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In situ measurements and FE-analysis of a deep excavation

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ABSTRACT: An elasto-plastic finite element analysis has been performed to analyse the deformation behaviour of an excavation for an underground car park. Although the excavation is not very deep (about 6.3 m below the surface) it has been a very difficult project because of the presence of soft soil layers and the vicinity to existing railway lines. In addition to numerical analyses prior to construction it was decided to refine the analysis during construction utilizing results from in situ measurements obtained at the early stages of construction. Results from these analyses are presented together with measured displacements. It follows from this comparison that the numerical model proved to be a valuable tool in assessing the deformations to be expected at critical phases of excavation.

1 INTRODUCTION

Finite element calculations are widely used to assess ground deformations to be expected at various stages of the construction of underground structures such as subway lines and underground car parks. Numerical studies covering many important aspects of numerical modelling have been presented in the literature (e.g. Potts & Fourie 1986, Matsuzawa & Hazarika 1997, Simpson 1992, Whittle & Hashash 1994, Hashash & Whittle 1996, Freiseder & Schweiger 1997). In addition to these rather fundamental contributions analyses related to practical applications have also been presented (e.g. Whittle et al. 1993, Hsi & Small 1993, Ou & Lai 1994). Comparison with field measurements generally shows that simple elastic-perfectly plastic constitutive models are not able to represent the deformation behaviour observed in the field and the importance of estimating the in situ stiffness, especially under small strains, has been pointed out e.g. by Hashash & Whittle (1996).

In general it is not possible to include the complete construction sequence into the model in great detail especially if 2-D analyses are performed, an approach which is still commonly adopted in practice although 3-D analyses become more and more feasible (e.g. Ou et al. 1996). It is therefore often necessary to refine the numerical analysis undertaken in the design stages based on measurements obtained at the early stages of construction. More reliable predictions of deformations for later, possibly crucial, construction stages can then be made with sufficient time available for taking auxiliary measures if required. As such numerical methods can play a significant role in applying the observational method in geotechnical engineering. A successful example of this approach will be given in this paper.

2 PROBLEM DESCRIPTION AND GROUND CONDITIONS

In the city of Salzburg, Austria an underground car park was planned in the immediate vicinity of an existing underground railway line, which had been constructed as open pit excavation with two 24 m deep diaphragm walls as retaining walls. These walls are part of the final construction. In addition the site was close to the the main railway station. The layout follows from Figures 1 and 2, also indicating the overal dimensions of the project.

The basin of Salzburg is of Quaternary origin. During the last glaciation period a more than 250 m deep basin was created by erosion which on melting and receding of the ice cover was filled with sediments of fine sands and clayey silts of varying thickness, the latter locally known as "Seeton" which are at places up to 70 m thick. In the later period the "Seeton" was covered by a 4 m to 6 m thick layer of compact gravel and sand (Fig.1). For the construction of deep basements in Salzburg, dewatering with vacuum wells, combined with special excavation techniques, is generally necessary for an efficient excavation.

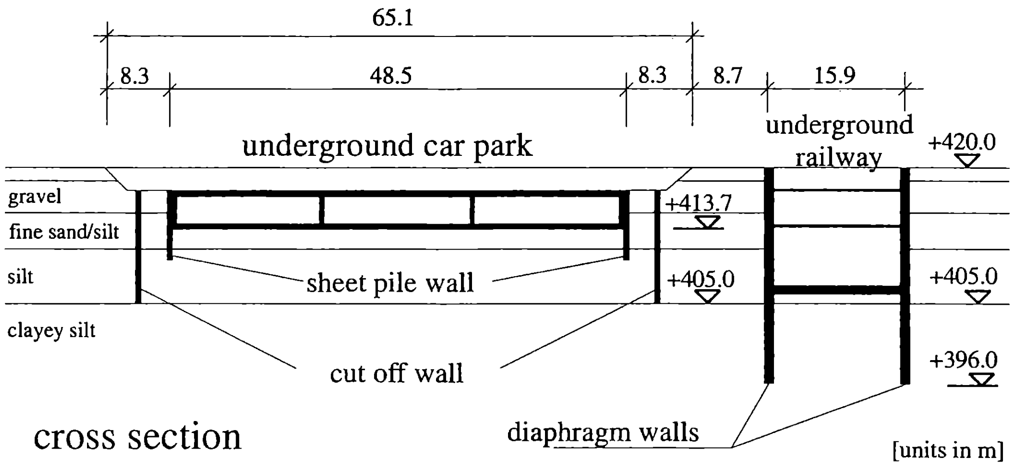

Figure 1. Cross section of underground car park.

