Activities

2005

4 - 6 January, 2005
Short course on Computational Geotechnics + Dynamics (English)
New York, USA

7 January, 2005
1st North American Plaxis User Meeting

17 - 19 January 2005
International Course Computational Geotechnics (English)
Northwich Bank, The Netherlands

24 - 26 January 2005
Geosciences
Austin, Texas

1 - 2 March 2005
Plaxis seminars in London and Manchester
United Kingdom

7 - 9 March 2005
Short course on Computational Geotechnics (German)
Stuttgart, Germany

21 - 24 March 2005
International Course for Experienced Plaxis Users (English)
Northwich Bank, The Netherlands

May 2005
Short course on Computational Geotechnics (Italian)
Napoli, Italy

7 - 12 May 2005
ITA-AITES 2005
Istanbul, Turkey

17 - 20 May 2005
Short course on Computational Geotechnics (Spanish)
Bogotá, Colombia

19 May 2005
French User Meeting
Paris, France

15 - 17 June 2005
5th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground
Amsterdam, The Netherlands

19 - 22 June 2005
11th IACMAG
Turin, Italy

August 2005
Short course on Computational Geotechnics (English)
Houston, USA

August 2005
Short course on Computational Geotechnics (English)
Sydney, Australia

12 - 16 September 2005
12th International Conference on Soil Mechanics and Geotechnical Engineering
Osaka, Japan

24 - 27 October 2005
International Course for Experienced Plaxis Users (English)
Santiago de Querétaro, Gto. México

9 - 11 November 2005
12th European Plaxis User Meeting
Karlsruhe, Germany

November 2005
Short course on Computational Geotechnics (French)
Paris, France

CONTROL OF GROUND MOVEMENTS FOR A MULTI-LEVEL-ANCHORED DIAPHRAGM WALL DURING EXCAVATION

Deep excavation in soft soils and complex groundwater conditions in Bogotá
Application of the Random Set Finite Element Method (RS-FEM) in Geotechnics
The Plaxis Bulletin is the combined magazine of Plaxis B.V. and the Plaxis Users Association (NL). The Bulletin focuses on the use of the finite element method in geotechnical engineering practise and includes articles on the practical application of the Plaxis programs, case studies and backgrounds on the models implemented in Plaxis.

The Bulletin offers a platform where users of Plaxis can share ideas and experiences with each other. The editors welcome submission of papers for the Plaxis Bulletin that fall in any of these categories.

The manuscript should preferably be submitted in an electronic format, formatted as plain text without formatting. It should include the title of the paper, the name(s) of the authors and contact information (preferably email) for the corresponding author(s). The main body of the article should be divided into appropriate sections and, if necessary, subsections. If any references are used, they should be listed at the end of the article. The author should ensure that the article is written clearly for ease of reading.

In case figures are used in the text, it should be indicated where they should be placed approximately in the text. The figures themselves have to be supplied separately from the text in a common graphics format (e.g. tif, gif, png, jpg, wmf, cdr or eps formats are all acceptable). If bitmaps or scanned figures are used the author should ensure that they have a resolution of at least 300 dpi at the size they will be printed. The use of colour in figures is encouraged, as the Plaxis Bulletin is printed in full-colour.

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In Plaxis Practice, the focus this time is on deep excavations. Two projects are described in detail. The first paper describes the 23 m deep excavation for the Dana Farber research tower in Boston. Here tiebacks were used to limit the soil deformation successfully, but still surface settlements occurred, damaging nearby properties. Finite element calculations gave insight into the unexpected cause of these deformations.

Another excavation was constructed in Bogotá, Colombia. Here construction was hindered by the groundwater flow downhill into the excavation.

Accurate prediction of the behaviour of an excavation is strongly influenced by the uncertainty of the soil properties. The third article in Plaxis Practice introduces a method to handle this uncertainty using the Random Set Method and extends the deadline to June 1st, 2005. Hopefully this will encourage more Plaxis users to try on the challenge and send the results to Helmut Schweiger at Helmut.Schweiger@tugraz.at

In 2004 we released two 3D products; 3DFoundation Version 1 and 3DTunnel Version 2. Furthermore, update packs were released for the different Plaxis programs. These update packs contain both new functionality to Plaxis that was requested by users and bug fixes. Read more on this in the recent activities column. This process will continue over the coming months. What new functionality you may expect in the 3D foundation program for example is detailed in new developments.

Embedded pile element in volume element

Figure 1: Embedded pile element in volume element

After the release of the 3D Foundation program in March 2004, a great effort has been put into improvement and further developments of this program. Meanwhile Version 1 of the 3D Foundation program is used more and more by geotechnical engineers in practical applications, as well as for research problems. The program proves to be useful for many applications involving raft foundations, large diameter piles, soil reinforcements, and excavation projects. Nevertheless, there is a strong requirement for additional features. In this issue I will describe the new features we are working on and that will become available in future upgrade versions.

A missing feature in Version 1 is the K0-procedure for a quick generation of initial stresses. This well-known procedure, as available in all other Plaxis products, will also be available in the next upgrade of the Foundation program. The K0-procedure also allows for the generation of over-consolidated stress states when using advanced soil models.

In addition to the Mohr-Coulomb and the Hardening Soil model, the next upgrade will have the Soft Soil Creep model. This model allows for the calculation of time-dependent behaviour due to creep (secondary compression). Moreover, consolidation will be available as an additional type of calculation to analyse the generation and dissipation of excess pore pressures in time. These features will enhance the foundation program especially for settlement analysis of foundations in soft soils.

Although piles and pile groups can be modelled in Version 1, it has been realised that the first version is not very convenient for the evaluation of computational results for these types of structures. For piles modelled using volume elements, there will be a facility to integrate the stresses in the pile elements into structural forces (axial forces, shear forces and bending moments). Another new pile-related feature is the option to apply a radial expansion to the pile to simulate the installation effects of soil displacement piles (lateral stress increase around the pile).

In order to model a large group of slender piles in different configurations, a new one-dimensional pile element has been developed that can be placed arbitrarily in the finite element mesh (see Fig. 1). This so-called embedded pile element interacts with the surrounding volume elements using a special type of interface. Regarding the pile-soil interaction, a shear force can be specified for the interface. This shear force can vary from the pile top to the base. In addition, an additional base resistance can be given. Hence, to use the embedded pile element information is required regarding the expected maximum allowable shear stress along the pile and the expected maximum base resistance. This information can be obtained from pile loading tests for the corresponding project or from local expertise. Although the element has been implemented, further research is needed to be able to provide sufficient information to enable the appropriate use of this element in practical applications. As a result, this element will not be immediately available in the next upgrade but it will come at a later date.

In addition to the features as described above, we are working on new input and output facilities to enhance the modelling, calculation and evaluation of results for foundation projects and other types of applications. In this respect the following features will also become available:

- Enhanced water pressure generation options
- Output of plastic points
- Animations
- Stress paths and stress-strain diagrams

By the time that this bulletin is available to you, all these features will have been implemented and are being tested extensively. The next upgrade is expected to be released in the second quarter of 2005, whereas the embedded piles are expected to become available around the end of 2005.
Validation and Verification of numerical models is an important issue in computational geotechnics. The developments of Plaxis in the past five years have advanced the capabilities of numerical modelling far beyond simple elastic-perfectly plastic analysis. Very complex models involving soil/structure interaction problems can now be solved with relatively little effort and thus these analyses are perfectly feasible for daily engineering practice. However, despite the effort of providing a knowledge transfer to the users by organizing courses for beginners and advanced Plaxis users, many possible shortcomings and pitfalls of numerical analyses may not be appreciated in practice, especially under the given time and money constraints of large projects, which usually prohibit a comprehensive study of all modelling aspects to be performed.

In order to address specific problems in finite element modelling of geotechnical problems the section benchmarking is introduced. It is the aim of this section to create awareness for the sensitivity of results on particular assumptions which have to be made in numerical modelling, and which are sometimes not given sufficient attention. It is understood that this section provides an additional support to the PLAXIS community in order to improve the reliability of computational models and increase the confidence in numerical predictions.

The format of this section will be as follows: An example specification will be published and everybody interested is invited to solve the problem and send me the results. In the following bulletin some of the results will be presented, a more detailed coverage will be provided via the Plaxis Webpage.

All results will be kept strictly confidential, names of authors who submit solutions will not be disclosed neither in the bulletin nor in the Webpage!

The examples will be such that they do not require a lot of time to create the model and also run times on the computer will not be excessive, although it is anticipated that problems get slightly more difficult once this section is well established.

