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Excavation in Deep Soft Lacustrine Soil Deposit

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ABSTRACT

The paper presents a case study of a supported excavation in a deep soft lacustrine soil deposit located in the city of Constance, southern Germany. Besides demonstrating the construction aspect, the field measurement and monitoring results of the excavation project, the paper illustrates a back analysis of the project using the finite element method. It was revealed that in addition to the understanding of the soil behaviour, the soil-structure-interaction behaviour, and their simulation using an advanced constitutive soil models, the understanding and simulation of each detail of the construction process is also equally important. The study underlines the important of performing a 3D- analysis for reasonably prediction of the soil movement in and around excavations, or at least the inclusion of the 3D effect in the 2D analysis of the excavation.

INTRODUCTION

The area of the lake Constance, locally known as Bodensee, is known to consist a thick layer of post glacial soft lacustrine deposit. Its thickness is believed to exceed 25 m. Excavation on such thick soft soils in urban areas is usually prone to movements of soils which damages the nearby structures. Now a days the possible movement of soil in excavations in urban areas is predicted by means of the Finite Element Method. However, experiences show that predicted deflection of wall and settlement behind the wall does not usually match with the measured values. The aim of this paper is to perform a back analysis of a practical excavation project in soft soil with the help of the finite element method, so that to identify the possible cause of the deviation of the calculated and measured results and to calibrate the soil parameters accordingly.

The soil exploration, monitoring and construction information of the project are documented in *Kempfert + Partner (1994-1998)* and are partly reported by *Berner (1997)* and *Gebreselassie (2003)*. Hence, more emphasis will be given to the numerical analysis of the project after a brief introduction to the site condition, the instrumentation and the recorded data.

GENERAL DESCRPTION OF THE EXCAVATION SITE

The Site

A multi-storey building for the purpose of apartments and shopping centre which include 2 floor underground park was built in 1997/98 in the old part of the Constance city, southern Germany. The site was fairly level (397.50 MSL), measured about 40 x 60 m and surrounded by old and relatively new 3 to 5-storey buildings (Fig.1).

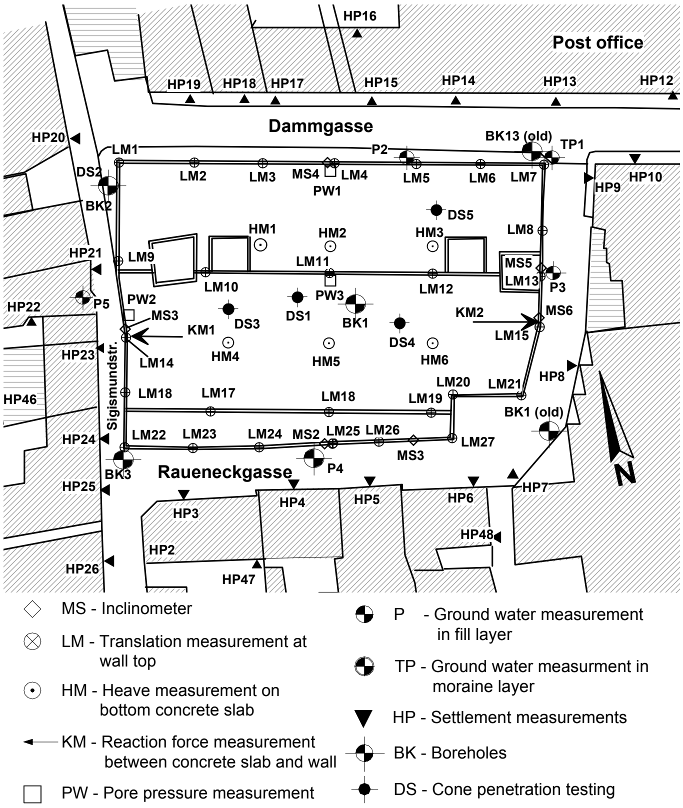

Fig. 1. The site plan, borehole location and instrumentation.

Soil Condition

Altogether 3 boreholes were drilled, 5 test pits were excavated, 5 cone penetration test were conducted on the site for the purpose of exploring the soil and investigating the ground water condition. Two additional bore logs were also available from old soil exploration on the site. Their locations are shown in Fig. 1. The site investigation revealed 2.6 to 5.3 m fill material comprising gravel, sand and rubble from old buildings, overlaying soft lacustrine deposit to a depth of 12 to 30 m. Beneath the lacustrine soil, a moraine comprising sandy and silty gravel was encountered to a depth of 20 m up to a depth greater than 30 m. Though it was not bored at this site, a boulder clay is believed to be found beneath the moraine. Figure 2 is a typical bore log and cone penetration result along with the distribution of the water content with the depth.

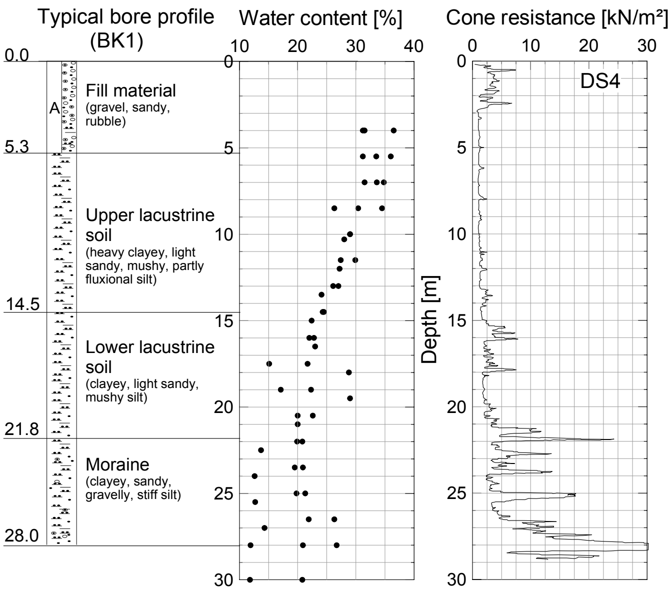

Fig. 2. Typical soil profile and properties.

Support System

The retaining structure used was a sheet pile wall of the type Hoech 134. There were a double row of sheet pile walls on the northern and southern part of the site. Fig. 3 is a section through the site. The walls were temporarily supported by wooden strut ϕ = 26 cm (south wall S1 & S2), propped wooden plumbs ϕ = 36 cm (north wall S1), propped I-Steel beam IPB 360 (west and east walls S3) and bottom concrete slab d = 25 and 30 cm.

Construction Stages

The excavation was proceeded in slices in a daily output basis according to the construction phases shown in Fig. 4. After each slice was excavated, a fast hardening concrete had been placed providing support to the walls (see also Fig.5), before the next slice excavation had begun.

