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Erosion experiments on fine grained soils and their theoretical evaluation

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1 Introduction

1.1 Literature review

Seepage forces in soils can lead to local or global deformation of their structure. In geotechnical practice there are three main processes which can be observed in soils, when they are subjected to hydraulic forces, namely suffusion, erosion and colmation. Ziems (1968) defines suffusion as a process where the fine particles of a soil with instable grading get detached. It just remains the skeleton of the coarse soil. Colmation is the inverse process where the hydraulic forces are so small, that fine particles get deposited in the soil. At last erosion is a process where nearly all particles of a soil get detached, which may lead to local or global failure. Contact erosion is a phenomenon happening at the interface between a sealing material (fine grained soil) and a filter material (coarse grained soil). The discharge of particles may lead to the loss of the sealing effect of fine grained layers in dams or of soil layers above confined aquifers, resulting in possible damages for overlying structures.

Safety against contact erosion depends on two criteria. First there is the geometric criterion, defining whether it is possible for the fine particles to be eroded into the coarse material or not. This can be checked using various approaches for comparing representative grain diameters of the sealing and the filter material.

When a combination of sealing and filter material is geometrically unstable, the second influence on contact erosion is the hydraulic force due water flow. To ensure safety the critical hydraulic gradient i_{cr} has to be higher than the actual gradient i_{act} in the sealing layer.

To calculate the critical hydraulic gradient different suggestions have been made. The approaches of Rehfeld (1972) or Davidenkoff (1976) consider d_p of the filter and the tensile strength σ_t of the sealing material as important factors.

$$\eta_h = \frac{a \cdot \sigma_t}{d_p \cdot (i_{act} \cdot \gamma_w - \gamma')} \geq 1.5 \quad (1)$$

In Rehfeld (1972) the tensile strength is multiplied by $a = 1.5$ and in Davidenkoff (1976) $a = 6$. For the determination of the representative void diameter of the filter material d_p , an empirical equation (2) from Pavcic (1961) can be applied. It uses the uniformity U of the soil, the grainsize diameter at 17% of mass d_{17} and the void ratio e .

$$d_p = 0.455 \cdot \sqrt[6]{U} \cdot e \cdot d_{17} \quad (2)$$

An improved approach considering the effective stress state and the arrangement of particles is presented in Zou (2000) (equation 3). Here the resistance of the fine grained material is calculated using φ' and c' , making it possible to account for the actual stress state in the fine grained layer. Zou (2000) also introduces a parameter T_1 to take the direction of water flow and the arrangement of particles into account. For triaxial stress states the effective stress σ_{x0}' at the sides of the piping channel and the effective normal stress p' at the interface between sealing and filter material can be represented by $p' = \sigma_{x0}' = \sigma_3'$. Furthermore the parameter ζ describes the dependence of the maximum shear stress on actual stress state.

$$\eta = \frac{4 \cdot c' + (\sigma'_{x0} - i_{act} \cdot \gamma_w \cdot d_p) \cdot \tan \varphi'}{2 \cdot \zeta \cdot p' + \left(i_{act} \cdot \frac{\gamma_w}{T_1} - \gamma' \right) \cdot d_p} \quad (3)$$

A refined approach of the ideas from Zou (2000) is shown in Schmitz (2007). Here also samples in an overconsolidated state have been investigated. The results show, that the unit weight of the soil has nearly no influence, so it is not considered in equation (4).

$$\eta = \frac{2 \cdot c' + \left(\frac{\xi_0}{\xi_1} \cdot \sigma'_{x0} + \xi_0 \cdot \gamma_w \cdot \frac{d_p}{2} \cdot i_{act} \right) \cdot \tan \varphi'}{\zeta \cdot p' + \frac{d_p}{2T_1} \cdot \gamma_w \cdot i_{act}} \quad (4)$$

The dimensionless values ξ_0 , ξ_1 , ζ are used to describe the stress state in the sealing layer and can be determined using design charts. Schmitz (2007) recommends a relation of ξ_0/ξ_1 from 0.2 to 0.6. This allows some adjustment of the results to conducted experiments, but makes the prediction of a critical gradient without experiments more complicated. As in Zou (2000) the influence of the value of cohesion on the factor of safety seems quite high.

