Time and stress path dependant performance of excavations in soft soils

P. Becker
Institute of Geotechnics and Geohydraulics, University of Kassel, Germany
p.becker@uni-kassel.de

ABSTRACT

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 is of fundamental importance in the design of excavations in soft soils. The state of the art demonstrates that there is a possibility of undrained failure in excavation, although the drained condition is most critical. The actual state of an excavation lies between the cases of drained and undrained conditions. 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 affect the development of the excess pore pressure and thus the rate of change of the effective stress in an excavation in soft soils. The paper outlines the ongoing research work on this basic soil mechanics problem, namely the time dependant effective stress development in excavations in soft soils. Excess pore pressure, total stress development and movements in and around an excavation in soft soil has been studied with small- and large-scale model tests. Results from the model tests are presented with due consideration of the effects of wall support, surcharge load and construction stages of an excavation in soft soils and compared with a FEM analysis using advanced constitutive soil models.

Keywords: excavation, soft soil, stress path, retaining structures, excess pore pressure, effective stresses

INTRODUCTION

One of the elementary question in excavations in soft soils is the effective stress development 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, see also Gebreselassie (2003). The application of the effective stress analysis, however, requires the knowledge of the excess pore pressure, which is crucially affected by 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.

Figure 1 demonstrate that unlike footing or embankment foundations, where the foundation becomes stronger with drainage, the safety of retaining walls supporting an excavation will deteriorate with time.

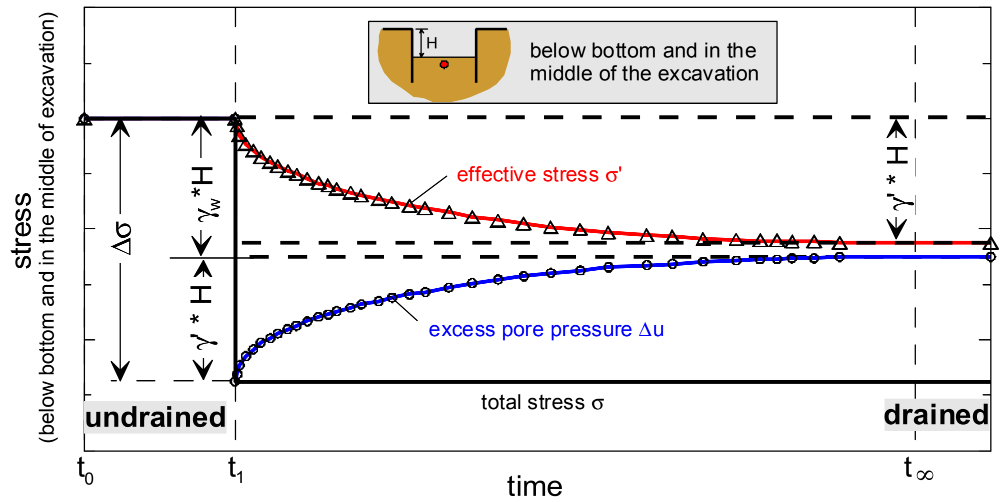

Figure 1. Idealized time dependant stress development below bottom of excavations in soft soil

SOIL-STRUCTURE-INTERACTION OF EXCAVATIONS IN SOFT SOILS

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 embedment 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 behaviour of the soil to an extent which cannot be fully quantified yet.

The soil-structure-interaction of excavations in soft soils is controlled by many factors. Mana (1978) classified the factors into: parameters under designer control, parameters partially under designer control and fixed parameters not subject to designer control (Table 1). The effects of the listed parameters on the performance of excavations in soft soils are described in Gebreselassie (2003), Puller (2003) and Kempfert & Gebreselassie (2006).

The most unfavourable condition for excavations in soft soils is apparently the drained (end) condition, since the relief of stresses due to excavation produces a negative excess pore pressure, which contributes to the stability of the excavation at initial state. It should be noted however that by temporary 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.

