FE-analysis of five deep excavations in lacustrine clay and comparison with in-situ measurements

H.F. Schweiger
Graz University of Technology, Graz, Austria

H. Breymann
Radstadt, Austria

ABSTRACT: In this paper finite element analyses of deep excavations in soft ground in the city of Salzburg, Austria are presented. The regional subsoil situation in Salzburg can be described as fully saturated soft soil (referred to as "Seeton") overlain by a quaternary gravel fill. The excavations addressed in this paper differ in the support measures chosen for the retaining diaphragm wall (ground anchors, struts, berms) and in the depth of the quaternary gravel fill, i.e. the depth the final excavation level reaches into the soft soil layer. From the comparison of computed results with in situ measurements it follows that the employed elasto-plastic constitutive model is capable of representing the behaviour of the different soil layers with sufficient accuracy from a practical point of view. Finally it is shown that calculated and measured wall displacements compare well with values reported in the literature.

1 INTRODUCTION

Soft subsoil deposits in Austria are mainly fresh water deposits, sedimented in the post-glacial lakes after the boulder periods. These deposits are known as lacustrine clays on the foothills of the Alps. One example for a widespread lacustrine clay deposit is the basin of Salzburg, where the city of Salzburg is situated on subsoil sediments, which partly show a thickness up to 70 m, called 'Salzburger Seeton'. The poorly graded Seeton can be classified as clayey silt and shows unfavourable soil properties with respect to the deformation behaviour of deep excavations.

To improve the design of constructions on soft soil, finite element calculations are a useful tool for the optimisation of the design and to obtain a realistic prediction of the deformations expected. Common calculation methods for retaining structures based on failure criterions like Mohr-Coulomb cannot take into account the complex material behaviour of soft soils adequately. Therefore a more advanced constitutive model, namely the Hardening Soil model as implemented in the finite element code Plaxis (Brinkgreve 2002), has been used in analysing the five different deep excavations discussed in this paper.

The projects addressed differ in the support measures chosen for the retaining diaphragm wall (ground anchors, struts, berms) and in the depth of the quaternary gravel fill, i.e. the depth the final excavation level reaches into the soft soil layer.

The input parameters for the constitutive model have been determined not solely from site investigations but also from previous experience of finite element analyses under similar conditions. It is pointed out that all analyses for the different projects have been made with the same set of parameters for the individual soil layers.

Due to space limitations a detailed description of all projects will not be given. Instead, one of the examples will be described, and basically the same modelling strategy has been applied to all other projects. However, differences in the support structure and soil layers will be highlighted, in particular the thickness of the quaternary gravel fill in relation to the final excavation depth. Finally a summary will be provided where characteristic displacements for all projects are compared to measured values and to published data from the literature.

2 DESCRIPTION OF PROJECTS

2.1 *Schematic representation of geometric outline of all projects considered*

In this section the differences in the support system and the soil profile for all projects addressed in this paper are summarized (Figures 1 to 5). The most important issue is the depth of the gravel layer below surface as highlighted by the bold line, which varies between

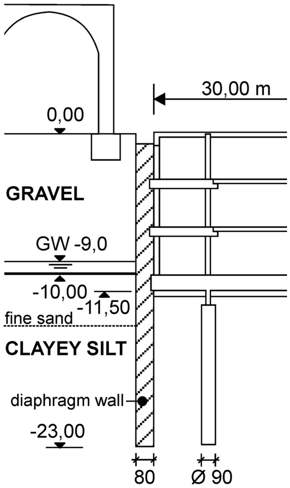

Figure 1. Layout of project "Toskanatrakt".

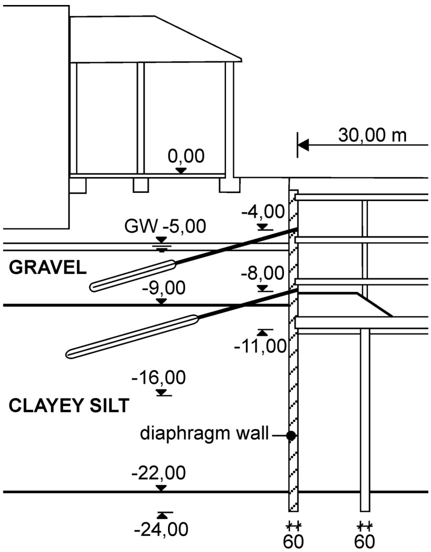

Figure 2. Layout of project "Hypobank".

