Hydraulic Heave in Cohesive Soils

Rupture hydraulique du sol en terrain cohérent

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ABSTRACT:

According to DIN EN 1997-1:2009, the shear parameters of the soil are not relevant for stability analysis due to hydraulic heave. Resistance is only activated by the weight of the soil of the failure area. However, during tests on cohesive soils the failure sequence was visually detected as a series of pore widening, initial crack formation and hydraulic induced structure decomposition of the soil in the failure area followed by downstream sudden uplift. On the basis of the test observations a new design approach under consideration of soil weight, shear parameters and an optionally mobilised earth pressure is proposed. The used reference volume represents both the construction as well as the flow situation. Processes that occur before the final uplift are transmitted by the hydraulic gradient i_{crit} on the failure area. The final limit state is interpreted as a shear failure.

RÉSUMÉ:

Pour l'évaluation d'une défaillance par rupture hydraulique du sol, les normes DIN EN 1997-1:2009 sur les propriétés de cisaillement du sous-sol ne sont pas pertinentes. Les résistances sont activées seulement par le poids du sous-sol. Des essais de visualisation du déroulement de la défaillance en sol cohérent ont montré que le processus se caractérise par une succession d'effets de dilatation des pores, de formation initiale de fissures, de destruction structurelle du continuum dans la zone de défaillance, suivis d'une rupture brutale de la surface. Conformément au déroulement de la défaillance, les calculs présentés prennent en compte, en tant que résistances, le poids mort du sol, sa résistance au cisaillement de rupture ainsi que, le cas échéant, l'existence d'une contraction du sol. La mise en évidence se fait sur une zone de défaillance-modèle reproduisant la situation de construction et de courant. Les phases précédant la rupture finale sont transférées à la zone défaillante par les gradients hydrauliques i_{crit}. L'état limite final est interprété comme rupture par cisaillement.

KEYWORDS: stability analysis, design approach, cohesive soil behaviour, test method

1 INTRODUCTION

Stability analysis against hydraulic heave considers in accordance with Eurocode 7 (DIN EN 1997-1:2009) exclusively the soil weight as stabilizing influence. Thus, especially for cohesive soils the design approach takes not the available shear strength into account. In accordance with current regulations, a consideration of shear properties is only possible if special experience with the material is available.

In order to optimize the design approach of hydraulic heave for cohesive soils within the constraints of the valid standard, it is necessary to examine various questions. This includes:

- How to characterize the failure mechanism of hydraulic heave in cohesive soils?
- Which is the characterizing difference of this for non-cohesive soils?
- How to describe or determine an appropriate reference volume for the limit state?
- Which are the controlling material parameters of the limit state?
- What is the effect of the water content on these soil properties?

In order to develop a new design approach, it is also necessary to consider temporal aspects of the failure sequence. An adaptation of the design approach to the problem of a flow around excavation wall must be considered for both drained and undrained soil properties. In addition, a specific consideration of the supporting effect of the subsoil abutment at sheeting location is possibly positive.

2 CURRENT DESIGN

Changes in groundwater regime are critical for hydraulic heave and for particle transport processes in the soil. The stability of earth structures, excavations and foundations is influenced by both phenomena during and after construction.

From a global point of view, changes in ground water level cause changing pore water pressures, which might affect the general balance within the soil continuum. A violation of the limit state condition leads in this context to failure. On the other hand an increased flow of water though the voids leads to changes in the soil structure due to particle transport. To evaluate hydraulic heave a limit state condition is critical. Particle transport processes have only an indirect effect on the overall stability.

Referred to Eurocode 7 (DIN EN 1997-1:2009) for a stability analysis against hydraulic heave for each possible soil prism the limit state condition according to Equations (1) or (2) has to be evaluated.

$$u_{dst;d} \leq \sigma_{stb;d} \tag{1}$$

$$S_{dst;d} \leq G'_{stb;d} \tag{2}$$

The limiting criterion in Equation (1) is formulated as a comparison of the design values of total stresses $\sigma_{stb;d}$ and pore water pressures $u_{dst;d}$. Equation (2) refers the context to the flow force $S_{dst;d}$ and the buoyancy weight $G'_{stb;d}$ and therefore to the effective stress.

