Consolidation process and stress-path-dependant deformations in excavations in soft soils

Berhane Gebreselassie & P. Becker
Institute of Geotechnics and Geohydraulics, University of Kassel, Germany

ABSTRACT: Back analysis of well instrumented excavations showed that the short term behavior of excavations is best expressed in terms of effective stress analysis. The three-dimensional consolidation theory, introduced by *Biot (1941)*, assumes solely a one-dimensional loading, namely a change of stresses in vertical direction but drainage of the pore water in all directions. The complex relationships between the three-dimensional change of stresses due to the excavation and the resulting wall movements are not yet considered in the current available computation methods. Therefore, the simultaneous influence of the stress relief due to excavation and the stress-path-dependent displacements on the consolidation process of an excavation has to be yet quantified. The paper introduces the research work currently running at our institute that investigate this problematic based on small scale model tests and controlled stress path triaxial tests.

1 INTRODUCTION

One of the elementary question in excavations in soft soils is the consolidation behaviour in the initial condition. The knowledge of the influences of the deformation of the excavation wall and the relief of stresses due to excavation on pore pressure development and therefore the resulting time dependant change of effective stresses in soft soils is of fundamental importance in the design of excavations.

Comparative calculations of well monitored excavations show that the initial condition can best and realistically be described with the effective stress analysis. The application of the effective stress analysis, however, requires the knowledge of the excess pore pressure, which is crucially affected by the stress path dependant deformations. Both the redistribution of the stresses due to the excavation and the resulting wall deformation determine the consolidation behaviour and thus the rate of change of the effective stress in an excavation in soft soils.

2 PROBLEM FORMULATION

In an excavation, the soil transferred into a new equilibrium condition because of the relief of stresses due to excavation. The excavation wall supports the soil above the bottom of the excavation, which in turn get its support through the mobilization of the passive resistance throughout the embendement depth of the wall and through struts or anchors. Thus, there arise an additional changes in the horizontal stresses apart from the vertical stress changes due to excavation. The amount of the mobilized horizontal earth pressure depends in turn on the wall movement, which can not be in principle fully avoided in an excavation in normally consolidated soft soils. The resulting multidimensional deformation state is therefore dependent on the stress paths and affects the consolidation behavior of the soil in as much as so far not quantifiable extent.

The fundamental of the solution of consolidation problems in soils was first carried out by Terzaghi & Fröhlich (1936) based on the well known diffusion equation in physics. Thereby, however, a number of assumptions has been taken to simplify the real mechanical behaviour of soils. Among other things, for example, the compressibility of the soil is not constant as supposed by Terzaghi, but in reality it changes with the effective stress and also depends on the pre-consolidations history. A possible influence of the viscosity of the soil is also not considered. Moreover, the Darcy flow law is formulated with the assumption of a constant permeability. Investigations by Scherzinger (1991) show, however, that the permeability is not constant, rather it is dependant on the void ratio *e*. The equilibrium of the forces also neglects the weight of the soil itself. In addition, the volume balance assumes an incompressibility of the solid and pore fluid.

Apart from the general simplifications of the consolidation theory, there are additional conflicting assumptions by excavations. These are:

- Beside the vertical deformations, horizontal deformations can be induced, which result from the wall deformation due to the redistribution of stress as a result of excavation.
- The permeability of the soil depends on the volume change, i.e. on the deformation behavior of of the soil. The relief of stresses due to excavation is to be understood as a negative load, which causes a volume increment of the soil and an increase of the pore volume. Depending on the permeability and the stiffness of the soil, a negative pore pressure can be developed. The unloading stiffness of the soil is taken in this case as governing stiffness. A question may however arise that to what extent will the permeability of the soil be affected by the heave of the soil due to the excavation
- The assumed linear elastic material behavior does not consider the irreversible deformations and the possible coupling between shearing and volume change. Moreover, the linear elasticity theory is independent of the stress paths.

To date no theoretical approach is available that reasonably describes the consolidation process in excavations.

