PART I: RESULTS OF BENCHMARKING
PART II: REFERENCE SOLUTION AND PARAMETRIC STUDY

Helmut F. Schweiger

Institute for Soil Mechanics and Foundation Engineering
Graz University of Technology, Austria
# CONTENTS

## PART I: RESULTS OF BENCHMARKING

ABSTRACT ................................................................................................................................. Page 3

1 INTRODUCTION .................................................................................................................. Page 3

2 BENCHMARKING IN GEOTECHNICS ............................................................................. Page 4

3 PROBLEM SPECIFICATION: DEEP EXCAVATION IN BERLIN SAND .................. Page 5
   3.1 Assumptions and geometry .......................................................................................... Page 5
   3.2 Material parameters ..................................................................................................... Page 7

4 COMMENTS ON SUBMITTED SOLUTIONS .................................................................... Page 8

5 COMPARISON OF RESULTS .......................................................................................... Page 11
   5.1 Comparison of all analyses submitted .......................................................... Page 11
   5.2 Comparison of selected results ........................................................................... Page 13
      5.2.1 Construction stage groundwater lowering to -17.90 m .................... Page 13
      5.2.2 Construction stage first excavation step to -4.80 m ....................... Page 14
      5.2.3 Final construction stage .............................................................................. Page 16
   5.3 Comparison elastic-plastic vs. elastic-perfectly plastic analyses .................. Page 20

6 SUMMARY AND CONCLUSION ....................................................................................... Page 24

7 REFERENCES ....................................................................................................................... Page 24

## PART II: REFERENCE SOLUTION AND PARAMETRIC STUDY

ABSTRACT ................................................................................................................................. Page 26

1 INTRODUCTION .................................................................................................................. Page 26

2 PROBLEM DEFINITION ................................................................................................. Page 27
   2.1 Geometry, basic assumptions and computational steps ................................ Page 27
   2.2 Brief description of PLAXIS Hardening Soil model ........................................ Page 29
   2.3 Material parameters for Hardening Soil model .................................................. Page 31
   2.4 Material parameters for structural elements ......................................................... Page 31
3 RESULTS FOR REFERENCE SOLUTION .......................................................... Page 32

4 INFLUENCE OF VARIOUS MODELLING ASSUMPTIONS ................................. Page 34
   4.1 Influence of groundwater lowering .......................................................... Page 34
   4.2 Influence of wall friction ......................................................................... Page 36
   4.3 Influence of discretisation ....................................................................... Page 37
   4.4 Influence of domain analysed .................................................................. Page 39
   4.5 Influence of modelling ground anchors ................................................... Page 40
   4.6 Influence of free anchor length ............................................................... Page 41
   4.7 Influence of anchor prestress force .......................................................... Page 43
   4.8 Influence of wall thickness ....................................................................... Page 44
   4.9 Influence of variation in stiffness and strength parameters of soil ........ Page 45
   4.10 Wall deflection and anchor forces for all construction stages .......... Page 48

5 INFLUENCE OF CONSTITUTIVE MODEL .................................................... Page 50
   5.1 Elastic analysis ....................................................................................... Page 50
   5.2 Elastic - perfectly plastic analyses (Mohr-Coulomb) .............................. Page 53
      5.2.1 MC-model 1 and 2 ............................................................................ Page 53
      5.2.2 MC-model 3 and 4 ............................................................................ Page 55
      5.2.3 Parameter variation with MC-model 1 ............................................. Page 57

6 SUMMARY AND CONCLUSION .................................................................... Page 59
PART I: RESULTS OF BENCHMARKING

ABSTRACT

In part I of this report the results of a benchmarking exercise are presented. A deep excavation problem in Berlin sand has been specified by the working AK 1.6 of the German Society for Geotechnics and sent to various university institutes and consulting companies known to deal with numerical analysis in practical geotechnical engineering. The results summarized in this report clearly emphasize the need for these types of exercises in order to improve the reliability and validity of numerical models. The wide range of results submitted is by no means acceptable and therefore it is argued that guidelines and recommendations have to be formulated in order to assist numerical analysts in avoiding unrealistic modelling assumptions, which may lead to consequences the user of a particular code may not be aware of.

