Pore water compressibility and soil behaviour – excavations, slopes and draining effects

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Abstract
Soils under water may be considered as an unsaturated porous medium containing gas, water and solids. Observations and laboratory measurements have shown that under natural conditions pore water contains microscopic air bubbles which change pore water compressibility considerably. These bubbles are embedded in the pore fluid of the soil skeleton and may play a key role in soil behaviour. Soil failure and delayed soil deformation may be explained by pressure changes applied on such unsaturated submerged soils, causing f. e. heaving or shrinking. By using a three-phase model, the mechanical behaviour of such soils may be described by an extended consolidation equation in order to calculate transient pore water pressures, induced by external pressure changes. Selected case studies in low permeable soils are presented. Effects of rapid external pressure reduction are discussed, such as excavation, pore pressure reduction to improve slope stability and fully soil embedded draining structures. Results of numerical simulations, laboratory and field investigations are presented with special regard to such structures acting as draining facilities.

1 Introduction
Observations and laboratory measurements have shown that pore water under natural conditions contains a quantity 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 in transient states, explaining soil failure and structure deformation. Pressure changes applied to unsaturated submerged soils initiate changes in volume of the embedded air bubbles inside the gas-water mixture (Skempton 1954). 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 and 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 regions and even interweaving soil motion (translation and rotation) may be induced. The decrease of external pressures acting on unsaturated submerged soils may therefore induce fluidisation (Schwab et al. 2003). In low permeable soils such as silt and clay, excess pore water pressure may be induced, decreasing with time until the necessary amount of pore water has been expelled out of the deforming soil. This causes pre-failure deformations and finally even failures. Using finite element simulations the consequences of the pore water-gas interaction of unsaturated submerged soils may be examined.
Consequences should be drawn from the fact that natural pore water may always contain gas entrapments and these have to be taken into account especially during and after rapid load changes, when deformation characteristics and failure conditions of the soil and structure have to be investigated. Possible failure conditions caused by changing water pressures and solutions that may be used in the design practice for safety assessment considerations can be explained by using numerical simulations dealing with embankments, water front structures, excavations and tunnels. Solutions originating from these investigations are presented.

In the past unsaturated soil analysis has mainly considered soil regions above the water level, where capillary suction occurs. Taking the content of micro-sized occluded gas bubbles in natural pore water into account, the region of unsaturated soil conditions needs to be extended to soil areas under the water table. Introducing a pressure dependent degree of saturation $S$ of the submerged soil below the phreatic level, it can be elaborated, that within a certain depth of soil, a transition zone between the unsaturated zone (above water level) and fully saturated soil (located deeply below the phreatic level) may be defined. If external pressure fluctuations are acting on such soils, even a small amount of gas volume changes the soil behaviour in an extreme way.

Figure 1(left) shows a schematic view of a section cut through the soil above and below the phreatic level. The phreatic level is defined as the location where pore water pressure equals barometric (atmospheric) air pressure. The figure depicts the zones of a so called water saturated submerged soil, filled by pore water, which contains occluded gas bubbles and larger macro pores filled by gas. In this case the fluid phase below the phreatic level remains continuous. It has to be regarded as being more instantaneously compressible due to the locally prevailing gas content as compared with the state of a completely saturated soil. Below this transition zone containing pore fluid with occluded gas bubbles one will find pore water in the deeper regions without these gas bubbles due to collapse from high water pressure. The region above the phreatic level also consists of two zones. First the capillary fringe of a continuous water phase containing occluded gas bubbles or even macro pores and secondly the adjacent unsaturated soil zone with a continuous gas phase reaching far above
the capillary fringe. The continuous gas phase pressure is always controlled by the currently acting barometric pressure. The line where the pore pressure equals barometric pressure is defined as the phreatic level. Figure 1(right) displays the phreatic level which divides the soil into a pore water pressure area (below phreatic level) and a suction area (above phreatic level). Due to climate influence delayed pore pressure spreading below phreatic level may be accounted for (Schulze et al., 1999). Hydrostatic pore pressure distributions however may consequently often not be expected due to the gas content and the externally acting pressure changes. Deviating pressure contours may easily appear.

