Geotechnical Aspects of Underground Construction in Soft Ground – Viggiani (ed)
© 2012 Taylor & Francis Group, London, ISBN 978-0-415-68367-8

How to analyse walls for deep excavations

A. Hettler
University of Sciences and Technology of Dortmund, Germany

T. Schanz
Ruhr-Universität Bochum, Germany

ABSTRACT: The paper gives a survey how to analyse walls for deep excavations according to the rules of the German committees for excavations (EAB) and for numerical calculations in geotechnics (AK 1.6). In most cases relevant in engineering practice it is sufficient to use a classical linear elastic beam model. This can be improved introducing a subgrade reaction at the foot of the wall. For complex systems—this means for example complex in geometry and/or initial and boundary conditions—the finite element method is recommended to be used. The paper closes with open questions and gives some ideas for future research, for example how to perform a full design with FE-method and how to take into account different stages of construction processes.

1 INTRODUCTION

During the last years the investigation of serviceability limit states has gained more and more importance. Especially in the case of inner city excavations adjacent to buildings the admissible deformations are often very small. Therefore the engineers have to select carefully the design tools. High demands in design do not implicate automatically finite-element-calculations. Depending on the boundary conditions also classical models may be sufficient or even equivalent.

Additional difficulties may rise in combination with so called deep excavations. Deep may not be defined only by the geometrical depth. Deep can be understood by the degree of technical difficulties in design. This may include for example the geometrical depth, soil conditions, magnitude of water pressure, groundwater conditions and the condition of adjacent buildings. It is supposed that a precise definition of a deep excavation is difficult to be found but in a qualitative sense an approach for practical purposes is possible. For example in Berlin sand and groundwater conditions an excavation of more than 20 meters deepth may be called a deep excavation. For the soft clay at the lake of Konstanz a depth of more than 5 to 6 meters can already cause extreme difficulties in the construction process. For details, see Hettler (2010).

The aim of the paper is to give a survey about the calculation methods which are used in German practice and which are compatible with Eurocode 7. Mainly three models are used:

– the classical beam model
– the beam model combined with subgrade reaction at the foot of the wall
– finite-element-analysis

It is also intended to initiate a discussion and to compare the different methods in different countries within Europe.

2 SAFETY CONCEPT

Eurocode 7 admits three ways to investigate safety. In Germany procedure 2* was introduced. This means the following steps:

– design of the structural system
– determination of characteristic loads and calculation of characteristic effects
– determination of design values for effects by multiplying the characteristic values with the partial safety factors
– investigation that the design values of effects are smaller or equal to the design values of resistences.

It is believed that the calculation with characteristic values is advantageous, especialy for nonlinear systems. In such cases it may happen that the calculations for example with design values for φ and c lead to unrealistic results and the interpretation of the results may be difficult. Only for linear elastic systems calculations with design values for loads and resistences are mathematically equivalent. Then the user can decide whether to calculate with characteristic or design values. Sometimes it is easier to find the minimum dimensions by applying design loads. For details, see Hettler (2009) or Weißenbach and Hettler (2010). See also the discussion in section 5.

3 CLASSICAL BEAM MODEL

According to the German recommendations for excavations (EAB, 2006, 2008) the following simplifications are assumed when investigating the limit state:

- The static system is an elastic beam with unyielding supports.
- Each step in the construction process is analysed separately, neglecting previous steps.

The earth support may be classified into the following systems:

- A free earth support, which is fixed at the toe with a force $C_{h,k} = 0$ and where the resultant of the soil reactions $\sigma_{h,k}$ is the supporting force $B_{h,k}$ (Fig. 1a). The embedded length is t_0.
- A partly fixed earth support with length t_1' (Fig. 1b).
- A fixed earth support with length t_1 and the boundary condition $M_c = 0$ and a vertical tangent at the support (Fig. 1c).

The extra length Δt_1 in Figure 1b and c is necessary to transfer the support forces $C_{h,k}$ to the soil as proposed by Blum. For details, see Weißenbach, Hettler, Simpson (2003) or Hettler and Weißenbach (2010).

A particular feature in German design of excavations is how to calculate the earth pressure.

In a first step a triangular distribution according the Rankine's theory is assumed. Then following Weißenbach's proposals it is redistributed, as shown by the example in Figure. 2.