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 out that the readjustment of stresses and pore water pressures to 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 behaviour 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 overestimate the factor of safety of the failed slopes.

Table 1. Parameters affecting the soil-structure-interaction of excavations in soft soils

Parameter		**examples**
Under designer control	Type of support system	• type of wall (sheet piling, bored pile walls, diaphragm walls, caissons) • type of wall support (struts, ground anchors1), bottom slab, berm, etc.) • construction (bottom-up, top-down, core-wise)
	Stiffness of support system	• excavation depth, vertical distance of support • cross section and properties
	Degree of wall embedment	• free, partially or rigid
	Degree of pre-loading	
Partially under designer control	Method of support system construction	• loosening and mining of soil by cast-in-place concrete • pressing, vibrating, driving by sheet piling
	Quality of excavation works	• unexpected over excavating • progress in excavating
	Method of supporting structures construction	• e.g. construction of lean concrete, bracing base
	Construction period	• duration, schedule
	Surcharge loads	• construction equipment
	Groundwater conditions	• e.g. Confined groundwater, dewatering, etc.
	Weather	• safety measures (e.g. rain, insolation)
Not under designer contr.	Subsoil conditions and properties	• stiffness, shear strength, permeability, etc. • soil stratification • initial stress condition
	Surrounding structures	• effect on supporting structure
	Excavation shape and depth	• size, depth

1) in deeper subsoil with bearing capacity

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.

Kempfert & Gebreselassie (2002) and Gebreselassie (2003) came to the conclusion that the drained analysis results the most unfavourable 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 excavation 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.

Based on the principle that the behaviour of soils is primarily governed by the effective stresses independent of the drainage condition, Kempfert & Gebreselassie (2002) and 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, see also EAB (2006).

Due to anisotropy effect the time dependant performance of excavations in soft soils is additional influenced by the orientation and direction of the stresses extensively. The change in stresses in an excavation can be shown using a typical stress path diagram (Fig. 2.). Attention should also be paid to the discontinuity in the stress paths in different areas and due to successive construction stages.

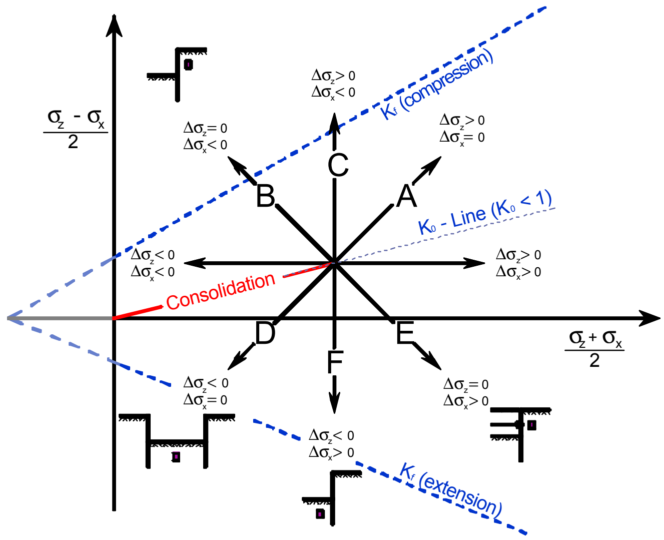

Figure 2. Idealized total stress paths of excavations in soft soils

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

EXPERIMENTAL STUDY

General

One of the strategies of the research work is to analyse the soil-structure-interaction of excavation in soft soils using small- and large-scale model tests. Beside the deformation behaviour, the excess pore pressure and total stress developments are the basis for learning more about the time and stress path dependant performance of excavations in soft soils. Furthermore, stress path controlled triaxial tests and numerical studies using advanced soil models are performed.