2 Materials

2.1 Classification

For the erosion tests a fine grained material (clay with high plasticity) and three coarse grained materials (sand and gravel) were used. The coarse material was

used as filter material and the fine material as the sealing material, which should be eroded.

Three different gradings were used as filters to investigate the influence of the equivalent pore diameter on the critical hydraulic gradient. Material F1-2 is a well graded Sand (SW) with grain sizes from $d = 1-2$ mm. On the other hand material F2-4 is a well graded gravel and F1-4 is a mixture of both materials.

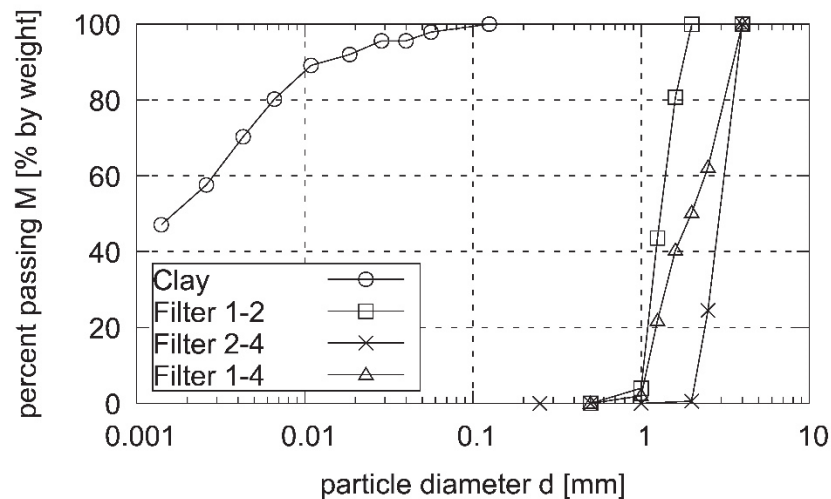


Fig. 1: Grain size distributions of the tested soils

The fine grained material can be classified as clay with high plasticity, with a liquid limit of $w_L = 50.5$ % and a plastic limit of $w_p = 21.3$ %. The unit weight of the material is $\rho_s = 2.634$ g/cm³ with an organic content of 7.5 %. Figure 1 shows the grain size distributions of the materials used for the erosion experiments.

To investigate the influence of preconsolidation stresses on the critical hydraulic gradient, one clay sample was oedometrically consolidated up to $\sigma_v' = 200$ kPa. In the experiments these specimens are labeled as normally consolidated (C1, nc). The other part of the fine grained material was preconsolidated up to $\sigma_v' = 6000$ kPa (C1, oc).

2.2 Shear parameters

The shear strength of the fine grained material was determined from direct shear tests. Shear parameters were calculated for a linear and a nonlinear limit stress condition using the shear stresses τ at normal stresses of $\sigma_N' = 100, 200$ and 400 kPa. From the consolidation phase of the tests, the permeability of the material was determined.

First, the results of the direct shear tests have been evaluated with the linear limit stress condition after Mohr-Coulomb (equation 5).

$$\tau = \sigma'_n \tan \varphi' + c' \quad (5)$$

The cohesion c' marks the intersection of the linear stress envelope with the vertical axis and is not necessarily the shear stress at vanishing normal stress, since the linearity of the envelope is an just an assumption. For overconsolidated soils the limit stress condition is normally curved (e.g. Atkinson 1993). Therefore the results were approximated with a power function.

$$\tau = A \cdot \left(\frac{\sigma'}{\sigma'_{ref}} \right)^b \quad (6)$$

The parameters A and b can be obtained from an $\ln \tau' - \ln \sigma'$ - diagram. Here A is the intersection of the line with the vertical axis and b is its slope. Table 1 shows the values for φ' and c' , as well as A and b which were derived from the direct shear tests. The value of σ'_{ref} was chosen 1 kPa.

