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Deep open excavation in soft plastic ground in Salzburg, Austria

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ABSTRACT: Probably one of the most difficult ground condition pertaining to the design and construction of deep excavation in built-up areas is exhibited when a relatively shallow stiff layer of soil (gravel and sandy gravel) overlies a deep layer of soft plastic soil (silt and clayey silt), especially, when the latter is prone to liquefaction. This is the case for the Quaternary deposits within the basin of Salzburg in Austria. In this case the choice of the foundation system and of the construction process as well as of the support system for the open excavation is very much dependent on the short and longterm deformation behaviour of the soft plastic subsoil, apart from the size and the founding depth of the structure and the vicinity of adjacent buildings on account of potential damage during the construction phase. In this paper the problem of deep open excavations using the diaphragm wall technique with its associated deformations is highlighted in respect of the time-dependent construction activities, the bulk excavations and the relevant support system. Two case histories are presented, each employing a different method for the temporary supports of the retaining structure. In all instances a careful monitoring of deformations by means of various measurement techniques during all phases of construction has been made. Most revealing is the evaluation of the monitoring during the construction of a short stretch of an underground railway line in the subsoil of Salzburg. Finally, reference is also made to a 3D-FEM analysis which compares measured results with a calculated prediction.

1 INTRODUCTION

Deep open excavations in densely built-up areas are nowadays even more frequently required than before on account of the ever-growing need for better utilization of available building sites in sometimes very confined spaces, even in traditionally poor ground. This requirement may be for the construction of deep basements, underground storage spaces, pump installations, and increasingly, for underground railway stations. Normally open excavations require lateral support for the side walls unless the design provides for a free cantilever construction of the retaining walls, or it relies on the self-supporting effect of a cylindrical structure in the ground. Using diaphragm walls as retaining structures with adequately designed temporary supports provides in general a very effective ground support on account of the intimate contact of the concrete wall and the surrounding soil and their ability to control the seepage into the excavation by providing a cut-off below formation level. The overall stiffness of such a retaining system can be very great, especially when the temporary supports are prestressed and a top-down construction is being used whereby the exterior diaphragm walls and floors serve as part of the finished structure.

The construction of a deep excavation induces at all stages of construction a change of the stresses in the surrounding soil and in consequence thereof deformations in the vicinity of the excavation. Such deformations may be critical on account of potential damage to neighbouring structures.

Apart from the overall stiffness of the ground support system, also influencing the deformations are the construction sequence for the bulk excavation and the excavation process itself. This is particularily evident if the ground consists of soft plastic soil overlain by a stiff layer of sands and gravel. Two case histories from the urban district of Salzburg (Austria) are presented below as typical examples.

2 GEOLOGICAL ASPECTS AND GROUND CONDITIONS

The basin of Salzburg is of Quaternary origin and lies at the confluence of two rivers, the Salzach and Saalach. 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. These deposits are fully saturated with a strong anisotropy in permeability ($k_h = 10^{-5}$ to 10^{-6} m/s and $k_v = 10^{-6}$ to 10^{-9} m/s), and when unloaded by removal of overburden exhibit a liquefaction-like behaviour upon the influence of dynamic loads. In the later period the "Seeton" was covered by a 4 m to 6 m thick layer of compact gravel and sandy gravel. Old buildings have usually been founded within this gravel layer and only more recent deep bulk excavations brought the problem of time-dependent deformations and the liquefaction behaviour of the underlying "Seeton" into the open. Therefore for the construction of deep basements in Salzburg, dewatering with vacuum wells, combined with special excavation techniques, is generally necessary for an efficient excavation.

The behaviour of the subsoil is characterized by the soil parameters established from a number of laboratory and in-situ tests, summarized in Fig. 1 and Table 1, below. Of particular significance for the deformation behaviour of the soft-plastic "Seeton" is the deformation modulus E_s, gained from compression tests on undisturbed soil samples after pre-loading with the in-situ stress of the relevant depth. Using the relationship for the tangent modulus $E_s = a.\sigma_n^b$ $[MN/m^2]$, parameters a and b are shown in Fig. 2 in respect of depth (Breymann 1995).