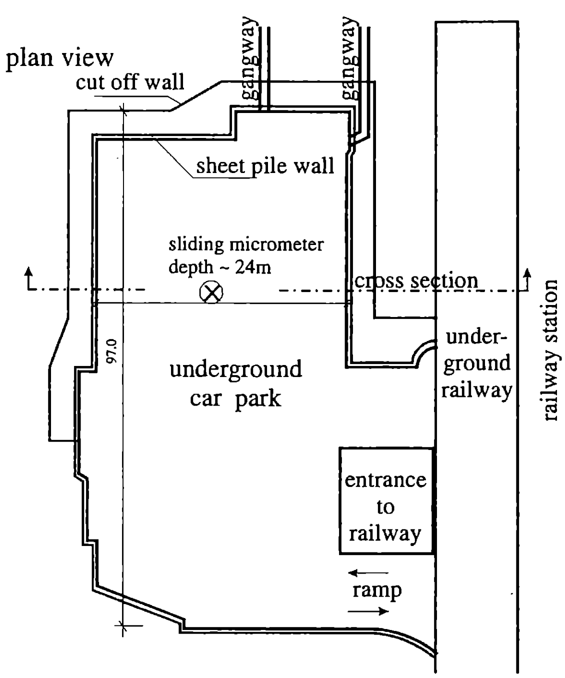

Figure 2. Plan view of car park.

The behaviour of the subsoil is characterized by the soil parameters established from a number of laboratory and in situ tests. Of particular significance for the deformation behaviour of the soft-plastic "Seeton" is the deformation modulus E_s, gained from onedimensional compression tests on undisturbed soil samples after pre-loading with the in situ stress of the relevant depth. The approximate relationship $E_s = a.\sigma'^{\,b}_n$ for the tangent modulus has been established by Breymann (1995) and has been used to arrive at appropriate input parameters for the numerical analysis. The finite element code PLAXIS V7 has been employed and the so-called Hardening Soil Model has been used to model the different soil layers. The basis of the model is the well known hyperbolic stress-strain relationship proposed by Duncan & Chang (1970) which is modified in such a way that it can be formulated in the framework of plasticity thus allowing a consistent transition from primary loading to unloading/reloading. The model incorporates the feature of shear hardening but does not consider compression hardening in the formulation employed here. A detailed description of the model can be found in Brinkgreve & Vermeer (1998). A short summary of the most important equations is given however in the following.

The hyperbolic relationship between axial strain and deviatoric stress for primary triaxial loading is written as

$$\varepsilon_1 = \frac{1}{2E_{50}} \frac{q}{1 - q/q_a} \tag{1}$$

where

$$E_{50} = E_{50}^{ref}\left(\frac{c\cot\varphi - \sigma_3'}{p^{ref}}\right)^m \tag{2}$$

with E_{50}^{ref} being a reference Young's modulus at a reference pressure p^{ref}, usually taken as 100 kN/m^2. The amount of stress dependency on the minor principal stress σ_3' is introduced by the power m in Equation 2. The ultimate deviatoric stress q_f is derived from the Mohr-Coulomb criterion.

$$q_f = (c\cot\varphi - \sigma_3')\frac{2\sin\varphi}{1-\sin\varphi} \tag{3}$$

where

$$q_a = \frac{q_f}{R_f} \tag{4}$$

and $R_f = 0.9$ may be adopted in most cases.
For unloading/reloading stress paths the following relationship is used

$$E_{ur} = E_{ur}^{ref}\left(\frac{c\cot\varphi - \sigma_3^{'}}{p^{ref}}\right)^m \tag{5}$$

where E_{ur}^{ref} is a reference Young's modulus for unloading and reloading corresponding to the reference pressure p^{ref}.
The yield function is given by Equation 6 and again triaxial loading conditions are postulated.

$$F = \bar{F} - \gamma^p \tag{6}$$

with

$$\bar{F} = \frac{1}{E_{50}}\frac{q}{1 - q/q_a} - \frac{2q}{E_{ur}} \tag{7}$$

and

$$\gamma^p \approx 2\varepsilon_1^p \tag{8}$$

Equation 8 is obviously an approximation and assumes that plastic volume changes are small compared to axial strains. An extension of the model introducing compression hardening as well has been presented recently by Brinkgreve & Vermeer (1998).