For the experimental study Kaolin is used instead of natural soft soil, e.g. lacustrine clay, in order to get repeatable test results and homogenous conditions, see also Soumaya (2005). The Kaolin is in its soil mechanic behaviour similar to the lacustrine soft clay. Grain size distribution and basic index properties are shown in Fig. 3 and Table 2.

Table 2. Plastic properties of Kaolin and typical lacustrine soft clay

Soil	Liquid limit w_L [%]	Plastic limit w_P [%]	Plasticity index I_p [%]	Grain size distr. *C/Si/Sa* [%]
Kaolin	36,4	18,6	17,8	28/55/17
lacustrine clay (mean values after Soumaya 2005)	38,5	20,5	18,0	42/53/5
lacustrine clay (Konstanz, after actual project)	35,4	18,3	17,1	30/61/9

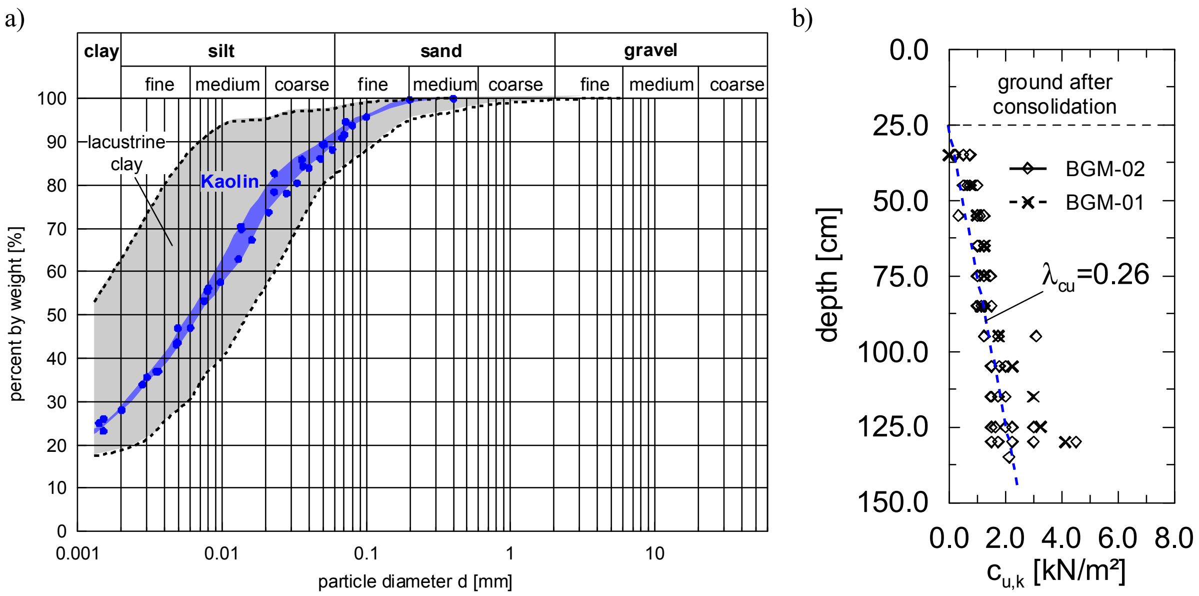

Figure 3. Grain size distribution of Kaolin (a) and undrained shear strength at large scale model tests (b)

Excavation model tests

The soil-structure-interaction of excavations in soft soil has been studied using an extensive small scale model test series and two large scale model tests at our institute. The models include several stages of excavation.

The test program contains 12 small scale and 2 large scale model tests and it is listed in Table 4 and 5. In the case of the small scale tests, the Kaolin had been consolidated under self weight or surcharge load for about 5 and 18 weeks till settlements ceased and the degree of consolidation U_c reached greater than 0.90.