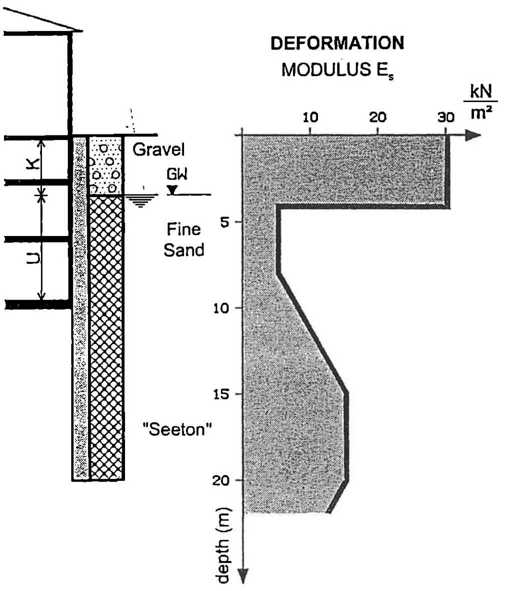

Figure 1. Variation of E_s with depth

Table 1. Soil parameters

	E_s (MN/m^2)	ϕ' (°)	c' (MN/m^2)
gravel	20 to 35	35 to 37	0
sand/silt	10 to 14	26 to 30	0
"Seeton"	5 to 25	22 to 27	0 to 0.02

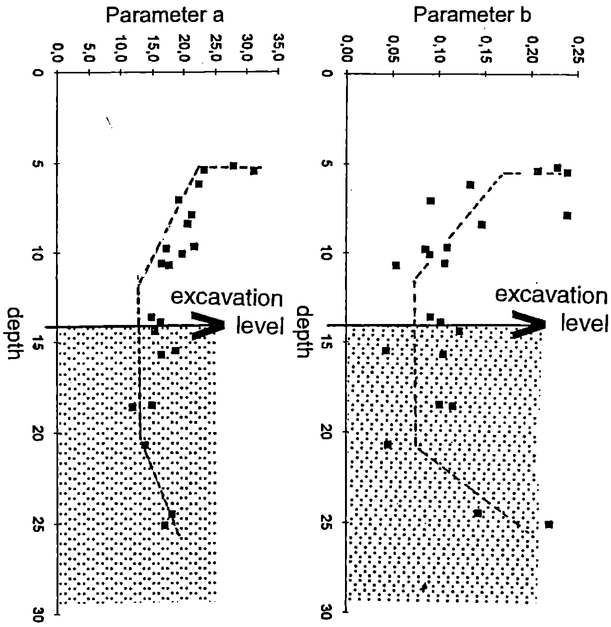

Figure 2. Variation of parameters a and b with depth

3 FOUNDATION PROBLEMS AND CONSTRUCTION

As is well known, there exists a strong relationship between the various phases of bulk excavation and the consequential deformations of the sides of the excavation and of the adjoining ground surface, as well as with the amount of heave at the bottom of the excavation. This is even more pronounced when a deep layer of soft plastic soil is covered by a relative shallow stiff layer of dense gravel as in the present case.

Prediction of the magnitude of such deformations and their pattern of distribution around the excavation by standard calculations is a futile undertaking on account of the many influencing factors. Only finite element analysis may provide in such instances reasonable results.