The following additional considerations of how to make assumptions and define the boundary conditions for hydraulic heave stability analysis and their interpretations are given in DIN 1054:2010. For example, in flow around an excavation wall conform to Terzaghi's approach the reference volume can be taken into account as a rectangle with a width of half of the wall embedment depth. The hydraulic impact may be differentiated on the basis of the soil type. Accordingly, the partial safety factors can be distinguished for soils with flow unfavorable or favorable characteristics. For at least stiff cohesive soils the consideration of cohesion or tensile strength as a material-specific parameter is permitted, if special expertise and experience are available.

In particular, the reference to a possible consideration of additional material specific strength properties indicates that the valid approach lead to a conservative interpretation of the relevant boundary conditions. Thus, according to the current experience the approach is suitable especially for non-cohesive soils. The existing shear strength of cohesive soils is neglected.

3 EXPERIMENTAL ANALYSIS

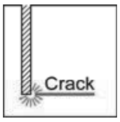
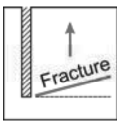
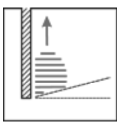
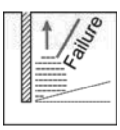
3.1 *Visual failure state determination*

The valid failure mechanism of cohesive soils is not so far comprehensively analysed. Therefore experiments were carried out to describe the structural failure during a hydraulic heave in cohesive soils on the basis of a visual identification (Wudtke and Witt 2006). The experimental setup and procedure rebuild the water flow around a pit wall. The dividing wall was fixed on frame sides.

The following questions were examined by the experiments:

- Which sequence of events can be characterized as the most important attribute of hydraulic heave in cohesive soils?
- What phenomena are announcing the limit state?
- How the hydraulic decomposition of a cohesive soil can be described?
- Where does the initial damage occur?

Table 1. Failure sequence during hydraulic heave in cohesive soils (Wudtke 2013)

	Phase	Description
Failure sequence		- water content increase, - development of an initial crack with origin at the foot of the dividing wall
		- continuing pore widening, - heaving of the downstream surface, - development of a deep-lying fracture
		- continuous crack initiation and closure; - continuing heaving, - locally limited decomposition of the soil structure, dissection of the soil into aggregates
		- continuing rapidly increasing elevation of the soil surface, - sudden failure as hydraulic heave, - local potential equalization, failure body erosion

From visual observations it can be stated that the failure sequence is characterized as pore widening effects, soil structure destroying crack opening and closing followed by a final failure of the downstream surface.

Relative to a real excavation pit, the decomposition of the soil structure at the foot of the flow around pit wall leads to changing hydraulic routeing and flow distribution in the soil. Consequence of the changed hydraulic boundary conditions is increasing hydraulic impacts at the excavation side of the pit wall.

The type of failure sequence is independent of the shear strength of the soil. The failure principle is subjected to the availability of cohesion and tensile strength. The activation of resistances is related to the shear strength of the soil and the stress state.

For low-cohesive soils in the course of crack opening and closing processes a comparatively small resistance is activated. At the same time the influence of shearing increases. How much the resistance can be mobilized depends on the stress state of the reference volume and the hydraulic head difference. An increase of the water content causes plasticity changes and is therefore sensitive to the occurrence of the failure.

3.2 *Critical hydraulic gradient*

Conventional tensile strength tests show that the tensile strength of cohesive soils decreases with increasing of water content. Basically it can be noted that tensile strength is available only for soils with at least stiff plastic consistency. In these tests due to the mechanically induced sample destruction the conventionally determined tensile strength is only partially representative for the detected hydraulic induced loss of structural integrity of cohesive soils, (Wudtke and Witt 2010).

The development of a new testing method was required to identify adequately the hydraulic induced limit state condition. Objective of the testing method is to identify and quantify for a certain construction status valid critical hydraulic gradient i_{crit}. The parameter represents the hydraulic impact required for a structure decomposition of the soil. There are three main influences on the test results: 1 – representative stress state, 2 – pore water pressure, 3 – soil water content. In contrast to conventional testing methods the test method considers a hydraulic effect as the driving force for specimen destruction. (Wudtke 2013)