Tab. 1: Classification and shear parameters of the fine grained material

	C1, nc	C1, oc
$\varphi' [^\circ]$	20.8	23.6
$c' [\text{kPa}]$	6.9	19.2
$A [\text{kPa}]$	0.663	1.678
$b [-]$	0.913	0.789

2.3 Nonlinear modifications

Due to the strong influence of the value of cohesion on the calculated critical gradient in the approach of Zou (2000), a curved limit stress condition (equation 6) has been introduced into equation 3. The aim is to have a better representation of the behaviour of the tested clay, especially in the overconsolidated state.

$$\eta = \frac{4 \cdot A \cdot \left(\frac{0.25}{\sigma'_{ref}} \cdot \left(\sigma'_{x0} - \gamma_w \cdot i_{cr} \cdot d_p \right) \right)^b}{2 \cdot \zeta \cdot p' + \frac{d_p}{T_1} \cdot \gamma_w \cdot i_{act}} \quad (7)$$

The calculation of the critical gradient i_{cr} results in an iterative procedure, nevertheless convergence is reached very fast. For the approach of Schmitz (2007) a nonlinear limit stress condition can also be applied. However calculation of i_{cr} is not straight forward due to determination of ξ_0 , ξ_1 and ζ , requiring the stress dependent friction angle.

3 Experimental Procedure

3.1 Experimental setup

The experiments have been conducted in a permeability cell, making it possible to control the cell pressure and the pore water pressure at the top and at the bottom of the sample. To represent a realistic soil composition, layered samples consisting of two filter layers including a clay layer have been prepared. Water flow was imposed in upward direction by increasing the pore water pressure at the bottom of the sample. The water pressure was increased stepwise, until a breakthrough in the clay layer could be observed as a jump in the measured permeability.

3.2 Sample preparation

A sketch of the used specimens is shown in Figure 2. They consist of three layers being prepared separately and put together on the bottom plate of the permeability cell. The filter material was mixed up to a water content of 10 %, put into a preparation cylinder and frozen for minimum 12 h. After freezing, the material was dismantled from the cylinder and put on the bottom plate of the permeability cell. Due to the low water content, freezing did not lead to an increase in soil volume, therefore this procedure made it easy to determine the void ratio of the filter before the tests.

The fine grained specimen was cut out of bigger preconsolidated sample and its natural water content was measured. To enable high gradients of water flow, a height of $h = 1.2$ cm was used for the sealing layer. This is in accordance with the experiments done by Zou (2000) and Schmitz (2007). After the layers were stacked, the specimen was surrounded by two membranes to avoid leakages.

3.3 Testing procedure

All specimens were saturated using a pore water pressure of $u = 200$ kPa and a cell pressure of $\sigma_3 = 220$ kPa. The saturation phase lasted for minimum 12 hours allowing the coarse grained material to thaw. The specimens were consolidated to effective stresses of $\sigma_3' = 200$ kPa or 400 kPa.

After consolidation the pore water pressure at the bottom of the sample u_u was increased stepwise ($\Delta u = 20$ kPa), imposing an upward flow of water through the specimen. As an increase of the gradient decreases the effective stresses in the specimen, one gradient was kept constant for minimum 4 hours to ensure stable conditions in the soil.