Because of the great depth of the clayey silt deposits a "floating" raft foundation is generally used in Salzburg for buildings with deep basements. This may be a box-like structure of reinforced concrete installed inside an open excavation with sloping sides or as common in built-up areas, constructed as a raft tied into R.C.-diaphragm walls around the perimeter of the site. In the latter case the principal construction phases are as follows:

- Construction of the diaphragm walls
- Conventional dewatering of the gravelly soil and vacuum dewatering of the clayey silt within the walls. Although only a 3 to 5 % reduction of the degree of saturation of the silt is achieved the liquefaction potential is significantly reduced and facilitates bulk excavation.
- Bulk excavation top-down in stages whilst installing the temporary supports for the side walls, usually consisting of prestressed struts or parts of the permanent floor slabs.
- Construction of the bottom raft in section, after reaching the formation level, tied into the side walls.

Depending on local site and soil conditions different temporary methods of side-wall supports have been employed at various projects in Salzburg.

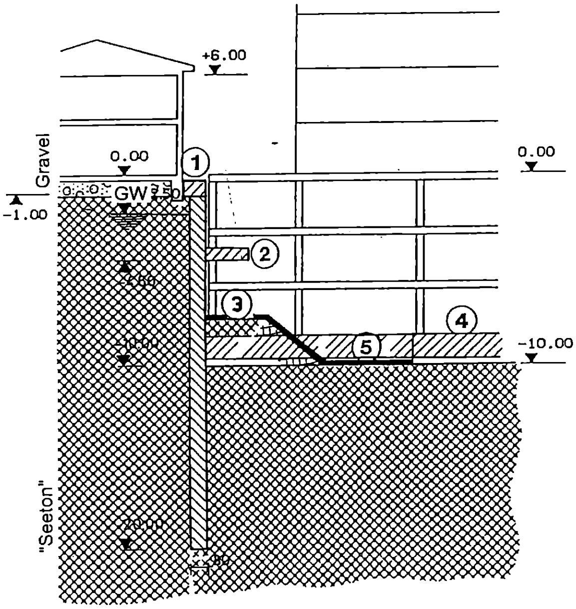

Figure 3. Typical cross-section and construction sequence

4 CASE HISTORIES

The following typical case histories underline the above stated problems. In all cases a comprehensive field instrumentation was installed which permitted careful monitoring of ground and structure deformations during every phase of construction.

4.1 Case 1 - Project AMV

For the nearly square basement plan, the perimetral diaphragm walls with a capping beam were supported during the top-down internal excavation by a substantial 2.5 m wide R.C. waling beam, cross-braced at the corners. The bracing force of 200 kN/m