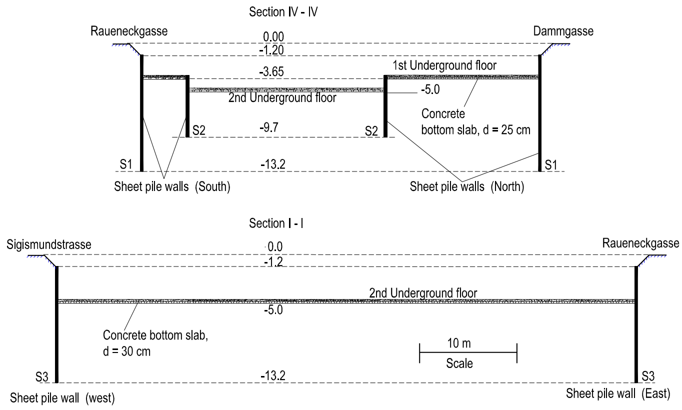

Fig.3. Section through site.

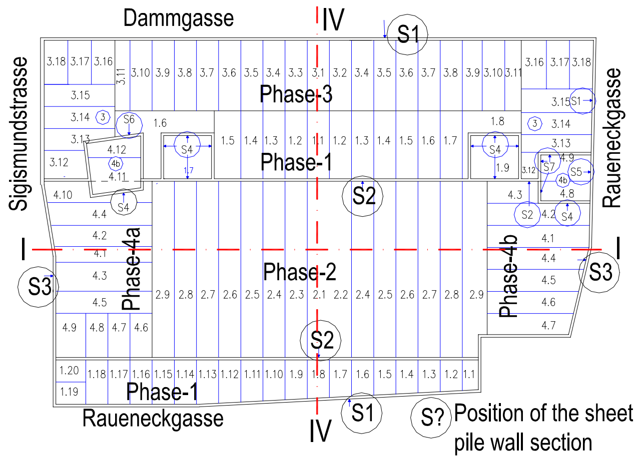

Fig. 4. The construction phases in plan.

Fig. 5. Excavation and placement of the bottom slab in slices.

Instrumentation

Instrumentation was used to record the wall movement, the settlement of the nearby existing buildings, the heave of the basement slab, the pore water pressure in the lacustrine soil layer and the reaction forces between the wall and the basement slab and the propped support. Wall movement was measured in two ways. The top of the wall was monitored using Geodometer at 27 points. The wall deflection was measured using 6 inclinometers which provides a result to an accuracy of ±1 mm. The settlement at 50 locations near the existing buildings were monitored using surface surveying starting prior to the beginning of the construction activities at the site. The potential heave of the basement slab was controlled using surface surveying at 6 points. In addition to the ground water observation, pore water pressure was measured at the inner side of the sheet pile wall at 3 points.

The result of the monitoring will be presented later together with the result of the finite element analysis.

BACK ANALYSIS USING THE FINITE ELEMENT

The back analysis was carried out using the two-dimensional FE-program PLAXIS 8.1 professional version. This program was specifically developed for geotechnical purposes and it provides material models from a simple elastic to advanced elasto-plastic cap models. The hardening soil model (HSM) was used to simulate the behaviour of the soils of all the layers. This HSM is a versatile model capable of describing the behaviour of all type of soils except a time dependant behaviour. It is based on isotropic hardening and its basic characteristics are: stress dependant stiffness according to the power low, plastic straining both due to primary deviatoric loading (shear hardening) and primary compression (compression hardening, cap yield), elastic un/reloading, dilatancy effect, failure according to the Mohr-Coulomb. The contact behaviour was simulated with the Mohr-Coulomb model (MCM). This model is a simple elastic-perfect plastic model. Detail description of the HSM and the MCM can be found in PLAXIS handbook by *Brinkgreve/vermee (1998)* or *Brinkgreve (2002)* and *Schanz, et al. (1999)*. The structural elements were assumed to behave elastically. A plain strain analysis was adopted using 15 node triangular elements. These elements provides a fourth order interpolation for displacements and it involves twelve numerical integration stress points (Gauss points).

Model Geometry

The first step in any FE-analysis of geotechnical problem is to convert the data from the geotechnical reports to a simplified soil profile, idealise the structural elements and to determine the extent of the model geometry. Figure 6 shows the section through the site (south - north section) showing the idealised soil profile, arrangement of the structural elements and sequences of the excavation. A similar west - east section is also shown in Fig.7. These sections are identified as section IV-IV and I-I respectively in Fig.4.

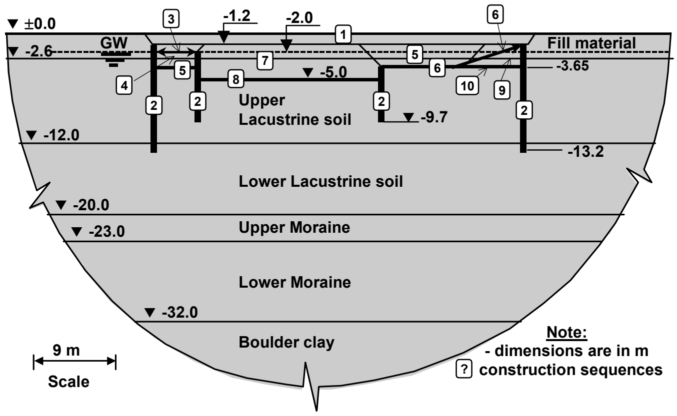

Fig. 6. Section through the site (south - north), soil profile and construction sequence.

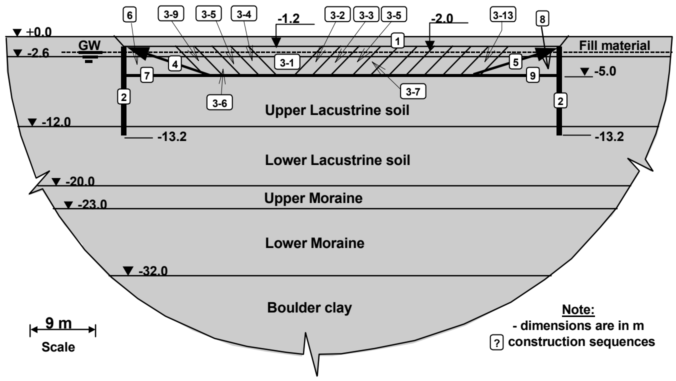

Fig. 7: Section through the site (west - east), soil profile and construction sequence.