The German recommendations for excavations (EAB) propose different earth pressure figures, depending for example on the stiffness of the wall, the stiffness of the props and the number of props. For details, see EAB (2008).

For investigating serviceability limit states the beam model has to be improved:

- The supports may be assumed as elastic springs.
- The predeformations before installing the next prop are taken into account and added up during the installation process.

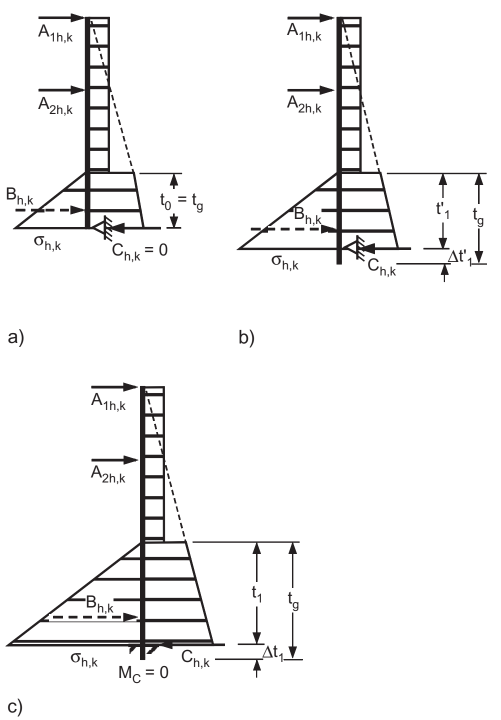

Figure 1. a) Elastic beam with free earth support b) partly fixed earth support c) fixed earth support.

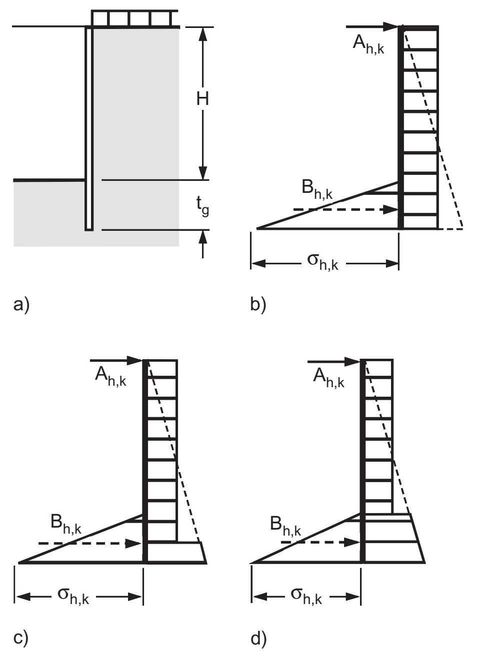

Figure 2. Excavation wall with free earth and propped at the top a) statical system, b) redistribution of the classical earth pressure up to the toe of the wall c) up to the gravity center of the supporting force $B_{h,k}$ d) up to the bottom of the excavation.

– Subgrade reaction is used for the earth support, see section 4.

In very simple cases the improved beam model may also be sufficient to estimate roughly the deformations adjacent to the wall and not only for the wall itself. But generally if the deformations in the surrounding have to be investigated numerical methods are recommended, see section 5.

4 CALCULATIONS USING SUBGRADE REACTION

The non yielding earth supports in section 3 lead to a contradiction between the earth pressure distribution and the displacements at the embedded part of the wall. These deficencies can be avoided by models using a subgrade reaction. Following the German recommendations for excavations the preloading by the weight of the excavation can be taken into account as illustrated in Figure. 3. As initial stress state the earth pressure at rest $e_{0g,k}$ is assumed, calculated for the original ground surface. For physical reasons the horizontal stress may not surpass the passive earth pressure $e_{ph,k}$. A subgrade reaction is only admitted in the embedded part t_B in Figure. 3. A further limitation is that the sum of the initial stress and the stress from subgrade reaction may not surpass the passive earth pressure.