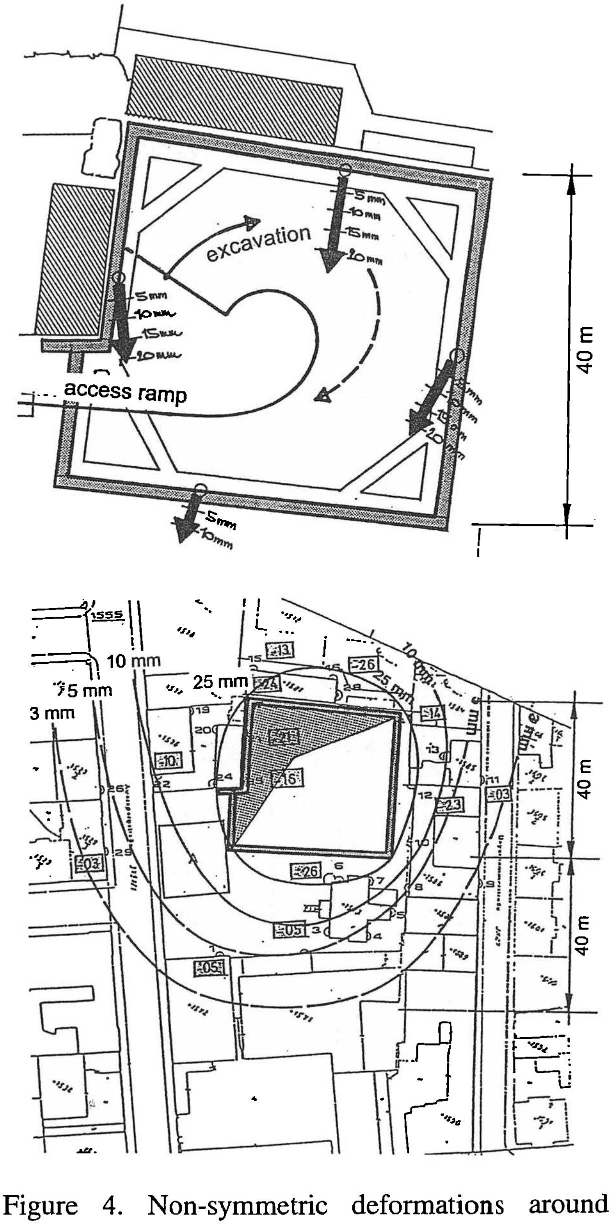

Figure 4. Non-symmetric deformations around excavation

of this ring-like beam was maintained at a constant level by a row of flat jacks.

Fig. 3 shows a typical cross-section indicating the sequence of construction. This site deserves particular attention due to the fact that bulk excavation could only be made via a steep access ramp in an non-symmetric screw-like fashion. The uneven unloading of the perimeter walls and of the base of the excavation pit resulted in unusual wall movements at the top and in an non-symmetric pattern of settlement distribution around the site (Figure 4). A similar case was reported by Burland and Hancock (1977) during the construction of the underground car park at the Houses of Parliament in London.

4.2 Case 2 - Project Lokalbahn

Most revealing was the evaluation of the monitoring during the construction of a short stretch of an underground railway line in the subsoil of Salzburg (Breymann 1995).

A well instrumented cross-section (Figure 5) allowed the monitoring of the movements of various measuring points during every phase of bulk excavation and construction. This included, perhaps for the first time, the observation of the settlement of a fixed point at a nearby structure as the sectional diaphragm wall construction approached the building. Fig. 6 shows the time-settlement behaviour of measuring point MP fixed onto a building which is founded on a R.C. raft, about 3 m below ground level and having a sole pressure of 145 kN/m². This MP is at a distance of about 2.50 m from the line of the 1 m wide diaphragm wall which was constructed in a hit-and-miss sequence in 3.60 m long panels, 24 m deep. The diagram demonstrates that the first measurable deformations begin when the panel excavation is about 2 to 3 m distant from a normal axis through MP and reach a final value only after about 4 weeks. If one considers the time necessary for the excavation and concreting of a panel to be one working day, this means that the soil adjoining the diaphragm wall has to undergo various stress changes during this period (conditions at rest - bentonite supported excavation - liquid concrete pressure - new stress state), and one can see that with this kind of soft plastic subsoil there is a significant time-delay until an equilibrium stress state is finally established.