The corresponding finite element model and its mesh are shown in Fig. 8 and 9. These figures are zoomed in order to show the main part of the model that consist the structural elements, loads, soil layers and the excavation section. The finite element mesh at south-north section was extended to a depth of 82 m where a fixed boundary was imposed and a zero horizontal displacement was imposed at a distance of 82 m from the edge of the wall. The size of the model as a whole was 204 m wide and 82 m high (Fig. 8). Similarly, The finite element mesh at west-east section was extended to a depth of 125 m and to a distance of 120 m from the edge of the wall (all in all 300 × 125 m) (Fig. 9). A surcharge load of 33 kN/m² and 23 kN/m² at the southern and northern side (Fig. 8) respectively were applied at the level of the underground floor to represent the building loads. Similarly, a load of 20 kN/m² and 32 kN/m² at the western and eastern side of the excavation (Fig. 9) respectively were applied. In both cases a traffic load of 10 kN/m² was applied between the wall and the existing buildings.

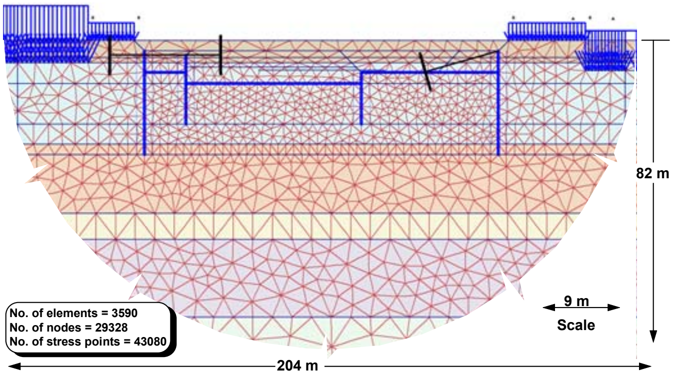

Fig. 8. Main part of the finite element mesh: South - North Section.

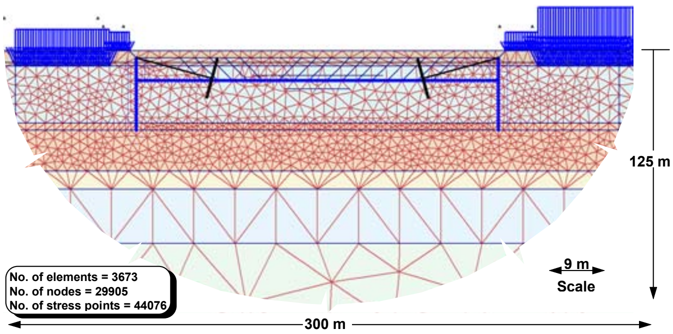

Fig. 9. Main part of the finite element mesh: West - East Section.

Material Properties

The soil parameters required for the FE - computation with the hardening soil model under drained condition are given in Table 1. PLAXIS provides the option of performing an undrained analysis using the effective strength and stiffness parameters. Hence, the parameters in Table 1 can also be used for the consolidation analysis. The program accepts the effective stiffness parameter and calculates the balk modulus according to the Hook's law of elasticity. The excess pore pressure are calculated from the volumetric strain rate, the bulk modulus of water and the porosity of the soil medium. The layers of the lacustrine soils and the fill material only were assumed undrained in the undrained analysis, where as the other layers remain drained.

The soil parameters of the lacustrine soil layers were directly taken from intensive laboratory tests on undisturbed soil samples from the site and are calibrated using a finite element simulation of the laboratory tests (*Gebreselassie, 2003*). The soil parameters of the remaining layers were derived from penetration and sounding field tests documented in the geotechnical reports after converting them to suit for HSM. A separate material set was defined for the interface elements. The shear parameters of the contact elements were adopted from the corresponding layers of soils after reducing the values by a factor of 0.33, whereas the stiffness of the soil layers were adopted as it is (*Gebreselassie, 2003*). The structural properties of the structural elements are given in Table 2.

Table 1. Soil parameters for the HSM

a) Unit weight and permeability

Soil layer	depth [m]	γ_{sat} [kN/m³]	γ_{unsat} [kN/m³]	$k_x = k_y$ [m/d]
Fill material	00.0-02.6	17.0	20.0	8.64E-5
Upper lacustrine soil	02.6-12.0	19.5	19.5	8.64E-5
Lower lacustrine soil	12.0-20.0	19.5	19.5	8.64E-5
Upper moraine	20.0-23.0	20.0	20.0	8.60E-3
Lower moraine	23.0-32.0	21.0	21.0	1.0
Boulder clay	32.0-82.0	22.0	22.0	8.60E-6

b) Stiffness parameters

Soil layer	E_{50}^{ref} [MN/m²]	E_{oed}^{ref} [MN/m²]	E_{ur}^{ref} [MN/m²]	p^{ref} [MN/m²]	ν_{ur} [-]	m [-]
Fill material	6.0	6.0	30.0	0.1	0.20	0.70
Upper lacustrine soil	3.6	2.7	19.2	0.1	0.20	0.73
Lower lacustrine soil	5.4	4.0	29.0	0.1	0.20	0.73
Upper moraine	24.0	24.0	120.0	0.1	0.20	0.50
Lower moraine	28.0	28.0	140.0	0.1	0.20	0.50
Boulder clay	40.	40.0	200.0	0.1	0.20	0.80

c) Shear strength parameters

Soil layer	c' [kN/m²]	φ' [°]	ψ' [°]	R_f [-]
Fill material	10.0	25.0	0.0	0.90
Upper lacustrine soil	13.2	25.3	0.0	0.82
Lower lacustrine soil	14.1	27.0	0.0	0.82
Upper moraine	5.0	32.5	2.5	0.90
Lower moraine	5.0	36.5	6.5	0.90
Boulder clay	10.0	30.0	0.0	0.90

Table 2. Material properties of the structural elements.

Structural element	EA [kN/m]	EI [kNm²/m]	w [kN/m/m]	ν [-]
Sheet pile wall (Hoech 134)	3.591E6	53550.0	1.3	0.30
Concrete Bottom slab:				
d = 25 cm	7.500E6	39063.0	6.3	0.20
d = 30 cm	9.000E6	67500.0	7.5	0.20
Strut (Wood ϕ = 26 cm)	9.557E4	$L_{spacing}$ = 2.5 m		
Propped support:				
Wood ϕ = 32 cm	1.448E5	$L_{spacing}$ = 2.5 m		
I-Steel beam (IPB 360)	9.503E5	$L_{spacing}$ = 4.0 m		

Preliminary Analysis and Results

A preliminary analysis of the excavation was carried out using the finite element models shown in Fig. 8 & 9 and the material properties in Table 1 and 2. In all the computation cases present in this section, a hydrostatic ground water was assumed.

Both drained and undrained (consolidation) analysis were performed separately.

The new professional version of PLAXIS v8.1 provides an option of performing a consolidation and simultaneous loading in the sense of changing the load combination, stress state, weight, strength or stiffness of elements activated by changing the load and geometry configuration or pore pressure distribution by means of stage construction. This is very important in regard to excavation, because the excavation usually takes some days or weeks or in extreme case also some months, and the pore pressure has the possibility to dissipate already during the excavation. This option of consolidation and simultaneous excavation was utilised in the back analysis of the project. The undrained behaviour was ignored in the first 4 calculation phases.