In order to model dilatant behaviour the following relationship is incorporated

$$\dot{\varepsilon}_v^p = \sin\psi_m \dot{\gamma}^p \tag{9}$$

whereas ψ_m (Eq. 10) is the mobilised dilatancy angle, φ_m is the mobilised friction angle and φ_{cv} is critical state friction angle.

$$\sin\psi_m = \frac{\sin\varphi_m - \sin\varphi_{cv}}{1 - \sin\varphi_m \sin\varphi_{cv}} \tag{10}$$

$$\sin\varphi_m = \frac{\sigma_1^{'} - \sigma_3^{'}}{\sigma_1^{'} + \sigma_3^{'} - 2c\cot\varphi} \tag{11}$$

As shown by Schanz & Vermeer (1996) these equations correspond to the stress-dilatancy theory by Rowe (1962) and therefore compaction is obtained for small stress ratios whereas dilatancy is predicted for high stress ratios. The material parameters for the soil layers are summarized in Tables 1 to 3.

Table 1. Depth of layers and permeabilities.

Soil	level	γ / γ_{sat}	k_x	k_y
	m	kN/m^3	m/d	m/d
Top layer	420.0-418.5	20.00	2.6	2.6
Gravel	418.5-415.0	18.95	2.6	2.6
Silt/fine sand	415.0-412.0	21.40	1.0	1.0
Silt	412.0-405.0	20.40	0.086	0.086
Clayey silt 1	405.0-385.0	20.20	8.6E-3	8.6E-3
Clayey silt 2	385.0-350.0	20.20	8.6E-3	8.6E-3

Table 2. Stiffness parameters.

Soil	E_{50}^{ref}	E_{ur}^{ref}	ν	m	p^{ref}
	kN/m^2	kN/m^2	-	-	kN/m^2
Top layer	52 000	208 000	0.20	0	30.0
Gravel	52 000	208 000	0.20	0	30.0
Silt/fine sand	44 000	176 000	0.20	0	60.0
Silt	20 800	83 200	0.25	0.3	100.0
Clayey silt 1	37 600	150 400	0.25	0.3	100.0
Clayey silt 2	75 200	300 000	0.25	0.3	100.0

Table 3. Strength parameters.

Soil	c	φ	ψ
	kN/m^2	°	°
Top layer	2.0	27.0	5.0
Gravel	2.0	35.0	5.0
Silt/fine sand	23.0	28.0	0.0
Silt	30.0	26.0	0.0
Clayey silt 1	30.0	26.0	0.0
Clayey silt 2	30.0	26.0	0.0

Structural elements such as walls, foundation slabs and struts have been modelled as linear elastic materials (Table 4).

Table 4. Parameters for structural elements.

Structural element	EA	EI
	kN/m	kNm^2/m
Foundation slab	2.2E7	1.173E6
Diaphragm wall	2.2E7	1.173E6
Cut off wall	8.0E5	167
Sheet pile wall	3.275E6	62870.0

It should be mentioned that stiffness parameters given in Table 2 are the ones eventually applied. They have not been determined solely from experimental investigations but also from previous experience of finite element analyses under similar conditions and measurements obtained from this site at early stages of construction. It is worth noting that in particular for the value of m, the power for stress dependency, one might expect a higher value for this type of soil. However the assumption has been varified in the laboratory for the "Seeton" at moderate depths. The initial stress state was assumed as $\sigma_z = \gamma.z$ and $\sigma_x = K_o\sigma_z$, z being the depth below surface and K_o has been assumed to 0.55 for all layers.

3 CONSTRUCTION STEPS MODELLED

In order to arrive at a reasonable initial stress state prior to construction of the car park a rough simulation of the construction of the underground railway line has been performed to account at least approximately for stress redistributions having taken place (this construction had finished more than 2 years before). Displacements were set to zero after this analysis and the construction of the car park was modelled in steps as given below. A fully coupled stress/consolidation analysis was performed whereas the individual construction step was modelled as undrained situation followed by a consolidation analysis allowing dissipation of excess pore pressure for a certain amount of time, roughly matching the actual construction time. The computational steps have been defined as follows:

1. first excavation step to level 417.5 (level of ground surface is 420.0)
2. consolidation 20 days
3. lowering of the groundwater table inside cut off wall (in practice this was achieved by means of vacuum wells)
4. consolidation 10 days
5. centre excavation to level 413.7, leaving berms in front of sheet pile walls
6. consolidation 10 days
7. construction of the foundation slab for centre part of car park
8. consolidation 30 days
9. final excavation removing berms
10. consolidation 190 days

The cut off wall (see Fig. 1), made of bentonite slurry, serves as hydraulic barrier only and is therefore introduced into the analysis as a beam element with low stiffness. The final construction steps, i.e. completion of the car park and backfilling was not significant for the displacement of the diaphragm wall for the underground railway and for this reason these steps have not been analysed.

