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IMPORTANCE OF OBSERVATIONAL METHOD IN VIEW OF NUMERICAL ANALYSES FOR RETAINING STRUCTURES IN SOFT SOILS

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ABSTRACT

The soil-structure-interaction of retaining structures is complex and dependent on many factors. In particular excavations in soft soils and urban environments with adjacent buildings are always subjected to deformations which are not fully avoidable. Therefore, numerical analyses can be a powerful tool for predicting the stress path and time dependent deformation behavior of retaining structures in soft soils. But the quality of these numerical predictions is directly related to the used constitutive soil models, the estimation of their material parameters and a realistic simulation of the soil-structure-interaction. The fact that Class-A predictions of the deformation behavior of retaining structures in soft soils tend to be rare highlights the current limitations of numerical analysis. Therefore, the observational method has still to be carried out as superior controlling tool for the construction of retaining structures in soft soils.

This contribution stresses the importance of the observational method based on a case history of an 8m deep excavation. The excavation is located in the City Constance in southern Germany and was successfully constructed in deep soft lacustrine clay deposits in 2008. The monitoring program is described in detail with its instrumentation consisting of vertical inclinometers, geodetic deformation points, pore pressure and strain transducers. Furthermore, the concept of the observational method is explained by means of limiting values of threshold for deformations and forces depending on the construction process.

Additionally, the evaluation of an a priori numerical analysis which was used for the determination of the threshold values is presented together with the measurement results and a numerical back analysis. The limitations of numerical analysis of retaining structures are shown based on the presented case history and successful application of the observational method. As a result recommendations are presented for numerical simulations of the soil-structure-interaction of excavations in soft soils.

INTRODUCTION

The prediction of deformations from excavations in soft soils is generally performed using the finite element method (FEM) in combination with the application of advanced constitutive soil models. The fact that a reliable estimation of deformations in advance of the project is generally difficult due to the variety of factors on soil-structure-interaction can be judged by the rare number of Class-A predictions published in literature. The FEM is rather applied for retaining structures in soft soils for more than three decades in conjunction with the observational method.

The a priori numerical analysis can be used to establish the design based on a working hypothesis of behavior anticipated under most probable conditions. In that case should the numerical calculation lead with suitable variations of material parameters and construction stages to both alarm and limit values of deformations and forces, taking into account a realistic modeling of the boundary value problem. Furthermore, the time-dependent material behavior and the characteristic stress paths in excavations, which differ from those of standard laboratory tests, require a high degree of experience of the geotechnical engineer for estimating the material parameters. If there are no appropriate laboratory test results available and calibration of measuring results in the planning phase is not yet possible, uncertainties regarding the numerical analysis increase significantly and hamper as well the design of structures and the optimization of the construction stages.

Therefore, the observational method has to be used for the validation of the working hypothesis based on the numerical analysis. The observational method enables a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis.

In the following the importance of the observational method in view of numerical analyses for retaining structures in soft soils is illustrated using a case study from literature, Becker (2009).

OBSERVATIONAL METHOD

The observational method is a combination of common geotechnical investigations and predictions with continuous measurements of the soil-structure-interaction during construction and, if necessary during its use.

In general the observational method has to be adopted in cases where it is not possible to predict the soil-structure-interaction based solely on previous ground investigations and geotechnical predictions. In particular, this includes a large range of geotechnical constructions which can be specified according to the national German standard (EN 1997-1):

- very complex construction projects;
- construction projects with pronounced soil-structure interaction, e.g. mixed foundations, raft foundations, flexibly anchored retaining walls;
- construction projects with substantial and variable water pressure action, e.g. trough structures or waterfront structures in tidal areas;
- complex interaction systems consisting of subsoil, excavation structure and neighboring buildings;
- construction projects in which pore water pressures may reduce stability;
- construction projects on slopes.

The observational method by Peck makes use of conventional geotechnical predictions which are based in the case of retaining structures in soft soils on numerical analysis. The observational method embodies according to Peck (1969) the following steps:

a) Exploration sufficient to establish at least the general nature, pattern and properties of the deposits, but not necessarily in detail.
b) Assessment of the most probable conditions and the most unfavorable conceivable deviations from these conditions. In this assessment geology often plays a major role.
c) Establishment of the design based on the working hypothesis of behavior anticipated under the most probable conditions.
d) Selection of quantities to be observed as construction proceeds and calculation of their anticipated values on the basis of the working hypothesis.
e) Calculation of values of the same quantities under the most unfavorable conditions compatible with the available data concerning the subsurface conditions.
f) Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted on the basis of the working hypothesis.
g) Measurement of quantities to be observed and evaluation of actual conditions.
h) Modification of design to suit actual conditions.

The observational method is based on a working hypothesis of behavior anticipated under the most probable conditions. Therefore, both the highly non-linear and time dependent material behavior of the soft soils and the soil-structure-interaction of retaining structures have major impact on the geotechnical predictions. Here, an experienced geotechnical engineer has to consider the uncertainties of soil conditions and construction stages for the design of the retaining structure in soft soils.

Based on the numerical analysis threshold values of deformations and forces have to be specified which allows the validation of the numerical prediction and the more important intervention in case of exceeding the specified threshold values.