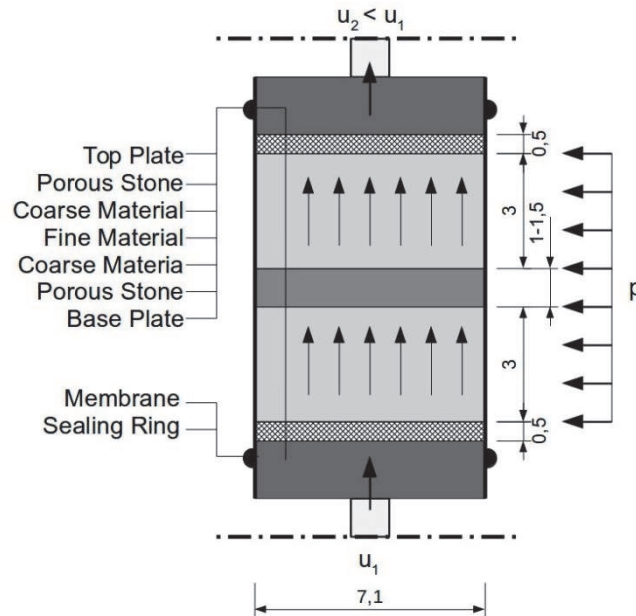


Fig. 2: Setup of specimens in the permeability cell

An experiment was finished, when the measured conductivity of the sample increased with a jump, or when eroded material was visible in the burette of the permeability apparatus. For all the tests a bypass of water flow between the membrane and the soil can be excluded, as the measured permeability in the beginning of the erosion tests and from the consolidation phase of the direct shear tests, was the same. Figure 3 shows the clay layer of specimen 7 after the experiment with a visible piping channel in the middle of the sample. This position of the piping channel confirms that the flow took place through the soil material and not along the membrane.

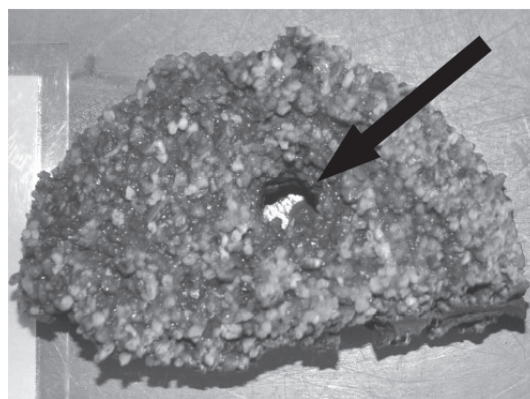


Fig. 3: Specimen of erosion test 7 after the experiment. The piping channel is clearly visible at the top of the arrow. The lower part of specimen was cut off to measure the water content.

4 Results and Discussion

4.1 Normally consolidated samples

Figure 4 shows the results of the erosion experiments on the normally consolidated samples. The horizontal line $\sigma' = 0$, marks the gradient were the pore pressure u is equal the cell pressure σ_3 , which is the highest gradient possible in the experiments. It can be seen, that the measured values of i_{cr} increase with increasing effective stress and decreasing diameter of voids in the filter layer. For an effective cell pressure of $\sigma_3' = 200$ kPa it is possible to compare the experimental values with the criteria by Zou (2000) and Schmitz (2007). However for an effective stress of 400 kPa the criterion by Zou (2000) is far away from with the experimental values. It also can be seen that the nonlinear modification (mod. Zou 2000) has just a slight influence on the evaluated results. The best prediction can be obtained, by the approach from Schmitz (2007), when the proper values of ξ_0 , ξ_1 and ζ , are chosen.