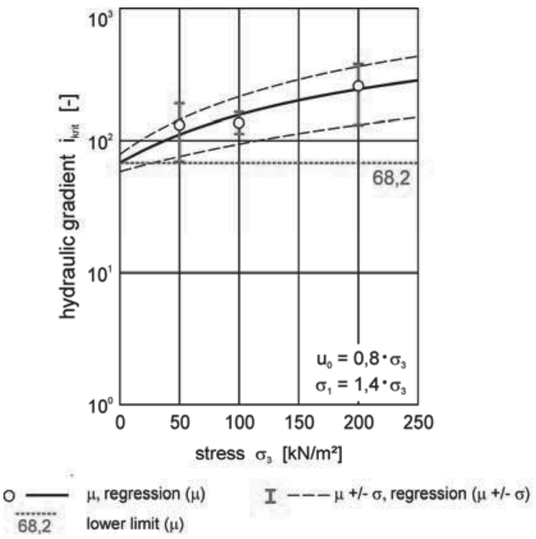


Figure 1. Stress-dependent critical hydraulic gradient, mean and variation.

Examplary for an inorganic clay of medium plasticity (φ' = 23.5° und c' = 13.7 kN/m²) the evaluation of the stress-dependent development of i_{crit} is shown in Figure 1. As experimental conditions a principal stress ratio of $\sigma_1 = 1.4 \cdot \sigma_3$ at an initial pore water pressure of $u_0 = 0.8 \cdot \sigma_3$ were defined. From the test results, the lowest limit value of i_{crit} can be determined for this kind of soil, $i_{crit} \geq 68.2$ (Figure 1).

The experimental determination of the parameter i_{crit} is basis for defining the representative reference volume for calculating the limit state of the hydraulic heave. Considering a construction state depending flow distribution and a valid limit value of i_{crit} there is a failure area, which can be distinguished as hydraulic induced structure decomposition and shear failure. Therefore the failure process consists of two phases, the structure decomposition followed by shear failure.

4 RECOMMENDATION OF A DESIGN APPROACH

4.1 Reference volume

The definition of a representative reference volume is essential for transferring the typical failure sequence of hydraulic heave in cohesive soils to the design approach. For a failure process given resistance activation the spatial arrangement of the resistances shall prevail.

The reference volume is based on the flow situation, which can be determined by the construction state and the soil stratification. The areas of hydraulically induced structure decomposition of the soil, as well as of shear failure are defined as part of the reference volume (Figure 2). The outer boundary of the reference volume is represented by the flow line, which is tangent to the curve of i_{crit}. For determination of the shear body, the relevant flow line is approximated as a parabolic function. A widening of the reference volume beyond the base of the wall, can only occur if i_{crit} is realized by the given flow situation.

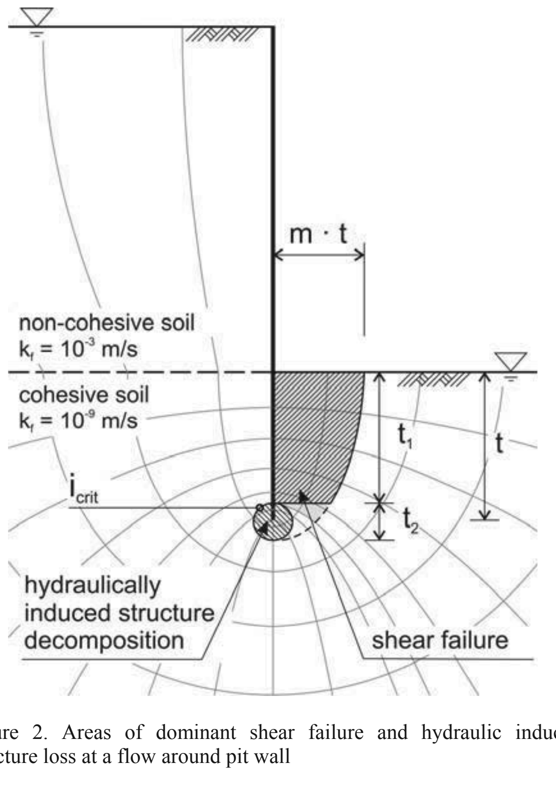


Figure 2. Areas of dominant shear failure and hydraulic induced structure loss at a flow around pit wall

4.2 Hydraulic induced soil structure decomposition

In the first step of safety analysis against hydraulic heave in cohesive soils, the maximum possible hydraulic gradient max i relevant for soil structure decomposition is estimated. Max i is related to a construction stage and has to be compared with soil-specific i_{crit}.