The three-dimensional consolidation theory of Biot (1941) considers the same assumptions and conditions as Terzaghi's one-dimensional theory. The differential equations of the three-dimensional consolidation analysis are based on the equilibrium and continuity conditions of a volume element of a soil. The continuity condition describes the relationship between the volume change of the soil particles of a given volume element and the amount of water flowing out of the element. It should be noted, however, that the volumetric deformations due to the drainage of the pore water in soil in all directions in three-dimensional consolidation theory are caused by loading in one direction only, namely the vertical direction. The increase in volume is described here by the Poisson's ratio ν. The complex behaviour of a three-dimensional change of stresses due to excavation cannot therefore be described on the basis of the Biot theory.

3 STATE OF THE ART

The most unfavorable condition for excavations in soft soils is apparently the drained (end) condition, since the relief of stresses due to excavation produces a negative pore pressure, which contributes to the stability of the excavation at initial state. It should be noted however that by temporarily supported excavations the construction time may not be long enough to define the state of the excavations as drained or may not be short enough to define it as undrained. The real condition of an excavation in soft soils lies somewhat between these two extreme cases, which can only be solved with the help of an accurate consolidation theory. Such theory, however, is not available at present for analytical computation of excavations.

It is a well known fact that the behaviour of soils is primarily governed by the effective stresses independent of the drainage condition. Based on this principle, Kempfert and Gebreselassie (2002); Gebreselassie (2003) attempted to develop an approach to calculate the active and passive pressure using the effective shear parameters and the pore pressure coefficient at failure A_f. They used the effective angle of the over all shear strength φ'_s instead of the φ' and c'. This assumption is a common practice in Germany.

On the basis of stress paths of a consolidated undrained triaxial test, relationships have been derived between the angle of the overall shear strength φ'_s and the normalized undrained strength λ_{cu} and A_f for various type of stress paths according to Figure 1 (Table 1). This relationships have the advantage of making use of the huge experience and data available on the undrained strength c_u in the field of geotechnical engineering. The possible ideal stress paths are shown in Fig. 1.

Table 1. Relationship between φ'_s and c_u for different stress paths according to Fig. 1 (after Gebreselassie (2003))

Stress-path	$\sin\varphi'_s =$
(OA)	$= \dfrac{1}{\frac{1}{\lambda_{cu}}(K_0+(1-K_0)\cdot A_f)-2\cdot A_f+1}$
(OB)	$= \dfrac{1}{\frac{1}{\lambda_{cu}}(1+A_f\cdot(1-K_0))-2\cdot A_f-1}$
(OE)	$= \dfrac{1}{\frac{1}{\lambda_{cu}}(1+A_f\cdot(1-K_0))+2\cdot A_f-1}$
(OD)	$= \dfrac{1}{\frac{1}{\lambda_{cu}}(K_0+A_f\cdot(1-K_0))+2\cdot A_f-1}$
$\Delta\sigma_3\uparrow$; $\Delta\sigma_1\downarrow$ and $\Delta\sigma_1=-n\Delta\sigma_3$ *	$= \dfrac{1}{\frac{1}{2\cdot\lambda_{cu}}(1+K_0+\beta(K_0-1)-A_f(1-K_0))+2A_f-\beta}$

*n is proportionality constant between $\Delta\sigma_v$ and $\Delta\sigma_H$; and $\beta = (n-1)/(n+1)$.

It should be noted that the stress path within the passive zone of an excavation can not easily be identified to the ideal stress paths in Fig. 1. Since there is a change of the vertical stress in addition to the change of the horizontal stress during construction (undrained), the total stress follows another path than that normally assumed for passive case (OE). If failure has to occur during excavation, the changes in vertical pressure should also be taken in to account. The relative rate of change of the vertical

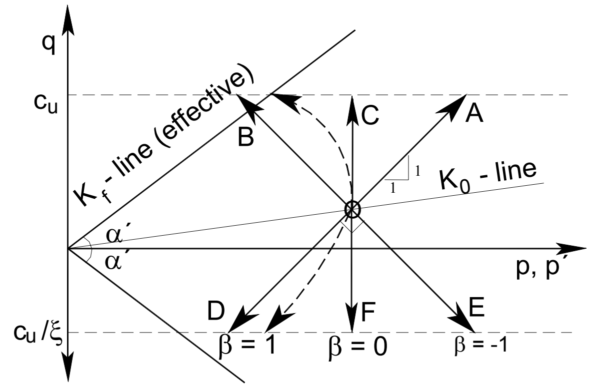

Figure 1. Idealized stress paths

stress and horizontal stress during excavation is not yet known, but it lies somewhat between the two extreme cases: the passive case (OE) and the extension case (OD) and it is dependant on on the type of the soil, the excavation depth and the wall type.