The difficulty is how to determine the subgrade reaction modulus. During the last years the EAB-committee has evaluated a number of field cases and proposed a range of values for the subgrade reaction modulus, which will be published in the next time. Furthermore nonlinear mobilisation curves may be used as proposed for example by Besler (1998) or Vogt (1984) and which give very reasonable results (Hettler et al. (2005), Hettler & Maier (2004). In practice it may be helpful to determine the subgrade reaction modulus by a finite-element analysis for one case and to use it afterwards in design for other load cases. By this way much time can be economised without really loosing accuracy.

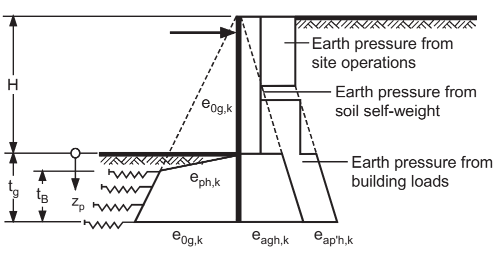

Figure 3. Load model for subgrade reaction in sand.

5 FINITE-ELEMENT ANALYSIS

In Schanz (2006a, 2006b) the current German status of application of numerical methods to calculate retaining walls is documented. By illustrative practical examples it is shown following the recommendations the numerical results regarding global stability are comparable to those using classical approaches. Stability in the numerical approach is defined as the ratio of the actual system resistance related to the system resistance at failure. For situations where the system resistance is determined just by the strength of the soil this approach (phi-c-reduction) is resulting in the classical Fellenius definition. It is equivalent to the definition of factors of partly safety (GZ 1C) following DIN 1054:2005-01. In practice additional structural elements (the wall itself, anchors, struts & nails) contribute to the system resistance. Therefore their resistance considering relevant failure modes (see Fig. 7) must be taken into account in the determination of global safety. A straight forward concept includes the reduction of characteristic resistance of structural elements (loading capacity of anchors, maximum bending moments of the wall) simultaneously to the soil strength. During reduction different levels of safety for the soil and these structural elements must be carefully considered.

6 DEFORMATIONS INDUCED BY CONSTRUCTION PROCESSES

Modern commercial finite-element software allows for taking into account a stepwise construction procedure. This may be called a staged construction analysis. Modelling the construction process itself is up to now rather reserved to scientific investigations for special cases. But there is still a gap in modelling the effects of constructing for example vibrated piles, so called RI-piles (RI = german abbreviation for vibrated and grouted) or anchors. In those cases the deformations from the construction processes may be the dominant factor as the following example may show.

Deep excavations in Berlin were often constructed by installing under water a concrete bottom slab which was anchored to resist against uplift. In the case of the Debis excavation at Potsdamer Platz the horizontal displacements of the wall caused during the installation of the so called RI-piles were 4 times higher than predicted by a classical analysis (Triantafyllidis, 1998).

After the final excavation under water the maximum deformation in the cross section MV 1 and MV 2 was about 10 to 12 mm (see phase 2 in Fig. 4b and c). It increased to about 38 to 58 mm

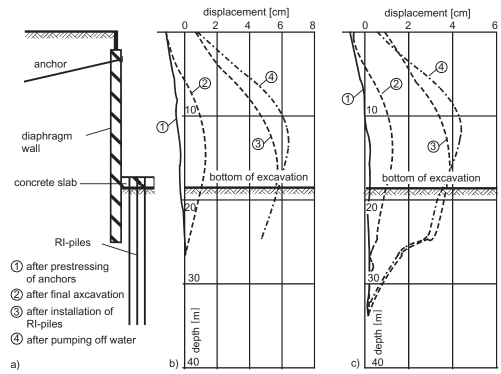

Figure 4. Debis-excavation at Potsdamer Platz: a) cross section of excavation b) displacements in cross section MV 1 c) displacements in cross section MV 2.

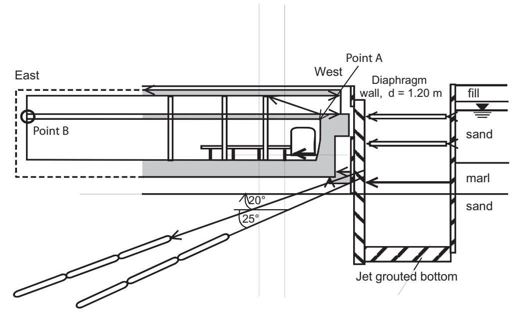

Figure 5. Cross section of city railway station and excavation wall at Potsdamer Platz in Berlin.