<u>South-North Section (Section IV-IV)</u>. The main construction phases are shown in Fig. 4 in plan and Fig. 6 in section. The construction stages followed in the FE-computation were the same as those describe above, but they are simplified and optimised as shown in Table 3 (consolidation analysis). The construction stages in drained analysis (13 phases) were the same as in Table 3 in the absence of the consolidation time.

Table 3. Construction stages (Section IV-IV).

Phase 00:	generate the initial stresses
Phase 01:	activate the surcharge and traffic loads
Phase 02:	1st excavation to a depth of -1.2 m (1)
Phase 03:	wall installation (2)
Phase 04:	2nd excavation (3) [4 days]
Phase 05:	strut installation (3) [4 days]
Phase 06:	3rd excavation (4) [7 days]
Phase 07:	installation of bottom slab (5) [8 days]
Phase 08:	4th excavation (5) and strut removal (3) [4 days]
Phase 09:	bottom slab installation (6) and strut (6) [5 days]
Phase 10:	consolidation time [6.5 days]
Phase 11:	5th excavation (7) [11 days]
Phase 12:	bottom slab installation (8) [12 days]
Phase 13:	consolidation time [6 days]
Phase 14:	6th excavation (9) [17 days]
Phase 15:	bottom slab installation (10) [17 days]
Phase 16:	removal of the strut (6) [12 days]

N.B.: numbers in () are construction sequences (Fig. 6) and [] are consolidation and execution time

Figure 10 shows the calculated displacement of the wall at the end of the excavation stage. Both drained and consolidation analyses results are presented in the diagram. It can be seen from Fig 10 that the FEM - computation shows an excessive displacement than it was measured at the field. A maximum displacement of 112.3 mm (drained) and 79.3 mm (consolidation) at the top of the southern outer wall was computed compared to the measured top displacement of 23 mm. Similarly, 30.9 mm (drained) and 28.4 mm (consolidation) was computed at the top of the southern outer wall compared to a measured value of 16 mm. As would expected the consolidation analysis had led to a lesser displacement than the drained analysis.

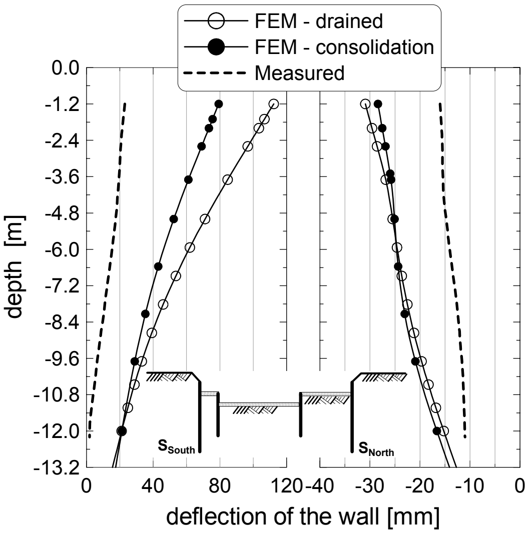

Fig. 10. Calculated and measured displacement of the wall (South - North Section).

<u>West - east section (Section I-I)</u>. The excavation of the west-east section consists of several slices of trenches that were executed on the daily output basis (Fig. 4 & 7). After each slice of the trench had been cut, it followed immediately the placement of fast hardening concrete bottom slab securing a bottom support to the walls in the south-north direction. The excavation was started at strip No. 2.5 (Fig. 4) or sequence No. 3-1 (Fig. 7) and proceeded to the left and right alternately towards the berms supporting the walls. An attempt had been done to simulate these excavation and construction processes in the FEM-computations. The simulation of excavation of each trench and placing of the slab immediately may have no much influence on the results of a drained analysis. The calculation phases followed during the consolidation analysis are shown in Table 4.

The computed and measured displacement of the walls at the end of the excavation stage for the west-east section are shown in Fig. 11. Surprisingly, both the drained and consolidation analyses had led to almost the same wall displacement, but they are far from the measured value. Compared to the measured value of the displacement (18 mm) at the top of the west wall, a maximum displacement of 76.6 mm (drained) and 78.1 mm (consolidation) were obtained from the FEM-computations. Similarly, displacements 76.4 mm (drained) and 71.2 mm (consolidation) were computed at the top of the eastern wall compared to a measured value of 16 mm. It can also be seen from Fig. 11 that the amount and shape of the displacement of the wall are almost identical, as if the excavation was symmetrical in respect to the loading and geometry.

It would appear from the preliminary analysis and comparison of the measured and computed displacement of the walls that the results did not match to each other. The possible cause of the deviation of the FEM - results from the measured values may be summarised in three points as follows: 1) Measured values are incorrect, 2) Soil parameters are weak, or 3) Construction details are not simulated correctly.

Table 5. Construction stages (Section I-I)

Phase 00:	generate the initial stresses
Phase 01:	activate surcharge load
Phase 02:	1st excavation to a depth of -1.2 m (1)
Phase 03:	installation of the walls (2)
Phase 04:	excavation of the first strip 2.5 (3-1) [1 day]
Phase 05:	excavation of the next strip 2.4 (3-2) [1 day]
Phase 06:	consolidation time [1 day]
Phase 07:	excavation of the next strip 2.3 (3-3) [1 day]
Phase 08:	consolidation time [2 days]
Phase 09 - 13:	excavation of the strips 2.6, 2.1+2.7, 2.8, 2.2, 2.4 in succession (3-4) to (3-8) [each 1 day]
Phase 14:	consolidation time [3 days]
Phase 15:	excavation of the next strip 2.5 + 2.9 (3-9) [each 1 day]
Phase 16:	consolidation time [8 days]
Phase 17:	excavation of the next strip 2.6 (3-10) and installation of the left support (4)[1 day]
Phase 18:	consolidation time [4 days]
Phase 19 - 21:	excavation of the strips 2.7, 2.8 2.8, in succession (3-11) to (3-13) [each 1 day]
Phase 22:	installation of the right support (5)[1 day]
Phase 23:	excavation of the strips 4.1-4.9 (6) [10 days]
Phase 24:	placement of the bottom slab (7) [11 days]
Phase 25:	consolidation time [10 days]
Phase 26:	removal of the left support (4)[2 days]
Phase 27:	consolidation time [3 days]
Phase 28:	excavation of the strips 4.1-4.9 (8) [7 days]
Phase 29:	placement of the bottom slab (9) [8 days]
Phase 30:	consolidation time [22 days]
Phase 31:	removal of the right support (5)[2 days]
Phase 30:	consolidation time [4 days]

N.B.: 1) numbers in () are construction sequence (Fig.?) and numbers in [] are consolidation and execution time.
2) placement of the bottom slab in the previous trench followed during the excavation of the next strip.