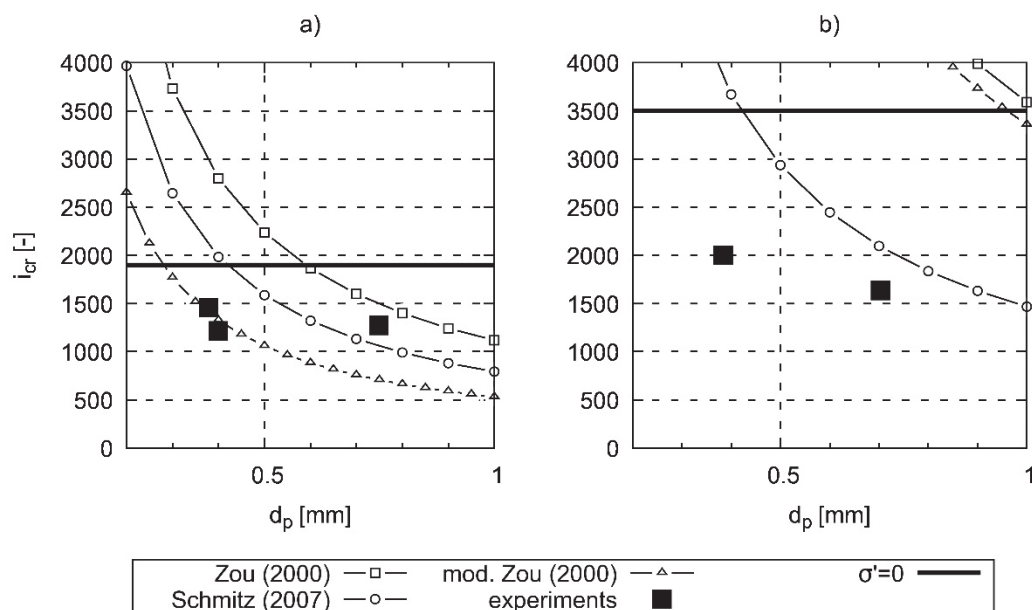


Fig. 4: Results of erosion experiments on normally consolidated samples.
a) $\sigma_3' = 200$ kPa; b) $\sigma_3' = 400$ kPa

4.2 Overconsolidated samples

The overconsolidated samples show similar trends like the normally consolidated ones. For an effective stress of $\sigma_3' = 200$ kPa, there is no influence of d_p visible in the experiments. It is also not possible to reproduce the experimental values with the presented approaches. All calculations highly overestimate the critical hydraulic gradient. This may be due to the strong influence of cohesion on the

calculation results. As for the normally consolidated samples, the best fit can be obtained using the approach of Schmitz (2007), although the range of ξ_0/ξ_1 from 0.2 to 0.6 makes it hard to use for predictions.

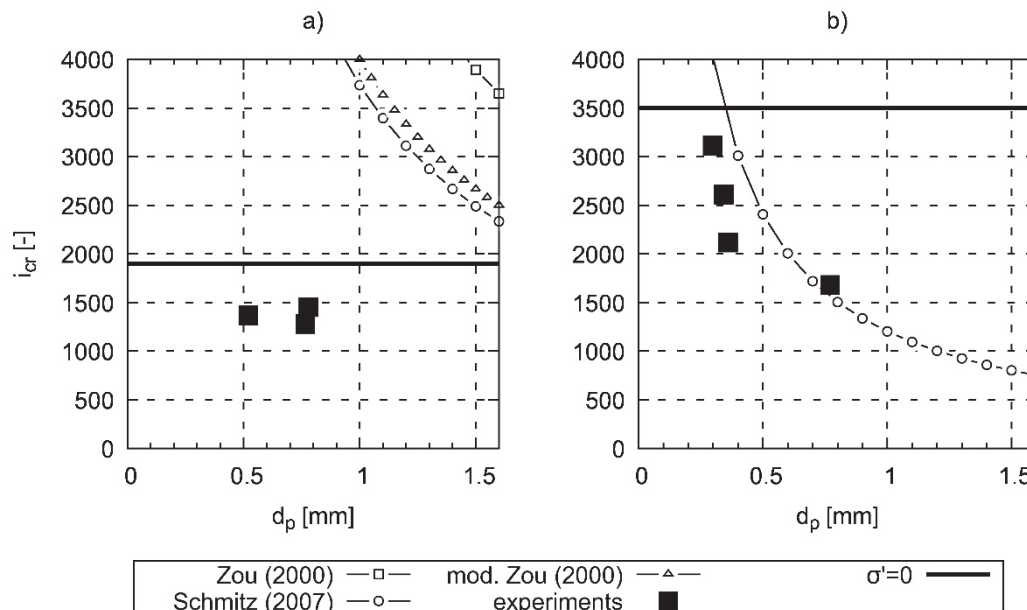


Fig. 5: Results of erosion experiments on overconsolidated samples
a) $\sigma'_3 = 200$ kPa; b) $\sigma'_3 = 400$ kPa

4.3 Dimensional analysis

To get a better understanding of the factors influencing the contact erosion problem for fine grained soils, a dimensional analysis was performed. The main idea of the dimensional analysis is, that “physical phenomena go their way independently of the units we measure them” (Palmer, 2008). This means it is possible to use different units for variables of a physical problem, as long as the dimensions of the variables stay the same. With the help of the Pi-Theorem (Buckingham 1914) it can be stated that a physical problem involving a number of n physical variables can be rewritten in a set of $p = n - k$ dimensionless groups constructed from the original variables, when k is the number of the involved dimensions. Nevertheless when the dimensional analysis is performed successfully, it is possible that important physical values have been forgotten or that unimportant values have been used. In the present case, the analysis is used as a tool to identify the main relations governing the process of contact erosion.