The pore pressure coefficient at failure A_f is not a unique value for a given soil, rather it is dependent on the direction of the principal stresses, the initial stress state and the plastic volumetric strain (Franke (1980); Schweiger (2002)), while the effective shear parameters, in this case the angle of the overall shear, may be assumed independent of the stress path to a large extent. On the other hand the order of the magnitude of A_f is well-known for the standard triaxial compression case (OA). Therefore, it is worthwhile to develop a relationship between the A_{fs} value for standard triaxial case (OA) and other stress paths (Table 2) (Gebreselassie (2003); Kempfert and Gebreselassie (2002)).

Table 2. Estimation of the pore pressure coefficient for different stress paths according to Fig. 1 (after Gebreselassie (2003))

Stress-paths*	A_{fs} [-]
(OA)	$= A_{fs}$
(OB)	$= A_{fs} - 1_f$
(OE)	$= \dfrac{K_0 - \xi + A_{fs}(1 - K_0 - 2\lambda_{cu,s})}{(\xi(1-K_0) + 2\lambda_{cu,s})}$
(OD)	$= \dfrac{K_0(1-\xi) + 2\lambda_{cu,s} + A_{fs}(1 - K_0 - 2\lambda_{cu,s})}{(\xi(1-K_0) + 2\lambda_{cu,s})}$
$\Delta\sigma_3\uparrow$; $\Delta\sigma_1\downarrow$ and $\Delta\sigma_1=-n\Delta\sigma_3$	$= \dfrac{\beta}{2} + \dfrac{\lambda_{cu,s} - \xi/2 + A_{fs}(1 - K_0 2\lambda_{cu,s}) + K_0(1-\xi/2)}{(2\lambda_{cu,s} - \xi(K_0 - 1))}$

A_{fs} are $\lambda_{cu,s}$ are the pore pressure coefficient at failure and the normalized undrained shear strength respectively for a standard triaxial compression test (Path OA). $\beta = (n-1)/(n+1)$

Since the un/reloading modulus of elasticity of soft soils is 5 to 7 times higher than the modulus of elasticity in primary loading, the time required to reach an approximate steady condition after excavation is usually measured in weeks or months, i.e. during the normal construction periods. Freiseder (1998) presented a field pore pressure measurements during the construction of a 6.3 m deep excavation for underground park in Salzburg. It appears that the excess pore pressure stabilizes to a steady state in a time of 10 to 60 days after excavation.

The effective stress path A´→ B´ in Fig. 2 below corresponds to undrained loading and B´→ C´ corresponds to swelling and reduction in the mean normal effective stress. The pore pressure immediately after construction u_i is less than the final steady state pore pressure u_c and so there is an initial excess pore pressure which is negative. As time passes the total stresses remains approximately unchanged at B but the pore pressure rises. The wall will fail in some way if the states of all elements along the slip surfaces reach the failure line; if B´ reaches the failure line the wall fails during the undrained excavation and if C´ reaches the line the wall fails some time after construction. The figure demonstrate that unlike footing foundations or embankment foundations or retaining walls loaded by fill, where the foundation becomes stronger with drainage, the factor of safety of a retaining walls supporting an excavation will decrease with time.

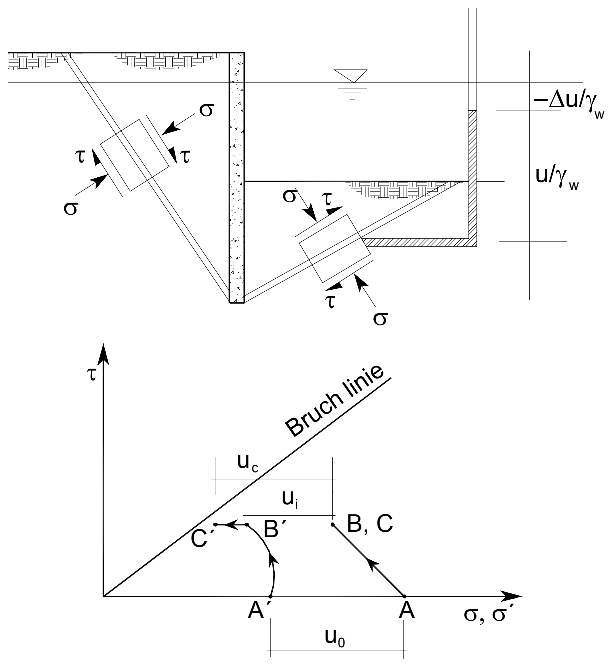

Figure 2. Change of stresses and pore pressure in an excavation (after *Atkinson (1993)*