The approach aims to identify a critical hydraulic loaded soil area. For the relevant part of the reference volume in the subsequent shear failure analysis no resistances are activated. Basic idea of the proposed design approach is the assumption of continuously decreasing shear strength of the soil in dependency of increasing hydraulic gradient. In the second step, the shear failure analysis, reduced shear resistances can be considered.

To analyze possible soil damage and soil structure decomposition, the following relationships have to be investigated.

Damage criterion

$$i_{crit} \geq max\ i \geq i_{damage} = \frac{\gamma'}{\gamma_w} \quad (3)$$

Failure Criterion

$$max\ i \geq i_{crit} \quad (4)$$

Soil damage starts when the effective stress disappears due to the hydraulic impact. This occurs first when $\sigma'_3 \leq 0$. Here the potentially shear failure body has to be identified. Depending on the size of max i, i_{damage} and i_{crit} different results of the reference volume are possible.

4.3 Shear failure

As second part of the design approach the analysis of the safety against shear failure must be considered. The analysis is performed on the soil body remaining after the possibly hydraulic structure decomposition of the soil continuum. For the calculation a time-consistent failure state is valid, i.e. it is necessary to provide a strict separation of drained and undrained soil properties and flow conditions.

The stability of the reference volume to shear failure is determined by the following equation.

$$S \leq G' + C_v + F_v \quad (5)$$

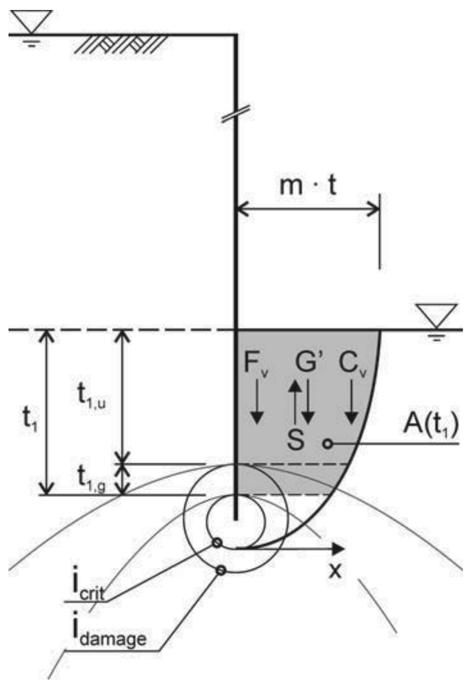


Figure 3. Parameter definition – shear failure

The limit state analysis is carried out as a balance of vertical forces of the flow impact and the acting resistances at the reference volume (Figure 3). Resistances are represented by the buoyant soil weight G', the shear resistance acting at the shear surface C_v and the bracing acting at the pit wall F_v.

The resistance of soil weight is determined by the submerged unit weight of the soil γ' and the area of the shear body A(t_1). To determine the soil weight a consideration of soil damage is generally not required. As a conservative assumption a consideration of hydraulic resistances from damaged soil areas can be dispensed. For the calculation a shear body height of $t_{1,u}$ is decisive. For example in a construction state with no filter load, the equivalent resistance can be determined according to Equation (6). In stratified soil conditions, a layer-related determination of the area proportions of the shear body is relevant.

$$G' = \gamma' \cdot A(t_1) \tag{6}$$

In the excavation area near to the sheet pile after a definite construction stage, there will be bulging of the surface (uplift effect), this leads to activation of shear resistances in dependency of soil shear parameters. Damaged and undamaged soil areas have to be distinguished considering failure velocity depending resistance activation. Damaged soil areas, which have a comparatively lower shear resistance, are considered by available hydraulic gradient $i \geq \gamma'/\gamma_w$ (see Equation (3)).

The permeability of soil and the excavation velocity respectively load relieving governs the resistance activation of the soil during hydraulic heave. Due to reduction of soil load in excavation area, the pore water pressure is released slowly so that the undrained shear parameters are relevant. In contrast, drained conditions are relevant for long phases without excavation progress and relatively permeable soils. In the stability analysis for drained conditions the effective cohesion c' and for undrained conditions the undrained cohesion c_u must be considered. The friction of soil should be neglected.