1 INTRODUCTION

The significant progress made in the understanding of the behaviour of geomaterials would not have been possible without the use of numerical methods. In particular, developments in constitutive modelling are closely related to advances made in the field of numerical analysis and therefore finite element (and other) methods have had a significant impact on geotechnical research since the 1970s. However, numerical methods have not been widely used in practical geotechnical engineering, possibly with a few exceptions, such as tunnelling, where these methods have a long tradition also in practice, at least in some parts of the world. But this has changed dramatically over the last decade. Developments in computer hardware and, more importantly, in geotechnical software enable the geotechnical engineer to perform very advanced numerical analyses at low cost and with relatively little computational effort. Commercial codes, fully integrated into the PC-environment, have become so user-friendly that little training is required for operating the programme. They offer sophisticated types of analysis, such as fully coupled consolidation analysis with elasto-plastic material models. However, for performing such complex calculations and obtaining sensible results a strong background in numerical methods, mechanics and, last but not least, theoretical soil mechanics is essential. This is sometimes overlooked in practice because glossy brochures give the impression that achieving reliable results is as easy as operating the programme and this is certainly not true.

The potential problems arising from the situation that geotechnical engineers, not sufficiently trained for that purpose, perform complex numerical analyses and may produce unreliable results have been recognized within the profession and some national and international committees have begun to address this problem, amongst them the working group AK 1.6 "Numerical Methods in Geotechnics" of the German Society for Geotechnics (DGGT) and working group A "Numerical Methods" of the COST Action C7 (Co-Operation in Science and Technology of the European Union). One of the main goals of
AK 1.6 of the DGGT is to provide recommendations for numerical analyses in geotechnical engineering. The group has published general recommendations (Meissner 1991), recommendations for numerical simulations in tunnelling (Meissner 1996) and recommendations for deep excavations will be published shortly. In addition benchmark examples are specified and the results obtained by various users employing different software are compared. It is claimed that these efforts will improve the validity of numerical models in geotechnics. More recently the problem has also been addressed by Potts and Zdravkovic (2001), Carter et al. (2000) and an initiative has been started via a bulletin of a commercial software company by Schweiger (2001).

In this report solutions for a benchmark problem, namely a deep excavation in Berlin sand, specified by the AK 1.6 of the DGGT, will be presented. The specification was sent to various research institutions and consulting companies known to be involved in numerical analysis of practical problems. A number of solutions have been submitted and the results of this exercise are discussed in part I of this report.

In order to study various modelling assumptions without the influence of code specific implementational details a comprehensive parametric study has been performed in addition to the benchmark exercise utilizing one particular code only (see part II of this report).

2 BENCHMARKING IN GEOTECHNICS

Relatively little attention has been paid in the literature on validation and reliability of numerical models in general and on specific software in particular, although some attempts have been made (e.g. Schweiger 1991, Schweiger 1998, Schweiger 2000). So far three benchmark examples have been specified by the working group AK 1.6 of the DGGT and discussed in two workshops. The first two of them, a tunnel excavation and a deep excavation problem, have been rather idealized problems with very tight specifications so that little room for interpretation was left to the analysts. Despite that significant differences in the results were obtained even in cases where the same software has been utilized by different users (Schweiger 1998). Based on these results and the ones which will be shown in this report it is argued that there is a strong need for defining guidelines and procedures to arrive at reliable numerical models in practical geotechnical engineering. Another aspect, equally important but not addressed in detail here, is the identification of appropriate input parameters from available experiments.

Benchmarking is therefore of significant importance in geotechnical engineering, probably more so than in other disciplines such as e.g. structural engineering. The reason for that may be summarized as follows

- the domain to be analysed is generally not clearly defined by the structure
- it is not always clear whether continuum or discontinuum models are more appropriate for the problem at hand
- a wide variety of constitutive models exist in the literature but there is no "approved" model for each type of soil
- in most cases construction details cannot be modelled very accurately in time and space (e.g. 2D-modelling of excavation sequence, anchors etc.) within the financial and time constraints given in practice
- soil/structure interaction is often important and may lead to numerical problems (e.g. certain types of interface elements)
- implementation details and solution procedures may have a significant influence on the results of certain problems but may not be important for others
- there are no approved implementation and solution procedures for commercial codes (implicit vs. explicit strategies, return algorithms etc.)

All of these aspects leave ample room for personal preferences both on developers and users side respectively. From a practical point of view this situation is by no means desirable because it proves to be very difficult to obtain consistent results due to the numerous assumptions involved in establishing a numerical model for a given practical problem. Modelling techniques vary depending on the personal experience of the user and, to a certain extent, also on the code utilized.

3 PROBLEM SPECIFICATION: DEEP EXCAVATION IN BERLIN SAND

3.1 Assumptions and geometry

The example is closely related to an actual project in Berlin. Slight modifications have been introduced in modelling the construction sequence such as the groundwater lowering which has been performed in various steps in situ but is modelled here in one step prior to excavation. The choice of constitutive model to be used has been left to the analysts. Some basic material parameters have been taken from the literature and additionally results from one-dimensional compression tests on loose and dense samples have been given together with triaxial tests on dense samples. As it is the aim of this section to demonstrate the necessity of performing these kind of exercises only the most relevant part of the specification will be given here (Figure 1).