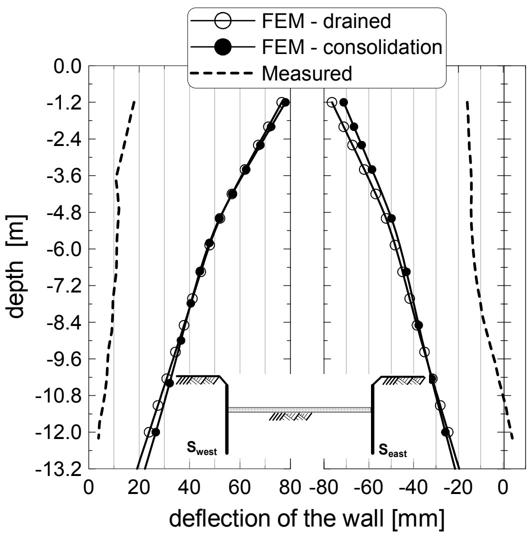

Fig. 11. Calculated and measured displacement of the wall (West - East Section).

The probability of mistakes during measuring and interpretations may not be fully ignored. However, the probability of making a mistake that lead to more than 300% differences at all measuring points is very unlikely. The deflection of the wall was measured using inclinometer with an accuracy of ±1 mm according the geotechnical report. Moreover, the construction was successfully completed without no remarkable damages on the surrounding structures. Therefore, point 1 may be ruled out as possible source of deviation of the results. To investigate the other two points as the main causes of the problem, a parameter study on a simplified geometry had been carried out and is presented in the section below.

PARAMETER STUDY ON SIMPLIFIED MODEL GEOMETRY

In order to investigate the possible causes of deviations of the FE-computation results and the measured values, a simplified model geometry (Fig 12) was selected for further parametric study. The parameter study includes influence of the interface properties, the ground water conditions, the stiffness of the soil, and an aspect of constructional procedures. All the FEM-computations had been performed under drained condition. Hence, the material properties in Table 1 were adopted as a reference parameters.

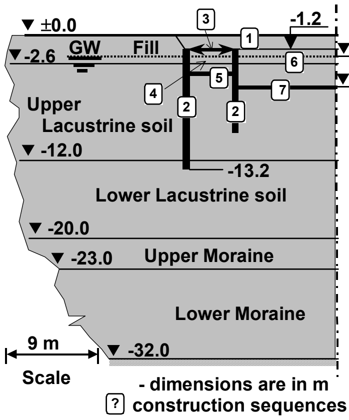

Fig. 12. Simplified model for parametric study.

The Interface Behaviour

In this section the possible influence of the interface properties had been examined by varying the shear parameter of the interface and the virtual thickness of the interface. The following cases had been investigated:

Case 0:	Reference case (R_{inter} = 0.33 and Virtual thickness factor = 0.10 (default)
Inter-case 1:	R_{inter} = 0.50
Inter-case 2:	R_{inter} = 0.75
Inter-case 3:	R_{inter} = 1.00
Inter-case 4:	Virtual thickness factor = 0.05
Inter-case 5:	Virtual thickness factor = 0.20

Note that the factor R_{inter} was only applied on the shear parameters.

Figure 13 shows the influence of the variation of the interface properties at three selected points, namely, at wall head (A), at wall toe (B) and around 6.5 m behind the wall (C) . It can be noted that varying the value of R_{inter} from the reference value of 0.33 to 1.00 led to a reduction of wall displacement at A by about 25%, no significant change at point B, up to 40% reduction of the surface settlement at C. These changes are not large enough to bring the computed displacements to the measured values. Compared to the reference values, no significant change was observed for the cases of varying the virtual thickness of the interface from the default value 0.1 to 0.05 and 0.20. Therefore, one may rule out the interface properties as a main source of the differences between the calculated and measured results, though it might contributes its part to the problem.

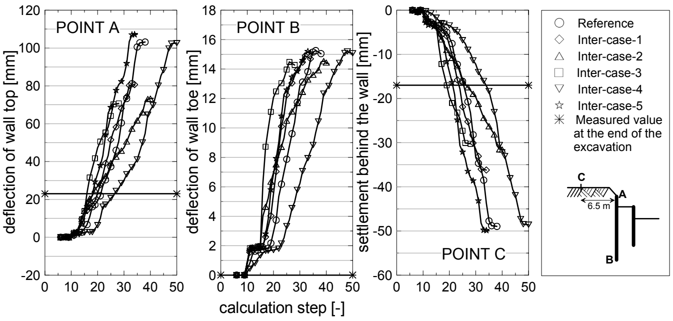

Fig. 13. The influence of the interface properties.

The Stiffness of the Soil

It is well known that the modulus of deformation of the soil strongly influences the calculated behaviour of the excavation. The differences in the measured and calculated deformations, not only by excavations but also by shallow foundations, are usually related in the literature to the luck of estimating the deformation modulus correctly from laboratory test results. To examine the effect of the stiffness parameters based on the practical project, the following cases had been investigated:

Case 0:	Reference case
SS-case 1:	Increase the E_{50} and E_{oed} of the fill, the upper and the lower lacustrine layers by a factor of 1.5 and the E_{ur} by a factor of 1.3
SS-case 2:	Increase the above stiffness parameters by a factor of 4
SS-case 3:	Increase the stiffness of the fill layer by a factor of 4

Figure 14 shows that the stiffness values of the upper three layers should increase by a factor as high as 4 in order to arrive at computed deformations that are fairly comparable with the measured values. An efficiency of deformation reduction up to 73% was achieved in the case of increasing the stiffness values by a factor 4 (*SS-case-2*). The assumption that the fill layer can be stiffer than usually would have taken due to the existing buildings and asphalt streets (SS-case3) does not help to reduce the deformations to a level of the measured values. The parameter variations had clearly shown the influence of the stiffness parameters, however, whether this was the main source of the difference between the computed or the measured values should be proven.

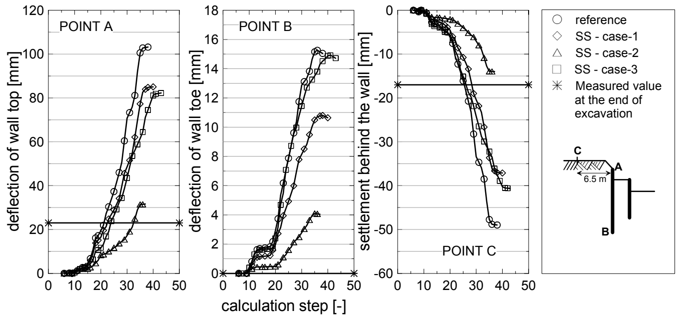

Fig. 14: The influence of the stiffness parameters.