4 RESULTS

The crucial point for the construction was the fact that the railway authorities did not allow more than 10 mm of horizontal displacement of the top of the diaphragm wall constructed for the underground railway line. A first finite element analysis carried out at the design stage revealed that this target (maximum 10 mm horizontal displacement) is difficult to achieve but it was believed it could be met taking appropriate measures during construction. In order to increase the confidence in the numerical model it was decided that the analysis would be refined during construction utilizing measurements from the site as soon as they became available. This approach was thought to be successful and would provide more accurate predictions of displacements for the final and most critical construction stages. Thus this project serves as an excellent example of applying the observational method by not only including in situ measurements but also advanced finite element analysis.

In all of the following diagrams the full line is the finite element result and the arrows depict measurements at a certain time, which are not always exactly identical with the construction step analysed and therefore measured values have not been put on the line but approximately where appropriate. Figure 3 shows results from the sliding micrometer inside the excavation together with calculated values for the first construction steps. The measured heave of the base of the excavation due to the first excavation step and the groundwater drawdown provided valuable information and as can be seen the "back analysis" was quite successful. Figure 4 shows the same for a point in the ground 11.5 m below surface, i.e. below the final level of excavation. Please note that all values for construction stages 4 to 10 are now predicted and not backcalculated. Figure 5 depicts the crucial point, the head of the diaphragm wall of the underground railway. The two calculations shown in the diagram differ only in the way the last excavation step (removal of berms) is modelled. The analysis resulting in higher displacement assumes that the berm is removed over the whole length (this is the standard assumption in plane strain analysis) whereas the lower value is obtained by taking into account the fact that the excavation is done in sections. This has been achieved in the analysis by applying only a percentage of the nodal forces due to excavation in a first calculation step and account for the foundation slab as a partial support when applying the remaining nodal forces. Again it is to mention that from excavation step 3 onwards all calculated values are predictions utilizing information obtained up to stage 3. The agreement for the final excavation stages is seen to be almost perfect and is certainly exceptional for a geotechnical problem. Figure 5 also indicates that consolidation effects do not play a significant role for the displacement of the diaphragm wall of the existing underground railway, consolidation having actually a positive influence. The effect however is slightly underestimated by the analysis (compare measurements 17.07.98 and 03.08.98 in Figure 5 against finite element predictions), although the trend is predicted correctly.

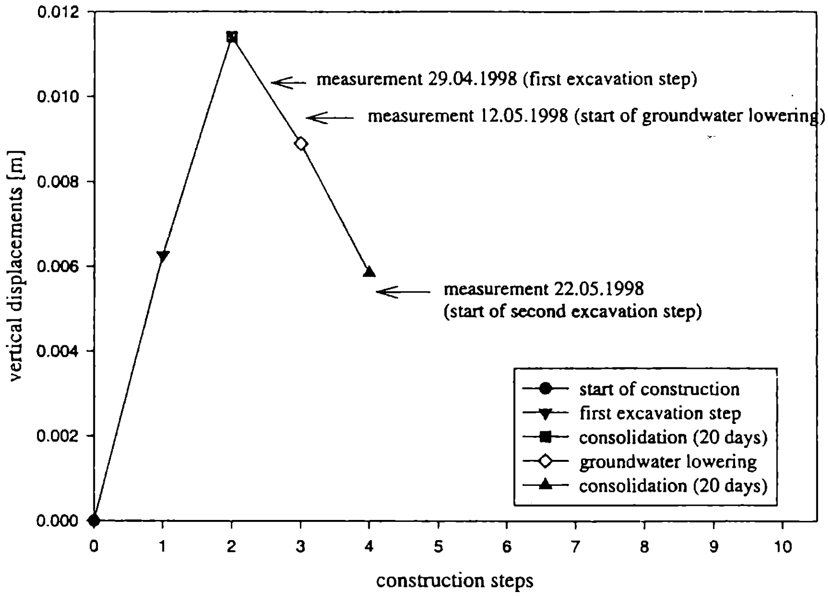

Figure 3. Vertical displacements of base of excavation: comparison sliding micrometer – finite element analysis.