After reviewing six open excavations (5 to 11m deep) case records in clay, *Janbu (1977)* concluded that the stability is best expressed in terms of effective stress analysis. He reasoned that the readjustment of stresses and pore water pressures to correspond with a state of steady seepage may often take place in the course of few days or few weeks or at most some months. He further commented that the simple total stress analysis for excavations in clay will frequently lead to erroneous results both for safety factor and shear surface location.

Lafleur et al. (1988) had also examined the behavior of a well instrumented, 8 m deep excavation slope in soft Champlain clay. They reported that the effective stress approach may give a reasonable estimate of the factor of safety. On the other hand, all the stability analyses using a short term approach overestimated the factor of safety of the failed slopes.

Kempfert and Gebreselassie (2002); Gebreselassie (2003) came to the conclusion that the drained analysis results the most unfavorable situation for excavations in soft soils in most cases. For the proof of the safety of the excavation in short term, it is advisable to use the effective stress analysis together with the pore pressure coefficient than the total analysis with undrained shear strength c_u. Based on an excvation with strut support at the top of the wall and free end support condition, Hettler et al. (2002) also showed that the total analysis method using c_u may lead to large interpretation difficulties in the determination of the penetration depth of the wall.

4 PRELIMINARY EXPERIMENTAL STUDY

4.1 *Excess pore pressure development in excavations*

The development of the excess pore pressure in an excavation in soft soil has been studied using an extensive small scale model tests at our institute. The models contain three stages of excavtion. In the first series of model tests the effects of wall support, the surcharge load and the construction stages on the development of the pore pressure has been investigated. In the case of the wall support, it was differentiated between fixed end and a free end support condition with additional strut support at wall head. The pore pressure change was measured at representative locations in the excavation model.

The following results present the development of the excess pore pressure change in an excavation model with a fixed end support condition in comparison with analytical and numerical analysis results (Becker (2003)).

The analytical analysis for the primary approximation of the consolidation process was performed based on the one-dimensional consolidation theory of Terzaghi.

Figure 3 shows typical course of the excess pore pressure during and after the execution of the excavation for the three selected measuring points and its comparison with the results of the analytical and numerical analysis.

The general tendency of the excess pore pressure at the measuring point A (Fig. 3a), which is located on the passive side at the level of the assumed sliding surface below the bottom of the excavation, corresponds to the computation results. It appears from this figure that the vertical relief of stresses are dominant in this area of the excavation model, which can be explained by the immediate drop of the excess pressure. This drop of excess pore pressure corresponds to the effective change of the vertical stress due to excavation, i.e., $\Delta u = \Delta\sigma' = \gamma' z = 2{,}0$ kN/m². On the basis of the gradual pore pressure drop in model test V5 in the first 24 hours, however, a possible influence of the wall movement and the resulting deformation in the soil body can be observed, although it can not be yet quantified. The result of the analytic computation supplies a further indication of the influence of the horizontal change of stress due to the excavation process on the development of the excess pore pressure after the first 24 hours. Here a pure relief of stresses is assumed and a stronger dissipation of the negative pore pressure is calculated than it can be confirmed experimentally. Therefore, the one-dimensional consolidation theory is not in a position to describe clearly the influence of the wall deformation, which produces a positive excess pore pressure in this zone, on the permeability and therefore on the drainage behavior of the soil.