The Groundwater Condition

The reference computation was carried out by assuming a hydrostatic groundwater level at 2 m below the ground surface for all soil layers. In the geotechnical report, however, different ground water levels are indicated for different layers. A free water in the fill layer at -2 m depth on average and a confined groundwater in the moraine layer was encountered, but it was not well known which water table governed in the relatively impermeable layer of the lacustrine soft soil. A drainage filter was also constructed just behind the sheet pile wall to a depth of 1.2 m below the wall, in order to relieve the wall from groundwater fluctuations. These variations of the ground water condition together with groundwater flow analysis had been considered in the study and they are categorised as follows:

GW-case 1:	GW flow analysis instead of hydrostatic groundwater
GW-case 2:	Consider the drop of the GW due to drainage filter
GW-case 3:	Assume the GW table at -2 m for the fill layer, at -1 for the moraine layer and for the lacustrine layers interpolation between the two
GW-case 4:	Assume the GW table at -2 m for the fill layer, at -12 for the moraine layer and for the lacustrine layers interpolation between the two.

Figure 15 shows no significant change of the deformations at point A and C due to the groundwater flow analysis but there is up to 45% reduction of the deformation at point B when compared to the reference values. The second case of the GW analysis shows a 22% and 17% reduction of the wall displacement at points A and B and an increase of the settlement at C by about 11%. The GW-case 3 shows no significant influence at points A and C but a reduction of the wall toe displacement by about 31%. The most noticeable effect can be observed from the GW-case 4 analysis results. As it can been seen from the Fig. 15 there is a 50 - 65% reduction of the deformations at the given points. In general, the above results show that the importance of the detailed information of the groundwater condition and the type of analysis.

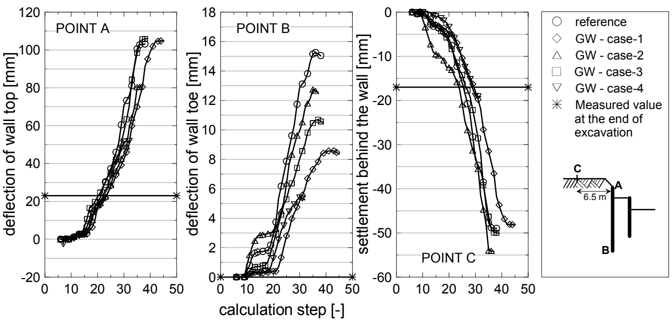

Fig. 15. The influence of the groundwater conditions.

Other Factors

To this group belong the effect of the plastic behaviour of the sheet pile wall, the no surcharge effect, the effect of considering the surcharge as a rigid body in stead of distributed load, the effect of replacing the bottom slab with a fixed strut, the effect of shifting the transition between the upper and lower lacustrine layer to -9.7 m from the -12.0 m depth below the ground surface and, increasing the stiffness of the lower lacustrine layer by a factor of 3 (after shifting the layer).

Others-case 1: Plastic behaviour of the sheet pile wall. Most often the assumption of the elastic behaviour of the wall might be sufficiently enough for practical purposes. To examine the possible effect of the plasticity, the material behaviour of the wall was defined as elastoplastic by defining an additional parameters: the maximum bending moment of M_p = 505 kNm/m and the maximum axial force of N_p = 3612 kNm/m (Profile ARBED AZ18-240). The analysis result (Fig. 16.-top) shows no influence of this variant on the displacement of the wall and the settlement of the ground surface.

Others-case 2: Without surcharge load. A surcharge load of 24.4 kN/m² and a traffic load of 10 kN/m² was assumed in the computation of the reference case at a depth of -2 m and at the ground surface respectively. To see the possible effect of these loads, they are set to zero. The result (Fig. 16-top) shows about 20% less displacement at A and about 30% less settlement at C, but no effect on the wall displacement at B.

Others-case 3: Simulation of the building load with a rigid porous body. A cluster was defined which is equal to the building width and 2.0 m deep below the ground surface with a porous linear elastic material property (γ = 18 kN/m² and E = 1.0×10^6). This is equivalent to the total weight of a 3 storey building (≈ 36 kN/m²). This assumption led to a negligible effect at point A and B, but to about 40% settlement reduction at C (Fig. 16-top).

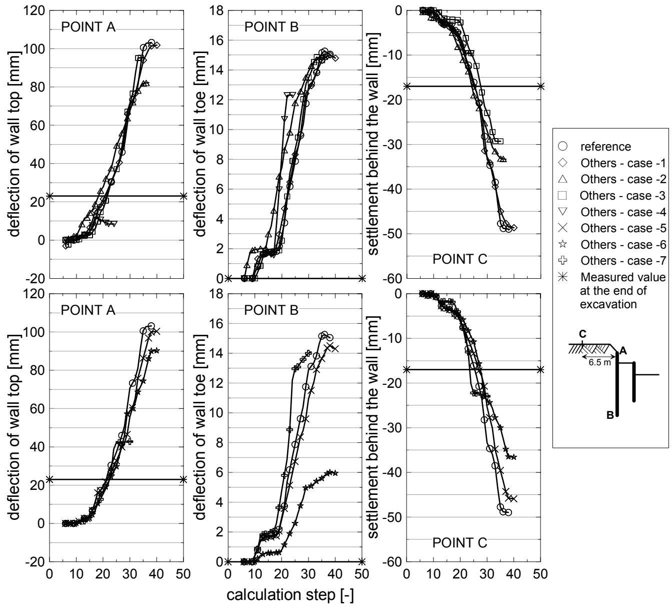

Fig. 16. The influence of other factors on deformation behaviour of the wall and the soil.

Others-case 4: The bottom slabs assumed as fixed struts instead of plates. The bottom slabs was represented by a plates in the reference case. This may have a negative effect, because the slab plates are automatically connected (either fixed or hinged) to the wall, i.e., the wall may move upward when the slab moves, which does not mirror the situation in the field. The hinge connection was chosen in this study. The advantage of such plates is their flexibility and stability effect because of their weight. An alternative to the simulation of the bottom slab is to introduce a fixed support, but this option do not allow any horizontal movement.

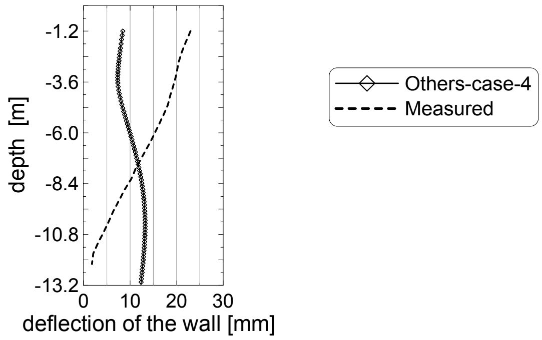

Fig. 17. Comparison of the computed and measured displacement of the wall (Others-case-4).