All results have to be presented in Excel sheets. Plaxis input files are optional. Users are kindly requested to mail the results of the benchmark to Helmut.Schweiger@tugraz.at.

Deadline for Benchmark no. 4: June 1st, 2005

The geometry of the pile load test follows from Figure 1. The tested pile has a diameter of 1.5 m and a length of 11.5 m. It is assumed that the groundwater table is below the base of the pile and can be ignored in the analysis. The soil layer is a homogeneous layer of medium dense sand and the Hardening Soil model with parameters as given in Table 1 should be used. The pile is loaded by hydraulic jacks supported by reaction anchors which can be assumed to have no influence on the load displacement behaviour of the loaded pile. The specification is based on an actual pile load test but has been simplified for this exercise. Therefore comparison will only be made between the numerical analyses submitted and not with measured data.

The following computational steps have to be performed:
- Initial stresses with $\sigma'_v = \gamma h$ and $\sigma'_h = \sigma'_v K_0$
- Installation of pile
- Loading of pile

Results to be presented:
- Load-settlement curve of pile (total resistance)
- Load-settlement curve separating tip and shaft resistance

Figure 1: Geometry of the pile load test

<table>
<thead>
<tr>
<th>Material</th>
<th>Properties</th>
<th>Sand</th>
<th>Pile</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma$</td>
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<td>21.0</td>
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<td>$c'$</td>
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<tr>
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<tr>
<td>$\psi'$</td>
<td>[$^\circ$]</td>
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<td></td>
</tr>
<tr>
<td>$K_0$</td>
<td>[-]</td>
<td>0.426</td>
<td></td>
</tr>
<tr>
<td>$\nu_{us}/\nu$</td>
<td>[-]</td>
<td>0.2</td>
<td></td>
</tr>
<tr>
<td>$E_{SHP}/L$</td>
<td>[kPa]</td>
<td>4.5E4</td>
<td></td>
</tr>
<tr>
<td>$E_{min}$</td>
<td>[kPa]</td>
<td>1.35E5</td>
<td></td>
</tr>
<tr>
<td>$m$</td>
<td>[-]</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>$p''$</td>
<td>[kPa]</td>
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<tr>
<td>$R_{min}$</td>
<td>[-]</td>
<td>0.7</td>
<td></td>
</tr>
</tbody>
</table>

Table 1: Material parameters for soil layer and pile
Control of ground movements for a multi-level-anchored diaphragm wall during excavation

Dimitrios C. Konstantakos, Mueser Rutledge Consulting Engineers, U.S.A.
Andrew J. Whittle & Carlos Regalado, Massachusetts Institute Of Technology, U.S.A.
Bernhard Scharner, Graz University of Technology, Austria

INTRODUCTION

The Dana Farber Research Tower (Smith Laboratory) is located in the Longwood Medical Area in Boston, Massachusetts. The tower has 14 above-ground stories devoted to office and research laboratory uses and five underground parking levels. Figure 1 shows the site plan, the building has a rectangular footprint (approx. 43m x 34m) and is bounded to the south by the MATEP (Medical Area Total Energy Plant) power plant, by Binney Street and Deaconess Road on the east and north sides, respectively, and by two low rise brick buildings to the west. In order to protect the delicate (and expensive) medical experiments carried out in the research tower, the basement structure is isolated from a permanent lateral earth support system comprising a 0.9m thick perimeter diaphragm wall braced by four to six levels of prestressed rock anchors.

This paper summarizes the performance of the lateral earth support system based on field monitoring data measured during excavation of the basement (January – October 1995). Back analyses are then used to evaluate and interpret the wall and ground movements.

SITE DESCRIPTION

The original site investigation comprised a series of 14 deep borings (with conventional blowcount data) within the footprint of the new building, supplemented with boring data from three adjacent structures. There was a very limited program of laboratory tests (index properties, water content and UU strength testing in the clay, and particle size distributions for the granular layers) while permeability properties were reported from borehole falling head tests.

Figures 2a and b summarize the soil profiles for the two orthogonal cross-sections A-A and B-B shown in Figure 1. The site is almost level with surface elevation El. 11.6m – 12.2m. The subsurface profile comprises 20m – 27m of post-glacial sediments overlying the Roxbury conglomerate bedrock. The bedrock was described as medium to hard, slightly weathered gray to purple, coarse-grained conglomerate with closely spaced dipping joints and RQD = 28% - 40%. The surface of the rock is quite irregular and dips from north to south across the site (Fig. 2a).

The overlying soils can be sub-divided into four main units: 1) surficial fill (up to 5m thick at the southern edge of the site), 2) low plasticity (Ip = 10—15%) marine clay (Boston Blue Clay) ranging in consistency and coloration and ranging in thickness from 10m – 17m (the upper clay contains discontinuous pockets of sand), 3) deposits of underlying sands and silts which taper to the north and south of the site but appear more continuous in the east-west plane; and 4) a drape of glacial till (0.3m – 3.0m thick) comprising very dense sand with silt and gravel.

The clay deposits range in consistency from a very stiff, oxidized crust (yellow coloration) to stiff (with gray coloration), and corresponding undrained shear strengths from UU tests, su = 90kPa, 60kPa, respectively. The clays are clearly overconsolidated, but no consolidation tests were performed for this project. The underlying silts, sands and till layers were found to have relatively high permeability in the range, k = 0.3 – 3.0m/day. The ‘silty fine sand’ and glacial till layers were classified as very dense layers based on SPT data. In some locations (Fig. 2) the sand there are looser deposits of ‘silt with sand’ (N = 12 – 37bpf) directly beneath the marine clay.

Groundwater conditions were measured by a series of piezometers screened within the underlying sands, till and rock layers. These data consistently show the groundwater head at El. +3m. There is a perched groundwater table in the overlying fill which is typically 1m - 2m below the ground surface.
The excavation performance was monitored by 1) weekly optical surveys of surface and property line in the glacial till and sand layers (3 locations). Axial forces and bending pressures below the excavation in the underlying rock (2 locations), and outside the slurry wall and extending 5m into the underlying Roxbury conglomerate. The tieback anchors were installed through steel sleeves cast into the slurry wall. Each anchor extends a minimum of 0.6m into the underlying Roxbury conglomerate. The tieback anchors were designed with zero tolerance against water leakage in order to protect the steel tendons from long term corrosion problems. This design requirement presented a particular challenge for the three lowest levels of anchors (P4 – P6) which were installed through the highly permeable sand and silt layers with an artesian/over pressure condition (i.e., the anchor head elevation is less than the piezometric head within these layers). The final anchor design used three levels of waterproofing including a flexible water-swelling seal (Shields, pers. comm.), further details can be found in Konstatakos (2000).

The MATEP power plant, immediately adjacent to the South slurry wall, is founded on a 1.5m thick concrete mat foundation at El. +2.7m. This foundation was expected to distribute the ground movements and hence, was less likely to suffer any damage during excavation. Similarly, minimal effects of the excavation were anticipated for the existing Dana Farber Cancer Institute, located 12m from the North slurry wall and founded on a shallow mat at El. 9.1m.

**LATERAL EARTH SUPPORT SYSTEM**

The permanent lateral earth support system comprising a 0.9m thick perimeter slurry wall with up to six levels (P1 – P6) of rock anchors, was designed to resist the lateral earth and pore water pressures, seismic loads and surcharge loads from construction equipment and adjacent structures. The tieback anchors were also specified with zero tolerance against water leakage in order protect the steel tendons from long term corrosion problems. This design requirement presented a particular challenge for the three lowest levels of anchors (P4 – P6) which were installed through the highly permeable sand and silt layers with an artesian/over pressure condition (i.e., the anchor head elevation is less than the piezometric head within these layers). The final anchor design used three levels of waterproofing including a flexible water-swelling seal (Shields, pers. comm.), further details can be found in Konstatakos (2000).

The perimeter slurry wall was constructed in a series of 6m long slurry panels each extending a minimum of 0.6m into the underlying Roxbury conglomerate. The tieback anchors were installed through steel sleeves cast into the slurry wall. Each anchor was inclined at 45° with minimum fixed anchor lengths of 6m in the bedrock. Horizontal spacing of the anchors ranged from 1.65m to 3.2m and each tendon comprised from 9 to 16 strands of 1.5cm diameter high tensile strength steel. Table 1 summarizes the average axial stiffness, free-length and measured lock-off loads for each level of anchors (per unit length along the wall) around the excavation. It should be noted that the top two levels of anchors (P1, P2) were absent along the South wall, due to the proximity of the MATEP mat foundation. The top of the East wall was braced by an edge beam cast at grade in order to avoid interference with the Binney Street utility tunnel.