The main part of the observational method is of course the observation of the soil-structure-interaction by the means of measurements and their evaluation. The measuring intervals, duration between the measurement and the evaluation of the observations have to be adjusted to the construction progress and possible developments in the behavior of the soil-structure-interaction.

NUMERICAL ANALYSES

Numerical methods enable the analysis of complex geotechnical problems with an emphasis on the soil-structure-interaction whereas conventional geotechnical methods, e.g. limit equilibrium, stress fields, empirical methods, are not sufficient anymore. In general, numerical methods can be suitable for the geotechnical analysis of:

- stresses and deformations,
- groundwater flow,
- stability analysis,
- design of geotechnical structures.

In particular are numerical methods a powerful tool for the analysis of stresses and deformations due to the soil-structure-interaction. The soil-structure-interaction of retaining structures in soft soils is governed on the one hand by the material behavior of the soils and therefore the appropriate selection of constitutive soil models and then again by the spatial influences of support elements, e.g. struts, base slab, etc., and construction stages, e.g. slice wise excavation, time effects. The later influences have to be considered by the discretization of the boundary value problem and require in some cases 3D simulations.

It is obviously important that in any analysis realistic, and appropriate, constitutive models are used to represent the behavior of the structural components and the behavior of the ground. The constitutive soil model used for the numerical analysis of retaining structures in soft soils should be able to predict the following features of soil behavior:

- non-linear stress-strain relation,
- plastic strains,
- hardening and softening behavior,
- stress path dependency, e.g. primary loading, unloading, reloading,
- anisotropy,
- time dependency.

For retaining structures in soft soils it is not sufficient to use appropriate constitutive models which are validated only on element level, i.e. element tests. Because of the complex soil-structure-interaction the numerical analysis has to be validated on its own. Gudehus (2004) pointed out the importance and consequences of validated numerical analysis (prognosis) for the observational method. In EAB (2012) it is recommended to calibrate the numerical analysis beforehand as back analysis of comparable projects of retaining structures with similar soil conditions and to validate the prediction with measured quantities.

The quality of each numerical analysis is directly linked to the quality of the soil parameters and state variables, the constitutive soil model and the discretization of spatial influences and construction stages. Each part has to be considered for the evaluation of the geotechnical prediction as basic concept of the observational method. For more details about the constitutive soil models and numerical modeling of retaining structures see also Hettler & Schanz (2008), Kempfert & Gebreselassie (2006), Wood (2004), EAB (2012), etc.

CASE STUDY – DEEP EXCAVATION IN SOFT SOIL

<u>General Description</u>

<u>Introduction.</u> The excavation site is located in the City Constance in southern Germany and was intended for two basements floors of a multi-story shopping center. The retaining structure of an 8m deep excavation was successfully constructed in deep soft lacustrine clay deposits in 2008. The layout of the retaining structure has an almost quadratic shape with site lengths of 21.6 m and 24.65 m. The site plan is shown in Fig. 1.

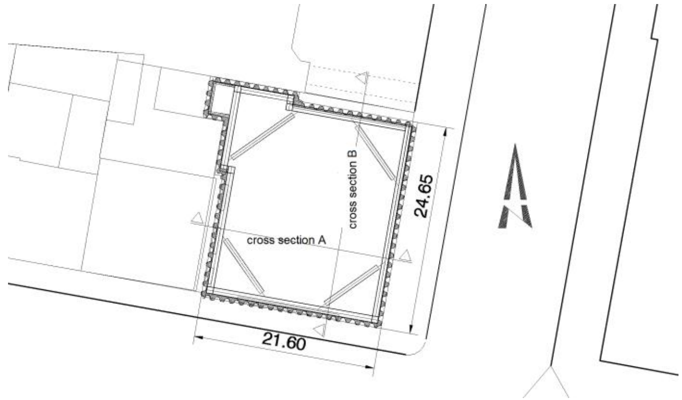

Fig. 1. Site plan.

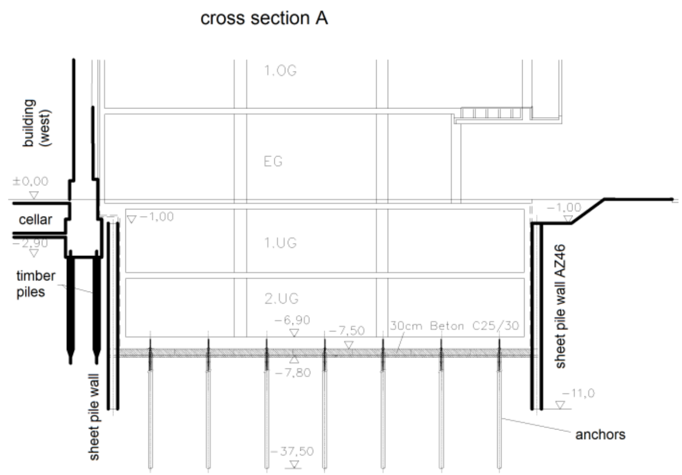

Fig. 2. Cross section A.