10.0 m ("Toskanatrakt", Figure 1) and 1.0 m ("AMV", Figure 3).

2.2 *Project Hypobank*

In this section the numerical analysis of one of the examples, namely the project "Hypobank" is described in more detail. Figure 6 shows the model that has been derived from the geometry depicted in Figure 2 including the soil layers and the support system: a

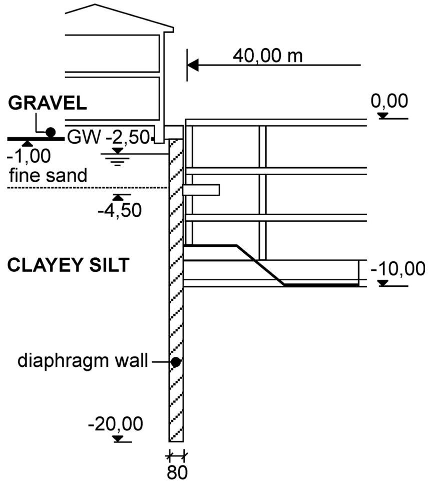

Figure 3. Layout of project "AMV".

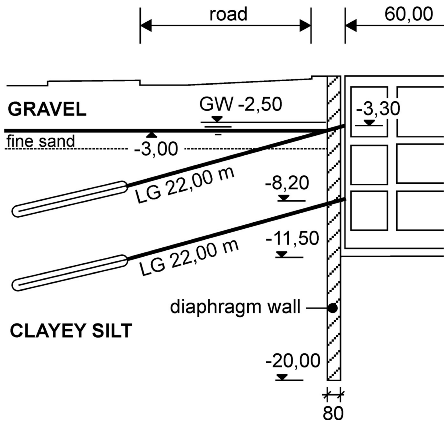

Figure 4. Layout of project "Kiesel".

diaphragm wall with a thickness of 0.6 m and a depth of 24.0 m is supported by two rows of anchors and at the final excavation level, which is at −11.00 m below surface, with a berm. The width of the excavation is 30 m but only half of the system has been analysed. The finite element code Plaxis V8.1 has been used for all analyses presented in this paper (Brinkgreve 2002). Typically about 1 300 15-noded triangular elements have been used for discretisation (Figure 6). As mentioned previously, the Hardening Soil model, an elasto-plastic constitutive model including deviatoric and volumetric hardening is used to describe the material behaviour of all soil layers. The main feature of this model is a stress dependent stiffness and a distinction in stiffness between primary loading and

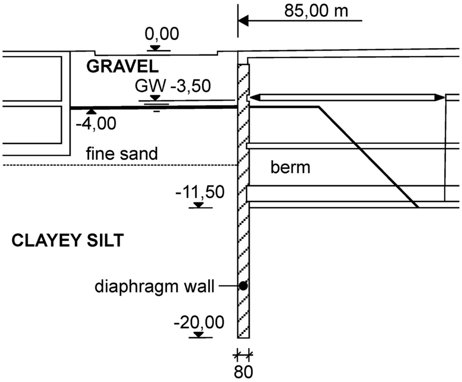

Figure 5. Layout of project "Penta".

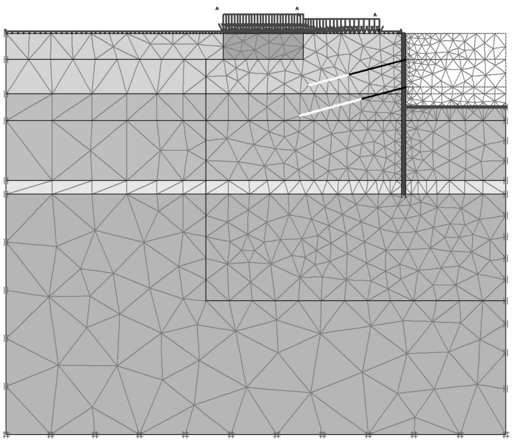

Figure 6. Plaxis-model for project "Hypobank".

unloading/reloading. The failure strength is described by a Mohr-Coulomb failure criterion. Structural elements have been assumed to behave as linear elastic-perfectly plastic materials. This is however not relevant when discussing the deformation behaviour at working load conditions as in this paper. The soil parameters used in the analysis for the silty gravel layer and the clayey silt are summarized in Tables 1 and 2. As mentioned previously, parameter determination is not only based on site investigations and laboratory experiments but also from experience of back analyses of other deep excavations in Salzburg not discussed here. The clayey silt layers have been assumed to behave as undrained material. This is justified given the low permeability of these layers in comparison to the time required for completing the excavation.