The excavation performance was monitored by 1) weekly optical surveys of surface and building settlements (accurate to within 3mm), 2) inclinometer measurements of wall deflections from a series of 4 inclinometers (IN1 – IN4, Fig. 1) installed through the slurry wall and extending 5m into the underlying rock (these are accurate to within 2.5mm at ground surface), and 3) vibrating wire piezometer measurements of water pressures below the excavation in the underlying rock (2 locations), and outside the property line in the glacial till and sand layers (3 locations). Axial forces and bending moments in the slurry wall were also interpreted from two clusters of strain gauges and pressure transducers attached to the steel reinforcement at El. –22.5m on the East and North slurry walls (close to IN-3 and IN-2, respectively).

**MEASURED PERFORMANCE DURING EXCAVATION**

Figure 3 summarizes the timeline of excavation and anchor prestressing for average conditions along each of the four sides of the excavation (following prior slurry wall installation over a period of approximately 5 months). The accumulated time in this figure refers to the completion of each excavation/prestressing phase. It should be noted that there was no dewatering carried out during the excavation although there was a passive relief system of two wells installed in the till within the footprint of the excavation. Total water inflows ranged averaged 75 l/min, and maximum inflows (190 l/min) occurred during the first stages of excavation. Piezometers outside the excavation measured less than 0.6m change in head throughout the excavation.

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Parameter</th>
<th>North</th>
<th>South</th>
<th>East</th>
<th>West</th>
</tr>
</thead>
<tbody>
<tr>
<td>P1</td>
<td>EA (10^5 kN/m)</td>
<td>0.79</td>
<td>-</td>
<td>0.016</td>
<td>0.79</td>
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<tr>
<td>El. 9.0m</td>
<td>L_f (m)</td>
<td>24.4</td>
<td>-</td>
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<tr>
<td>P2</td>
<td>EA (10^5 kN/m)</td>
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<td>-</td>
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<tr>
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<td>2.30</td>
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<td>1024</td>
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<td>7.9</td>
<td>9.1</td>
</tr>
</tbody>
</table>

Notes: 1. F_L — lock-off load, L_f — estimated free-length of anchor
2. Edge beam installed at ground surface (Fig. 1).

Table 1: Summary of rock anchors

Figures 4 and 5 compare the measured wall deflections and surface settlements at the South and North walls (i.e., section A-A, Fig. 1, 2a) at selected stages of the excavation. The results show a small (5mm) inward cantilever deflection of the North wall (Stage L1, Fig. 4b). However, subsequent excavation and prestressing of anchors causes a reversal in wall deflections such that there is a small net outward movement (up to 6mm at El. 5m) at the end of the excavation (stage L7, Fig. 5b). The toe of the wall rotates but shows no net displacement. The behavior of the South wall is strongly influenced by the proximity of the MATEP foundation mat. There is no space for the P1 and P2 anchors. However, the inward cantilever deflections (6mm) at L3 are similar in magnitude to those found for the North wall at L1. This behavior can be attributed to the stiffening effects from grouting that was carried out to mitigate shallow soil cave-ins that occurred during installation of the South wall. Prestressing of anchors P3 – P6 is able to control subsequent inward wall deflections to less than 18mm. The proximal edge of the MATEP power plant settled a similar amount (approx. 15mm) and there is less than 10mm of differential settlement across of the foundation mat. Settlements up to 20mm occurred behind the North wall, but movements of the Dana Farber Cancer Institute were less than 10mm.

Figure 6 and 7 show analogous results for the East and West walls (i.e., section B-B, Fig. 1, 2b). Both walls show excellent toe fixity and both are prestressed such that there are maximum net outward movements up to 15mm (and quite similar deflection mode shapes). Settlements behind the East wall were measured at a series of reference points inside the utility tunnel (Figs. 6b, 7b). These data show very small settlements through the early phases of excavation (less than 3mm at stage P3). However, there are large increments of settlements associated with installation of tieback anchors at level P4 and further increases during the remainder of the excavation. Maximum settlements of the utility tunnel beneath Binney Street exceeded 70mm by the end of excavation (L7, Fig. 7b). There was a similar pattern of measured surface settlements for the Redstone Building, with maximum settlements up to 64mm.
The utility tunnel suffered some cracking as a result of these settlements and only slight damage was reported for the Redstone building. Nevertheless, the measured settlements exceeded prior predictions by more than 40mm and 50mm for the Redstone and utility tunnel, respectively. The behavior on the East wall was attributed, in large part, to ground losses that occurred during installation of the P4 anchors. Air pressures used during drilling of one anchor forced sand and water out through an adjacent anchor hole (soil was ejected approximately 5m according to the construction records). The ground was then stabilized by grout injections and subsequent drilling was performed in a more controlled fashion. Although, this event can explain some of the observed settlements for the East wall, there were no comparable incidents on the West wall. It is unclear why much larger surface settlements occurred for the East and West walls compared to the North or South walls (even allowing for the presence of the MATEP foundation adjacent to the South wall). It seems likely that small ground losses occurred for all anchors drilled through the overpressured silt and sand layers. These layers thin out to the North of the project site and were not present beneath the Dana Farber Cancer Institute (Fig. 2a) and this may explain the more satisfactory performance noted in Figs. 4b and 5b.

FINITE ELEMENT SIMULATIONS

A series of finite element simulations have been carried out to obtain better insight into the performance of the excavation support system for the Dana Farber research tower. The calculations have been carried out using the Plaxis finite element code (Brinkgreve, 2002) and comprise a series of four 2-D, plane strain models representing each of the four sides of the excavation. Given the almost complete lack of site specific data on soil deformation and shear strength properties, these parameters have been estimated based on prior experience, published correlations and case studies in the Boston area (e.g., Duncan et al., 1980; Johnson, 1989; Ladd et al., 1999; Altabba & Whittle, 2001; Hashash & Whittle, 1996). The soil parameters have subsequently been refined in back-analyses for the North wall section. Each of the soil layers has been simulated using the Hardening Soil (HS) model (Schanz et al., 1999). This model represents an updated version of the well known Duncan-Chang model (Duncan et al., 1980), formulated using elasto-plasticity. The non-linear shear-stress strain behavior in loading is represented by a hyperbolic function (with average secant modulus, $E_{50}$, Fig. 8); while a much stiffer linear response in unloading is described by the parameter, $E_{ur}$. The shear strength is characterized by conventional Mohr-Coulomb parameters ($c'$, $\phi'$). The HS model enables a realistic description of the stiffness of the retained soil relative to the excavated material with minimal additional parameters.

Table 2 summarizes the input parameters used to model the soil layers. Values of hydraulic conductivity were derived from field pumping tests, while undrained shear strength parameters for the clay were based on UU triaxial shear test data. Each of the four FE models considers a profile of horizontal soil layers (cf. profiles shown in Figs. 2a, b) with initial K0 conditions listed in Table 2. The perimeter slurry wall was modeled using elastic beam elements (with axial and bending stiffnesses; $EA = 2.52 \times 10^{7}$ kN/m and $EI = 1.7 \times 10^{6}$ Nm2/m, respectively), while elastic properties and prestress loads for the rock anchors are given in Table 1.
Figure 5: Measured and predicted behavior of Section A-A walls at end of excavation

<table>
<thead>
<tr>
<th>Layer</th>
<th>Model</th>
<th>( \gamma_t ) [kN/m³]</th>
<th>( c' (s_u) ) [kPa]</th>
<th>( \phi' ) (°)</th>
<th>( E_{50} ) [MPa]</th>
<th>( E_{50} ) [MPa]</th>
<th>( m^* )</th>
<th>K₀</th>
<th>k [m/day]</th>
</tr>
</thead>
<tbody>
<tr>
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<td>20</td>
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<td>0.5</td>
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<td>0.1</td>
</tr>
<tr>
<td>Yellow Clay</td>
<td>HS(U)</td>
<td>20.0</td>
<td>90</td>
<td>--</td>
<td>25</td>
<td>100</td>
<td>1.0</td>
<td>0.8</td>
<td>5x10⁻⁴</td>
</tr>
<tr>
<td>Gray Clay</td>
<td>HS(U)</td>
<td>18.5</td>
<td>60</td>
<td>--</td>
<td>25</td>
<td>100</td>
<td>1.0</td>
<td>0.6</td>
<td>5x10⁻⁵</td>
</tr>
<tr>
<td>Silt &amp; Sand</td>
<td>HS</td>
<td>20.0</td>
<td>--</td>
<td>35</td>
<td>35</td>
<td>105</td>
<td>0.5</td>
<td>1.0</td>
<td>0.001</td>
</tr>
<tr>
<td>Silty sand</td>
<td>HS</td>
<td>20.0</td>
<td>--</td>
<td>40</td>
<td>55</td>
<td>165</td>
<td>0.5</td>
<td>1.0</td>
<td>0.1</td>
</tr>
<tr>
<td>Glacial Till</td>
<td>HS</td>
<td>22.0</td>
<td>--</td>
<td>43</td>
<td>60</td>
<td>180</td>
<td>0.5</td>
<td>1.0</td>
<td>0.01</td>
</tr>
<tr>
<td>Bedrock</td>
<td>MC</td>
<td>23.6</td>
<td></td>
<td></td>
<td>2.8x10⁴</td>
<td>n = 0.15</td>
<td>--</td>
<td>1.0</td>
<td>0.01</td>
</tr>
</tbody>
</table>