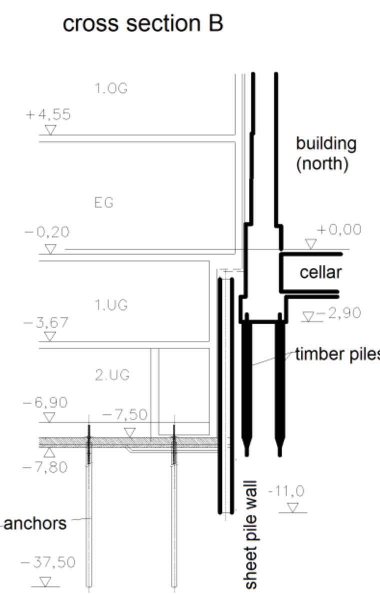

Fig. 3. Cross section B.

The excavation is at a street corner of the old town of Constance and is surrounded on two sites by the main road and on the other two sites by existing multi-story buildings in direct proximity of the retaining structure, see also Fig. 2 and Fig. 3.

<u>Site Condition.</u> The site investigation revealed a ground comprising below made ground and basin sand at the top 4 m upper lacustrine silty clay of thickness 4 m with low to medium plasticity and soft to stiff consistency overlying very soft lower lacustrine clay of thickness 17 m. Beneath the lower lacustrine clay layer is a low plastic lacustrine clay mixed boulder clay of thickness 5 m overlaying moraine gravel.

The groundwater is located at 2 m below the ground level.

An undrained shear strength profile from field vane tests and cone penetration test from the site is shown in Fig. 4.

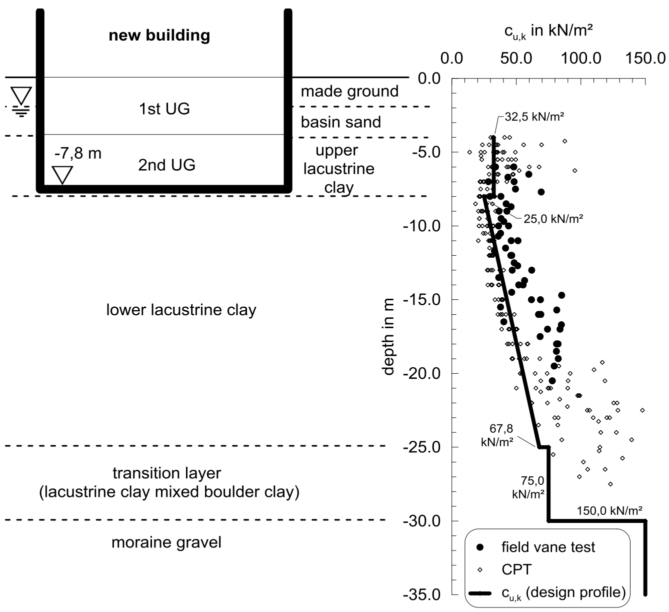

Fig. 4. Soil layer and undrained shear strength profile.

Additional information about soft clay deposits in Constance area and case studies of retaining structures is reported in Becker et al. (2008), Becker (2009), Kempfert & Gebreselassie (2006), Gebreselassie (2003) and Scherzinger (1991).

<u>Support System.</u> Sheet piles (type AZ 46) with a length of 11 m were used as retaining structure and supported with an upper frame at -1.80 m and a lower frame at excavation base at -7.80 m. The upper frame support consisted of a circumferential waler line of steel profiles HEB 800 which were supported by diagonal steel struts HEB 600 combined with IPB 450. The lower frame support was intended by the construction in slices of diagonal concrete base slabs.

