Transient excess pore water pressures causing soil deformation and hydraulic failure

Etats transitoires de surpression interstitielle causant déformations et rupture hydraulique du sol

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ABSTRACT: The soil below the piezometric level should be regarded as a 3-phase medium, containing gas, water and solids. Therefore protection measures against hydraulic soil failure should take into consideration not only steady but also transient states. The design procedure and the calculation results for a chosen example are presented aiming to encourage engineers to apply such design methods for open pit foundations in order to avoid potential failure caused by changing water pressures.


1 INTRODUCTION

Soils under water should be considered as an unsaturated porous medium containing solid particles, pore water and gas bubbles. Using a three-phase model, the mechanical behaviour of such soils may be described by an extended consolidation equation to calculate transient excess pore water pressures, induced by rapid external pressure changes such as water level draw down loading or excavation. Observations and laboratory measurements have shown that pore water under natural conditions contains a remarkable amount of microscopic air bubbles up to even more than 15 % embedded within the pore fluid inside the soil skeleton. These bubbles play a key role in soil behaviour, explaining soil failure and structure deformation. Pressure changes applied on unsaturated submerged soils initiate changes in volume of the embedded air bubbles inside the gas-water mixture. The immediate reaction of these bubbles is to cause local transient pore water flow. This process is hampered by low permeability thus creating a delayed pore water pressure response, initiating excess pore water pressures. The effective stress level is reduced, leading to loss of friction. In non-cohesive soils microscopic small structure changes may take place. Heaving and settling of soil segments and even interweaving soil movements (translation and rotation) may be induced. The decrease of external pressures acting on unsaturated submerged soils may therefore induce fluidisation. In low permeable soils such as silt and clay, excess pore water pressure may be induced inside the pore volume, decreasing with time until the necessary amount of pore water has been expelled out of the deforming soil, thus causing pre-failure deformations and finally even structure failures. Using finite element simulations the consequences of the pore water-air interaction of unsaturated submerged soils have been examined (Köhler et al., 1999).

A new and easy method of dimensioning against hydraulic soil failure at transient state may be introduced into the stability calculations. To attain sufficient stability against hydraulic failure the design procedure was developed based on the results of pore water pressure measurements (in situ and 1:1
scale model tests) and numerical calculations, taking the subsoil as a 3-phase system into account. The design method is described and is supported by a practical application example. The design procedure and the calculation results for a chosen protection structure are presented aiming to encourage engineers to apply such designs for the dimensioning purpose of open pit foundations.

2 DESIGN METHOD AND PHYSICAL BACKGROUND

It was found, that stability of retaining walls is very much related to transient pore water pressure developments and time dependent local flow conditions, regarding the soil as a three phase system. One of the main impact loading in relation to subsoil stability is the rapid water level draw down loading and/or the excavation itself, which may cause transient excess pore water pressure in the soil (Fig. 1).

The determination of the time dependant acting excess pore water pressure is a rather complex process, because its development is connected with the expansion of microscopic small gas bubbles embedded in the pore fluid. In addition it is coupled by the influence of the simultaneously deforming soil, caused by the external pressure drop. Despite the fact, that theoretical solutions are known, a realistic development of the induced excess pore water pressure cannot easily be modeled with these solutions because of the high non linearity of the process.

Therefore as a key for a practical approach at limit state conditions, the induced excess pore water pressure may be described by a simplified exponential function with only one parameter: the pore water pressure parameter b. This parameter is linked to the permeability of the subsoil (Fig. 2). Its value is given on the basis of an extended consolidation equation (Biot, 1941), additionally slightly corrected to the account for the non linear influence of soil behaviour by the results of in situ measurements, model tests and numerical simulations, undertaken by the author.