Notes: 1. HS – Hardening soil; (U) – ‘undrained’ capability to develop excess pore pressures during shearing
2. Default parameters used: \( E_{50} = E_{10} \) R₁ = 0.9 and n₁ = 0.2. MC – linearly elastic perfectly plastic with Mohr-Coulomb yield
3. All stiffness parameters \( E \sim (\sigma^3/p_{ref})^m \)

Figure 6: Measured and predicted behavior of Section B-B walls at early stages of excavation
The magnitudes of lateral wall deflection are very well predicted for the South Wall (Figs. 4a, 5a), with maximum inward deflections up to 10mm at the top of the wall. However, the analysis does not describe the measured rotations of the wall within the glacial till and rock layers. The analysis also gives very reasonable predictions of settlements for the MATEP mat foundation (grouting of surficial soils immediately behind the wall prevented large local settlements reported from the analysis).

Figures 6 and 7 show similar comparisons of predicted and measured wall deflections for the West and East walls at representative stages of the excavation (after 3rd level prestressing, P3, Fig. 6; and at the end of excavation, L7, Fig. 7). Note that deflections in Figure 7 are shown at a reduced scale. Again the overall magnitudes of wall deflections are well estimated by the FE analyses, while differences in the deflection modes shapes can be explained by simplifications in modeling (such as the representation of the edge beam for the East wall) or uncertainties in the selection of soil parameters. As expected, the predicted surface settlements remain very small (less than 10mm) through the end of the excavation.

Figure 9: Comparison of computed and measured bending moments in North wall
Further analyses have been performed to evaluate the possible effects of ground loss (associated with tieback installation) on the predicted wall deflections and ground movements. The analyses are performed by specifying a volumetric strain within particular clusters of finite elements (Brinkgreve, 2002), a technique that has previously been applied to model processes such as compensation grouting around tunnels (Schweiger & Falk, 1998). Calculations for the East wall assume that ground loss occurs within the silt and sand layers (but not in the clay or glacial till) during installation of the 4th and 5th level tiebacks. Figure 10 compares the finite element analyses with the timeline of settlements measured at a reference point BPI-5 on the utility tunnel (Fig. 1). The original analysis is in close agreement with the measured settlements through CD230 (L3 excavation, Fig. 3) and entirely consistent with the performance measured a similar distance behind the South wall (BP-44). By then assuming a 2% volume strain during anchor installation (prior to stages P4 and P5), the analyses achieve very good matching with the large settlements measured in the utility tunnel (71mm at end of excavation, L7). This simulation is equivalent to a ground loss of 0.52 m³/lin-m (which can be compared with the 1.5m³ of grout injected to remediate the one recorded occurrence of ground loss).

Figure 7b shows that the effects of the local ground loss (occurring within 6-7m of the wall) affects the surface settlements up to 20m behind the East wall. The ground loss accounts for an additional 60mm of surface settlement but less than 10mm of incremental wall deflection by the end of excavation (Fig. 7b). A similar procedure has been used to simulate ground losses for the West wall. In this case, smaller volume strains have been included prior to prestressing for P3 through P6 (the results in Fig. 6a assumed 1%, 1%, 1.3% and 1.5% following excavation of stages L3 through L6, respectively, amounting to a ground loss of 0.36m³/lin-m). The computed surface settlement beneath the Redstone Building increases from 8mm to 65mm as a result of these assumed ground losses, producing very good matching with the measured data. The simulated ground losses also produce a net outward deflection of up to 14mm of the wall, leading to a very good agreement with the 10-15mm of movement observed.

CONCLUSIONS

Deep excavations for the Dana Farber research tower were supported by a permanent lateral earth support system comprising a perimeter diaphragm wall embedded in the underlying conglomerate bedrock and 4 to 6 levels of prestressed rock anchors. The lateral earth support system was very successful in controlling lateral wall movements to less than ±15mm, however, surface settlements exceeded 65mm – 71mm occurred on two sides of the excavation causing minor damage to adjacent structures. These effects were attributed to local ground losses during anchor installation through overpressured sand and silt deposits.

Back-analyses of the excavation performance using 2-D finite element analyses were able to give consistent estimates of the measured wall deflections on each of the four sides of the excavation. Ground losses have also been simulated in the FE analyses by including local volumetric strains in clusters of soil elements around the tiebacks. These simulations are able to replicate the measured surface settlements with relatively small volume strains, generating additional surface settlements in the range, 50mm – 60mm. The simulated ground losses also cause the perimeter wall to deflect 10mm –15mm from the excavation under the action of the applied anchor pre-stress loads. This result improves the overall agreement between the computed and measured behavior. The results appear to confirm the hypothesis that local ground losses during anchor installation can explain the unexpectedly large surface settlements that occurred on two sides of the excavation.

REFERENCES

**PRODUCT NEWS**

Update pack 5 of Plaxis V8 is now available from the secure download area of our website. Major new features since the launch of Plaxis V8 are Modified Cam-Clay Model, Elasto-plastic geogrids and axial symmetric groundwater flow. All bug fixes and additional functionality made in previous Update Packs have been incorporated in Update Pack 5. Also the manual has been updated and new functionality is integrated in the latest version, which can be downloaded from our website at the Product area. After the official release of UP5 we concentrate on the release of the Chinese version of Plaxis V8. Please check out our webpage regularly for up to date information on the latest developments. On our website you can also find the digital version of the Spanish reference manual (PDF-format). In addition to our English, German, French and Italian reference manual we just released this Spanish version and are currently working on a Portuguese reference manual.

**AGENTS**

Plaxis appointed Institute of Foundation Engineering (IFE) as sales and support representative for China. IFE, a wholly owned subsidiary of China Academy of Building Research, is engaged primarily in geotechnical engineering, design and research. IFE is headquartered in Beijing and has branch offices in all major cities of China. Major activity for this year is the introduction of the Chinese version of Plaxis V8.

**DUTCH RESEARCH AWARD FOR PROF. PIETER VERMEER**

During the Dutch Annual conference on Geo-engineering in October 2004, Prof. Vermeer was honoured with The Research Award by the Royal Dutch Society of Engineers. He was awarded this medal for his efforts in Research and Development of Numerical geo-engineering in the last decades, many of which have been implemented in Plaxis.

**COURSES AND USER MEETINGS**

Following the successful yearly user meetings in Karlsruhe for European users mainly, Plaxis presented a similar event for the North American users group. This session was hosted by the Polytechnic University at Brooklyn on January 7, 2005. Presentations were made by staff members of Massachusetts Institute of Technology, Mueser Rutledge Consulting Engineers, (see also Plaxis Practice article in this bulletin) and Plaxis bv.

We hope to meet you at our oncoming user meetings, at one of our courses or at Geotechnical Conferences. Please check out our presences at the Agenda.
INTRODUCTION

A large construction project for a store was completed in December 2004 in the area of the 53th Street and Caracas Avenue in a centric commercial area in the city of Bogotá. The building has two store levels and four parking basement levels down to 14 m depth, covering 7500 m² of terrain.

Coluvial deposits with a depth of 26 to 36 m composed of layers of sands, silts and organic clays are found at the site overlying claystones. The ground water level was about 4 m deep with the sand layers having water pressures.

The great construction challenge besides the short time for the construction of the store was to be able to develop a project under the complex geotechnical conditions that are present in the area. Two projects had been attempted previously at the site, in 1971 and in 1988, that had failed because of stability problems with the excavation and construction of the piles. The problems were mainly induced by instability of the soils by water flow.