Table 4. Small scale model tests

Model tests	consolidation time [d]	surcharge load	support system	measurement[1)]
V-1D-I	63	-	cantilever	8 x set / 6 x pp
V-1D-II	56	-	cantilever	8 x set / 6 x pp
V-1D-III	60	-	cantilever	8 x set / 6 x pp
V-2D-I	35	5 kN/m²	braced	4 x set / 6 x pp
V-2D-II	33	5 kN/m²	cantilever	4 x set / 6 x pp
V-2D-III	33	5 kN/m²	cantilever	4 x set / 6 x pp
V-3D-I	35	5 kN/m²	braced	4 x set / 8 x pp
V-3D-II	35	5 kN/m²	braced	4 x set / 9 x pp
V-3D-III	33	5 kN/m²	cantilever	4 x set / 9 x pp
V-4D-I	70	-	cantilever	5 x set / 7 x pp / 3 x wd
V-4D-I	91	-	cantilever	5 x set / 8 x pp / 3 x wd
V-4D-I	130	-	cantilever	5 x set / 8 x pp / 3 x wd

[1)] set – settlement, pp – pore pressure, wd – wall deformation

In the first series of small scale model tests the effects of wall support, the surcharge load and the construction stages on the development of the pore pressure had been investigated. In the case of the wall support, it was differentiated between fixed end (cantilever) and a free end support condition with additional strut support at wall head. The pore pressure change and settlement of the surface was measured at representative locations in the excavation model. In the second model test series additional measurements of wall deflection and total stress development had been conducted. The arrangements of the measuring points are shown in Fig. 4 and 5.

The measurement program includes

- pore pressure transducers,
- total stress transducers,
- settlement on the surface,
- heave of bottom of excavation,
- wall deflection.