The following construction sequence has been modelled in the analysis:

1. initial stresses (a K_0-value of 0.5 has been assumed for all layers)

Table 1. Stiffness parameters for soil layers.

	E_{50} [MPa]	E_{oed} [MPa]	E_{ur} [MPa]	[m]	p_{ref} [kPa]	ν_{ur}
Silty gravel	52	52	208	0.0	100	0.2
Silty, fine sand	44	44	176	0.0	100	0.2
Clayey silt 1	37.6	37.6	150.4	0.30	100	0.2
Clayey silt 2	75.2	75.2	300.0	0.75	200	0.2

Table 2. Strength parameters for soil layers.

	c (kPa)	φ (°)	ψ (°)
Silty gravel	2	35	5
Silty, fine sand	5	28	0
Clayey silt 1	30	26	0
Clayey silt 2	30	26	0

2. loads representing buildings applied (displacements set to zero)
3. diaphragm wall wished-in-place
4. excavation to −3.8 m
5. activation of first anchor row and prestressing to 156 kN/m
6. lowering of groundwater table inside excavation to −8.1 m
7. excavation to −8.1 m
8. activation of second anchor row and prestressing to 156 kN/m
9. lowering of groundwater table inside excavation to −11.0 m
10. excavation of central part (berm left for support) and construction of concrete slab
11. removal of berm
12. construction of remaining part of concrete slab.

In Table 1 E_{50} denotes a reference secant stiffness for triaxial compression stress paths, E_{oed} a reference stiffness from one-dimensional compression tests and E_{ur} is the unloading/reloading stiffness, again at the given reference stress p_{ref}. m controls the stress dependency of the stiffness and ν_{ur} is the Poisson's ratio for unloading/reloading. The fine sand layer is not present in the "Hypobank" project but appears in varying thickness in the other examples not describe in detail here. Table 2 lists the strength parameters.

2.3 *Results for Project "Hypobank"*

Figure 7 shows the deformed mesh for the final excavation stage. A maximum horizontal displacement of 16 mm is obtained and the horizontal movement of the top of the wall is 11 mm, the measured one being 13 mm. Figure 8 confirms the good agreement between measured and calculated displacements of the top of the wall for all construction stages. Only in construction step 7 a mismatch is observed but this can be attributed to slight deviations in the construction activities as compared to the original design.

All other projects discussed in this paper have been analysed in a similar and the most relevant results from these analyses are summarized in the following chapter.

3 COMPARISON OF ALL PROJECTS

In this section a comparison is made between measured and calculated deformations for the five projects included in this study in the same way as presented by Breymann et al. (2003). Thus the horizontal displacement (u) of the top of the wall as defined in Figure 9 (Table 3), the settlement (v) in a distance of 7 m behind the wall (Table 4) and the extension of the settlement trough (Table 5) as illustrated in Figure 10 have been chosen for comparison. In addition the ratio of maximum calculated horizontal wall displacement to excavation depth (t) is listed in Table 6. In all tables some characteristic geometric data are also provided,

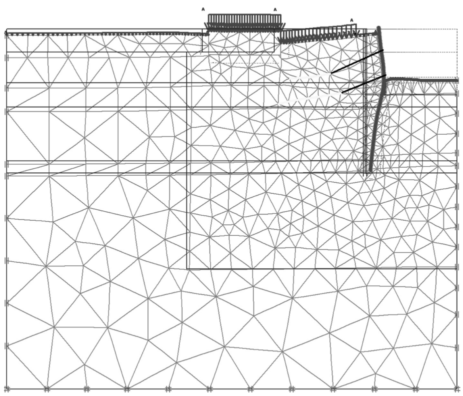

Figure 7. Deformed mesh for project "Hypobank".

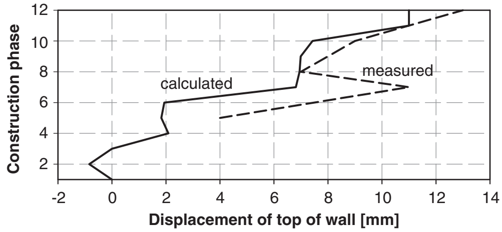

Figure 8. Comparison of calculated and measured horizontal displacements of top of wall.

namely the ratio of the gravel layer to the clayey silt layer (including the fine sand layer if present) up to the bottom of the excavation (GR/SI). Furthermore the excavated area (L*W) and the excavated soil volume is included in all tables.