The problem of a consolidation analysis of an excavation becomes particularly clear at the measuring point B (Fig. 3b). This point is located at the passive side near the wall toe. The exact knowledge of the development of the excess pore pressure is particularly important for calculation of the stability of the excavation around this point. The influence of the displacement and rotation of the wall toe on the consolidation process can be demonstrated on the basis of the test results. Depending on the horizontal deformation of the wall a positive excess pore pressure may develop which may overlay the developed negative excess pore pressure due to the excavation and thus reduces the development of the excess pore pressure as in the case of the model tests V1 and V2 or even fully dominates the negative excess pore pressure as in the case of the V3 and V4. Such phenomenon can by no means be simulated by any analytical computation methods.

The measured excess pore pressure at the measuring point C (Fig. 3c), which is located on the active side of the wall, confirms the validity of the above statements. For example, the rotation of the wall toe causes a pore pressure increase on the active side of the wall, which produces a positive excess pore pressure in the case of model test V1. The rise of the positive excess pore pressure is an indication of the influence of the wall deformation in this zone in contrast to the influence of the stress relief due to excavation.

Therefore, the model tests confirm the moderate agreement of the computation results based on the generally accepted one-dimensional consolidation theory with the experimentally measured development of the excess pore pressure at the middle of

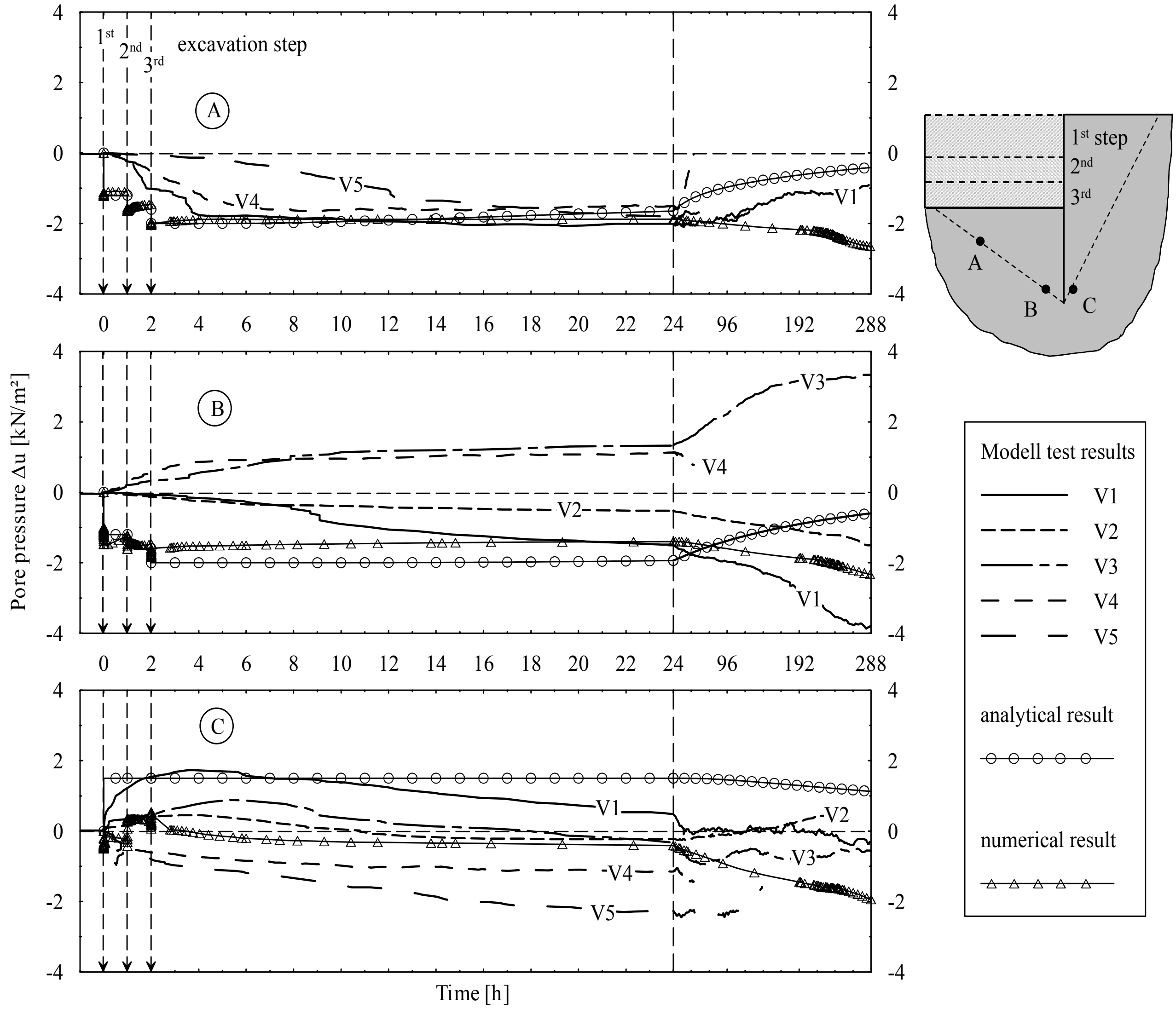

Figure 3. The develpement of the excess pore pressure in an excavation amodel

the excavation just below the bottom of the excavation. The effect of the horizontal wall deformation on the change of the effective stress in an excavation remains a determining factor in the consolidation analysis of excavations in soft ground, which is still not quantified.

4.2 *Stress path controlled Triaxial tests*

A series of stress path dependent triaxial tests on normally consolidated lacustrine soil had been carried out at our institute to investigate the development of the excess pore pressure (Gebreselassie (2003)).

Typical test results for the total stress paths OB and OC according to Fig. 1 are presented as follow. The postulated total stress path OB at a slope of 1:1 was effected by decreasing the horizontal stress and maintaining the vertical stress constant. Similarly, the postulated total stress path OC at a slope of infinity was effected by increasing the vertical stress and simultaneously decreasing the horizontal stress. The development of the excess pore pressures are shown in Fig. 4a & 5a. The excess pore pressure for the stress path B is negative for the effective con-

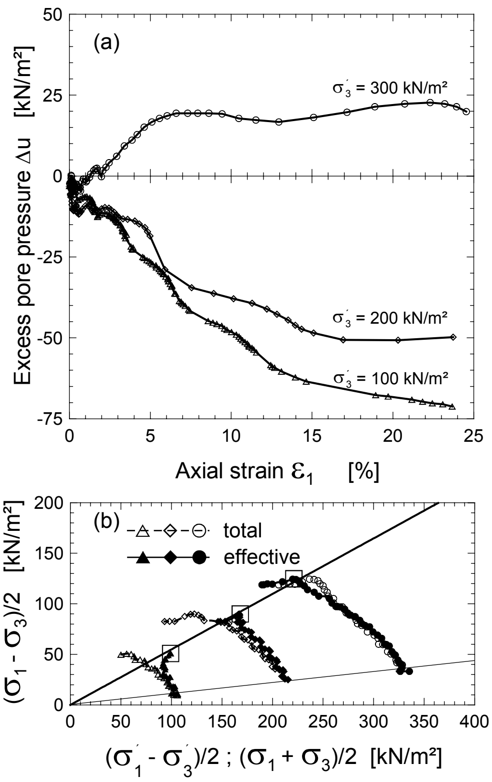

Figure 4. Excess pore pressure development and the stress paths in a stress path (OB) controlled triaxial test (Gebreselassie 2003)

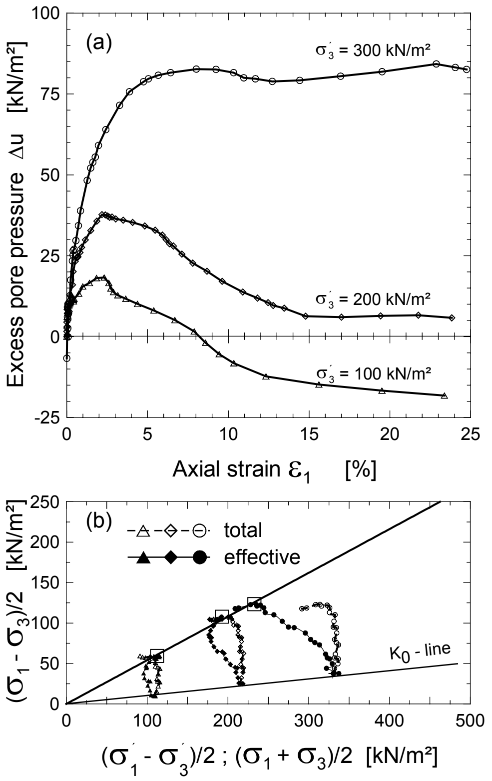

Figure 5. Excess pore pressure development and the stress paths in a stress path (OC) controlled triaxial test (Gebreselassie 2003)