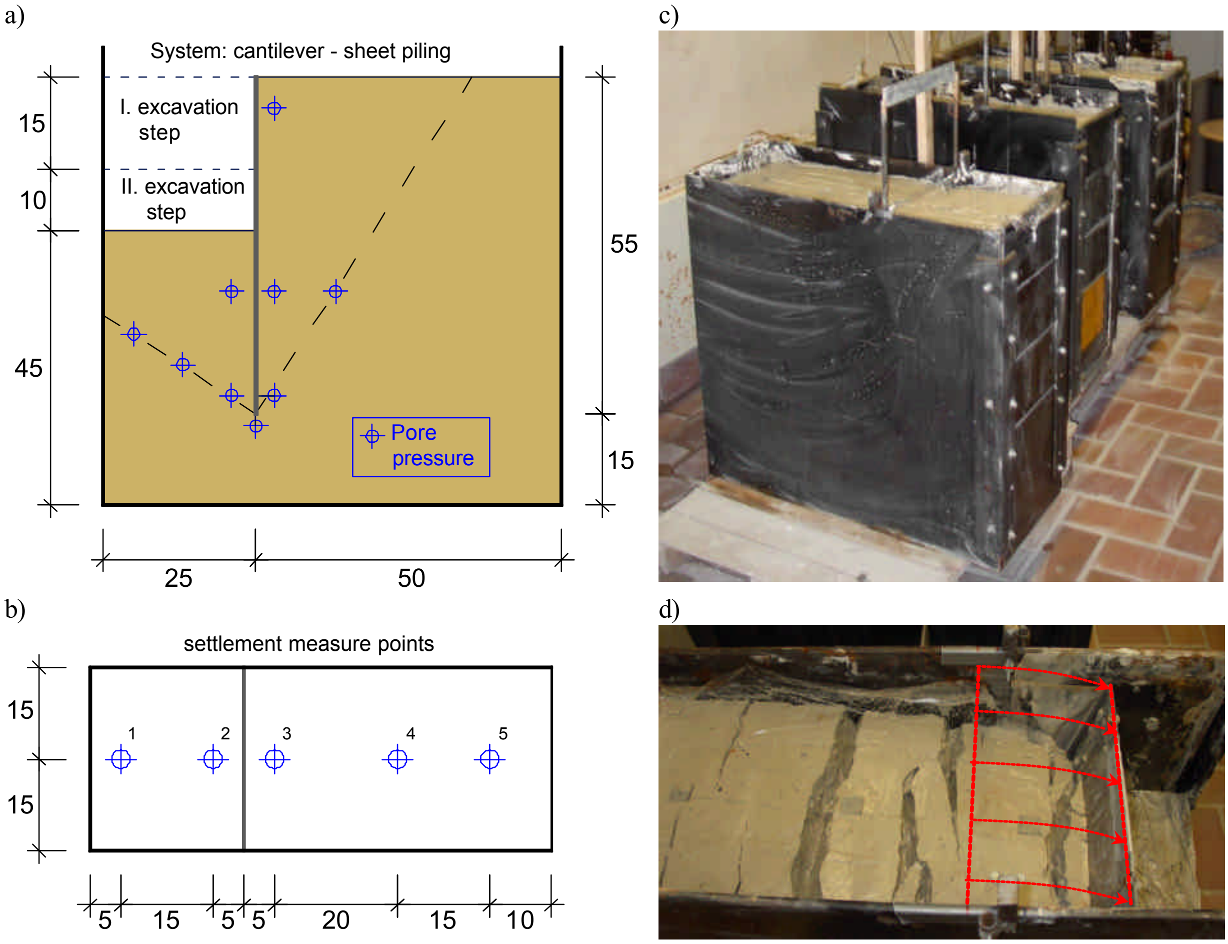

Figure 4. Small scale model test cross section (a) and horizontal projection (b) with points of measurement, cantilever test series before (c) and after (d) excavation

Table 5. Large scale model tests

Model tests	consolidation time [d]	surcharge load	support system	measurement[1)]
BGM 1	328	-	braced	24 x set / 16 x pp / 9 x ts / 3 x wd
BGM 2	800	-	braced	24 x set / 16 x pp / 8 x ts / 3 x wd

[1)] set – settlement, pp – pore pressure, ts – total stress, wd – wall deformation

Excess pore pressure development in excavation

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

Figure 6 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.