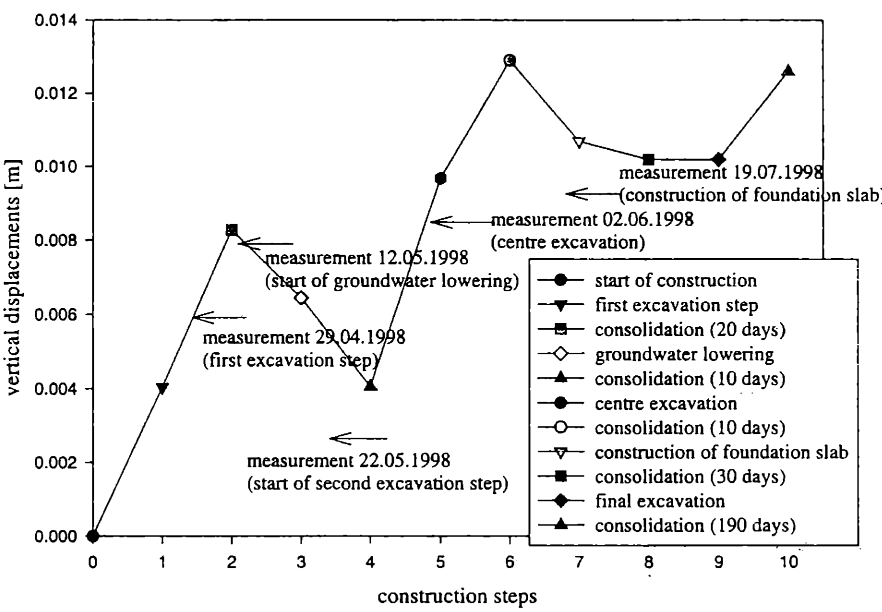

Figure 4. Vertical displacements of point below final level of excavation: comparison sliding micrometer – finite element analysis.

Although the numerical model presented here has been found to be extremely useful as far as predicting certain critical displacements are concerned other measurements have not been matched so well. In particular calculated pore water pressures did not compare well with measurements. However some problems have been reported on site with these mesurements so these may be not very realiable. It is anticipated that further insight will be provided by monitoring upcoming excavations under similar conditions where the same procedure of interplay between monitoring and finite element analyses as described here will be followed.

5 CONCLUSION

Application of a shear hardening plasticity model to a practical deep excavation problem has shown that the model is suitable for analysing these types of problems from a practical point of view. The case study presented here highlights the important role of

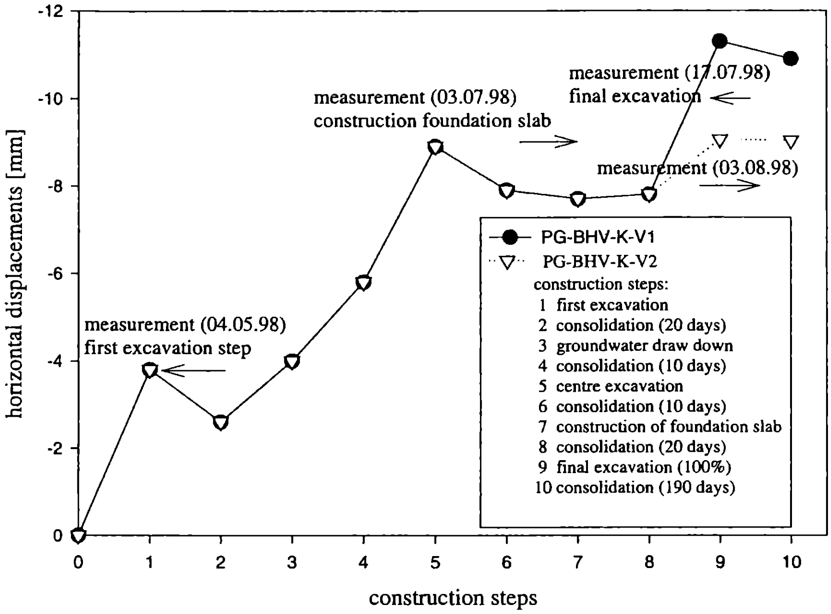

Figure 5. Horizontal displacement of top of wall of underground railway: comparison geodetic survey – finite element analysis.

numerical modelling in the framework of the observational method. Following an analysis prior to construction the numerical model was refined at the early stages of excavation utilizing in situ measurements which gave valuable input for improving the estimation for the actual in situ stiffness of the ground. This approach led to excellent predictions of displacements of existing structures for critical construction stages which have been a key point also from a contractual point of view. The numerical model provided the confidence during construction that one could go ahead as planned indicating at the same time that one would come very close to the limits of displacements

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