For calculating the shear resistance, shear body height t_1 is divided into two different heights depending on the soil structure (Equation (7) and Figure 3). For the height of damaged area, the relevant shear property and for undamaged area its shear resistance must be assigned.

$$t_1 = t_{1,u} + t_{1,d} \tag{7}$$

$$C_v = C_{v,u} + C_{v,d} = c_u \cdot t_{1,u} + c_{u,d} \cdot t_{1,d} \tag{8}$$

$C_{v,u}$ and $C_{v,d}$ represent the activated shear resistances in undamaged and damaged areas. Equation (8) shows in this case a soil with undrained conditions.

Additionally, the soil bracing, supporting system of the pit wall and other ground supports can be considered as a further resistance. The consideration of abutment as an additional resistance effect can be described as a vertical force which is transmitted by the sheeting system. In accordance with DIN 1054:2010 this represents the difference between the stress state without soil deformation and the actual or planned stress at the excavation abutment.

The consideration of abutment resistance requires the stability of sheeting system. It must be ensured that the construction has sufficient safety to permit a deformation free transferring of vertical forces at sheeting foot.

Transferring of vertical forces by the sheet pile is depending on frictional component of shear strength. The drained or undrained shear properties must be assigned in relation to the anticipated load conditions. A definite amount of earth pressure can be transferred to sheeting foot by consideration of contact friction between soil and sheeting. The vertical load component of the active earth pressure is essential for the determination of the resistance and generally should be estimated conservatively. If the hydraulic impact does not lead to hydraulic soil damage the abutment effect F_v corresponds to the vertical load of the active earth pressure.

$$F_v = E_{a,v} \tag{9}$$

The approach for determining the resistance of the abutment effect of sheeting represents a general solution to the problem, and requires the full activation of the passive earth pressure in the pit. The most liable solution can be achieved by determining the difference stress distribution between an unstressed, not deformed soil and the stress condition of the planned construction at the excavation abutment.

According to equation (10), the flow force S is determined depending on area of the shear body $A(t_1)$, the available mean hydraulic gradient i_M at the bottom side of $A(t_1)$ and the weight of water γ_w.

$$S = i_M \cdot \gamma_w \cdot A(t_1) \tag{10}$$

5 CONCLUSIONS

The presented design approach for the stability analysis against hydraulic heave in cohesive soils explicitly considers the available resistance due to cohesion. In addition, the consideration of a ground support effect acting at the sheeting is integrated.

The design approach is divided in two steps. A first step considers the hydraulic induced structure decomposition of the cohesive soil beneath the foot of the sheet pile. The condition is essential to determine the reference volume of the failure area. The critical hydraulic gradient i_{crit} depends on the cohesion, density and stress history of the soil. I_{crit} can be determined experimentally or might be estimated by experience.

In the second step the equilibrium of the vertical forces acting at the reference volume is analyzed. In addition to the current design approach (DIN EN 1997-1:2009), side forces and cohesion of the appropriate shape of the failure area are taken into account. The second part of the design approach corresponds to the verification of uplift failure.

During hydraulic induced structure decomposition of the soil the flow conditions will change. Thus an update of the distribution of pore water pressure should be taken into account.

The assumption of resistances from the weight and the available shear resistance in accordance with current design practice (DIN EN 1997-1:2009) leads to a possible classification as design situations HYD and GEO. The assignment of coherent partial factors is still an open question under discussion.

6 REFERENCES

DIN 1054:2010. Baugrund – Sicherheitsnachweise im Erd und Grundbau – Ergänzende Regelungen zu DIN EN 1997-1.

DIN EN 1997-1:2009. Eurocode 7: Geotechnical design, Part 1: General rules; German version EN 1997-1.

Wudtke, R.-B. (2013). Hydraulischer Grundbruch in bindigem Baugrund. Dissertation, Bauhaus-Universität Weimar. Weimar.

Wudtke, R.-B. and Witt, K. J. (2006). A Static Analysis of Hydraulic Heave in Cohesive Soil. 3rd International Conference on Scour and Erosion, Amsterdam, 1 - 3 November 2006, 251.

Wudtke, R.-B. and Witt, K. J. (2010). Hydraulischer Grundbruch im bindigen Baugrund - Schadensmechanismen und Nachweisstrategie. 9. Geotechnik-Tag in München - Wechselwirkungen Boden - Wasser - Bauwerk, München, 19.02.2010, 33 - 44.