solidation pressure of 100 and 200 kN/m², and it is positive for 300 kN/m². It shows a general tendency of shifting the excess pore pressure from negative to positive as the effective consolidation pressure increases. The excess pore pressure for the stress path C with effective consolidation pressure of 100 and 200 kN/m² reached their maximum value at an axial strain of 2.5% before they dropped gradually to minimum value at large axial strain, while the excess pore pressure with 300 kN/m² reaches its maximum value at 7.5 % axial strain and remains almost constant through out the test.

The stress paths are shown in Fig. 4b & 5b. The test paths in all the specimens are very similar to the postulated stress paths till they approximately reach the failure stress $((\sigma_1-\sigma_3)_{max})$, after which the paths begins to deviate from the postulated paths. Since the horizontal and vertical stresses had been effected in a stepwise fashion, the actual stress path reproduced in the laboratory have a zig-zag pattern. However, the general trend of the stress condition is faithfully reproduced.

5 CONCLUSION

The presented results, in particular the model tests, show the substantial differences between the theory and practice concerning the computation of the development of the excess pore pressure in excavations in soft soils. As far as the current stand of the science is concerned, the exact knowledge of the stress path dependent excess pore pressure development is indispensable for a realistic effective stress analysis based on the effective shear strength for the initial condition of excavations in soft ground. On the basis of the identification of the typical total stress path zones in an excavation, which characterize the three-dimensional deformation behavior, it is possible to perform an effective stress analysis for the initial condition with the help of the derived relationships for the angle of the overall shear strength and the pore pressure coefficient at failure. In this way, the safety reserve in the computation of excavations in soft soils can be optimized

The research work to this basic soil mechanics problem, namely the consolidation process in an excavation in soft soils, is currently running and will be published in the near future.

LITERATUR

Atkinson, J. (1993). An introduction to the mechanics of soils and foundations. London: McGraw-Hill.

Biot, M. A. (1941). General Theory of Three-Dimensional Consolidation. Journal of Applied Physics, Vol. 12: 155-165.

Becker, P. (2003). Porenwasserüber- und Unterdruckverlauf bei Baugruben in weichen bindigen Böden. Diplomarbeit, Universität Kassel, Fachgebiet Geotechnik, unpublished.

Franke, E. (1980). Anwendbarkeit der undränierten Scherfestigkeit im Vergleich zur Anwendbarkeit der effektiven Scherparameter. Vorträge der Baugrundtagung 1980: 297-319.

Freiseder, M. (1998): Ein Beitrag zur numerischen Berechnung von tiefem Baugrund in weichen Böden. Institut für Bodenmechanik und Grundbau, Technische Universität Graz, Heft 3.

Gebreselassie, B. (2003). Experimental, analytical and numerical investigations of excavations in normally consolidated soft soils. University of Kassel, Institute of Geotechnics, Dissertation, Booklet No. 14.

Hettler, A. et al. (2002). Zur Kurzzeitstandsicherheit bei Baugrubenkonstruktionen in weichen Böden. Bautechnik 79, Heft 9: 612-619.

Janbu, N. (1977). Slopes and excavations in normally and lightly overconsolidated clays. Proc. of the IX ICSMFE, Tokyo, Vol. 2: 549-566.

Kempfert, H.-G. & Gebreselassie, B. (2002): Zur Diskussion von dränierten und undränierten Bedingungen bei Baugruben in weichen Böden. Bautechnik 79, Heft 9: 603-611.

Lafleur, J. et al. (1988). Behaviour of a test excavation in soft Champlain Sea clay. Can. Geotech. J. 25: 705-715.

Scherzinger, T. (1991). Materialverhalten von Seetonen – Ergebnisse von Laboruntersuchungen und ihre Bedeutung für das Bauen in weichem Untergrund. Veröffentlichung des Instituts für Bodenmechanik und Felsmechanik der Universität Karlsruhe, Heft 122.

Terzaghi, K. & Fröhlich, O. K. (1936). Theorie der Setzung von Tonschichten. Leipzig-Wien: F. Deuticke.