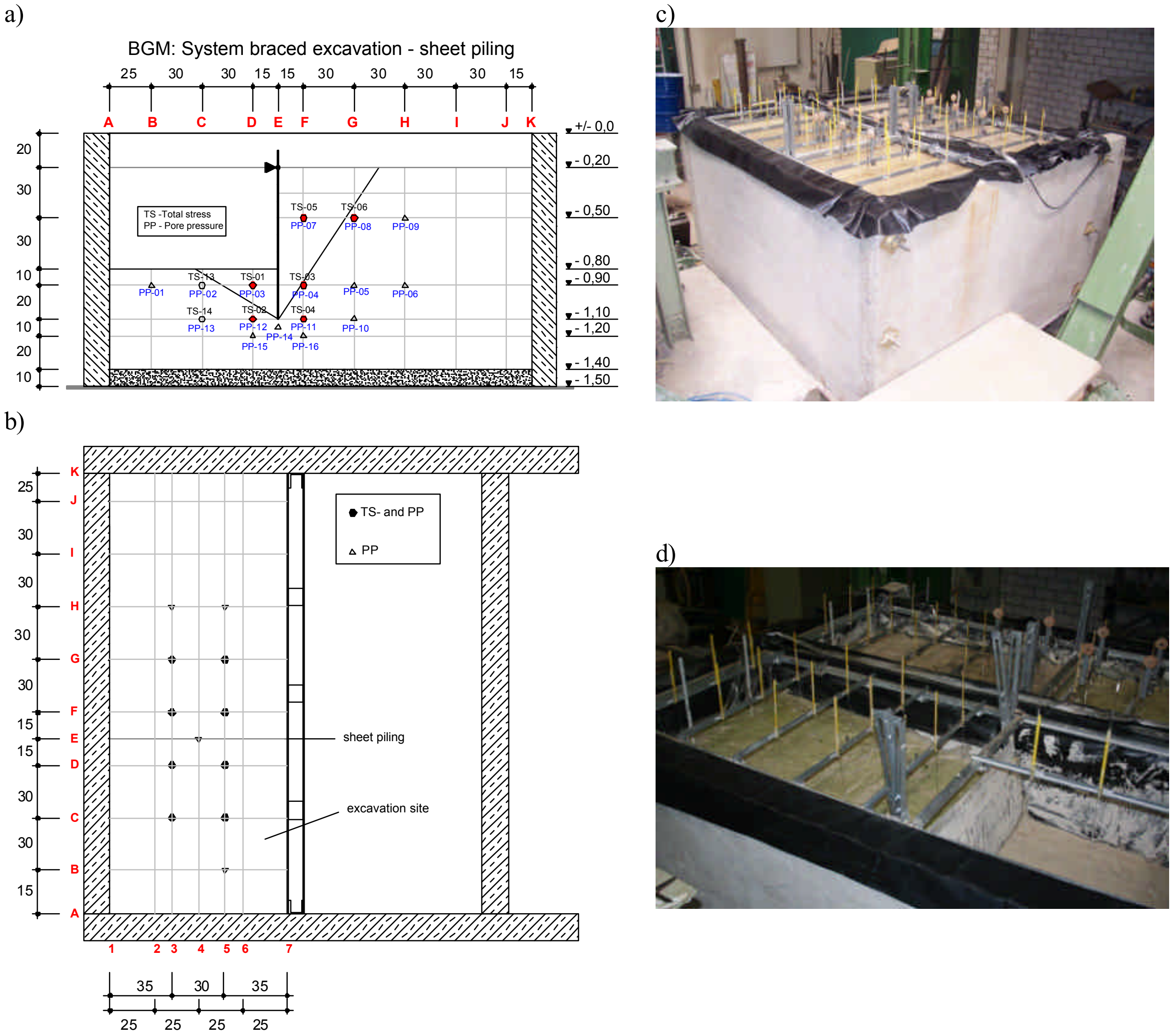

Figure 5. Large scale model test arrangements (a) cross section and (b) top view, (c) picture before and (d) after excavation

The general trend of the excess pore pressure at the measuring point A (Fig. 6a), 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, depending on the permeability and therefore on the drainage behaviour of the soil.

The problem of a consolidation analysis of an excavation becomes particularly clear at the measuring point B (Fig. 6b). 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. 6c), 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.