For the new project a seismic site response study was carried out due to the seismic hazard in the area as part of the additional geotechnical studies. This study provided valuable data on soil stiffness and stress-strain behaviour. Slurry walls enclosing the excavations down to the rock and bored cast-in-place piles were designed for the infrastructure of the building. It was assumed that water conditions would be controlled by the slurry walls and the assumed impervious rock at the bottom of the deposits.

During construction there were some difficulties due to water flow during the excavation of the slurry walls and piles. After the walls were built two pump wells were used to lower the ground water inside the excavation expecting to lower and empty the water underneath the project area. Pumping of water began in early 2004 and a steady flow condition was reached during the almost 12 months of construction indicating incoming flow towards the excavation, either through the slurry walls or underneath the rock. A detailed hydrogeologic study was carried out to identify the source of this problem.

Extensive geotechnical instrumentation was put in place to monitor the project. These included vibrating wire piezometers at different levels in locations inside and around outside the excavation reaching the sand layers and underlying rock, inclinometers built in the slurry walls, and settlement benchmarks around the excavation.

GEOTECHNICAL AND HYDRO-GEOLOGICAL CONDITIONS

The lot of the project is located in the piedmont of the hills that surround Bogotá along the east of the city. The deposit has variable thickness between 22 and 36 m, increasing towards the south-west. It is composed of hillside deposits with intercalations of sands, silts, clays and gravels. These strata have water to pressure and they are very unstable in excavations. Under these soils there are rocks of the tertiary Bogotá formation, which are predominantly claystones with locally interbedded sandstone layers less than one meter thick. These rocks were found dipping almost vertically at the site due to a reverse fault that runs along the project area under the deposits parallel to the hills. This fault and the effect on stratigraphy were not identified during the geotechnical studies.

Figure 1 shows a geological profile in the east-west direction. Due to the stratigraphy and faulting, there was hydraulic continuity of the sandstones at the bottom of the deposit and the recharge areas in the hills.
The analyses considered two scenarios. One in which the flow was through the slurry walls, and the other with the slurry walls completely impervious. The groundwater levels obtained from the analyses were compared with measured piezometric levels around the excavation. The piezometric measurements are shown in Figure 4. The total discharge considering the thickness of sand layers was compared with the actual discharge measured, which was 80 m³/day. Figure 5 shows a detail of the analyses indicating the effect of the variable permeability on the concentration of flow under the excavation. The areas of concentrated flow coincided with the places where there were difficulties during construction of the piles and slurry wall. The analysis showed that the discharge and water levels that were consistent with the measurements corresponded to the case where the slurry walls were impervious and the flow was under the walls through the sandstone layers. Based on these results the bottom floor of the excavation was redesigned and the slurry walls contractor was released from responsibility of the water flow problem.

ANALYSIS OF EXCAVATIONS

The special geotechnical conditions, the existence of nearby buildings and main roads, together with the fact that interferences with existing foundations in the lot should be managed and the stringent time constraints for the project, made the planning and construction of the excavations one of the critical points for the development of the project. To understand and to study what happened during the construction of the excavations of this project, it was sought to simulate the construction of the work by means of a model that involved the different soil materials, the constructive sequence, the existent previous structures, the soil-structure interaction and the variation of the piezometric levels, in a model that kept in consideration the non linear stress strength relationships for the materials. To carry out the analyses a finite elements model was developed using the program PLAXIS. The water flow conditions were obtained from hydrogeologic analyses of the excavations of this same project carried out by Rodríguez et al. (2004b), which were coupled in the model.

The soil model used for the analyses was the “hardening soil” (HS) model of plastic hardening developed by Schanz et al. (1999). This is an advanced model to represent the behaviour of different types of soft and hard soils. The plastic hardening refers to the generation of plastic deformations in the soil due to changes of stress. Two types of plastic hardening are distinguished: one due to shear deformations distinguished by a hyperbolic stress-strain relationship depending on the confining stress (Kondner, 1963, Duncan and Chang, 1970) and another due to volumetric compression. The model also considers a Mohr Coulomb failure envelope, as well as the generation of pore water pressures due to undrained shear.
The parameters for the soils in undrained conditions were obtained by the methodology outlined by Rodriguez et al. (2004a). The stress-strain curves were obtained from the shear wave measurements at very low strains, and available dynamic Q/Gmax -vs-deformation curves from the seismic site response study were integrated to define the hyperbolic stress-strain curves required for the model. Figure 6 shows the curve used to model the silty soils.

The analysis considered the following constructive stages:
1. Excavation and demolition of the existent structures in certain sectors of the lot.
2. Construction of the containment slurry wall with anchors with excavation to the level of the first basement (level -3.4 m).
3. Construction of anchors and excavation to the level of the second basement (level -6.8 m).
4. Excavation to the third basement (level -10 m). The wall is supported by the structure already built.
5. Excavation to the fourth basement (level -14 m).
6. This last stage was completed after the building was already in service.

Figure 7: Inclinometer readings, south side wall along 53rd Street. X-axis shows displacement in cm, Y-axis is depth in meters. Several measurements at different dates are shown. Maximum displacement at the end of construction was 8 cm at -10 to -14 m depths.

Figure 8: Inclinometer readings, north side wall along 52nd Street. X-axis shows displacement in cm, Y-axis is depth in meters. Several measurements at different dates are shown. Maximum displacement at the end of construction was 6 cm fairly uniform until 12 m depth.

Figure 9: Inclinometer readings, west side wall along Caracas Avenue. X-axis shows displacement in cm, Y-axis is depth in meters. Several measurements at different dates are shown. Maximum displacement at the end of construction was 10 cm with maximum at 12 m depth.

Figure 10: Settlements along Caracas Avenue, west side of project.
Figure 11: Settlements along 53rd Street, south side of project.

The instrumentation results showed maximum settlements around the excavation of some 6 cm. The settlements varied between 3 and 7 cm, especially along 53rd Street, probably due to local variations in the soil profile. The maximum displacement of the slurry wall was 8 cm towards the bottom of the excavation on the south side, where the depth of the rock layer was 23 m, and 10 cm at 12 m depth on the west side, where the depth of the rock layer is in the order of 33 m.

Figure 12: Computed deformations for the north-south cross section at the end of excavation. Maximum wall displacements as well as nearby settlements on the west side are 6 cm. On the east side these values are around 4 cm.

Illustrations of the results of the finite element analyses at the end of construction are shown in Figures 12 and 13. Figure 12 shows contours of computed deformations for a north-south section towards the mid part of the excavation. On both sides of the excavation the depth of the rock layer, and the actual contour of the rock depth are different according to the data obtained from construction records. The rock contact is dipping towards the south west. Therefore the deepest section of the wall is along Caracas Avenue, where the largest settlements and deformation were measured. The section shown is an intermediate cross-section.

These results show similar trend and similar orders of magnitude of displacements as those measured, both for the north as well as for the south walls. Also the magnitudes of soil settlements computed around the wall are in agreement with the measured data. No attempt was made to try and adjust soil properties to obtain a better fit to the data. These results indicate the capability of the method of analysis used to reasonably predict the behaviour of this complex excavation. It should be noted that other simpler models considered for the problem, particularly those without considering the coupling of groundwater conditions, or considering simplified construction sequences failed to give reasonable results.

The results shown are to the end of construction, at the end of January 2004. The behaviour of the construction is being monitored for ongoing long term deformations, particularly around the building. Although the models used compute excess pore water pressures due to the change in stress conditions, and can do consolidation analysis, no attempts have been made to compute long term settlements. It is foreseen, from other experiences using this approach for projects in Bogotá, that the undrained soil shear stress-strain curves used for the models are not suitable for long term deformation computations.

CONCLUSIONS

Valuable information has been gathered about the behaviour of slurry walls and the soil anchor system used for the excavation of the project in soft soil conditions with difficult water conditions in the piedmont of Bogotá eastern hills. This is an area undergoing renovation and new developments, with projects similar to the one considered in this paper.

From the analysis of this case it can be concluded that the computational model and the soil models used, considering the coupled problem of deformation and water flow, the highly non-linear behaviour of the soils and the construction sequence, allow detailed study of complex excavations in sectors with especially difficult geotechnical conditions in the short term. The results obtained are in good agreement with the data measured by means of the geotechnical instrumentation. This allows the use of these techniques for future complex projects in the area.