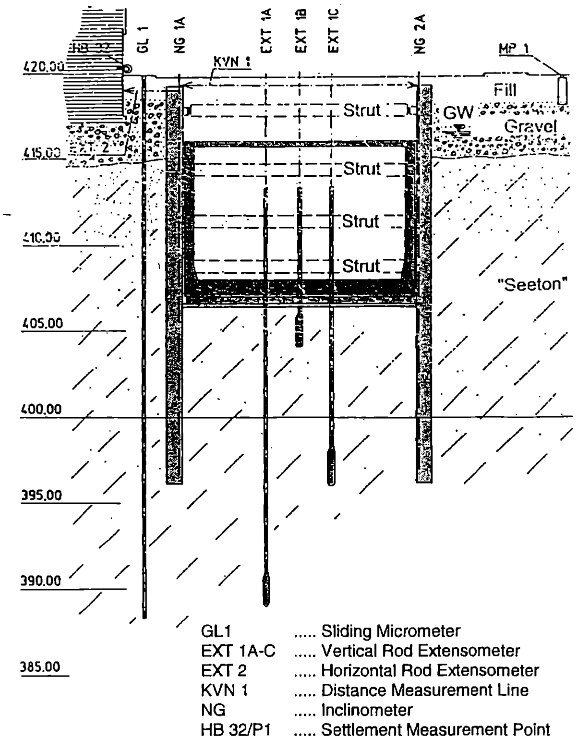

Figure 5. Instrumented cross-section

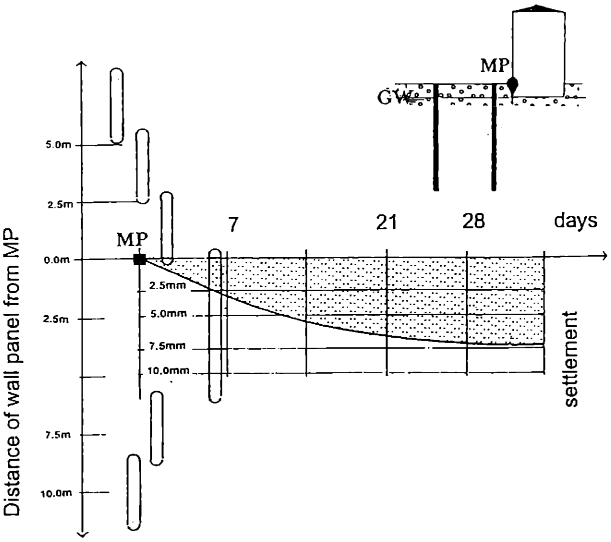

Figure 6. Settlement of point MP due to wall construction

This initial settlement forms the first part of the total fixed-point settlement resulting from the follow-on stages of bulk excavation, the strutting and the dewatering between the walls.

For an alongated construction site with open excavation there exists a similarity with tunnelling, since the various phases of construction (diaphragm walling, pre-excavation and initial strutting, dewatering, bulk excavation, more struts and finally floor slab) involve the whole cross-section and