The comparative effect of both options are investigated under the "*others-case 4*", and the results are shown in (Fig. 16.-top). At first glance, it would appear that this option would be the most effective means of reduction the displacements, as the displacements at A, B and C are reduced by about 92%, 18% and 80% respectively compared to the reference value.

There is even 50% less displacement of the wall at A. However, looking at Fig. 17, one can easily observe a shape of the computed displacement of the wall which completely oppose to the shape of the measured displacement. The fixed struts at the level of the bottom of excavation (sequence (5), Fig. 12) provide non-yielding support to both wall, even it holds back the internal wall. The whole system above the excavation level reacts rigid, but this does not show the reality, and therefore should be ruled out from the options.

<u>Others-case 5: Shifting the depth of the transition from the upper to the lower lacustrine soil layer to a depth of -9.7 m</u>. It can be seen from the measured values that almost no displacement was recorded at the toe of the outer sheet pile wall (south), indicating a possible fixed support of the wall around the foot. In order to provide a fixed support at the foot, the layer of the lower lacustrine soil was shifted upward to a depth of -9.7 m below the ground surface. If one closely examine the cone penetration field test results, there is a room for possible variation of the boundaries of the layers. However, this option alone did not help to avoid the computed wall displacement at the toe of the wall (Fig. 16-bottom). Further, one more variation was investigated by improving the stiffness of the lower lacustrine layer by a factor of 3 under the name *"Others-case 6"* in addition to the shifting of the layer. This option provides a 56% reduction of the wall displacement at B (Fig. 16 -bottom), whereas its effect at points A and B are moderate.

<u>Others-case 7: Allowing the bottom slab support to effect earlier (3D effect)</u>. At last but not least, the 3D effect was investigated. As shown in Fig. 4 & 5, the excavation was executed in slices of trenches based on daily output. Immediately after excavation of each trench, a fast hardening concrete was placed which provided a bottom support to the wall in south - north directions, before the next excavation had been proceeded. The measured wall displacements at different construction stages (*Gebreselassie, 2003*) also clearly show the effect of the bottom slab, in which a buckling of the sheet pile wall at the level of the bottom slab can be seen. In other words, the slab was already in effect before the end of the excavation. Such excavation procedure is a 3D problem, and can be best solved using 3D-finite element program. However, in this 2D study an earlier effect of the bottom slab was assumed by means of activating it after excavating half of the soil mass but before the end of excavation in each excavation phase.

It appears from Fig. 16-bottom that the computed result nears the measured value at point A and C, whereas it shows a little influence at point B. However, it should be noted that the assumption that the supporting effect of the slab starts after 50% excavation of the soil mass in the corresponding cluster, is purely a rough estimate. Therefore, additional investigation using FE-3D-programm is required before this factor can be used in the 2D-analysis of excavations.

END ANALYSIS RESULTS

Once the possible impact of the different parameters, construction and ground water condition had been studied, the next step was to make use of this parameter study to analysis the actual project. Although all the cases studied above might have impact on the deformation behaviour of the excavation, the last case "*Others-case-7*" in combination with the case *"Others-case-6"* has been identified as the major important factor. The earlier effect of the bottom slab has a major influence on the wall displacement at the top, and improving the stiffness of the soil around the toe will have an effect on the toe displacement. A combination of these two factor might lead to a result comparable with the measured values. Thus, the following cases had been considered for the final analysis of the excavation project in question:

Final-case-1:	the same as "Others-case-7" in the parameter study
Final-case-2:	the same as "Final-case-1" but with combination with "Others-case-6" in the parameter study
Final-case-3:	the same as " Final-case-2"but with the consideration of the drop of the ground water table behind the wall due to the drainage filter.

Both drained and consolidation analysis were carried out.

<u>Wall Displacements</u>

Figure 18 shows the computed and measured displacements of the southern and the northern walls for the cases described above. It appears from this figure that the consolidation analysis provides a lesser displacement than the drained analysis as would expected. It is also interesting to observe that the first case (*Final-case-1: consolidation)* led to a displacement at the top which is comparable to the measured values, but it shows more displacement at the toe. However, combining this effect with improving the stiffness of the lower lacustrine soil (*Final-case-2: consolidation*) still results more computed displacement at the toe of the southern wall and a lesser displacement at the toe of the northern wall than the measured value. The possible explanation can be the difference in soil profile at this particular points. The third variant (*Final-case-2*) show no significance effect, and it was neglected in the consequent presentation.

The computed displacement of the west-east walls for the variant *"Final-case-2"* only is shown in Fig. 19. It can be seen from this figure that the computed displacement from the consolidation analysis match fairly well the measured displacement of the wall. The shape of the measured displacement of the eastern wall does not match with the computed shape. The toe of the wall also shows movement in the direction of the soil mass. A difficulty of interpretation of the inclinometer measurement at this location was reported in the geotechnical report (*Kempfert + Partner, 1994-1998)*, hence the comparison with the computed displacement should take this into account.

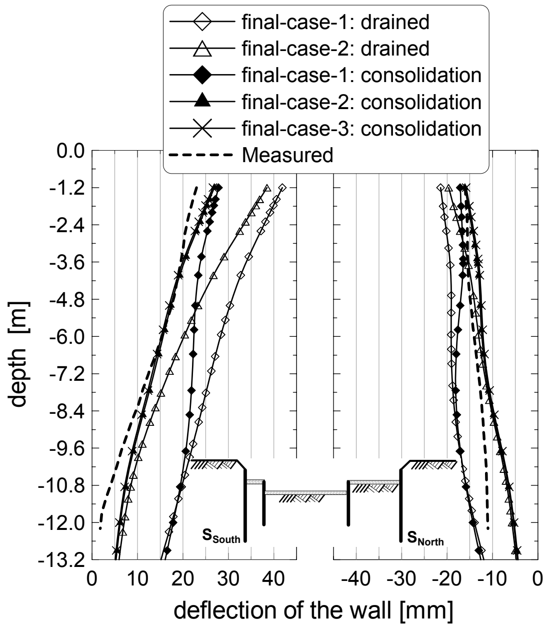

Fig. 18. Comparison of the computed and measured wall displacement (South - North Section).