The initial degree of saturation $S_0$ is suggested to be selected in a more realistic way between 0.9 and 1.0 at the entry level of the phreatic surface. An increase can be expected with soil depth $z$ up to a fully saturated soil region at a depth level, which may be estimated using the investigation results, presented by Hilf (Fredlund et al.). Using his formula a simplified calculation of the saturation distribution as a function of soil depth $z$ was performed (Köhler et al., 1999). The saturation degree increases with depth and may reach fully saturated soil conditions between approximately 5 and 80 m soil depth. The following chapters contain applications of unsaturated conditions in submerged soils (near-saturation) associated with external pressure changes.

2 Excavations

2.1 Possible failure modes

It was found that the 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 (Köhler et al., 1999). One of the main impacts in relation to subsoil stability is unloading due to rapid water level draw down and/or the excavation itself, which causes transient excess pore water pressures in low permeable soils.

Depending on the low stress level at ground surface and on the compressibility of the soil skeleton, the loading case of a rapid water level draw down and/or excavation 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 transient seepage needs to be taken into account in order to avoid hydraulic soil failure. Generally the time of lowering of the water level $t_A$ as well as the excavation of the pit itself may in practice range between days, weeks and even months, which of course has a direct influence on the excess pore water pressure development in the subsoil. The faster the external water level lowering $dh$ or excavation takes place, the more excess pore water pressures have to be taken into consideration in the transient state. The velocity $v_{zA} = dh/t_A$ of water level lowering or of the excavation rate needs to be kept smaller than the prevailing permeability $k$ of the subsoil ($v_{zA} < k$) in order to avoid transient pore pressure conditions.
which may endanger ground surface stability. The stability of a retaining wall at the loading case of excavation and de-watering of the open pit may fail due to hydraulic soil failure. Usually such loading is solely considering seepage at steady state. The hydraulic influence on the passive earth pressure in front of the retaining wall and a selected endangered soil prism is evaluated. In addition the stability of the wall may be highly endangered in transient pore pressure conditions (see fig. 2), especially at the critical time $t_A$ just after water level lowering and excavation. In order to withstand such loading cases, both failure conditions (transient and steady state) need to be investigated, to assure safe constructions (Köhler, 2003). The following three failure modes need to be investigated:

- stability against hydraulic soil failure due to seepage at steady state
- stability against hydraulic soil failure (hydrodynamic soil deformation) due to seepage at transient state in order to prevent uplift at the critical soil depth below the ground surface of the excavated pit
- sliding at the potential failure plane activating passive earth pressure by taking the time dependent acting excess pore pressure $\Delta u(z,t)$ into account (see fig. 2).

### 2.2 Heave

For excavations the unloading consists of a superposition of stress and pore pressure relief. In order to illustrate the effect of each loading type a 1D numerical example was computed. The simulation used a full coupled displacement-pore pressure transient analysis, based on Biot’s equation (Biot 1941), which is implemented in the Abaqus finite element code. In order to simulate the presence of air bubbles in the pore water the following form of fluid compressibility $\beta$ was adopted (Bishop & Eldin 1950, Fredlund & Rahardjo 1993):

$$
\beta = S \beta_w + \frac{1 - S + hS}{p_u + p_0}
$$

where $S$ is the degree of saturation; $\beta_w$ is the water compressibility ($4.58 \times 10^{-7}$ kPa$^{-1}$); $h$ is the volumetric coefficient of air solubility (0.02); $p_u$ is the air pressure; $p_0$ is the atmospheric pressure. The air pressure is assumed to be equal to the water pressure (no matric suction is present and the influence of the surface tension is neglected). Figure 3 shows the geometry and some results of the computed example. In case of a static unloading, the unsaturated
soil swells very quickly due to the deformation of the compressible pore fluid. The pore pressure dissipation follows a similar pattern as for saturated soils. A pore pressure change due to dewatering unloads a saturated soil in a neutral way, with no displacements. An unsaturated soil deforms under similar conditions because of the fluid compressibility. An initial excess pore water pressure dissipates slowly with a velocity controlled by soil permeability and drainage conditions.