The main goal of this exercise was to demonstrate potential difficulties in obtaining reliable and consistent results for a typical problem in engineering practice. Thus the following has been considered when specifying the problem:
- limited data on material properties of the soil (Berlin sand in this case) have been provided (in practice hardly ever all data required for numerical analysis are given in the geotechnical report)
- no analytical solution exists for the problem, i.e. the "true" solution is not known
the problem is related to a project actually constructed, so the order of magnitude of horizontal displacements of the wall is known from in situ measurements. 
- no restraints are imposed with respect to the constitutive model, discretisation, element types etc.

Thus the benchmarking exercise did not aim at the verification of particular software packages for solving a problem with a known solution but to see what range of numerical solutions for a geotechnical problem can be expected under conditions typically found in practice. For these reasons neither programme names nor authors of solutions submitted are disclosed in this report (they are denoted by B1 to B17 throughout this report).

Fig. 1 Geometry of benchmark example deep excavation in Berlin sand

General assumptions postulated:
- plane strain
- influence of diaphragm wall construction is neglected, i.e. initial stresses without wall, then wall "wished-in-place" (weight of wall e.g. as difference to soil weight)
- diaphragm wall modelling: beam or continuum elements
- interface elements between wall and soil (wall friction is specified as $\varphi/2$)
- domain to be analysed (suggested: see Figure 1)
- horizontal hydraulic cut off at -30.00 m not to be considered as structural support
  (same mechanical properties as surrounding soil)
- given anchor forces are design loads

The following computational stages had to be performed in order to simulate construction:

stage 0: initial stress state is given by $\sigma_v = \gamma z$, $\sigma_h = K_o \gamma z$
  (wall "wished-in-place", deformations = 0)
stage 1: groundwater-lowering to -17.90 m
stage 2: excavation step 1 (to level -4.80 m)
stage 3: activation of anchor 1 at level -4.30 m and prestressing
stage 4: excavation step 2 (to level -9.30 m)
stage 5: activation of anchor 2 at level -8.80 m and prestressing
stage 6: excavation step 3 (to level -14.35 m)
stage 7: activation of anchor 3 at level -13.85 m and prestressing
stage 8: excavation step 4 (to level -16.80 m)

Distance and prestressing loads for anchors follow from Figure 1.

3.2 Material parameters

Some reference values for stiffness and strength parameters from the literature, frequently used in the
design of excavations in Berlin sand, were given (z = depth below surface):

$$E_s \approx 20 000 \sqrt{z} \text{ kPa for } 0 < z < 20 \text{ m}$$
$$E_s \approx 60 000 \sqrt{z} \text{ kPa for } z > 20 \text{ m}$$
$$\varphi = 35^\circ$$
$$\gamma = 19 \text{ kN/m}^3$$
$$\gamma' = 10 \text{ kN/m}^3$$
$$K_o = 1 - \sin \varphi$$

The given (one-dimensional) compression moduli $E_s$ may be used to arrive at approximate values for
the Young's modulus for calculations with linear elastic - perfectly plastic material models (Mohr-
Coulomb) assuming an appropriate Poisson's ratio.

In addition to these values from literature, results from oedometer tests (on loose and dense samples)
and triaxial tests (confining pressures $\sigma_3 = 100, 200$ and $300 \text{ kPa}$) have been provided. It was not
possible to include a significantly large number of test results and thus the question arose whether the
stiffness values obtained from the oedometer test have been representative. If, for example, the
constitutive model requires a tangential oedometric stiffness at a reference pressure of 100 kPa as an
input parameter, a value of only $E_s \approx 12\,000\,\text{kPa}$ was found based on these experiments. If a secant modulus for a pressure range beyond 200 kPa is determined a value of about 40,000 kPa is obtained. This was considered as too low by many authors and indeed other test results from Berlin sand in the literature indicate higher values. For example from Ohde (1951) values of about 35,000 to 45,000 kPa could be estimated as reference loading modulus of a medium dense sand at a reference pressure of 100 kPa. However, sample disturbance and uncertainties in laboratory testing cannot be neglected, at least not in standard procedures usually performed for practical purposes, and therefore it has to be carefully judged whether stiffness values obtained in the laboratory should be used in numerical analysis without correction. These problems of determining appropriate stiffness parameters for numerical analyses are by no means desirable, but unfortunately it represents the situation in practice where geotechnical investigations and geotechnical reports often do not satisfy the requirements for numerical analysis. It can be anticipated however that more refined experimental investigations, including the measurement of stiffness at very small strains, will be employed increasingly for practical purposes and thus provide more reliable data for numerical analysis.