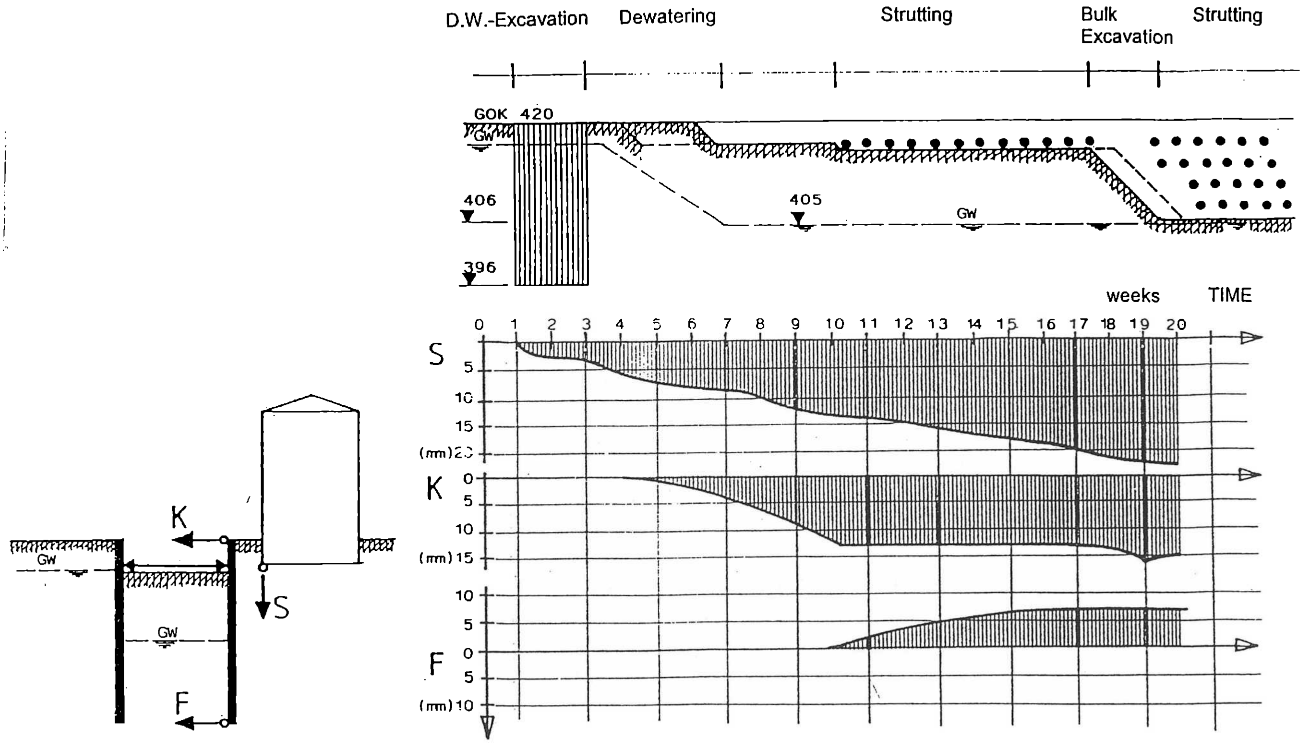

Figure 7. Time-dependent influence on settlements

progress in one direction only. Therefore the time-dependent deformations and stress-changes resulting from each distinct construction activity superimpose upon each other. The sequence "bulk excavation" and its progress rate (a bucket chain excavator was used in this case), before more pre-stressed struts are inserted, is critical in respect of deformations affecting the surroundings of the site and, of course, is on the critical path of the overall project.

Excavation from the top, immediately followed by placing the prestressed struts and the sealing of the base at formation level was essential to minimize deformations of the adjacent ground and of the side walls. Figure 7 shows the time-dependent influence of the construction precedure on the movements at three measuring points. This demonstrates that the uni-directional progression of the construction process in this soft plastic soil has a significant influence on nearby structures. Generally speaking, a fast progress combined with an efficient internal dewatering and an effective stiff strutting arrangement, preferably prestressed, is under the prevailing soil conditions the only way to make a project of this kind technically and economically viable. To achieve this a modern diaphragm wall technique is only but one of the contributing factors.

5 FINITE ELEMENT ANALYSIS

At the Institute of Soil Mechanics and Foundation Engineering of the Technical University Graz, an attempt was made to model the excavation and construction process and its resulting deformations in a nonlinear finite element analysis (Schweiger & Freiseder 1994, Schweiger 1994). Because of space limitations only ground deformations due to construction of the diaphragm wall calculated from a 3-D analysis are presented here.

The following construction sequence was assumed in the analysis (Figure 8):

i) calculation of the reference state (initial stresses and surcharge load due to existing buildings)
ii) excavation of panel No.1
iii) excavation of panel No.3, concrete in place in panel No.1
iv) excavation of panel No.5, concrete in place in panels No.1 and 3
v) excavation of panel No.2, concrete in place in panels No.1, 3 and 5
vi) excavation of panel No.4, concrete in place in panels No.1, 2, 3 and 5