The excess pore water pressure \( \Delta u(z,t) \), which is acting due to transient pore water flow caused by water level draw down, may be gained as a function of the depth z normal to the exit plane of the permeating pore water according to:

\[
\Delta u(z,t) = \gamma'_{soil} \cdot dh \cdot B(z,t)
\]
\[ \Delta u(z,t) = dh \cdot \gamma_w \left( 1 - e^{-b \cdot z} \right) \] (1)

For practical use the parameter \( b \) results out of numerical simulations and measurements, which have been combined in a simple diagram to ascertain the pore water pressure parameter \( b \) (Fig. 2). The dependency of the parameter \( b \) of the soil permeability \( k \), as shown in Figure 2, comprises the correlation between the gained measurement results and the practically almost unidentifiable actual gas content in natural pore water. According to the very low stress level at sea bed and the compressibility of the soil skeleton, the loading case of a rapid water level draw down in open pits may easily cause soil failure.

Especially in low permeable soils excavation of the open pit is usually performed without additional support of wells by using a pre-installed ground water head lowering. Although the water discharge is small in low permeable soils, the simultaneously acting pore pressure due to seepage needs to be taken into account in order to avoid hydraulic soil failure.

Generally the time of lowering of the water level as well as the excavation of the pit itself may in practice range between hours, days and even weeks, which of course has a direct influence on the excess pore water pressure development in the subsoil. The faster the external water level lowering or excavation takes place the more excess pore water pressures have to be taken into consideration at transient state. The velocity \( v_A = \frac{dh}{t_A} \, [m/s] \) of water level lowering or of the excavation rate need to be kept smaller than the prevailing permeability \( k \, [m/s] \) of the subsoil \( (v_A < k) \) in order to avoid transient pore pressure conditions, which may endanger soil bed stability.

\[
B(z,t) = \left( 1 - e^{-b \cdot z} \right)
\]

with

\[ z = \text{soil depth below water level} \]
\[ b = \text{pore water pressure parameter} \]
\[ B(z,t) \, [\text{-}] \]

Figure 2. Diagram of the pore water pressure parameter \( b \, [1/m] \) according to the prevailing soil permeability \( k \, [m/s] \) as a function of the acting draw down time \( t_A \, [s] \) (right hand side) and the transfer function \( B(z,t) \) (left hand side), which expresses the damping effect of the actual pressure spreading in the subsoil due to external pressure changes, valid for selected calculation parameters.

In order to attain safety against hydraulic soil failure at transient state, the uplift of the submerged unit weight \( \gamma' \) of a specified soil body with an endangered layer thickness at seabed of soil depth \( z_{crit} \) should at least be maintained at the critical time step \( t_A = t_{crit} \) (see fig. 1). The time dependent effect of the excess pore water pressures \( \Delta u(z,t) \) due to the water level lowering in the open pit, preventing up
lift or shear failure, should be compensated by the equivalent top load of the necessary soil weight, in order to maintain the requested factor of safety $f$:

$$ f = \frac{G'}{\Delta u(z,t)} $$

with $G' = \gamma' \cdot z_{\text{crit}}$ [kN/m$^2$] \quad (3) \quad \text{and} \quad \Delta u(z,t) = \gamma_w \cdot d_h \cdot B(z,t)$ [kN/m$^2$] \quad (4) \quad \text{and} \quad B(z,t) = 1 - e^{-bz}$ \quad (5)

With $z = z_{\text{crit}}$, the critical excess pore water pressure $\Delta u(z,t)$ has to be accounted for at the critical time step $t_A = t_{\text{crit}}$ with the aid of the following expression for the critical soil depth $z_{\text{crit}}$:

$$ z_{\text{crit}} = \frac{1}{b} \ln \left( \frac{\gamma_w \cdot d_h \cdot b}{\gamma'} \right) $$