For the investigated problem it seems reasonable to choose the maximum shear strength of the fine grained material τ in [Pa] as an influencing variable. For the dimensional analysis, it seems advantageous to use the shear strength τ , since stress state and stress history are also included in it. The second influencing variable is the equivalent hydraulic diameter of voids d_p [m], which characterises the filtering

material with a geometrical value. Furthermore the density of the fine grained material ρ in $[\text{kg}/\text{m}^3]$ and the gravitational acceleration g in $[\text{m}/\text{s}^2]$ are considered.

Finally i_{cr} itself has to be considered. The critical gradient is used with the dimension of $[\text{Pa}/\text{m}]$, representing the pressure drop of water in every meter of sample. This results in five independent variables containing the dimensions of length, weight and time. From the Pi-Theorem it is possible to derive two dimensionless groups (equation 8).

$$\frac{i_{cr}}{g \cdot \rho} = f\left(\frac{\tau}{d_p \cdot g \cdot \rho}\right) \quad (8)$$

With these two dimensionless groups it is possible to plot all experimental results in one normalized double logarithmic diagram. It can be seen, that the idea of two dimensional groups may be right, as all data for one particular soil can be connected with a straight line. Nevertheless, it is obvious that there has to be an unknown variable which would unify all points into a single line. For further investigations it can be assumed that the contact erosion problem is influenced by a parameter characterising the type of the fine grained soil.

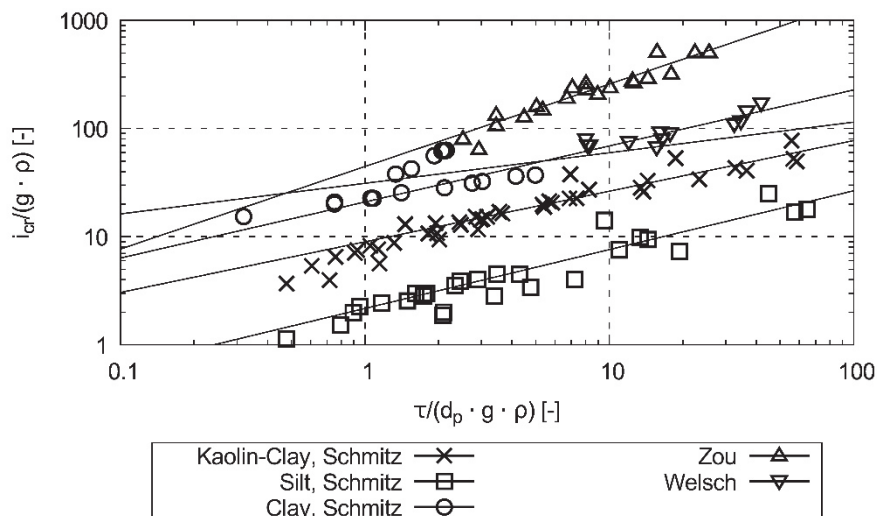


Fig. 6: Results of various erosion tests, normalized with dimensional analysis.

5 Summary

A series of seepage experiments on fine grained soils has been conducted to investigate the critical hydraulic gradient for contact erosion. A permeability cell with an upward flow through the specimen has been used. In the experiments the stress state and the stress history of the sample have been considered. It has been found that i_{cr} is higher for higher stress states in the sample or for higher overconsolidation ratios, as the maximum shear strength of the sample increases. i_{cr} also increases for voids of smaller diameter in the filter material (finer filters). A dimensional analysis has been conducted, showing that the main factors

influencing this problem are the maximum shear stress, the diameter of voids (filter material) and the soil density. An additional parameter needs to be identified which should depend on the type of fine grained soil. Owing to the measured very high critical hydraulic gradients, it was shown that contact erosion for fine grained soils with high plasticity only occurs for special boundary conditions.

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