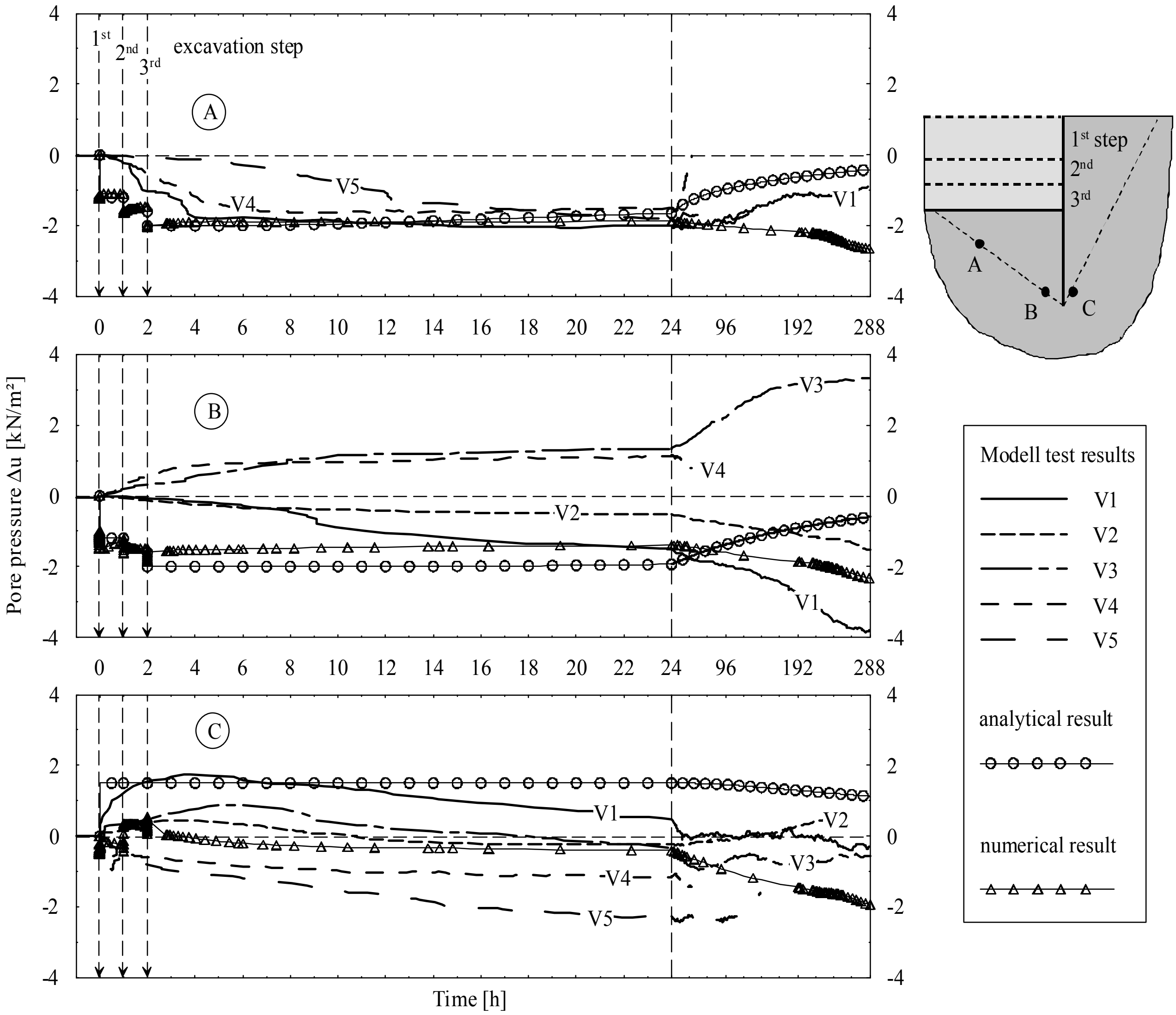

Figure 6. The development of the excess pore pressure in an excavation model

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 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.

The model test results (Fig. 6) demonstrate the difficulty in predicting the excess pore pressure development with an analytical analysis using the one-dimensional consolidation theory. Even a numerical analysis using an advanced constitutive soil model and a three-dimensional consolidation theory cannot predict the measured excess pore pressure taking into consideration the influence of the displacement and rotation of the wall toe.

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. By identifying the typical total stress path zones in an excavation, which characterize the three-dimensional deformation behaviour, 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 optimised.

REFERENCES

EAB (2006): Empfehlungen des Arbeitskreises „Baugruben". 4. Auflage. Deutsche Gesellschaft für Geotechnik (DGGT). Ernst & Sohn, Berlin.

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 Baugru-ben in weichen Böden. Bautechnik 79, Heft 9: 603-611.

Kempfert, H.-G. & Gebreselassie, B. (2006): Excavations and Foundations in Soft Soils. Springer, Berlin.

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

Mana, A.I. (1978): Finite element analysis of deep excavation behaviour. Thesis, presented in partial fulfilment of the Ph.D. Degree, Stanford University, Stanford, California.

Puller, M. (2003): Deep Excavations: a practical manual. 2nd edition. Thomas Telford Ltd, London.

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.

Soumaya, B. (2005). Setzungsverhalten von Flachgründungen in normalkonsolidierten bindigen Böden. University of Kassel, Institute of Geotechnics, Dissertation, Booklet No. 16.