Tables 1 to 5 show that a reasonable agreement between analysis and measurements could be achieved, in particular if one keeps in mind that the model can never simulate the details of the actual construction sequence on site. As far as horizontal displacements are concerned the disagreement in the project "Toskanatrakt" can be attributed to the fact that this

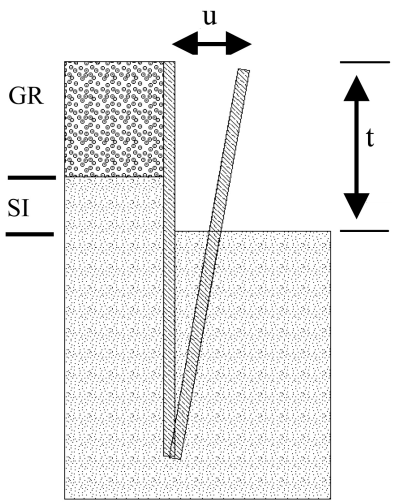

Figure 9. Definition of horizontal displacements and geometric data used for comparison.

Table 3. Comparison of horizontal displacement of top of wall.

	GR/SI [m/m]	L*W [m^2]	Vol [m^3]	u_{meas} [mm]	u_{cal} [mm]
Toskanatrakt	6.7	850	9800	5	0
Hypobank	4.5	1000	11000	13	11
AMV	0.1	1800	18000	17	19
Kiesel	0.4	2200	25000	20	19
Penta	0.5	6500	75000	100	47

Table 4. Comparison of settlement 7 m behind the wall.

	GR/SI [m/m]	L*W [m^2]	Vol [m^3]	v_{meas} [mm]	v_{cal} [mm]
Toskanatrakt	6.7	850	9800	8	1
Hypobank	4.5	1000	11000	11	9
AMV	0.1	1800	18000	26	17
Kiesel	0.4	2200	25000	28	13
Penta	0.5	6500	75000	30	27

Table 5. Comparison of extension of settlement trough.

	GR/SI [m/m]	L*W [m^2]	Vol [m^3]	R_{meas} [m]	R_{cal} [m]
Toskanatrakt	6.7	850	9800	25	40
Hypobank	4.5	1000	11000	25	30
AMV	0.1	1800	18000	30	35
Kiesel	0.4	2200	25000	35	80
Penta	0.5	6500	75000	45	100

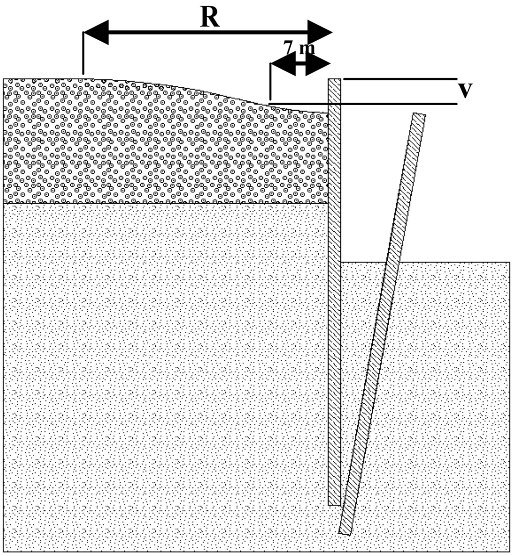

Figure 10. Definition of settlement and settlement trough as used for comparison.

example is governed by the silty gravel layer where less parameter validation has been possible. The difference for "Penta" can also be explained because the given 100 mm of horizontal displacement is the extreme value of all measurements over the entire length of the excavation. The average value was more like calculated.

When comparing settlements it is obvious that calculated ones are generally smaller than measured ones. The main reason for this is that the diaphragm wall was modelled as "wished-in-place" and thus settlements due to wall construction are not included in the analysis. They are however in the range of 5 to 10 mm for the given conditions.

With respect to the extension of the settlement trough it is well known that constitutive models which not do include effects such as small strain stiffness behaviour tend to overestimate the extension of the settlement trough. In addition there is also some room for interpretation from measurements.

In Figure 11 the results from this study are included in the diagram presented by Long (2001). It follows that the projects in Salzburg, all of them having similar excavation depths, cover almost the full range of case

Table 6. Comparison of ratio of calculated maximum horizontal displacement/excavation depth.