Properties for the diaphragm wall (linear elastic):

- $E = 30\,000\times 10^3\,\text{kPa}$
- $\nu = 0.15$
- $\gamma = 24\,\text{kN/m}^3$

4 COMMENTS ON SUBMITTED SOLUTIONS

A wide variety of programmes and constitutive models has been employed to solve this problem. Simple elastic-perfectly plastic material models such as the Mohr-Coulomb or Drucker-Prager failure functions (B1, B4, B5, B6, B7, B9, B12 and B16), still widely used in practice have been chosen by a number of authors. Several entries utilized the computer code PLAXIS (Brinkgreve & Vermeer 1998) with the so-called Hardening Soil model. This constitutive model is based on the well known formulation by Duncan & Chang (1970), but formulated within the theory of plasticity. It incorporates shear hardening and volumetric hardening, a stress dependent stiffness for primary loading and unloading/reloading and the stress dilatancy theory by Rowe (1962). One submission used a similar plasticity model with a simplified small strain stiffness formulation for the elastic range (B14). Three entries employed a hypoplastic formulation (B3, B3a and B13), B3 without and B3a and B13 with considering intergranular strains (Niemunis & Herle 1997).

Only marginal differences exist in the assumptions of strength parameters (everybody trusted the experiments in this respect), the angle of internal friction $\phi$ was taken as 36° or 37° and a small cohesion was assumed to increase numerical stability by some authors. A significant variation was observed however in the assumption of the dilatancy angle $\psi$, ranging from 0° to 15°.
For reasons mentioned earlier only a limited number of analysts used the provided laboratory test results to calibrate their material model. Most of the analysts used data from the literature from Berlin sand or their own experience to arrive at input parameters for their analysis assuming an increase with depth either by introducing some sort of power law similar to the formulation presented by Ohde (1951) which in turn corresponds to the formulation by Janbu (1963), or by defining different layers with different (constant) Young’s moduli. However the choice of the reference moduli for primary loading and unloading/reloading varied significantly. Table 1 summarizes the input parameters and other details of the numerical models for all analyses submitted. Some entries sent two calculations with different parameter sets or constitutive models, they are denoted by e.g. B3 and B3a respectively. The table clearly demonstrates how differently various authors interpreted the information provided in order to arrive at input parameters. It is apparent that most of them did not trust the low stiffness values from the oedometer test and increased the values, giving various reasons based on their experience. Some argued that the values are too low but still used them in the analysis (e.g. B17). Additional variation (see also Table 1) was introduced through different formulations for interface elements (zero thickness, finite thickness), element types (linear, quadratic), domains analysed (the width of meshes varied from 80 to 160 m, the depth from 50 to 160 m), modelling of the prestressed anchors, implementation details of constitutive models and the solution procedure with respective convergence criteria. The latter aspect is commonly ignored in practice but it can be easily shown that it may have a significant influence not only for stress levels near failure but also for working load conditions (Potts and Zdravkovic, 1999).

Some of the analyses ignored parts of the specification given, e.g. B15 performed a stepwise lowering of the groundwater table inside the excavation, which yields about 10 mm less horizontal displacements as compared to a one-step lowering. Furthermore, some small errors with respect to the unit weight have been discovered when examining the results and more importantly the prestressing of the anchors has not been modelled correctly by some of the authors. Despite that findings it was not possible to clarify all details of the analyses submitted from the information provided and thus the discrepancies in results could not be fully explored. In addition it turned out that some of the assumptions made in a model could be counteracted by another one made, and therefore it was not possible to isolate specific assumptions leading to a particular result.

In view of these comments one may argue that due to the different numerical models employed for solving this problem a comparison of these results is not meaningful at all. However, one cannot ignore the fact that this is the reality in practice. It is by no means acceptable that the results of an analysis for a given problem show a "user dependent" scatter of 100% and more.