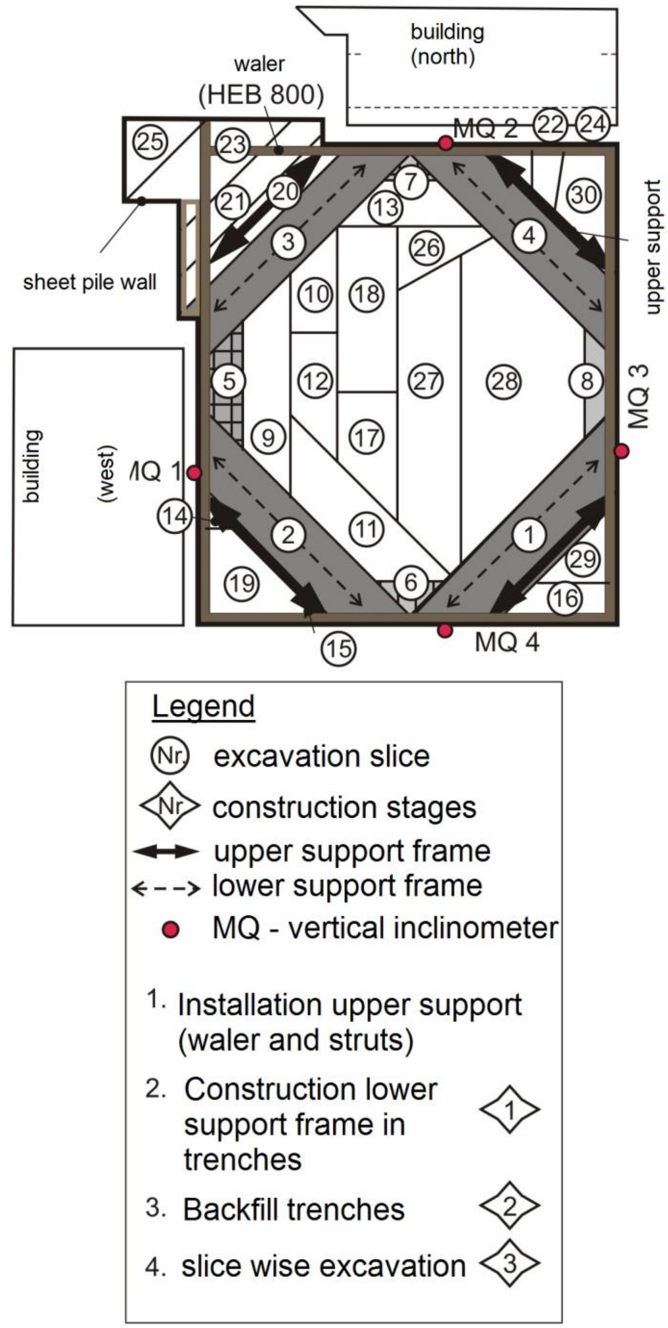

Fig. 5. Construction steps and instrumentation.

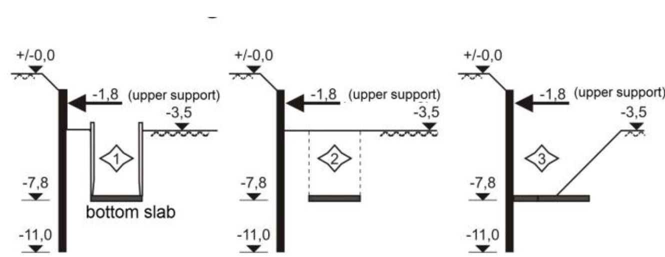

Fig. 6. Construction steps of lower support.

Construction Stages. The sheet piles (AZ 46) were installed by pressing after removal of obstacles.

The first support was realized with the upper support frame (HEB 800), see Fig. 5. The construction was conducted from a preliminary excavation level of -1.0m below ground surface in sloped trenches next to the retaining wall. The diagonal steel struts (HEB 600 / IPB 450) were installed afterwards and pre-stressed with an axial force of 2.5 MN.

After installing the upper support frame the lower support frame had to be constructed in a modified procedure to minimize the risk of deformations of the retaining structure due to a loss of lower support during excavation. Therefore, the lower support frame had to be installed without major excavation work. This was realized by diagonal reinforced concrete slabs which had to be constructed one at the time, step 1 to 4 in Fig. 5. A trench was excavated using a trench support system diagonal between the retaining wall and with a depth of 8 m and a width of 2 m. The trench was backfilled after the construction of the lower concrete slab and then the next trench could be excavated using a trench support system. The construction steps for the installation of the lower support frame are idealized in Fig. 5 and Fig. 6.

Thereafter, the excavation was conducted from an intermediate excavation level at -3.5 m above the soft lacustrine clay deposits in slices according to the excavation steps in Fig. 5. Each excavation step had to be finished on daily basis with installing the reinforced concrete slab to achieve immediate support at the excavation level.

The construction stages are summarized:

- Sheet piles installation by pressing,
- Upper support frame in sloped trench,
- Diagonal support of upper frame with steel struts,
- Excavation of a diagonal trench with trench support system,
- Construction of diagonal lower reinforced concrete slab one by one,
- Backfill of trench,
- Intermediate excavation level at -3.5 m,
- Excavation in slices and immediate construction of reinforced concrete bottom slab.

Additional support of the concrete bottom slab against basal heave failure was achieved by installation of 80 anchors with diameter of 15 cm and to a depth of -37.5 m, see also Fig. 2 and Fig. 3.

A priori Numerical Analysis as Geotechnical Prediction

General. The working hypothesis for the observational was established with a numerical analysis using the finite element method (FEM). The a priori numerical analysis (Class A prediction) could be calibrated and validated with soil properties from field and laboratory tests, back analysis of similar projects in the area of Constance and evaluation of measurement results from previous observational methods (Becker et al. (2008), Kempfert & Gebreselassie (1999), (2006), Becker (2009)).

The stress-strain behavior of soft soils depending on stress-paths due to the excavation process in this validated numerical analysis is considered using a commercially available FE-Program (PLAXIS 2Dv9) and advanced constitutive soil models. The elasto plastic hardening soil model (HS) was applied as constitutive soil model. For detailed information about the soil model see Schanz (1998) and Brinkgreve et al. (2008).

FE Model Geometry and Idealization of Spatial Effects. The 2D numerical analysis was performed for the monitoring section indicated in Fig. 3 (cross section B). The spatial influences of soil structure interaction on the deformation behavior, e.g. slice wise excavation, installation of supporting bottom slab and diagonal steel struts, are considered in the analysis using an idealized modeling of the construction progress. The time-dependent behavior was considered with the help of an undrained analysis using a coupled consolidation analysis which take into account the actual construction periods.

The structural components were simulated as structural elements (e.g. retaining wall, steel struts and micro piles) and continuum elements (e.g. bottom slab). The diagonal struts were idealized in the 2D simulation as fixed node anchors and influences from tangential forces at the retaining wall were neglected due to the quadratic pattern of the excavation. The FE model geometry is shown in Fig. 7.