	GR/SI [m/m]	L*W [m^2]	Vol [m^3]	u_{max} [%]
Toskanatrakt	6.7	850	9800	0.06
Hypobank	4.5	1000	11000	0.14
AMV	0.1	1800	18000	0.25
Kiesel	0.4	2200	25000	0.22
Penta	0.5	6500	75000	0.41

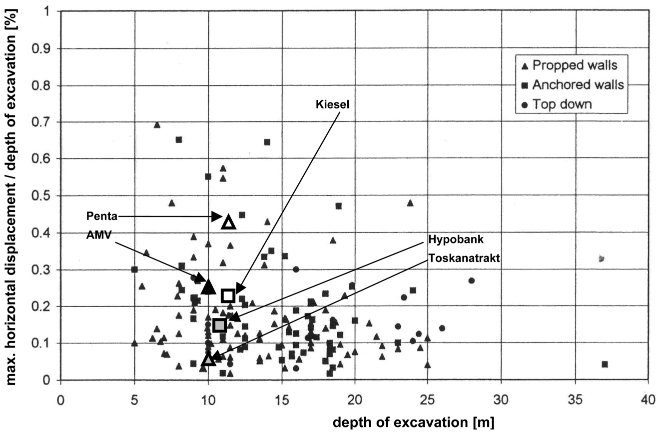

Figure 11. Comparison of ratio of maximum horizontal displacement to depth of excavation with data from literature (modified from Long 2001).

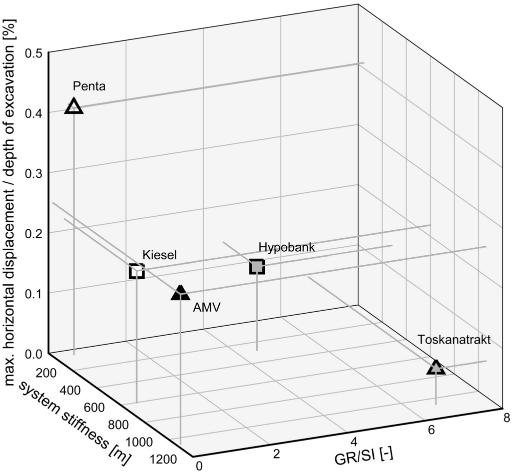

Figure 12. Ratio of maximum horizontal displacement to depth of excavation vs system stiffness and depth of gravel layer.

histories reported in the literature. In Figure 12 the reason for this wide range is illustrated. In this three-dimensional graph the ratio of calculated maximum horizontal displacement to excavation depth (u_{max}/t) is plotted against GR/SI (ratio of thickness of stiff to soft soil layer) and the system stiffness $EI/\gamma_w s^4$ (Clough et al. 1989). EI is the bending stiffness of the wall, s is the vertical distance of the support system (struts or anchors) and γ_w is the unit weight of water. One obvious, expected conclusion from Figure 12 is that horizontal displacements decrease for thicker layers of gravel, where already a moderate system stiffness leads to values for u_{max}/t of less than 0.15%. On the other hand, even a relatively high system stiffness cannot prevent u_{max}/t exceeding 0.2% if the gravel layer is thin. For the project "Penta" the high value of 0.4% for u_{max}/t can be attributed to the very low system stiffness (a berm provided the major support system) in combination with only an average value for GR/SI.

4 CONCLUSION

Results from finite element analyses of five different deep excavations in soft soil in the city of Salzburg have been compared with the measured performance in situ. An elasto-plastic constitutive model, namely the so called Hardening Soil model, has been employed to describe the behaviour of all soil layers and a consistent set of parameters has been used for all projects discussed in this paper. The agreement between analyses and measurements was on overall very good and one can conclude that these types of constitutive models are reasonably well suited for solving this kind of problems in practice.

The influence of various geometric parameters has been shown and a comparison with the deformation behaviour of published case histories has been provided.

REFERENCES

Brinkgreve, R.B.J. 2002. PLAXIS, Finite element code for soil and rock analyses, users manual. Rotterdam: Balkema.

Breymann, H., Moser, M., Premstaller, M. (2003). Die Gründung des Kongresshauses Salzburg, Felsbau 5, 10–17.

Long, M. (2001). Database for retaining wall and ground movements due to deep excavation, Journal of Geotechnical and Geoenvironmental Engineering, 203–224.

Clough, G.W., Smith, E.M., Sweeney, B.P. 1989. Movement control of excavation support systems by iterative design. Proc. Foundation Engineering-Current Principles and Practices, Vol. 2, ASCE, New York, 869–884.