The factor of safety $f$ against hydraulic soil failure at transient state follows as:

$$ f = \frac{\gamma' \cdot \frac{1}{b} \ln \left( \frac{\gamma_w \cdot d_h \cdot b}{\gamma'} \right)}{\gamma_w \cdot d_h \cdot \left(1 - e^{-bz_{\text{crit}}} \right)} $$

with the unit weight of the water $\gamma_w$

The pore water pressure parameter $b$ [1/m] may be taken out of the diagram from Figure 2 or Figure 3. The factor of safety against hydraulic soil failure at time step $t_A$ [s] just after water level lowering by the difference of water level $d_h$ [m] may be ascertained. In case of insufficient stability against failure an additional top load of a sufficient permeable granular filter layer $(G'_F = \gamma' \cdot d_F)$ or even the effective cohesion $c'$ of the soil may be taken into consideration, which leads to the following expression:

$$ f = \frac{\gamma' \cdot \frac{1}{b} \ln \left( \frac{\gamma_w \cdot d_h \cdot b}{\gamma'} \right) + \gamma' \cdot d_F + \langle c' \rangle}{\gamma_w \cdot d_h \cdot \left(1 - e^{-bz_{\text{crit}}} \right)} $$

Figure 3. Pore water pressure parameter $b$ [1/m] to be used for soils according to coefficients of permeability between $k = 1 \times 10^{-3}$ [m/s] and $k = 1 \times 10^{-7}$ [m/s] as a function of draw down time $t_A$ [s]
The term of soil cohesion $c'$ (here used as a parameter to withstand the requested tensile strength) in equation (8) is written in brackets in order to ensure, that it should only be adopted, if one can accept this parameter would hold its value for the loading case of a rapid water level lowering. This should be proved in laboratory investigations, especially carried out to this purpose.

3 STABILITY CONSIDERATIONS

The stability of a retaining wall at the loading case of excavation and de-watering of the open pit suffers by failure conditions such as hydraulic soil failure (see fig. 4), usually maintained against seepage at steady state only, but also by failure due to insufficient activation of the passive earth pressure in front of the retaining wall. The stability of the wall may be highly endangered by this failure mode (see fig. 5), especially at transient state at the critical time $t_A$ just after water level lowering and excavation. In order to withstand such loading case, both failure conditions (transient and steady state) need to be investigated aiming to assure the construction at limit state. The following three failure modes need be investigated:

- stability against hydraulic failure due to seepage at steady state (see fig. 4, without $\Delta u(z,t)$)
- stability against hydraulic failure (hydrodynamic soil deformation) due to seepage at transient state in order to prevent uplift at critical soil depth $z_{\text{crit}}$ (see fig. 1 and fig. 4, with $\Delta u(z,t)$)
- sliding at the potential failure plane activating passive earth pressure (see fig. 5, with $\Delta u(z,t)$)

Supposing a draw down value of $dh = 4.0 \text{ m}$ in a draw down time $t_A = 10 \text{ h}$ together with a permeability coefficient of the soil of $k = 2 \times 10^{-9} \text{ [m/s]}$ the pore water pressure parameter $b$ may be ascertained from diagram (see fig. 2) to the value of $b = 2$.

The requested factor of safety against hydraulic failure at steady state with $f = 2.4$ ($f_{\text{requested}} \geq 2$) would be satisfied.

Following equations (1) to (7) the factor of safety against hydraulic failure at transient state was ascertained with the pore water pressure parameter $b = 2$ (see fig. 2) at critical time $t_A = 10 \text{ [h]}$ and criti

Figure 4. Terzhagi’s model of a prismatic soil body used in describing hydraulic soil failure in a homogenous soil caused by the acting pore pressure $\Delta u$ induced by the seepage at steady state, here additionally extended for the loading case of a rapid water level lowering in the open pit, inducing transient excess pore water pressure $\Delta u(z,t)$
cal soil depth $z_{\text{crit}} = 1.06$ [m] (see fig. 1) to $f = 0.29$ ($f_{\text{requested}} \geq 1.2 \ldots 1.5$), which would be potentially unsafe.