It is the main purpose of this report to increase awareness that results from numerical analyses are very sensitive not only to the constitutive model and material parameters but also to modelling and implementation details, the user may not consider important. Again the strong need for guidelines and recommendations is emphasized.
<table>
<thead>
<tr>
<th>analysis</th>
<th>constitutive model</th>
<th>stiffness</th>
<th>reference stiffness (loading / unloading) [kPa]</th>
<th>$\varphi$ [°]</th>
<th>$\psi$ [°]</th>
<th>domain analysed width x depth [m]</th>
<th>element type wall</th>
<th>interface</th>
<th>note</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>elastic-perfectly plastic</td>
<td>stress dependent</td>
<td>$z &lt; 20$ m: 14 900, $z &gt; 20$ m: 44 700</td>
<td>35</td>
<td>5</td>
<td>100 x 64</td>
<td>quadratic</td>
<td>9 noded</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>yes</td>
<td>-</td>
</tr>
<tr>
<td>B2 / B2a</td>
<td>elastic-plastic ($z &lt; 40$ m)</td>
<td>stress dependent</td>
<td>$z &lt; 40$ m: 15 000 / 39 000 (B2), $z &lt; 40$ m: 60 080 / 180 000 (B2a), $z &gt; 40$ m: 253 000 (B2a), 227 000 (B2a)</td>
<td>36</td>
<td>6</td>
<td>100 X 100</td>
<td>quadratic</td>
<td>beam</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td>elastic-perfectly plastic ($z &gt; 40$ m)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B3 / B3a</td>
<td>hypoplastic without (B3) with intergranular strains (B3a)</td>
<td>constant, but 6 layers &gt; increase with depth</td>
<td>$z &lt; 2$ m: 10 500, $102 &lt; z &lt; 107$ m: 457 000</td>
<td>35</td>
<td>0</td>
<td>105 x 107</td>
<td>linear</td>
<td>4 noded</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B4</td>
<td>elastic-perfectly plastic</td>
<td>43 layers &gt; increase with depth</td>
<td>$z &lt; 2$ m: 10 500, $102 &lt; z &lt; 107$ m: 457 000</td>
<td>35</td>
<td>0</td>
<td>105 x 107</td>
<td>linear</td>
<td>4 noded</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B5</td>
<td>elastic-perfectly plastic</td>
<td>constant, but 6 layers &gt; increase with depth</td>
<td>$z &lt; 5$ m: 32 600, $32 &lt; z &lt; 60$ m: 303 000</td>
<td>35</td>
<td>15</td>
<td>80 x 60</td>
<td>-</td>
<td>beam</td>
<td>no</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B6</td>
<td>elastic-perfectly plastic</td>
<td>constant, but 20 layers &gt; increase with depth</td>
<td>$z &lt; 20$ m: 20 000 $\sqrt{z}$, $z &gt; 20$ m: 44 700 $\sqrt{z}$</td>
<td>35</td>
<td>15</td>
<td>122 x 90</td>
<td>-</td>
<td>-</td>
<td>error in prestress force</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B7</td>
<td>elastic-perfectly plastic</td>
<td>constant</td>
<td>60 000</td>
<td>40.5</td>
<td>13.5</td>
<td>90 x 60</td>
<td>quadratic</td>
<td>yes</td>
<td>c = 2.5 kPa capillary cohesion</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B8</td>
<td>elastic-plastic</td>
<td>stress dependent</td>
<td>$z &lt; 20$ m: 20 000 / 74 400, $z &gt; 20$ m: 60 000 / 120 000</td>
<td>35</td>
<td>10</td>
<td>90 x 70</td>
<td>quadratic</td>
<td>beam</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B9</td>
<td>elastic-perfectly plastic</td>
<td>constant, but 3 layers &gt; increase with depth</td>
<td>$z &lt; 20$ m: 39 400, $z &gt; 40$ m: 310 000</td>
<td>35</td>
<td>5</td>
<td>150 x 100</td>
<td>quadratic</td>
<td>beam</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td>elastic-perfectly plastic</td>
<td>stress dependent</td>
<td>25 000 / 100 000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B9a</td>
<td>elastic-perfectly plastic</td>
<td>constant, but 3 layers &gt; increase with depth</td>
<td>25 000 / 100 000</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B10</td>
<td>elastic-plastic</td>
<td>stress dependent</td>
<td>60 000 / 180 000</td>
<td>36</td>
<td>6</td>
<td>100 x 72</td>
<td>quadratic</td>
<td>beam</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B11</td>
<td>elastic-plastic</td>
<td>stress dependent</td>
<td>$z &lt; 20$ m: 20 000 / 100 000, $z &gt; 20$ m: 60 000 / 300 000</td>
<td>35</td>
<td>0</td>
<td>150 x 120</td>
<td>quadratic</td>
<td>beam</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B12</td>
<td>elastic-perfectly plastic</td>
<td>constant, but 9 layers &gt; increase with depth</td>
<td>$z = 5$ m: 23 000, $42 &lt; z &lt; 92$ m: 365 000</td>
<td>35</td>
<td>4</td>
<td>90 x 92</td>
<td>quadratic</td>
<td>beam</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B13</td>
<td>hypoplastic with intergran. strains</td>
<td>constant, but 9 layers &gt; increase with depth</td>
<td>$z = 5$ m: 23 000, $42 &lt; z &lt; 92$ m: 365 000</td>
<td>35</td>
<td>4</td>
<td>90 x 92</td>
<td>quadratic</td>
<td>beam</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B14</td>
<td>elastic-plastic with small strain stiffness</td>
<td>stress / strain dependent</td>
<td>$G_{\text{min}} = 30 000$, $G_{\text{near strain}} = 240 000$</td>
<td>35</td>
<td>5</td>
<td>120 x 100</td>
<td>quadratic</td>
<td>8 noded</td>
<td>continuum</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B15</td>
<td>elastic-plastic (0-20 m)</td>
<td>stress dependent</td>
<td>$z &lt; 20$ m: 32 000 / 96 000, $z &gt; 20$ m: 192 000 / 384 000</td>
<td>35</td>
<td>7</td>
<td>95 x 50</td>
<td>quadratic</td>
<td>beam</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td>elastic-plastic (&gt;20 m)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B16</td>
<td>elastic-perfectly plastic</td>
<td>constant, but 5 layers &gt; increase with depth</td>
<td>$0 &lt; z &lt; 3$ m: 13 000, $92 &lt; z &lt; 122$ m: 456 000</td>
<td>35</td>
<td>11.7</td>
<td>120 x 122</td>
<td>quadratic</td>
<td>8 noded</td>
<td>continuum</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B17</td>
<td>elastic-plastic</td>
<td>stress dependent</td>
<td>20 000 / 47 000</td>
<td>35</td>
<td>5</td>
<td>130 x 100</td>
<td>quadratic</td>
<td>continuum + beam</td>
<td>yes</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
5 COMPARISON OF RESULTS