REFERENCES

INTRODUCTION

Dealing with uncertainty, caused e.g. by material parameters varying in a wide range or simply by a lack of knowledge, is one of the important issues in geotechnical analyses. The advantages of numerical modelling have been appreciated by practitioners, in particular when displacements and deformations of complex underground structures have to be predicted. Therefore, it seems to be logical to combine numerical modelling with concepts for the mathematical representation of uncertainties. Recent theoretical developments and advances made in computational modelling have established various methods which may serve as a basis for a more formal consideration of uncertainties as has been done so far. Random set theory offers one of these possibilities for the mathematical representation of uncertainties. It can be viewed as a generalisation of probability theory and interval analysis. After a brief introduction of the basics of the proposed approach an application to a boundary value problem is presented. The results show that the assessment of the probability of damage of a building, situated adjacent to the excavation, is in line with observed behaviour.

About half a century ago, the latest phase in a debate about the mathematization of uncertainty started. Probability theory based on the neo-Bayesian school has been extensively employed (see e.g. Ang & Tang 1975, Wright & Ayton 1994) but more recently fuzzy methods and evidence theory have been established as legitimate extension of classical probability theory. It is important to realise that different sources of uncertainty exist, material parameters varying in a wide - but known - range are one of them but simply the lack of knowledge may actually be the more dominant one in geotechnical engineering. The full scope of uncertainty and its dual nature can be described by the definitions from Helton (1997): Aleatory uncertainty results from the fact that a parameter can behave in random ways whereas epistemic uncertainty results from the lack of knowledge about a parameter. Aleatory uncertainty is associated with variability in observable populations and is therefore not reducible, whereas epistemic uncertainty changes with ones’ state of knowledge and is therefore reducible. Despite the fact that it is well recognised that the frequentist approach associated with classical probability theory is well suited for dealing with aleatory uncertainty, it is common practice to apply probability theory to characterise both types of uncertainty, although it is not capable of capturing epistemic uncertainty (Sentz & Ferson 2002).

In view of having insufficient data available for a particular project, in practice alternative sources of information are usually utilised. These sources can be previously published data for similar ground conditions, general correlations from literature or simply expert knowledge. This data conventionally appear as intervals with no information about the probability distribution across the interval and therefore it seems to be appropriate to actually work with these intervals rather than assuming a density distribution function. One way of doing so is the application of the random set theory as presented here.

RANDOM SET THEORY

The theory of random sets provides a general framework for dealing with set-based information and discrete probability distributions. The analysis gives the same result as interval analysis, when only range information is available and the same result as Monte-Carlo simulations when the information is abundant.

Basic concepts

Let X be a non-empty set containing all the possible values of a variable x. Dubois & Prade (1990, 1991) defined a random set on X as a pair (X,m) where X = {A_i : i = 1,...,n} and m is a mapping, X → [0,1], so that m(Ø) = 0 and

\[ \sum_{A_i \in X} m(A_i) = 1. \]  

(1)

X is called the support of the random set, the sets A_i are the focal elements (A_i ⊂ X) and m is called the basic probability assignment. Each set, A ∈ X, contains some possible values of the variable, x, and m(A) can be viewed as the probability that A is the range of x. Because of the imprecise nature of this formulation it is not possible to calculate the 'precise' probability Pro of a generic x ∈ X or of a generic subset E ⊂ X, but only lower and upper bounds on this probability (Fig. 1): Bel(E) ≤ Pro(E) ≤ Pl(E). In the limiting case, when A is composed of single values only (singletons), then Bel(E) = Pro(E) = Pl(E) and m is a probability distribution function.

![Figure 1: Upper bound (Pl) and lower bound (Bel) on "precise" probability (Pro)](image)

According to Dempster (1967) and Shafer (1976) the lower bound Bel (belief function) and the upper bound Pl (plausibility function) of its probability measure are defined, for every subset E ⊂ X, by

\[ Bel(E) = \sum_{A_i : A_i \cap E \neq \emptyset} m(A_i), \]  

(2)

\[ Pl(E) = \sum_{A_i : A_i \subset E} m(A_i), \]  

(3)

which are envelopes of all possible distribution functions compatible with the data.

Bounds on the system response

Tonon et al. (2000a) showed that the extension of random sets through a functional relation is straightforward. Let f be a mapping X_1 × ... × X_N → Y and x_1, ..., x_N be variables whose values are incompletely known. The incomplete knowledge about x = (x_1, ..., x_N) can be expressed as a random relation R, which is a random set (X, m) on the Cartesian product X_1 × ... × X_N. The random set (R, m), which is the image of (X, m) through f is given by (Tonon et al. 2000b):
In the absence of any further information, a random relation (random set model) can be constructed by assuming stochastic independence between marginal random sets (Equ. 6).

\[ m(A_1 \times \ldots \times A_n) = \prod_{i=1}^{n} m_i(A_i) \quad A_1 \times \ldots \times A_n \in \mathcal{R} \]

(6)

If the focal set \( A_i \) is a closed interval of real numbers: \( A_i = [x_i, u_i] \), then the lower and upper cumulative probability distribution functions, \( F^*(x) \) and \( F^*(x) \) respectively, at some point \( x \) (Fig. 2) can be obtained as follows:

\[ F^*(x) = \sum_{i \geq x_i} m(A_i) \]

(7)

\[ F^*(x) = \sum_{i \leq x_i} m(A_i) \]

(8)

In the absence of any further information, a random relation (random set model) can be constructed by assuming stochastic independence between marginal random sets (Equ. 6).

![Focal sets Ai and upper and lower cumulative distribution function, \( F^*(x) \) and \( F^*(x) \).](image)

The basic step is the calculation by means of Equation 4 and 5 of the image of a focal element through function \( f \). The requirement for optimisation to locate the extreme elements of each set \( R_i \subseteq \mathcal{R} \) (Equ. 4) can be avoided if it can be shown that the function \( f(A_i) \) is continuous and also no extreme points exist in this region, except at the vertices, in which case the Vertex method (Dong & Shah 1987) applies. Assume each element through function \( f \) has to be calculated only twice for each focal element \( A_i \) (Tonon et al. 2000b).

\[ R_{ij} = \min_k [f(v_k) : k = 1, \ldots, 2N] \]

(9)

\[ R_{ij}^* = \max_k [f(v_k) : k = 1, \ldots, 2N] \]

(10)

Thus function \( f(A_i) \) which represents in this framework a numerical model has to be evaluated \( 2^n \) times for each focal element \( A_i \) where \( N \) is the number of basic variables. The computational effort involved can be reduced significantly if \( f(A_i) \) is continuous and a strictly monotonic function with respect to each parameter \( x_i \). In this case the vertices where the lower and upper bounds (Equ. 9 and 10) on the random set are located can be identified simply by consideration of the direction of increase of \( f(A_i) \) which can be done by means of a sensitivity analysis (Schweiger & Peschl 2004). Thus \( f(A_i) \) has to be calculated only twice for each focal element \( A_i \) (Tonon et al. 2000b).

**Combination of random sets**

An appropriate procedure is required if more than one source of information is available for one particular parameter in order to combine these sources. Suppose there are \( n \) alternative random sets describing some variable \( x \). Each one corresponding to an independent source of information. Then for each focal element \( A \in \mathcal{X} \)

\[ m(A) = \frac{1}{n} \sum_{i=1}^{n} m_j(A) \]

(11)

Alternative combination procedures have been proposed depending on different beliefs about the truth of the various information sources (e.g. Sentz & Ferson 2002, Hall & Lawry 2004).

**The reliability problem**

Basically, reliability analysis calculates the probability of failure \( p_f = P(g(x) \leq 0) \) of a system characterised by a vector \( x = (x_1, \ldots, x_N) \) of basic variables on \( X \) where \( g \) is called the ‘limit state function’ and \( p_f \) is identical to the probability of limit state violation. Utilising random set theory the reliability problem is reduced to evaluate the bounds on \( p_f \) subject to the available knowledge restricting the allowed values of \( x \). If the set of failed states is labelled \( FC \subseteq X \) the upper and lower bound on the probability of failure are the Plausibility \( Pl(F) \) and Belief \( Bel(F) \) respectively: \( Bel(F) \leq p_f \leq Pl(F) \).

**THE RANDOM SET FINITE ELEMENT METHOD**

The assessment of the stability of a geotechnical system is based on various sources of information such as ground conditions, construction procedure and others. Although it is well established that there is a significant scatter e.g. in the material parameters a deterministic approach with design values ‘on the safe side’ or a parametric study based on experience is commonly adopted. Sometimes worst case assumptions are postulated. However they are not always obvious, in particular for complex, highly nonlinear systems such as geotechnical structures. By using finite element codes in reliability analysis there are some advantages compared with limit equilibrium methods or other similar methods because more than one system parameter can be obtained without the need of changing the computational model. These can be used for the evaluation of the serviceability or the ultimate limit state of a geotechnical system or of the respective elements of the system. Examples are displacements in the soil or stresses in structural elements.