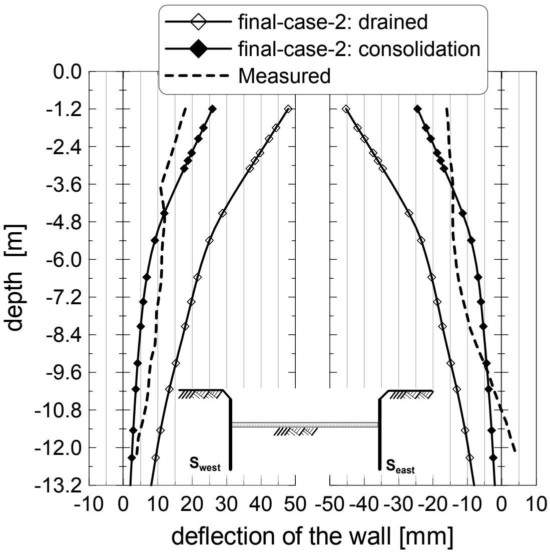

Fig. 19. Comparison of the computed and measured wall displacement (West - East Section).

Wall Head Displacement

The measured displacements of the wall head at selective points (LM23 to LM26) on the southern wall compared to the computed displacements are shown in Fig. 20. It can be seen from the figure that the consolidation analysis (*Final-case-2*) result fairly match with the observed displacements.

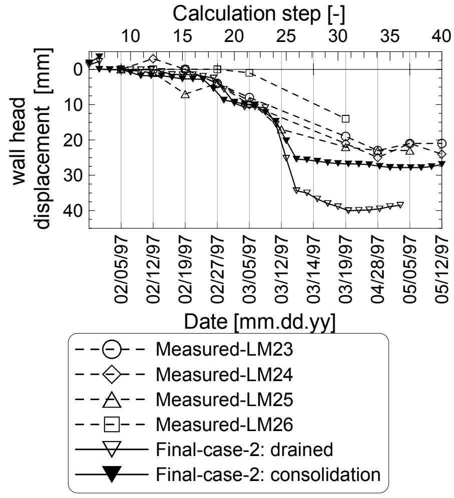

Fig. 20. The wall head displacement.

Surface Settlement Behind the Wall

Two representative computed settlements from drained and consolidation analysis are presented in Fig. 21 for the variant *"Final-case-2"*. The Fig. 21a shows the computed ground surface settlement at the location in front of the existing building which is 6.5 m behind the southern wall (Raueneckegasse) and the measured settlements at the locations HP4 and HP5 (see Fig. 1). The course of the computed and measured settlement curves is more or less similar. It should be noted that the computed settlements are given in terms of the calculation steps, whereas the measured settlements are drawn based on the real construction time. However, one can clearly identify the different construction phases from the course of the curves, and one may easily compare the computed and measured results. Moreover, the computed settlements up to the installation of the walls (inclusive) had been set to zero, whereas the measured settlements are displayed from the beginning of the construction. For example, a total settlement of about 5 mm was measured at HP4 immediately after the installation of the wall, and adding this value to the computed settlement will result even to a better agreement of the computed and measured settlements.

Similarly, Fig. 21b shows the computed ground surface settlement behind the western wall (Sigismundstrasse) at a distance of 10.5 m compared to the measured settlements at measuring points HP23 and HP21 (see Fig. 1). The consolidation analysis result shows a good agreement with the measured value as far as the course of the settlement curves is concerned.

As would be expected, the drained surface settlement is less than the settlement from the consolidation analysis. As can be seen from Fig. 21, the difference of the settlements is higher in the west-east section than that the south-north section, which indicates the effect of the simulation of the excavation of each slice, subsequent placement of the bottom slab and simultaneous consolidation.

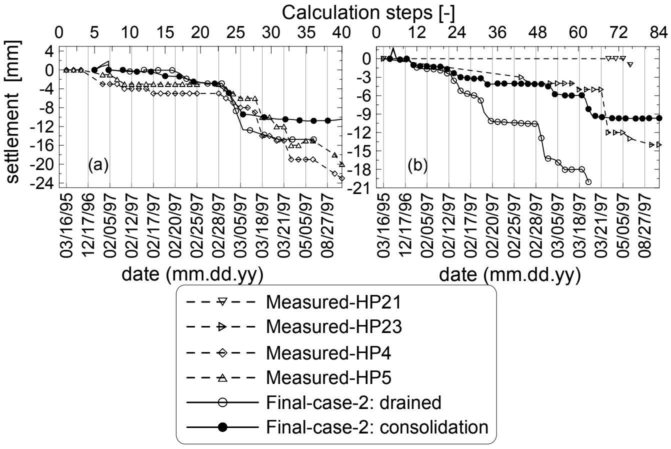

Fig. 21. Computed and measured settlements: a) behind the southern wall; b) behind the western wall.

Pore Pressure

The calculated excess pore pressure at three locations and at two different depths are shown in Fig. 22. It appears from the figure that there is fair agreement between the measured and computed values, though the excess pore pressure was not carried out to the end of the excavation.

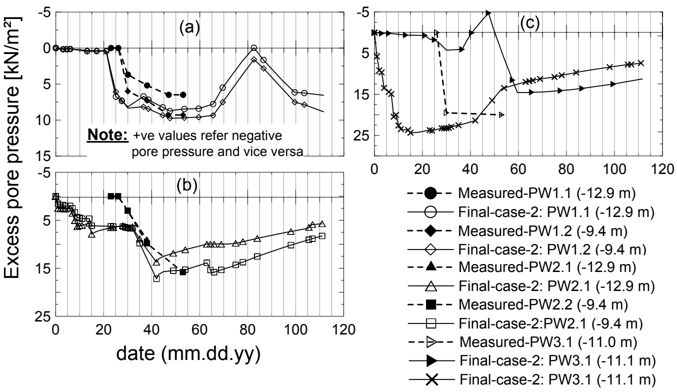

Fig. 22. Computed and measured pore pressure.

CONCLUSION

From the parameter study and analysis of the actual excavation project, it may be concluded that all the variants considered may exercise an influence on the performance of the excavation in one way or the other way, but the 3D-effect and the stiffness of the lower lacustrine soil showed the maximum influence. The authors believed that using soil parameters from carefully conducted laboratory tests for the soft soil layers and from field penetration or sounding tests for the relatively bearing layers, and with due consideration of the 3D effect may lead to a reasonable prediction of the deformation of the soil in and around an excavation.

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SYMBOLES AND ABBREVATIONS

γ_{sat}, γ_{unsat} = Saturated / Unsaturated unit weight
k_x, k_y = Permeability of the soil in x and y directions
c', φ' = Effective cohesion and friction angle
ψ' = Dilatancy Angel
E = Elasticity modulus
E_{50} = Secant modulus at 50% the deviatoric failure stress
E_{oed} = Constrained modulus
E_{ur} = Modulus of elasticity for un/reloading
p^{ref} = Reference pressure (atmospheric pressure)
ν_{ur} = Poisson's ratio for un/reloading
m = Exponent in the power law of the stiffness of soil
R_f = Failure ratio
A, I = Cross sectional area and moment of inertia
w = Weight of plate per unit area
R_{inter} = Interface strength factor
ref = Reference
HSM = Hardening Soil Model
MCM = Mohr-Coulomb Model