The foundation of the adjacent building was modeled with timber piles below the strip foundation of the gable wall. The timber piles were as well simulated with linear elastic continuum elements.

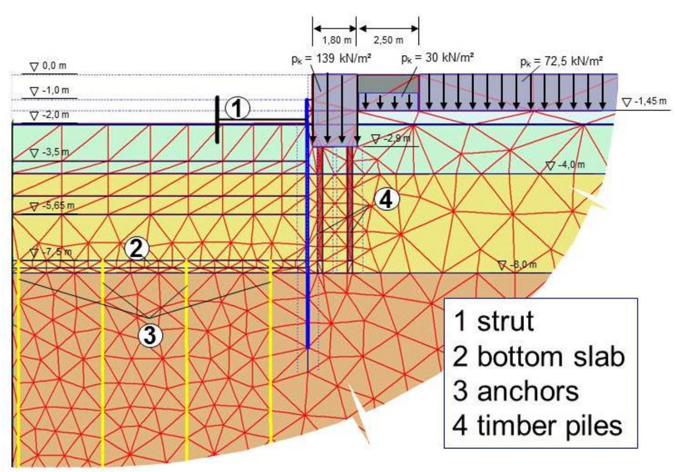

Fig. 7. FE model geometry.

<u>Calculation Phases.</u> The real construction process of the excavation at the simulated cross section has been distributed to a longer period to take account of temporal effects on the stress deformation behavior.

The main difficulty in a 2D analysis is to model the spatial effects of the supporting structures. For this case study, this is the installation of the upper support frame (diagonal steel strut), lower support frame (reinforced concrete slab in trenches) and the slice wise excavation and construction of the bottom slab. These effects were examined separately in a numerical analysis and simulated with so called mobilization factors, see Becker (2009).

The calculation phases are summarized in Table 1. For all calculation steps a groundwater flow calculation was carried out with a water table at a depth of -2 m behind the wall and at the bottom of excavation within the excavation. The above mentioned spatial effects were considered at the calculation phases 07 and 11. This changed the calculation phase in a purely plastic calculation and soil excavation was controlled by the program's internal factor m_{stage}, which reduces the stiffness of the excavation area, see also Schikora & Fink (1982). The subsequent calculation step was performed using a coupled consolidation analysis taking into account the conceptual construction period of the previous step, where the partial calculation step ($m_{stage} < 1.0$) has been completed.

Table 1. Calculation phases.

Phase	Type	Description	Time [d]
00	P	Initial stress (K_0-procedure)	
01	P	Simulation of load history (adjacent building)	
02	P	Simulation of load history (removal of old building on site)	
03	C	Preliminary excavation -1.0 m, activation of anchors	120
04	P	Installation of sheet pil wall (wished in place)	
05	C	Consolidation	12
06	C	Consolidation	70
07	C[1)]	1st excavation step –2.0 m	9
08	P	Installation of upper support frame	
09	P	Pre-stressing of steel struts	9
10	C	2nd excavation step –3.5 m	10
11	C[1)]	3rd excavation step –7.8 m	10
12	P	Installation of lower support	
14	C	Consolidation (min pore pressure)	

N.B.: P – plastic calculation
C – plastic consolidation with coupled consolidation analysis
1) Calculation with $m_{stage} < 1.0$ plastic (P) and following phase was plastic consolidation (C)

<u>Material Parameters.</u> The material parameters for the a priori numerical analysis were adopted from geotechnical reports and verified by parameters from adjacent projects. In Table 2 are the material parameters for the HS model. The material parameters of the structural elements are indicated in Table 3 and of the continuum elements in Table 4.

Table 2. Soil parameters for the HS-model.

a) Unit weight and permeability

Soil layer	γ_{sat} [kN/m³]	γ_{unsat} [kN/m³]	$k_x = k_y$ [m/d]
Fill material	21.0	21.0	0.086
Basin sand	19.0	20.0	1.730
Upper lacustrine clay	19.0	19.0	8.64E-4
Lower lacustrine clay	19.0	19.0	8.64E-4
Transition layer	20.0	20.0	8.60E-4
Ground moraine	22.0	22.0	8.60E-4

b) Stiffness parameters

Soil layer	E_{50}^{ref} [MN/m²]	E_{oed}^{ref} do.	E_{ur}^{ref} do.	p^{ref} do.	ν_{ur} [-]	m [-]
Fill material	6.0	6.0	24.0	0.1	0.2	0.70
Basin sand	8.0	8.0	32.0	0.1	0.2	0.50
Upper lacustrine clay	5.9	4.5	19.0	0.1	0.2	0.90
Lower lacustrine clay	6.0	5.5	24.0	0.1	0.2	0.90
Transition layer	8.0	8.0	32.0	0.1	0.2	0.50
Ground moraine	40.0	40.0	160.0	0.1	0.2	0.80

c) Shear strength parameters

Soil layer	c´ [kN/m²]	φ´ [°]	ψ´ [°]	R_f [-]
Fill material	0.01	30.0	0.0	0.90
Basin sand	0.01	27.5	0.0	0.90
Upper lacustrine clay	13.5	26.0	0.0	0.90
Lower lacustrine clay	0.01	24.0	0.0	0.90
Transition layer	0.01	25.0	0.0	0.90
Ground moraine	0.01	30.0	0.0	0.90

Table 3. Material properties of the structural elements.