The proposed method combines random sets to represent uncertainty and the finite element analysis (RS-FEM: random set finite element method). Figure 3 depicts an overview for a typical random set finite element calculation in geotechnics (basic procedure).
APPLICATION TO DEEP EXCAVATION PROBLEM

In order to demonstrate the applicability of the proposed method some results from a back analysis of a practical example, namely a deep excavation (Breymann et al. 2003) on a thick layer of post-glacial soft lacustrine deposit (clayey silt) are presented. An underground car park has been constructed as an open pit excavation with a 24m deep anchored concrete diaphragm wall as retaining construction. In addition a berm was left before excavating to full depth and the foundation slab in the centre of the excavation was constructed in sections. Figure 4 plots the cross section of the system and the soil profile. In this analysis particular attention is given to the assessment of the angular distortion of the building exceeding acceptable limits.

Subsoil conditions and material parameters

The behaviour of the subsoil is characterised by soil parameters established from a number of laboratory and in situ tests. In order to assess the applicability of the proposed approach in practical design situations only data available before the excavation started has been used. Of particular significance for the deformation behaviour of the soft-plastic clayey silt is the deformation modulus Es (denoted as Eoed in the following), gained from one-dimensional compression tests on undisturbed soil samples after pre-loading with the in situ stress of the relevant depth. The parameters used in the analysis performed with the finite element code PLAXIS V8 (Brinkgreve 2000) are summarised in Tables 1 to 3. The mesh consists of approximately 1,400 15-noded triangular elements and the Hardening-Soil model (HS) was used to model the behaviour of the different soil layers.

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Basic variables for the random set model

The material parameters for the soil layers, which were treated as basic variables are summarised in Table 5. The parameters have not been determined solely from experimental investigations (geotechnical report) but also from previous experience of finite element analyses and in situ measurements under similar conditions (expert knowledge). The table highlights the wide range of certain parameters which in itself to some degree contain engineering judgement of people involved, e.g. a significant capillary cohesion has been assigned to the sandy, silty gravel in the geotechnical report.

<table>
<thead>
<tr>
<th>Soil</th>
<th>c</th>
<th>$q'$</th>
<th>$E_s$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Information source</td>
<td>kN/m²</td>
<td>° MN/m²</td>
<td></td>
</tr>
<tr>
<td>Sandy, silty gravel</td>
<td>0 - 50.0</td>
<td>35.0 - 37.0</td>
<td>20.0 - 35.0</td>
</tr>
<tr>
<td>Expert knowledge</td>
<td>0 - 5.0</td>
<td>34.0 - 36.0</td>
<td>30.0 - 50.0</td>
</tr>
<tr>
<td>Clayey silt 1</td>
<td>0 - 20.0</td>
<td>22.0 - 30.0</td>
<td>5.0 - 25.0</td>
</tr>
<tr>
<td>Expert knowledge</td>
<td>10.0 - 30.0</td>
<td>24.0 - 28.0</td>
<td>20.0 - 40.0</td>
</tr>
<tr>
<td>Clayey silt 2</td>
<td>0 - 20.0</td>
<td>22.0 - 29.0</td>
<td>20.0 - 30.0</td>
</tr>
<tr>
<td>Expert knowledge</td>
<td>10.0 - 30.0</td>
<td>24.0 - 28.0</td>
<td>30.0 - 40.0</td>
</tr>
</tbody>
</table>

Table 5: Basic variables for material parameters (input values)

Soil properties do not vary randomly in space but their variation is gradual and follows a pattern that can be quantified using spatial correlation structures whereas the use of perfectly correlated soil properties gives rise to unrealistically large values of failure probabilities for geotechnical structures (see e.g. Mostyn & Soo 1992 and Fenton & Griffiths 2003). PLAXIS requires the soil profile to be modelled using homogeneous layers with constant soil properties. Due to the fact of spatial averaging of soil properties the inherent spatial variability of a parameter, as measured by its standard deviation, can be reduced significantly. The variance of these spatial averages can be correlated to the point variance using the variance reduction technique. In this framework the variance reduction technique by Vanmarke (1983) is applied which depends on the averaging volume described by the characteristic length and the type of correlation structure. The approach followed here (Peschl & Schweiger 2004) is certainly a simplification compared to real field behaviour. However, a more rigorous treatment of the spatial correlation requires computational efforts which are not feasible in practical geotechnical engineering at the present time, at least not in combination with high level numerical modelling.

For the material parameters given in Table 5 two published sources of information were available and these interval estimates were combined using the averaging procedure in Equation 11. As an example the random set for the effective friction angle $\phi'$ of the gravel layer is depicted in Figure 5. Most values for the vertical spatial correlation length for sandy gravel materials and clayey silt deposits recorded in the literature are in the range of about 0.5 up to 5.0m. Therefore, a value of about 2.5m is assumed in this case. The characteristic length has been taken as 55m, which is based on analyses investigating potential failure mechanisms for this problem.

Construction steps modelled

The analyses performed were calculated as 2D plane strain problems and do not consider 3D-effects. It could be reasonably assumed that consolidation effects do not play a significant role for the excavation-induced movements and therefore an undrained analysis in terms of effective stresses was performed for the clayey silt layers. The computational steps have been defined according to the real construction process.

CALCULATION RESULTS

Before the random set analysis as described previously is performed a sensitivity analysis quantifying the influence of each variable on certain results can be made (Schweiger & Peschl 2004). For the 9 variables shown in Table 5, 37 calculations are required to obtain a sensitivity score for each variable. In this case the horizontal displacement of the top of the diaphragm wall, $u_x$, the angular distortion, $d/l$, of the adjacent building at each construction step and the safety factor, determined by means of the $j/c$-reduction technique, at the final construction step is evaluated. Based on the results of the sensitivity analysis the following parameters were considered in the random set model: cohesion for the sandy, silty gravel layer and the stiffness parameters $E_{50}$, $E_{100}$ and $E_{ur}$ (but these are correlated) for the sandy, silty gravel and the upper clayey silt layer, i.e. 64 calculations are required.

Serviceability limit state

The angular distortion $d/l$ of a structure, with $d$ being the differential settlement and $l$ the corresponding length, is often used as measure to assess the likelihood of damage. The value tolerated depends on a number of factors such as the structure of the building, the mode of deformation and of course the purpose the building has to serve. A ratio of about 1:600 is used here as a limiting value for the evaluation of the limit state function in order to obtain the reliability in terms of serviceability.

![Figure 5: Random set for the friction angle of the gravel layer](image)

![Figure 6: Range of angular distortion $d/l$ after second excavation step](image)
As an example, Figure 6 depicts the calculated bounds (CDF's) of the angular distortion $\delta/l$ after the second excavation step. These discrete CDF's were fitted using best-fit methods like the method of least squares in order to achieve a continuous function (dotted line in Fig. 6), which are used for the evaluation of the limit state function by means of Monte-Carlo simulations. By doing so, results in ranges on the probability of failure are obtained as given in Table 6 for all construction steps. The probabilities show that damages of the adjacent building can be expected already during the second excavation step (crucial construction step) and continues to be critical throughout the following construction steps.

<table>
<thead>
<tr>
<th>Construction step</th>
<th>Fitted distribution</th>
<th>max $p_f$</th>
<th>min $p_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Upper bound</td>
<td>Lower bound</td>
<td></td>
</tr>
<tr>
<td>Excavation 1</td>
<td>Beta</td>
<td>Gamma</td>
<td>0</td>
</tr>
<tr>
<td>Anchor 1</td>
<td>Gamma</td>
<td>Beta</td>
<td>3.0E-5</td>
</tr>
<tr>
<td>Excavation 2</td>
<td>Triangular</td>
<td>Beta</td>
<td>1.3E-1</td>
</tr>
<tr>
<td>Anchor 2</td>
<td>Gamma</td>
<td>Beta</td>
<td>2.2E-1</td>
</tr>
<tr>
<td>Excavation 3</td>
<td>Exponential</td>
<td>Beta</td>
<td>7.2E-1</td>
</tr>
<tr>
<td>Excavation 4</td>
<td>Beta</td>
<td>Beta</td>
<td>9.7E-1</td>
</tr>
</tbody>
</table>

Table 6. Range of probability that $\delta/l \geq 1/600$

**ULTIMATE LIMIT STATE**

For the evaluation of the ultimate limit state, the shear strength reduction technique is applied. In this study, the diaphragm wall and the anchors have been modelled as an elasto-plastic material (Tab. 4) as suggested by Schweiger (2003). This means that for the evaluation of the factor of safety via $\phi/c$-reduction not only the shear strength parameters of the soil layers but also the strength parameters of the wall are successively reduced until a failure mechanism is developed, taking into account an increase in anchor forces until their ultimate capacity is reached. The range of the resulting factor of safety and a deformed mesh of the final excavation step after $\phi/c$-reduction is shown in Figure 7 and Figure 8.
CONCLUSIONS

Reliability analysis in engineering conventionally represents the uncertainty of the system state variables as precise probability distributions and applies probability theory to generate precise estimates of e.g. the probability of failure or the reliability. However, it has been recognised that traditional probability theory cannot capture the full scope of uncertainty (inherent variability and lack of knowledge). The presented approach offers an alternative way of analysis when insufficient information is available for treating the problem by classical probabilistic methods. It requires less computational effort but has the disadvantage that spacial correlation can be taken into account only in a simplified way by means of variance reduction factors. The applicability of the proposed method for solving practical boundary value problems has been shown by analysing the excavation sequence for a deep excavation in soft soil.