5.1 Comparison of all analyses submitted

Figure 2 shows the deflection curve of the diaphragm wall for the final excavation stage for all solutions submitted and it follows that the results scatter in a range which is by no means acceptable. The horizontal displacement of the top of the wall varies between -229 mm and +33 mm (-ve means displacement towards the excavation).

![Wall deflection at final excavation stage for all analyses submitted](image-url)
Looking into more detail of Figure 2 it can be observed that entries B2, B3, B3a, B9a, B7 and B17 are extremely off the "mainstream" of results. B2, B3, B3a, B9a and B17 are the ones which derived there input parameters mainly from the provided oedometer tests, which however showed very low stiffness as compared to values given in the literature. B7 was the only analysis using a constant Young’s modulus together with a Mohr-Coulomb failure criterion. As mentioned previously, some of the analysis did not show the correct prestress force in the respective construction stage, but not all of these showed a similar trend in behaviour. It was not possible to identify groups of analyses showing a similar deformation pattern with comparable input assumptions. Some others had small errors in the specific weight but these cannot account for the large differences. Even if the aforementioned six analyses are ignored the differences in magnitude of horizontal displacements and shapes of the deflection curves are striking.

Figure 3 shows the vertical displacements of the ground surface behind the wall. B7 calculates a surface heave of more than 40 mm, confirming the well known fact that elastic-perfectly plastic constitutive models with constant Young’s modulus are not suitable for predicting the correct pattern of deformations for these types of problems, in particular with respect to settlements behind the wall. On the contrary B3, a hypoplastic solution without consideration of intergranular strains, calculates settlements of approximately 275 mm.

Because the aforementioned six analyses were significantly out of the ranges as compared to all other solutions they are no longer considered in the further examination of the results.
5.2 Comparison of selected results

5.2.1 Construction stage groundwater lowering to -17.90 m

Figure 4 depicts lateral displacements of the diaphragm wall due to lowering of the groundwater level inside the excavation pit to -17.90 m below surface. Again no clear trend e.g. with respect to the constitutive model could be identified, B6 is an elastic-perfectly plastic model but so is B16, both on the opposite sides of the range of results. Observing this variety of results already in the first construction stage, it is of course not surprising that the scatter increases with further calculation steps as shown in Figure 2.

![Fig. 4 Wall deflection after groundwater lowering](image-url)
It should be emphasized at this stage that not only the assumption of the constitutive model and the parameters have a significant influence on the result of this construction stage but also the way the groundwater lowering is simulated in the numerical analysis. Again programme specific implementation details, the commercial user of a particular software may not be aware of, will contribute to the differences shown in Figure 4. The corresponding surface displacements depicted in Figure 5 show differences in settlements behind the wall from approximately 2 to 30 mm.