The requested safety factor may be maintained by an equivalent thickness $d_F$ [m] of an adequate filter top load, assuming submerged unit weight of an additional filter layer with a porosity of $n = 0.45$ [-] and a stone density of $\rho_s = 2.65$ [t/m³] to $\gamma' = 9.1$ [kN/m³]. In order to maintain sufficient stability against hydraulic soil failure, the time $t_A$ of water level lowering and/or excavation should be at least 12 days (even longer). The factor of safety could be ascertained together with this additional top load chosen as a 40 cm thick filter layer to $f = 1.27$ ($f_{\text{requested}} \geq 1.2$) using equation (8). If the filter layer is to be used as a filter drain for longer periods the chosen structure and construction time would meet benefit costs. Where the drain is not needed long term, the extra costs to maintain safety at transient conditions have to be counted as additional cost factors in order to provide sufficient safety during construction time.

The failure mode of sliding at the potential failure plane with $\Delta u(z, t)$ (see fig. 5) activating passive earth pressure needs to be investigated. It leads to the solution, that the additional top load of the filter layer would be necessary in order to provide sufficient safety against shear failure aiming to ensure the structure during construction time.

4 COMPARISON WITH A TRANSIENT GROUND WATER MODEL

Comparison calculations have been performed by using a commercial 2-D finite element program (GGU, 2002) in order to predict time dependent pressure release. Assumptions had to be taken into account concerning the compressibility of the pore water and the calculation model:
- uncoupled calculation (coefficient of volume compressibility of soil remains constant)
- application of a three-phase model, assuming unsaturated conditions below the piezometric line

These assumptions allow estimates to remain on the safe side. Usually such a program code would not be used in this way modelling a free aquifer.

The velocity $v_{zA} = 1.1 \times 10^{-4}$ [m/s] of the pressure release due to de watering of the open pit is rapid compared to the permeability $k = 1 \times 10^{-6}$ [m/s] of the soil ($v_{zA} > k$). Thus confined conditions may be...
applied. To select the specific storage \( S_s \), the following values have been adopted: porosity \( n = 0.405 \) [-], degree of saturation \( S = 0.9 \) [-], soil stiffness \( E_S = 22000 \) [kN/m²]. The specific storage \( S_s \) has been estimated using the well known storage equation:

\[
S_s = \gamma_w (1/E_s + n \cdot \beta_w) \quad [1/m] \quad (10)
\]

\[
\beta_w = 5 \times 10^{-7} + (1-S)/(p_{amb} + p_{hyd}) \quad [m^2/kN] \quad (11)
\]

where \( \gamma_w \) = unit weight of water; \( 1/E_s = \) coefficient of volume compressibility; \( \beta_w = \) compressibility of water-soil mixture, \( p_{amb} = \) mean barometric pressure; \( p_{hyd} = \) mean pore water pressure at the regarded depth below piezometric line.

Thus variation of hydraulic head (i.e. pore water pressure) with time and location have been calculated. The result of the pore water pressure at time \( t_A = 10 \) [h] dependent on time and location is depicted in Figure 6. The calculations show clearly that no significant reduction of pore pressures below the soil bed of the pit can be expected at transient state. Only small reductions at the ground surface of the de-watered pit take place. However the pore water pressure on the right hand side of the retaining wall remains unchanged from that of the state before the beginning of de-watering. The pore pressure release, gained out of the transient ground water model has been plotted over the soil depth \( z \) [m] (see fig. 6). Comparing this result shown as the thick black line drawn between the curves 4 and 5 in Figure 6 (left hand side), the pressure release due to the external pressure drop by water level lowering in the pit results in a damped pressure distribution, which may be expressed by the transfer function \( B(z, t) \).