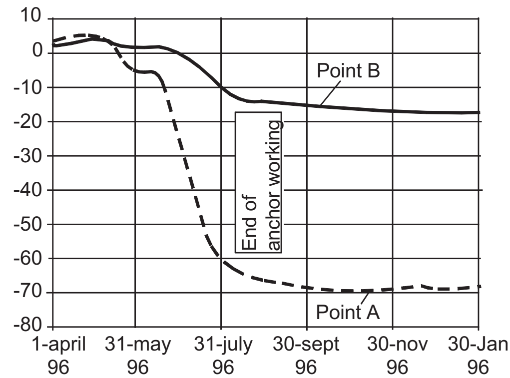

Figure 6. Settlements at points A and B of city railway station at Potsdamer Platz in Berlin caused by installation of grouted anchors.

in phase 3 when the RI-piles were constructed. Pumping off the water in phase 4 caused only a relatively small additional displacement of 10 mm to 12 mm. The maximum difference of about 20 mm in cross sections MV 1 and MV 2, where the geological conditions are quite similar, can be explained by the time sequence and the geometrical arrangement of the piling works. Obviously the embedment at the foot of the wall reduced and at the same time pore water pressure increased. For details see Triantafyllidis (2000).

The installation of grouted anchors may also cause damages in adjacent buildings particularly for multilevel anchored walls showing high density of anchor placement. Boring against groundwater with a hydraulic head is judged as specially unfavorable effect. Constructing the excavation walls for the long distance train station Potsdamer Platz in Berlin two rows of anchors had to be built very close under the tunnel and the station for the city railway, see Figure 5.

Although the material transport was carefully controlled during drilling settlements of the city railway tunnel were observed up to 7 cm, see Figure 6. For details see Borchert (2008).

7 FURTHER REMARKS ON FEM-ANALYSIS

Closely related to the definition of global safety as discussed above is the identification of the relevant reason for failure including the failure kinematics. Using the extended features of FE-analysis it must be asked which reason for failure is relevant and which different failure mechanisms must be analyzed. This important topic is studied in detail by Grabe et al. (2008) for sheet pile walls applying a so called "plastic-plastic-approach". In order to trigger the occurrence of plastic hinges they analyzed six different causes, among others the reduction of the strength of the wall, lowering the groundwater table and an increase of the specific gravity. Theoretically all possible causes must be considered, as for example some more are shown in Figure 7, in order to determine the minimum of safety. For example in Figure 7f, using FE-analysis, an increase of the vertical force P might result in rather large deformations. These deformations might change the direction of the action from $+2/3\varphi$ to $-2/3\varphi$. Therefore the earth pressure increases, the anchor might fail and/or the wall might fail before the wall sinks vertically. For further details, see Hettler and Schanz (2008).

The described task is not unique in the frame of application of FE-analysis, but also included when applying classical methods. In the frame of classical methods only a limited number of different causes for failure are analyzed. Based on long

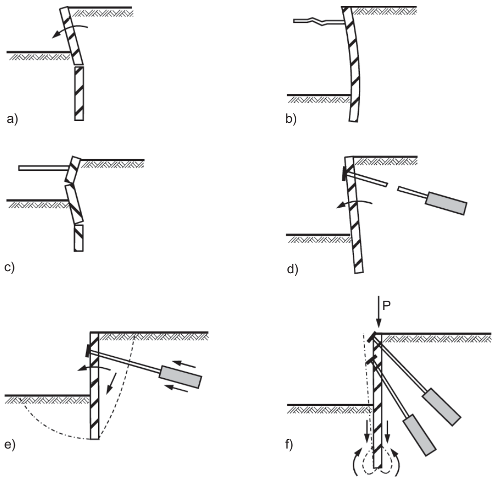

Figure 7. Examples showing effects producing failure and failure mechanisms.

term experience this decision seems to be sufficient. Brinch Hansen and Lundgren (1960) concluded, based on their work on anchored sheet pile walls: One might think, that comparable to the procedure to determine cause for failure in the soil the task must be extended to the global system including structural elements. Different from considering all possible cause it seems sufficient to study a limited number of causes to derive a safe construction.