5.2.2 Construction stage first excavation step to -4.80 m

Because of possible differences in modelling the groundwater lowering depending on the software used, it was investigated whether a more clear picture would evolve if a construction stage without the influence of the groundwater lowering is considered. For that purpose the wall deflection for excavation step 1 (to -4.80 m below surface) was plotted setting displacements to zero before this construction stage. The result follows from Figure 6 and the significant scatter already at this stage is obvious. Although most of the differences can be attributed to the stiffness parameters chosen as input, a few additional conclusions can be drawn. The largest horizontal displacement is obtained from the hypoplastic analysis, which was not the case in the previous construction stage (groundwater lowering). This indicates the strong response of these models on the stress paths, which are obviously quite different for these two construction steps. The effect of different stress paths is also observed in the other models but by far not to the same extent. The elastic-plastic models with stress dependent
stiffness (B2a, B8, B10 and B14) tend to give smaller displacements compared to the elastic-perfectly plastic models. Exceptions are B5 and B16, which show a distinctly different deflection curve although the Young's modulus chosen is similar to other entries. Most probably is due to the fact that they did not use an interface element for modelling the soil/wall interaction.

Fig. 6 Wall deflection for first excavation step
5.2.3 Final construction stage

Limited in situ measurements are available for this project and although some simplifications compared to the actual construction have been introduced for this benchmark exercise in order to facilitate the calculations, the order of magnitude of displacements can be assumed to be known. Figure 7 shows the measured wall deflection for the final construction stage together with calculated values.

It should be mentioned that measurements have been taken by inclinometer readings, fixed at the base of the wall, but unfortunately no geodetic survey of the wall head is available. It is very likely that the wall base moves horizontally and a parallel shift of the measurement of about 5 to 10 mm is
thought to reflect the in situ behaviour with reasonable accuracy, and therefore the measurement readings have been shifted by 10 mm in Figure 7. This is confirmed by other measurements under similar conditions.

The calculated maximum horizontal wall displacement for all results considered varies between approximately 10 to 65 mm (exception B6). The shape of the deflection curves is also quite different. Some results indicate the maximum displacement slightly above the final excavation level, others show the maximum value at the top of the wall. When comparing the results of the calculations with the measurements it has to be pointed out that the simplification introduced in modelling the groundwater lowering (one step lowering instead of step-wise lowering according to the excavation progress) leads to higher horizontal displacements. Further studies revealed that the difference in calculated horizontal displacements due to the difference in modelling the groundwater lowering is strongly dependent on the constitutive law employed and ranges in the order of 5 to 15 mm (see also part II of this report). This may be one of the reasons why B15, which is an elastic-plastic analysis with stepwise groundwater lowering, is close to the measurement, but it also means that all solutions predicting less than 30 mm of horizontal displacement are far off reality.

Figure 8 depicts the calculated surface settlements. Settlements of 45 mm (B11) have to be compared with a heave of about 15 mm (B4). Considering the fact that calculation of surface settlements is one of the main goals of such an analysis these results are not very encouraging.
Bending moments are presented in Figure 9 and again a significant scatter in maximum bending moments is observed, but also some rather peculiar distribution of bending moments over the length of the wall (in particular B13 and B16). B13 could be due to the fact that the anchors were fixed to the lateral boundary of the mesh and in B16 the reason could be an incorrect calculation of bending moments based in the stresses calculated in continuum elements. The consequences for design are obvious, with bending moments varying in the range of several 100%.

Fig. 9 Bending moments for final excavation step
Figure 10 compares calculated anchor forces at the final construction stage for all anchor rows. The significant differences cannot be attributed to the previously mentioned errors in defining the prestress forces only but are a result of all modelling assumptions made.

Figure 11 shows the development of anchor forces in the first row of anchors with progressing excavation. Three entries took the given prestress force as kN/m instead of kN, i.e. they had a significantly higher prestress force in their anchors (B4, B11, B12). One would assume that their calculated horizontal displacements of the wall would be small. However, that this is not necessarily true follows from examination of Figure 7. Another problem became apparent, namely the way the different programmes handle the prestressing of anchors. In some of the analyses part of the prestressing force is lost due to deformations occurring in this particular construction stage thus the design prestress force is not correctly taken into account in the analysis (B1, B5 and B6).
5.3 Comparison elastic-plastic vs. elastic-perfectly plastic analyses

It has been mentioned before that a number of assumptions have to be made when setting up a numerical model for a given problem. This not only involves obvious decisions such as the choice of the constitutive model and their input parameters but also the domain to be analysed, the element type used for representing soil and structural elements and so on. These latter modelling details are often considered to be of minor influence, at least in practice, where in general the suggestions of the user’s manual are followed. However, the sum of all these details may have a significant influence on the results as has been shown in the previous sections. In order to attempt to categorize results at least roughly, results from analyses employing elastic-perfectly plastic and elastic-plastic constitutive models are plotted separately.