Figure 3c shows the superposition of these two effects. The main difference between a saturated and an unsaturated soil is the presence of a reasonably large instantaneous heave in the latter case. This instant heave has been observed in many in-situ excavations. In order to investigate the time dependent swelling effect of a more complicated succession of excavation phases, which were well monitored during excavation at Lion Yard (Ng, 1998). A 1D finite element simulation was performed in a simplified way using elastic parameters which approximate to some extent the soil parameters described by Ng. Figure 4 shows measured and simulated pore pressure and heave developments caused by excavation. A reasonably good agreement was obtained for the excavation stages; the post-excavation behaviour is slightly different, possibly caused by the approximations, which in the simulation had to be carried out to describe soil permeability and drainage conditions and by neglecting the casting of concrete slabs. Due to the large length/depth ratio (approx. 4.7) of the Lion Yard excavation it was expected that the 2D effects in the excavation axis may be disregarded. In order to proof this assumption a comparative 2D calculation were carried out, using the geometry presented in (Ng & Lings, 1995). Figure 4c shows the results out of this comparison calculations in terms of the vertical strain distribution in 25 months after completion of the excavation. Practically no difference between the 1D and 2D simulation was observed. In addition the measured values are shown. The differences in the upper soil domain are caused by the use of non fully calibrate ed soil properties in the analyses. The target of this
The most interesting result is the occurrence of the instantaneous heave, which may only be gained by unsaturated simulation. A fully saturated simulation does not show this effect (Figure 4a). This result agrees well with the obtained ground deformation in the 10-m-deep excavation site at Lion Yard. The time dependent heaving and pore pressure development of the finite element simulation shows satisfactorily the typically measured soil behaviour.

### 3 Safety improvement of unstable slopes

Pore water pressure reduction is widely used to stabilise endangered slopes. This concept is applied with reluctance in low permeable soils such as clay. Usually there is great uncertainty concerning the estimation of transient pore pressure distributions which cover the time between initial and final steady state conditions. Pore pressure measurements undertaken in the field have shown the effectiveness of such concepts of pore pressure dissipation. To ease
application the three-phase model allows the estimation of transient pressure states which might encourage further utilisation of such methods.

It is widely assumed that drainage pipes need to withdraw water visibly in order to be effective. Thus apparently dry drainage pipes are often viewed incorrectly as being ineffective. Even the removal of small quantities of water (e. g. evaporation into the pipe, which is especially valid in low permeable soils) will allow a reduction of pore water pressure in the vicinity of the drain.

Pore pressure reduction may occur regardless of the inclination of the bore hole or the water level inside the bore hole. Furthermore a new approach is the estimation of the transient development of pore pressures, taking into account the compressibility of the pore fluid by regarding the submerged soil as a three phase medium containing gas, water and solids. It covers the gap between the steady state conditions of the initial and final state. The application of this model to estimate pore pressure dissipation in low permeable soil is described in detail in Köhler et al. 2002a, Schulze & Köhler 2003a, 2003b.

Even reversely inclined bore holes or bore holes entirely filled with water may be able to decrease pore pressures. Removing water from the bore holes increases effectiveness, because barometric pressure is allowed to be transferred directly into the soil. A successful application of this concept depends on the original magnitude of the pore pressures to be reduced. Looking at a bore hole filled entirely with water, the piezometric level (= pressure head + elevation head) all over the bore hole is constant and solely determined by the geodetic level of the bore hole mouth (flow velocity in the bore hole may safely be assumed to be negligible). In a bore hole which is filled with air (water removed), the local piezometric level (hydraulic boundary condition along the bore hole) will be the local geodetic level. These facts will often allow a much more effective placement of the drainage pipes into the shear

![Figure 6. Schematic results of pore pressure measurements in section A-A.](image)
In accordance with Terzaghi’s principle of effective stress the stability of the slope will be increased directly as pore water pressure is reduced.

Measurements have been carried out in an old cut (built in the 1920s) shown in figure 5. The soil consists of stiff, fissured Lower Lias clay (\(w_L = 0.58\), \(w_P = 0.22\), clay 40%, silt 60%). Narrow limestone bands are embedded occasionally. Although the fissures and limestone bands in the clay formation increase the large scale permeability of the soil, the permeability is still considered to be as low as about \(10^{-10}\) to \(10^{-11}\) m/s. Pore water pressure sensors have been installed well ahead in order to measure the initial pore pressure distribution. Later the
depicted seven bore holes were drilled (D1 to D7) between December 17th, 2001 and January 9th, 2002. Changes in pore pressures have been documented continuously.