Different from the calculation of wall deformations by using FE analysis the decision on the appropriate constitutive model is of less importance when performing stability analysis. For practical cases it can be shown that simple ideal-elastic-ideal-plastic models using the same failure criterion result in the same global safety as more complex models considering for example isotropic strain hardening. It can be concluded that the detailed stress path influenced by the individual soil models is of minor importance compared to the definition of the ultimate condition.

REFERENCES

Besler, D., 1998. Wirklichkeitsnahe Erfassung der Fußauflagerung und des Verformungsverhaltens von gestützten Baugrubenwänden. Schriftenreihe des Lehrstuhls Baugrund-Grundbau der Universität Dortmund, Heft 22.

Borchert, K. 2008. Innerstädtische Baugruben, Erfahrung zur Verformungen und Dichtigkeit, Bauen in Boden und Fels, Esslingen, Technische Akademie.

Brinch Hansen, B. Lundgren, H. 1960. Hauptprobleme der Bodenmechanik, Berlin, Springer-Verlag.

EAB 2008, Recommendations on Excavations, 2nd English Edition, Ernst & Sohn.

Empfehlungen des Arbeitskreises Baugruben (EAB), 2006. 4. Auflage, Berlin, Verlag Ernst und Sohn.

Grabe, J., Schürmann, B., Katzmann, A. 2008. Zur plastischen Berechnung von Baugrubenwänden, Bundesanstalt für Wasserbau, Kolloquium "Anwendung der Finite-Elemente-Methode" Karlsruhe, BAW.

Hettler, A. 2009. Einbindetiefen und Schnittgrößen bei gestützten Wänden mit beliebiger Fußauflagerung. Bautechnik, Heft 9, pp. 542–546.

Hettler, A. 2010. Bodenmechanische Herausforderungen bei tiefen Baugruben. RUB RuhrGeo Tag, Schriftenreihe des Lehrstuhls für Grundbau Boden- und Felsmechanik, Heft 43, Ruhr-Universität Bochum.

Hettler, A., Borchert, K. 2010. Herstellungsbedingte Verformungen bei tiefen Baugruben. Baugrundtagung 2010, München Deutsche Gesellschaft für Geotechnik.

Hettler A., Gutjahr, St. Vega-Ortiz, S. 2005. Nichtlinearer Bettungsansatz von Besler bei Baugrubenwänden, Bautechnik 82 (2005), Heft 9, pp. 593–604.

Hettler, A., Maier, T. 2004. Verschiebungen des Bodenauflagers bei Baugruben auf der Grundlage der Mobilisierungsfunktion von Besler. Bautechnik 81, Heft 5, pp. 323–336.

Hettler, A., Schanz, T. 2008. Zur Anwendung der Finite-Elemente-Methode bei Baugrubenwänden. Bautechnik 85, Heft 9, pp. 603–615.

Hettler, A., Triantafyllidis, Th. 2009. Deformations of deep excavation walls by construction process. 17th ICSMGE Tagung (DGGT), Alexandria, 5.9.-9.9.09.

Schanz, T. 2006a. Aktuelle Entwicklungen bei Standsicherheits- und Verformungsberechnungen in der Geotechnik. Geotechnik 29 (1), pp. 13–27.

Schanz, T. 2006b. Standsicherheitsberechnungen von Baugruben—Berechnungsbeispiele, Geotechnik, 29 (4), pp 359–368.

Triantafyllidis, T. 1998. Neue Erkenntnisse aus Messungen an tiefen Baugruben am Potsdamer Platz in Berlin, Bautechnik 75, pp. 133–154.

Triantafyllidis, T. 2000. Ein einfaches Modell zur Abschätzung von Setzungen bei der Herstellung von Rüttelinjektionspfählen, Bautechnik 77, pp. 161–168.

Vogt N. 1984. Erdwiderstandsermittlung bei monotonen und wiederholten Wandbewegungen in Sand. Baugrundinstitut Stuttgart, Mitteilung 22. Stuttgart.

Weissenbach, A., Hettler, A., Simpson, B. 2003. Stability of excavations. Geotechnical Engineering Handbook. Vol. 3, Ernst & Sohn, Berlin.

Weißenbach, A., Hettler, A. 2010. Baugruben—Berechnungsverfahren, Ernst & Sohn.