In Figure 12 horizontal displacements of the diaphragm wall are shown for the final construction stage for all analyses assuming elastic-perfectly plastic material behaviour for the sand (Mohr Coulomb: B1, B5, B6, B9 and B12, Drucker Prager: B4 and B16). It follows from this diagram that not even in the shape of the deflection curves similarities between the individual solutions can be observed. It can be concluded that the assumptions made with respect to the increase of Young’s modulus with depth, which in these models is somewhat arbitrarily done by introducing a number of soil layers with constant stiffness in each layer, significantly influence not only the magnitude of calculated...
displacements but also the shape of the deflection curve. Of course other assumptions such as choice of interface elements, domain considered play a role too.

Figure 13 reveals that significant surface settlements are obtained only from analyses B1, B9 and B12 whereas the others show only marginal settlements or significant heave behind the wall. With the exception of B16, which shows large horizontal settlements but heave of the surface, the predicted heave corresponds to small lateral displacements. Interesting is the fact that by comparison of analyses B1 and B12 one can see that they produce very similar settlement troughs behind the wall but have significantly different shapes of the deflection curve with a large difference in the maximum horizontal displacement. The latter can be explained by the fact that B1 used much lower values for the Young's modulus and B12 had much too high anchor forces. This again emphasizes that a
number of modelling details have significant influence on displacements, but these effects vary whether horizontal or vertical displacements are looked at.

Figure 14 summarizes results obtained from elastic-plastic analysis, utilizing the same software and the same constitutive law. In this case at least the shape of the deflection curves is similar, with exception of B11, which can be explained by an error in the prestressing force (too high). B10 assumed one homogeneous layer of soil whereas the others introduced 2 layers of soil. Again it is interesting to see that B2a and B8 produce very similar horizontal displacements although the stiffness parameters introduced are quite different. However the domain considered in the analysis also differs (in particular the depth of the mesh) and the combination of these assumptions lead to very similar results. However, B2a and B8 produce quite different surface settlements. None of the elastic-plastic solutions leads to the unrealistic heave at the surface behind the wall as observed in some of the elastic-perfectly plastic analyses (Figure 15).
Fig. 14 Wall deflection for final excavation step – elastic-plastic analyses

Fig. 15 Vertical displacements of surface for final excavation step – elastic-plastic analyses
6 SUMMARY AND CONCLUSION

Results from a geotechnical benchmark exercise have been presented. A typical problem of a deep excavation in Berlin sand, formulated by the working group 1.6 of the German Society for Geotechnics, has been solved by a number of geotechnical engineers from universities and consulting companies utilizing different finite element codes and constitutive models. Only limited results from standard laboratory experiments and typical material properties for the sand were provided. Thus it is claimed that the situation one faces in practice has been addressed where in most cases insufficient and sometimes not very reliable data, at least as far as stiffness parameters are concerned, are available and it was part of the game to see how this was handled. Indeed most of the analysts did not rely on these values and chose stiffness parameters based on their experience resulting in a wide scatter of input parameters representing the stiffness in the various models. Strength parameters did not vary significantly.

Thus the comparison of the solutions submitted showed a wide scatter in results and only the most extreme solutions on the far end of the range could be explained with respect to assumptions of input parameters made in the analysis. It is clearly evident that matching laboratory experiments with a particular constitutive model and corresponding material parameters is no guarantee at all to arrive at a sensible solution for a complex boundary value problem where the soil experiences significantly different stress paths as compared to the experiments. Some of the results showed obvious errors such as incorrect prestress forces of anchors but most analyses made reasonable assumptions for parameters, discretisation and other modelling details.

This benchmark exercise demonstrates the strong need for guidelines and recommendations how to model typical geotechnical problems in practice. Pitfalls and unrealistic modelling assumptions, the commercial user may not be aware of, have to be pointed out and procedures have to be developed to identify these. A strong demand is also put on software developers to thoroughly check their codes and to clearly document their solution procedures, implementation of constitutive models, structural and interface elements and other code specific details.

7 REFERENCES


Meissner, H. 1991. Empfehlungen des Arbeitskreises 1.6 "Numerik in der Geotechnik", Abschnitt 1, Allgemeine Empfehlungen. Geotechnik, 14, 1-10. (in German)