Structural element	EA [kN/m]	EI [kNm²/m]	w [kN/m/m]	ν [-]
Sheet pile (AZ46)	6.11E06	2.32E05	2.30	0.3
Upper support (HEB600 + IPB450)	7.04E05	-	-	-
Anchor	2.22E05	-	-	-

Table 4. Material properties of the continuum elements.

Continuum element	γ kN/m³	$k_x = k_y$ m/d	ν	E_{ref} MN/m²
Reinforced bottom slab (d = 0,30 m)	23.0	0	0.20	2.5E04
Timber piles	as soil	as soil	0.20	1.3E04

Geotechnical Prediction. The results of the numerical analysis as part of the observational method are presented briefly in Fig. 8 and Fig. 9. A more detailed discussion of the numerical analysis is given later together with the measurement results.

Based on the numerical analysis threshold values in form of warning levels and intervention levels were derived for the construction process of the observational method. The warning level of horizontal wall deflections is given at 3.2 cm and the intervention level at a maximum deflection of 4.0 cm, see Fig. 8.

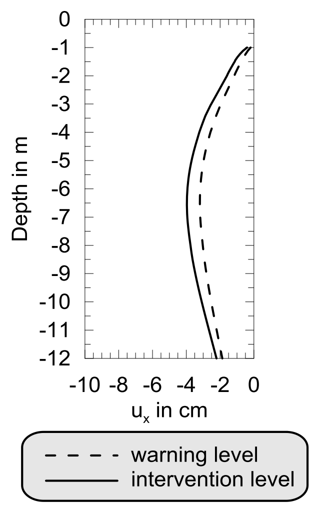

Fig. 8. Threshold values for horizontal wall deflection.

The settlements next to the retaining wall were defined by the warning level of 2.4 cm and the intervention level of 3.2 cm, see Fig. 9.

The strut forces of the upper support frame were defined with a warning level at 3100 kN and an intervention level of 3400 kN.

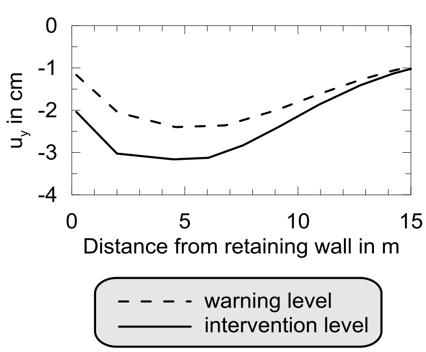

Fig. 9. Threshold values for settlements.

Observational Method

The monitoring program of the observational method contained deformation measurements of the surrounding buildings in a distance up to 50 m. Additional, deformation points were installed and monitored at the gable walls of the adjacent buildings at every floor level next to the retaining structure and at every site of the retaining structure at the top of the wall. The measured quantities were vertical and horizontal deformations.

Furthermore, 4 vertical inclinometers (V) were installed behind the sheet pile wall in the middle of each section, see Fig. 10. The horizontal deflection of the sheet pile wall was limited by the before mentioned threshold values.

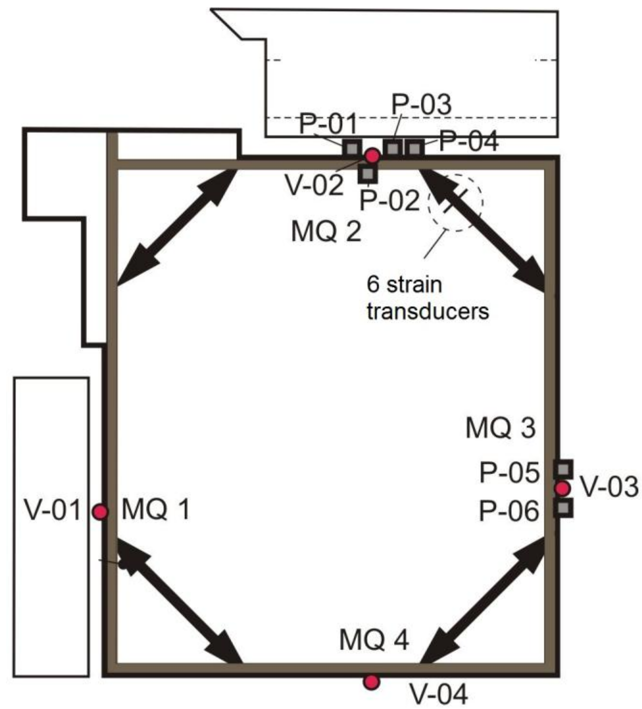

Fig. 10. Construction steps of lower support.

Monitoring section MQ 2 and MQ 3, see Fig. 11, was additional equipped with in total 6 pore pressure transducers and strain transducer for the observation of the strut force between MQ 2 and MQ 3, see Fig. 10.

The measurement results are presented and discussed together with the results of the numerical back analysis.

Numerical Back Analysis

General. This case study was reviewed and evaluated within an independent research project. The aim of this research project was to improve the numerical analysis of retaining structures in soft soils with stress path dependent material behavior based on commercial available FEM codes. For further information see Becker (2009).