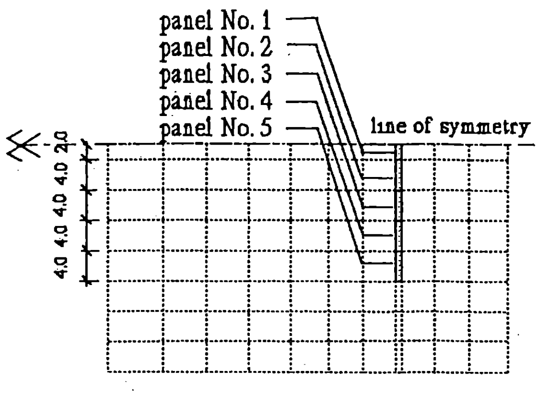

Figure 8. Plan view of diaphragm wall and domain analysed

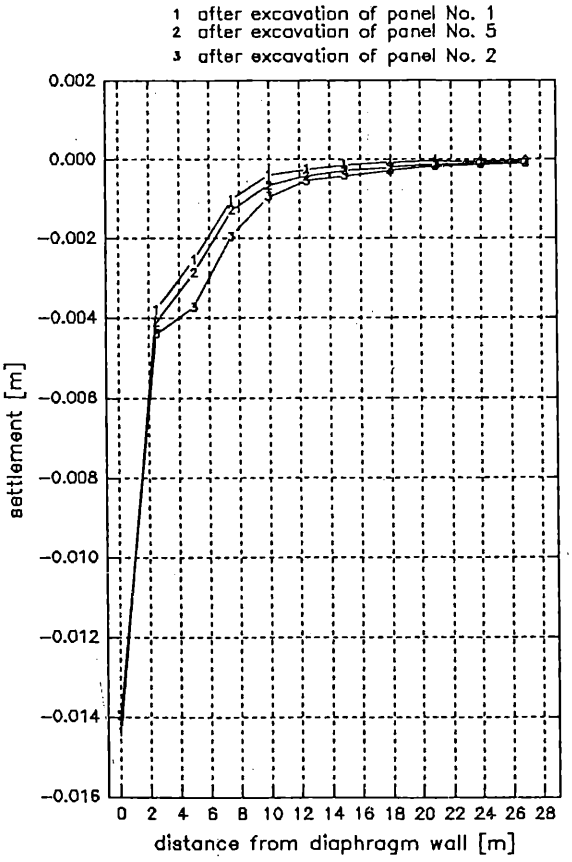

Figure 9. Settlement trough due to excavation of panels No. 1 to 5

A comprehensive parametric study has been performed evaluating e.g. the influence of the level of the bentonite slurry inside the panel on the settlements and the pressure of the viscous concrete during concreting on the horizontal stresses. These details are not discussed here, as they can be found in Freiseder 1993.

In Figure 8 the settlements in the line of symmetry normal to panel No.1 are shown. Most settlement occurs during construction of panel No.1 and it can be seen that excavation of panels No.3 and 5 has only a marginal influence on the settlement behind panel No.1. Excavation of panel No.2 increases settlements again. The value at the edge of the excavated panel is probably not realistic which is due to the somewhat crude mesh used and the fact that the guide walls necessary for wall construction have not been considered in the analysis. However settlements of approximately 5 to 10 mm seem be to be quite realistic for the given ground conditions and compare well with observed values (see Figures 6 and 7).

6 CONCLUSION

The unique subsoil "Seeton" within the basin of Salzburg exhibits unusual liquefaction-like behaviour when unloaded. Further the time-dependent deformations initiated by the construction requires special care when designing temporary works. Some results from 3D finite element analyses modelling diaphragm wall construction have been presented.

REFERENCES

Breymann, H. 1995. Tiefe Baugruben - Aufwand und Risiko. *Jubiläumsbroschüre bvfs-Salzburg*. 15-27.

Burland, J.B. & R.J.R. Hancock (1977). Underground car park at the House of Commons, London: Geotechnical aspects, *The Structural Engineer*, 55: 87-100.

Schweiger, H.F. 1994. Deep excavation in soft soil - Finite element analysis and in-situ measurements. In I.M. Smith (ed), *Proc. 3rd Europ. Conf. Numerical Methods in Geotechnical Engg., ECONMIG 94*: 369-376. Rotterdam: Balkema

Schweiger, H.F., & M. Freiseder 1994. Three dimensional finite element analysis of diaphragm wall construction. In H.J. Siriwardane & M.M. Zaman (eds), *Proc. 8th Intl. Conf. on Computer Methods and Advances in Geomechanics*: 2493-2498. Rotterdam: Balkema

Freiseder, M. 1993. Dreidimensionale FE-Modellierung einer Schlitzwandherstellung. *Diploma thesis*. Technical University Graz.