In this comparison study the function \( B(z, t) \) analogue equation (5) follows a pressure distribution plotted over the soil depth \( z \) [m] to be gained with a pore water pressure parameter \( b = 0.39 \) [1/m]. It should be mentioned, that the calculated time dependent pore pressure distribution at transient state results out of an uncoupled finite element model, which is based on following statements:

- the static unloading due to excavation would induce deformation of the soil in form of soil bed heaving causing simultaneous pore water pressure release, which is not usually included in a hydraulic FE-simulation yet
- changes in permeability and storage characteristics due to transient pore water pressure changes, which cause additional soil deformation, is also not taken into account in the used hydraulic model.
Instead of using the before mentioned diagrams for the parameter \( b \) (see fig. 2 and 3) the influence of soil deformation due to external pressure changes might also be disregarded by using a simple approximation for the calculation of the pore water pressure parameter \( b \) [1/m], taking the following simplified expression of equation (12) into consideration.

\[
b = \sqrt{\frac{\pi \cdot \gamma_W \cdot (n \cdot \beta_W + \alpha)}{2 \cdot k \cdot t_A}} \tag{12}
\]

Applying this simplified calculation in ascertaining the pore water pressure parameter \( b \) out of equation (12) using the following calculation parameters together with equations (10) and (11) would lead to the same parameter \( b = 0.39 \) [1/m] as gained out of the transient hydraulic model (see fig. 6).

Used calculation parameters: \( \alpha = 1/E_s \) [m²/kN] (stiffness \( E_s \) dependent on time and stress rate), Soil stiffness \( E_s = 22000 \) [kN/m²], pore volume \( n = 0.405 \) [-], degree of saturation \( S = 0.9 \) [-], soil permeability \( k = 1 \times 10^{-6} \) [m/s], draw down time \( t_A = 10 \) [h], unit weight of water \( \gamma_W = 10 \) [kN/m³], \( p_{atm} = 101.3 \) [kN/m²], \( \rho_{hyd} = (dh/2 + z_{crit}) \cdot \gamma_W = 30 \) [kN/m³]

Following equations (1) to (7) the factor of safety against hydraulic failure at transient state may be ascertained with the pore water pressure parameter \( b = 0.39 \) (calculated by equation (12)) at critical time \( t_A = 10 \) [h] and critical soil depth \( z_{crit} = 1.19 \) [m] (see fig. 1) to \( f = 0.79 \) (« \( f_{requested} \geq 1.2 ... 1.5 \)).

The draw down velocity \( v_{zA} \) [m/s] of the lowering water level exceeds the permeability coefficient \( k \) [m/s] of the subsoil about 110 times (\( v_{zA} = 110 \cdot k \)), which causes unstable soil bed conditions. The upper soil layer of the critical thickness \( d_{soil} = z_{crit} \) is endangered by hydrodynamic soil deformation, which may lead to fluidisation (liquefaction) and soil bed heaving. Vertical and horizontal soil deformation may be correlated with an increasing bending of the retaining wall, eventually even causing collapse of the structure by shear failure (see fig. 5).

In order to maintain sufficient safety against hydraulic soil failure at transient state, the draw down time of de-watering the pit should be chosen to \( t_A \geq 24 \) [h].

5 CONCLUSION AND OUTLOOK

Safety assessment calculations are suggested against hydraulic soil failure at transient state, based on simplified formulations for the temporarily acting excess pore water pressure due to external pressure drop such as rapid water level lowering and/or excavation in open pits. The new and easy design procedure may ensure structures at limit state, which could easily suffer from hydrodynamic soil deformation caused by transient pore water pressures. The soil under water should be regarded as a three phase medium, containing solid soil particles and gas bubbles embedded in the pore fluid, causing unsaturated submerged soil conditions. Effective protection structures and maximum loading parameters such as the time of de-watering and/or excavation rate may be estimated in order to provide the requested factor of safety at transient states. New protection methods may be investigated by the suggested design procedure. Special regard should be taken of structures including draining facilities, such as draining structures and pore pressure reduction borings (Köhler et al., 2002), which seem to be an effective counter measure against hydraulic soil failure.

6 REFERENCES


GGU (2002), finite element code, Johann Buß, Braunschweig, Germany