Two clusters of pore pressure sensors can be distinguished: The first cluster is located in the vicinity of the perforating bore holes as shown in figure 5b (section A-A). A second cluster was installed above the bore holes as depicted in figure 5c (section B-B). The shear zone has been identified using inclinometer measurements. The depicted piezometric line has been constructed for the initial state from pressure measurements. More sensors than those presented in figure 5 were involved in this process. Since fluctuations of the pressure distribution are common in natural slopes this line may only be a rough approximation. A single piezometric line will therefore not suffice to describe transient pore pressure states.

Results of the pressure measurements of the first cluster (see fig. 5b) are sketched in figure 6. Pore pressure dissipation is clearly taking place, with a major pressure drop associated with the drilling process (due to volume change and other effects caused by the drilling). After this major drop a second phase of continuous pressure dissipation has been observed which will last for a long period of time, finally approaching the level of the bore hole mouths, indicated by the dashed line. Results of sensor W30, which show a peculiar behaviour (discussed in Schulze & Köhler 2003b) have been omitted for clarity. Looking only at figure 6, it may be concluded that water filled fissures were simply drained by the bore holes.

Figure 7 indicates that this may not be the case: In figure 7a all sensors have been projected into an imaginary vertical section (located somewhere between section A-A and B-B). The measured pressures are shown at distinct time steps, starting with the initial state (before the bore holes were drilled (Dec. 12th, 2001)) and several time steps closing with the state on Nov. 7th, 2003. Furthermore theoretical steady states (initial and final) are depicted for comparison. Please note: The initial steady state is derived from a potential net analysis where the piezometric line is inclined. The final steady state assumes water filled bore holes calling for a hydrostatic pressure distribution. Figure 7 b shows a schematic summary of the measurement results of figure 7a. Although pore pressures in the vicinity of the bore holes were reduced considerably, the sensors above the area being directly influenced by the bore-holes show minor changes. This would not have been the case if only fissures were drained. Instead the pore pressure release above the perforated area will take many years and may approach final steady state conditions as indicated in figure 7. The measurements show that the pore pressures in the vicinity of the bore-holes (and the shear zone) have been reduced greatly, leading already to improved safety. Pore pressures in other parts of the soil mass are also influenced. However their contribution to slope stability is minor. In this region it will take a long time to reach final steady state.

An estimation concerning the velocity of pore pressure dissipation using the three-phase system has been performed in advance before the bore holes have been drilled. The distribution of the piezometric levels in a transient state are shown in figure 8. The specific storage $S_s$ has been estimated to be 0.0035 m$^{-1}$ and applied to confined conditions. Details on this calculation carried out using a 2-D finite element code may be found in Köhler et al. 2002a.

The influence of pore pressures concerning the safety of the slope have been assessed by using the finite element code PLAXIS. A factor of safety of 1,0 is assumed for the initial state (before drilling of the bore holes). The pore pressure conditions in the shear zone of the transient piezometric level of 75 m asl (reached in August 2002, see fig. 6) will increase the factor of safety to at least 1,05. Final steady state conditions will increase the factor of safety
to about 1,18. Further details on the performed calculations are presented in Schulze & Köhler 2003b. In the examined case the final factor of safety may not necessarily meet legal requirements concerning the safety level of the slope. But a significant reduction of slope movements may certainly be expected. A considerable increase in safety may be expected especially in high slopes where the initial piezometric level is located closely below the surface. An important finding is to extend the bore holes sufficiently deep into the slope in order to avoid new shear zones which may develop in non-drained sections.

Thus it is concluded: Bore holes which are installed in low permeable soil for reasons of pore pressure reduction need to be placed as closely as possible to the vicinity of potential shear zones in order to be effective in a reasonable time. If a proper placement cannot be accomplished, extremely long time periods may be necessary for pressure dissipation to reach the shear zone. The three-phase model of a submerged soil (gas, water and solids) allows a useful estimation of time dependent pore pressure dissipation.

Time dependent pore pressure spreading may further be applied to other aspects of slope stability: For example pore pressure reduction may be utilised in order to increase slope stability of existing river dikes, used for flood protection. Improvements in existing structures are often requested because of the increasing probability of higher flood waves than originally anticipated (Köhler et al. 2002b). Furthermore stability of unstable low permeable slopes may be influenced by fluctuating barometric pressure. Extreme drops of barometric pressure are able to trigger slope movements (Köhler & Schulze 2000).

4 Tunnels
An interesting effect of the near-saturated state is the time-dependent development of the pore pressures and subsequently produced ground movements after a tunnel excavation.

