel at the borderline

Herein y [m] is the coordinate of the pipe radius, ν [m²/s] the kinematic viscosity of the streaming fluid, τ_{fs} [kN/m²] the shear strength of the cohesive soil (determined by the vane test) and ρ_W [kg/m³] the density of the streaming fluid. One can assume a turbulent streaming in the pore channel for this theoretical model. It's generally known that the distribution of the velocity in the case of turbulent flow in average is nearly constant over the pipe cross section, so it should be assumed that the flow velocity $v_{solid,crit}$ should be determined in the middle of the pipe section. Now y in Equation 1 can be written as:

$$y = \frac{d}{2} \quad (2)$$

where d stands for the pipe diameter or the proper pore channel diameter respectively which can be determined according to Pavcic's equation (1961), see Equation 3.

$$d = 0,454 \cdot e \cdot d_{17} \cdot \sqrt[6]{U} \quad (3)$$

where e represents the void ratio, d_{17} the grain size at 17 % of mass passage and U the coefficient of uniformity of the grain size distribution.

Considering the fluid state of the cohesive soil, one can imagine a model of two fluids with different density lying upon each other. If both fluids move relatively together a critical wave length λ_{crit} can be defined according to Prandtl (1944). This wave length is directly dependent on the relative flow velocity ($\Delta v = v_1 - v_2$). When the critical wave length is underrun the waves begin to break at the border line. This can be equated with the intermixing of both fluids (Figure 9), which in turn can be understood as erosion at the contact zone.

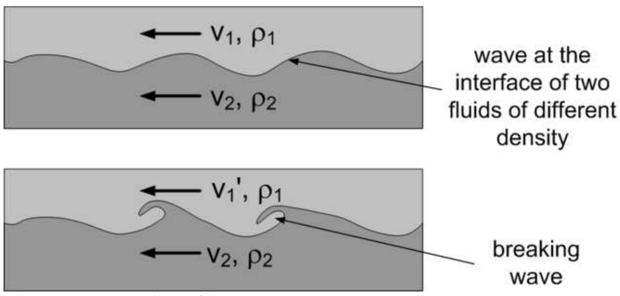

Figure 9. Interaction of inner waves

The description of the critical wave length λ_{crit} (White 1991) and Tollmien's considerations (1929) for the description of the instability of boundary waves allow to determine the critical stream velocity for the model representation of the boundary layer between two fluids with different densities und thus for the limitation between fluid cohesive soil and water.

$$v_{fluid,crit} = 10{,}53 \cdot \left(\frac{\rho \cdot \rho_W}{\rho^2 - \rho_W^2} \cdot g \cdot \nu \right)^{\frac{1}{3}} \quad (4)$$

In Equation 4 ρ [kg/m³] stands for the density of the cohesive soil, ρ_W [kg/m³] for the density of water, g [m/s²] for the acceleration of gravity und ν [m/s²] for the kinematic viscosity of water.

Both extrema of the consistency of cohesive soil were considered. In order to describe the transition between solid and fluid, a combination of both critical velocities (Equations 1 and 2) should be made according to Zanke (1982). Now the critical velocity v_m in the transition zone of the consistency is

$$v_m = v_1 + v_2 - \sqrt{v_1 \cdot v_2} \quad (5)$$

Because the limitation between cohesive soil and mineral filter material is about a pore water flow and with the aim to determine the critical hydraulic gradient, the velocity v_m has to be integrated in a non linear flow law as e.g. Forchheimer (1901) recommended, see Equation 6.

$$i = a \cdot v + b \cdot v^2 \quad (6)$$

To determine the parameters a and b, literature quotes different possibilities as e.g. the experimental determination according to Valentin (1970) and the semi-experimental determination according to Kovacs (1969).

For the determination of the critical hydraulic gradient one has to integrate Equation 6 into Equation 5. For this however one has to introduce both coefficients ψ and χ. A comparison of test and calculation results shows that one can expect good results using for the coefficient ψ = 0,01 and for the coeffient $\chi = 1{,}0 \cdot 10^{-4}$. Now one has to apply:

$$v_1 = \psi \cdot v_{crit,fluid} = \psi \cdot \left(10{,}53 \cdot \left(\frac{\rho \cdot \rho_W}{\rho^2 - \rho_W^2} \cdot g \cdot \nu \right)^{\frac{1}{3}} \right) \quad (7)$$

$$v_2 = \chi \cdot v_{crit,solid} = \chi \cdot \left(\sqrt{\frac{\tau_{fs}}{\rho_W}} \cdot \left(5{,}52 \cdot \log\left(\frac{y}{\nu} \cdot \sqrt{\frac{\tau_{fs}}{\rho_W}} \right) + 5{,}84 \right) \right) \quad (8)$$

Taking into account the porosity n the determined flow velocity v_m has to be converted into the real flow velocity v_r (Equation 9) which has to be integrated into the non linear flow law to obtain the estimation of the critical hydraulic gradient for parallel flow (Equation 10).

$$v_r = v_m \cdot n \quad (9)$$

$$i_{crit} = a \cdot v_r + b \cdot v_r^2 \quad (10)$$

4 APPLICATION OF THE THEORETICAL MODEL

The application of the theoretical model compared to the test results shows a conservative estimation. The model is now available to determine the critical hydraulic gradient i_{crit} with well known shear strength (τ_{fs}) and density (ρ) of the cohesive base material and with well known filter material (n, e, d_{17}, U). Figure 10 shows the test results compared to the theoretical model, in which the correlation between the shear strength τ_{fs}, determined by the vane shear test, and the critical hydraulic gradient i_{crit} is plotted. A constant density of 1,9 g/cm³ was assumed for the evaluation of the theoretical results.

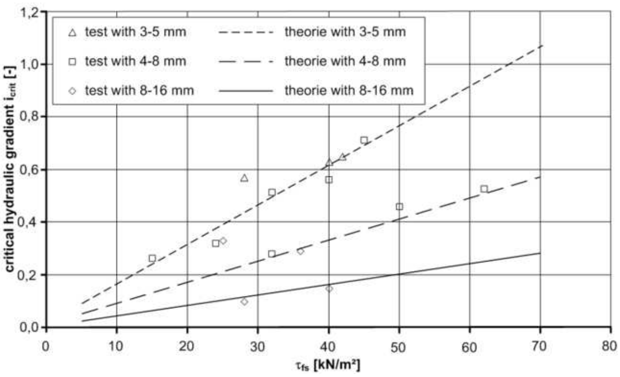

Figure 10. Theory and test results for the flow parallel to the layer

5 CONCLUSIONS

Schmitz (2007) showed that it is possible to determine the critical hydraulic gradient of any cohesive soil with this new theoretical model, which allows geotechnical engineers to give well-founded judgements on the stability of dykes.

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