The following results are based on a modified numerical analysis which takes stress path dependent material behavior of the anisotropic soft soil into account. For this purpose, stress path zones next to the retaining wall were identified with 1g model tests and numerical analysis of different construction stages. The anisotropic material behavior was studied with series of triaxial stress path test on undisturbed samples from this project, Becker (2009).

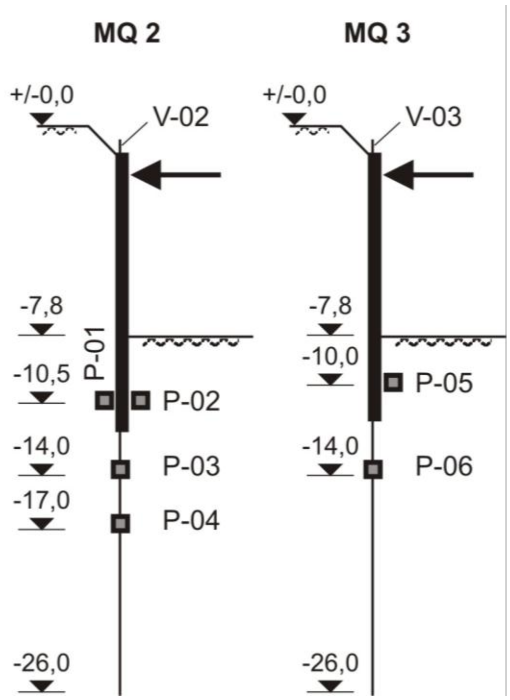

Fig. 11. Construction steps of lower support.

The numerical analysis is based on the small strain concept and the elasto plastic Hardening Soil Small model (HSS) was applied as constitutive soil model. For detailed information about the soil model see Brinkgreve et al. (2008) and Benz (2007).

Modifications had to be made for the material parameters of the soft lacustrine clay layers and the FE model geometry based on the a priori numerical analysis described before. The construction phases were not changed.

<u>FE Model Geometry for Back Analysis.</u> The FE model geometry contains as a result from the a priori numerical analysis both sides of the simulated cross section. This was improved because of the different stress history of the soil and different length of the sheet pile walls. Furthermore, stress path zones with modified material parameters were introduced. The FE model geometry is shown in Fig. 12 which contains a detail of the area of the retaining structure.

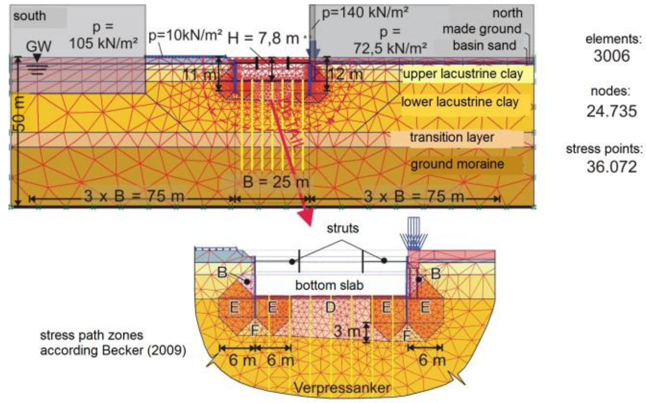

Fig. 12. Modified FE model geometry.

<u>Material Parameters.</u> The material parameters for the numerical back analysis were derived from triaxial stress path tests on undisturbed samples from this project. The material parameters for the HSS model are based on empirical methods and given in Table 5. The modified stiffness input parameters which are characteristic for the identified stress path zones (Fig. 12) are indicated in Table 6.

Table 5. Additional soil parameters for the HSS-model.

Soil layer	G_0^{ref} MN/m²	$\gamma_{0,7}$ -
Fill material	50,0	3,1E-04
Basin sand	60,0	2,4E-04
Upper lacustrine clay	31,0	3,0E-04
Lower lacustrine clay	44,0	2,0E-04
Transition layer	60,0	2,4E-04
Ground moraine	150,0	1,0E-04

Table 6. Modified stiffness in stress path zones.

SPZ	E_{50}^{ref} in MN/m²	E_{ur}^{ref} in MN/m²	G_0^{ref} in MN/m²	$\gamma_{0,7}$ -
B	17,50	87,50	87,50	1,7E-04
D	14,40	76,35	76,35	1,9E-04
E	13,00	69,10	69,10	2,1E-04
F	22,30	118,15	118,15	1,2E-04

<u>Numerical Results and Comparison with Measurements.</u> Figure 13 indicates the horizontal deflection of the sheet pile wall in monitoring section MQ 2. There is a good agreement between the observed deformations and the numerical predicted. The measurement result after reaching final excavation level lies within the predefined threshold values.

The observed settlements in Fig. 14 can be as well simulated by the numerical analysis.