The ground displacements have two causes: the stress relief due to excavation and the pore pressure reduction due to the draining effect of the construction. In order to isolate the evidence of the drainage effect of a newly built tunnel inducing a transient pore pressure reduction in the soil, a sequence of finite element simulations of an exemplary tunnel was performed. Figure 9 shows the finite element model. The soil consists of two layers: on the
top is a 4 m gravel layer followed by 21 m of clay. The tunnel with an outer diameter of 6 m was simulated as being very stiff to prevent further deformations of the soil caused by the excavation process. In practice this deformation cannot be prevented. Thus pore pressures in the vicinity of the tunnel is influenced by volume changes, often leading to negative pore pressures (suction). This process initiates a hydraulic gradient towards the tunnel lining. Assuming the suction dissipating with time, the pressure at the tunnel lining has been set to zero on purpose to focus only on the draining effect. The displacement boundary conditions permitted no horizontal displacements along the vertical mesh boundaries and no vertical displacements along the bottom horizontal boundary. Hydraulic boundary conditions require that hydrostatic pore pressure on the right vertical boundary and atmospheric pressure on the inner side of the tunnel lining be given. The hydraulic right boundary condition was artificially extended using a domain with a larger horizontal permeability.

Three simulations were performed: one fully saturated and two near-saturated cases with a saturation degree $S_R$ of 0.99 respectively 0.95.

The permeable gravel layer imposed a quasi-constant head condition on the clay layer top (Figure 9). The de watering effect of the tunnel is presented in Figure 10 in form of differences between the initial hydrostatic water pressure and the calculated pressure after 100 days. The tunnel influence zone extends much slower in the unsaturated conditions due to the presence of occluded gas bubbles in the pore water. The same aspect is evidenced by the evolution of the pore pressure in tunnel axis shown in Figure 11. The pore pressure reduction is substantially delayed by the presence of these gas bubbles in groundwater especially at shallow conditions. A similar trend was measured in-situ in the slope presented in section 3. In principle the draining effect of the tunnel is more or less identical to the presence of a small diameter bore hole or a cluster of bore holes. This effect is confirmed well by comparing the calculated pore pressure reduction shown in Figure 11 with the measurements of Figure 7b. Consequently the evolution of the settlements due to pore pressure changes is also damped. Figure 12 shows a comparison of the settlements of a point on the surface in tunnel axis for different saturation degrees – i.e. 95 %, 99 % and 100 %. The damping of the consolidation process is large, depending on the degree of saturation.

![Figure 11. Pore pressure evolution in the A-A axis (a – saturated, b- unsaturated)](image)
Thus pore pressure reduction is a long term process which leads to settlements. As shown in Figure 12 the time/settlement behaviour is highly dependent on the degree of saturation. However the size of the final settlements is independent on the degree of saturation. The process may influence large regions in long term. It should be mentioned that the deformation caused by stress release due to the excavation of the tunnel changes this evolution, if both effects (unloading of water pressure and effective stress change due to excavation) are taken into account. Considering both effects the time-settlement behaviour may be expected to be accelerated in the initial construction phase (comparatively see Figure 6: drilling period).

5 Conclusions
Deformation characteristics, hydraulic soil failure and investigations in slope stability in transient states have been presented, based on the formulations of temporarily acting excess pore water pressures due to external pressure changes such as rapid water level lowering and/or excavation in open pits or time dependent pore pressure release by drain borings. The calculation of such transient pore pressure release using drain facilities 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 required safety factor in transient states. New protection methods may be investigated by regarding the soil under water as unsaturated porous medium, new design procedures should be investigated. Special considerations should be given to structures including draining facilities, such as draining structures and pore pressure reduction borings (Köhler et al., 2002a, b), which seem to be an effective counter measure against hydraulic soil failure and slope movements.

During unloading a highly non-linear soil deformation of the unsaturated ground is induced. Most time dependent deformations may be expected directly at ground surface of the excavated pit, the least in deeper soil levels, which are directly dependent on the rate of the actual pressure change, the governing stiffness and permeability of the soil at the actual loading stages and at the governing local drainage conditions. The development of such transient ground deformation is typically accompanied by a reasonable large instantaneous deformation (i.e. rapid unloading – instantaneous heave), which is controlled by the consolidation characteristics and the degree of saturation of the soil.
6 References

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