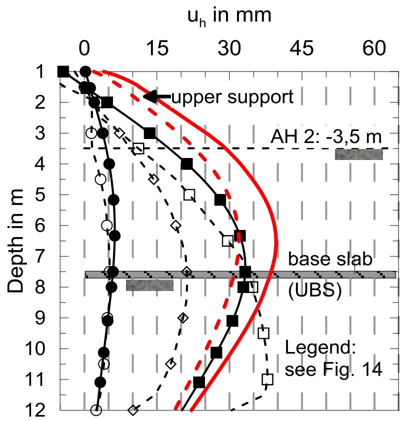

Fig. 13. Horizontal deflections of sheet pile wall in MQ2.

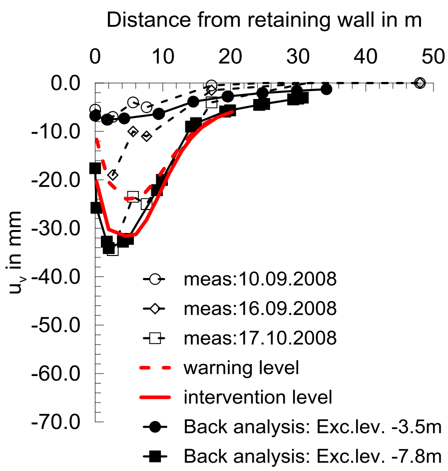

Fig. 14. Settlements in MQ 2.

The observed time dependent development of excess pore pressures is shown in Fig. 15. The numerical simulation describes the reduction of negative excess pore pressure due to the excavation in a very good agreement with the measurement results. The negative excess pore pressure dissipated within 3 months.

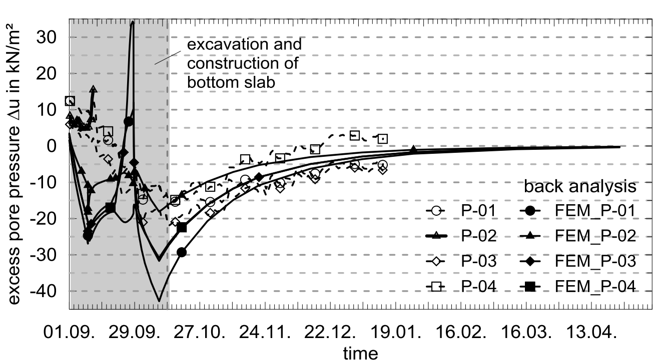

Fig. 15. Excess pore pressures.

The strut force is derived from strain transducers at one end of the combined steel profile. Although, there is a pronounced scatter of the measurement results evident the pre-stressing force of 2500 kN has to be compared with the measured 2200 kN. There are some uncertainties regarding the composite cross section of the steel profile, but the measurement results can still be evaluated. The trend of the strut force development can be approached with an over prediction of the numerical analysis. The influence of the excavation can be seen in both measurements and numerical simulation. The threshold value of the warning level (3100 kN) was reached during the excavation phase and the trend was still increasing. Therefore, the strut profiles were reinforced with additional steel profile sections as demanded from the observational method after reaching the critical intervention level (3400 kN).

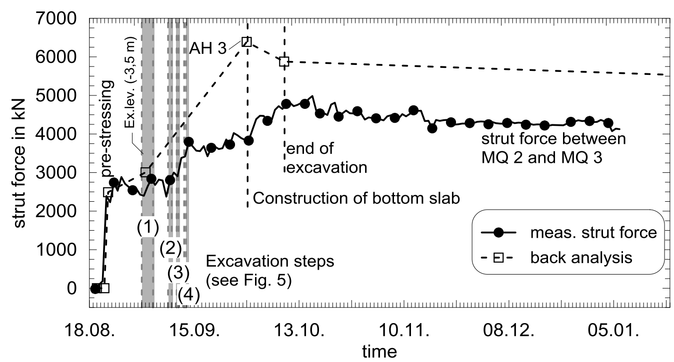

Fig. 16. Strut force between MQ 2 and MQ 3.

CONCLUSIONS

Design and construction of retaining structures in soft soils is very complex and in most cases deformations are not avoidable. The vast development of numerical methods in the last decades allows better and more reliable predictions with every development step. But the quality of these numerical analyses is directly linked to the quality of input parameters, the suitability of the constitutive models and the ability of geotechnical engineers.

Therefore, care has to be taken due to the user friendly commercial software codes and their possibilities. The numerical analysis is only as good as their validation.

Recommendations for the geotechnical prediction of the observational method after Peck (1969) can be concluded based on the presented case study with modifications required in view of the numerical analysis of retaining structure in soft soils:

- Identification of material properties and state variables of soft soils tailored to the constitutive soil model used for the numerical analysis (Field and laboratory tests are necessary).

- Evaluation of soil data with identification of the most probable conditions and the most unfavorable conceivable deviations from these conditions.
- Numerical analysis with appropriate and realistic constitutive soil model
- Variations based on most unfavorable conceivable deviations of soil data and construction stages of the numerical analysis.
- Validation of numerical analysis, e.g. back analysis of similar retaining structures.
- Design based on validated numerical analysis of behavior anticipated under the most probable conditions.
- Selection of quantities to be observed as construction proceeds
- Identification of threshold values for the selected quantities on alarm level and intervention level.
- Selection in advance of a course of action or modification of design for every foreseeable significant deviation of the observational findings from those predicted.
- Measurement of quantities to be observed and evaluation of actual conditions.
- Modification of design to suit actual conditions.

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