The significant innovation of the presented framework is that it allows for the allocation of a probability mass to sets or intervals and provides a consistent framework for dealing with uncertainties throughout the design and construction of a project, because the model can be refined by adding more information when available without changing the underlying concept of analysis. As a side effect worst case assumptions in terms of unfavourable parameter combinations have not to be estimated from experience but are automatically generated. The argument that engineering judgement will do the same much faster is not entirely true because in complex non-linear analyses, which become more and more common in practical geotechnical engineering, the parameter set for a highly advanced constitutive model leading to the most unfavourable result in terms of serviceability and ultimate limit state for all construction stages is not easy to define. From a practical point of view the concept of dealing with ‘ranges’ seems to be more appealing for engineers than working in density distribution functions.

REFERENCES

PARAMETER DETERMINATION

An often reported obstacle that causes users to forego use of the Soft Soil Creep model is the determination of the stiffness parameters $\lambda^*$, $\kappa^*$, $\mu^*$ and the initial OCR, especially since the creep behaviour depends on all four parameters.

In principal all four parameters can be derived from a standard oedometer test, provided that the test has been performed for long enough after applying the load step. Preferably, the test results are plotted as the volumetric strain, $\epsilon_v$, versus the natural logarithm of the mean effective stress, $\ln(p')$. The volumetric strain of an oedometer test sample can easily be calculated by dividing the settlement by the original sample height.

As usual, the graph of $\ln(p')$ vs. $\epsilon_v$ (Figure 1) will provide the $\kappa^*$ and $\lambda^*$ from the inclination of the settlement curve before and after the preconsolidation stress $P_c$. And the OCR can be calculated from this preconsolidation stress, if the mean stress level at the depth where the sample was been taken from, is known.

Further take into account that these parameters cannot be varied unlimitedly. The ratio of the unloading/primary loading stiffness, $\lambda^*/\kappa^*$, cannot be smaller than 1 and should normally be between 2 and 10. Users should be very wary of values outside this range; for most practical cases the value of the $\lambda^*/\kappa^*$ ratio falls within the range of 3 to 7.

Secondly there is the creep ratio, $(\lambda^* - \kappa^*)/\mu^*$, to consider. This ratio can have a wide range of values, normally between 5 and 25, where high values represent stiff soils with little creep and small values represent soft soils with a considerable amount of creep. For most practical cases the ratio falls within the range of 10 to 20, and if the creep ratio is over 25 one could reconsider the use of the creep model. For these relatively stiff soils creep is of minor importance and the Soft Soil model or even Hardening Soil model could be used instead.

Therefore, it is strongly recommended to check the correctness of the parameters by simulating the oedometer test in Plaxis and comparing the calculation results with the laboratory findings. In case the Plaxis simulation does not resemble the test results, check carefully which parameters should be changed.

The Plaxis Curves program can easily plot graphs of volumetric strain vs. mean stress or time if a stress point is selected before starting the calculation. First, if the graph of the logarithm of mean stress $p'$ vs. volumetric strain $\epsilon_v$ does not match the original oedometer test results, adjust $\lambda^*$ and $\kappa^*$ in order to match the slopes of the settlement curve before and after the preconsolidation stress. Possibly also adjust the OCR to match the preconsolidation stress point, where the transition between reloading and virgin loading takes place. Secondly, if the graph of the logarithm of time vs. $\epsilon_v$ differs from the oedometer results the parameter $\mu^*$ has to be adjusted. Certainly, the creep rate also depends on the values of $\lambda^*$ and $\kappa^*$, but changing these parameters will also affect the primary loading and unloading-reloading behaviour and thus seriously affect the results of the simulation.

The Soft Soil model and the Soft Soil Creep model have a different default setting for the slope of the Critical State Line, the $M$-parameter. For the Soft Soil model the default value of the $M$-parameter is chosen such that the simulation of an oedometer test will give the most realistic results. The Soft Soil Creep model on the other hand chooses the $M$-parameter such that the simulation of an undrained triaxial test gives the best results. When calculations using the Soft Soil and the Soft Soil Creep model are compared, one has to manually alter the default value of the $M$-parameter in order to obtain similar results. This can be done by changing $K_{v'c}$ in the Advanced material properties window.

Although the parameter determination for the SSC model will sometimes be hindered by a practical lack of long-term oedometer measurements, it is often worthwhile to determine these parameters if possible. Although there remains a gap between accurately modelling a oedometer test and making a prediction for a practical case the SSC model offers enough possibilities for the accurate prediction of long-term settlements of embankments on soft marine clays and peats.

CLOSING REMARKS

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22
Q: How do I use tension cut-off with the linear elastic soil model?
A: The linear elastic model does not have a tension cut-off option. If this is needed the only solution is to use the Mohr-Coulomb model with a zero friction angle and a cohesion high enough that the failure criterium will never be reached. The soil will then always react elastic and the tension cut-off can be used.

Q: Where do I specify different stiffness parameters for dynamic analysis?
A: Currently Plaxis does not have a separate soil model specific for dynamic analysis and no separate parameters for dynamic analysis can be specified. A solution that is sometimes used is to specify a material set with the desired dynamic parameters as the stiffness parameters and exchange material sets in a plastic nil step prior to a dynamic analysis. After the dynamic analysis the material sets will need to be switched back again to ensure that the any subsequent static analysis is performed with the correct static parameters.

Q: In Plaxis 7.x it was possible to use an interface as a drain; is that still possible in Plaxis V8?
A: In Plaxis 7.x an interface had a longitudinal and transversal permeability of its own and by specifying a high longitudinal permeability the interface could be used as a drain. In Plaxis V8 the function of interfaces with respect to groundwater flow and consolidation has changed. Now an interface is either switched on or off; an interface that is switched on is fully impermeable, an interface that is switched off is fully permeable. For modeling drains the user should now use the special drain element for perfect drains. Another option would be to model the drains as thin clusters with a different material set if full control over the drain's permeability is required.

Q: If I model a ground anchor with a node-to-node anchor as the anchor rod and a geotextile for the grout body, should I use an interface along the geotextile to take possible slip of the anchor into account?
A: A ground anchor is relatively small construction element that acts locally. A plane strain model is a 1 meter slice in out-of-plane direction of a geometry where in principle everything is present over an infinite length in the out-of-plane direction. So when adding an interface along the grout body this interface will be present over the full length in out-of-plane direction of the model and will therefore decrease the strength of the model by creating a possible unwanted slip surface. In reality this slip surface only exists around the grout body and not in all the soil in between the anchors. It is therefore not realistic to use an interface along the grout body, as this has impact on the soil in between the anchors too.

Q: After generation of the initial stresses with the K0 procedure I often get a warning that plastic points have been created. How is this possible?
A: The K0 procedure indeed gives this warning if plastic points are generated. These plastic points either can be Mohr-Coulomb points, cap points or hardening points. Generation of Mohr-Coulomb points is usually caused when the K0 value is very small or very large; the resulting stress state will then violate the Mohr-Coulomb criterium. This can also happen when a rather high POP is specified. The stresses just below the soil surface in such a case often violate the Mohr-Coulomb criterium, because the POP will cause an increase of horizontal stresses. For the Hardening Soil and Soft Soil (Creep) model cap and hardening points are generated when no POP or OCR is specified; the stress state then coincides either the cap or the shear hardening envelop and those points are also reported as plastic points.
In general the occurrence of cap and hardening points is not a problem. In case of Mohr-Coulomb points it is advised to investigate if the proper values for soil strength, K0 and POP have been used.

Q: When I use the Hardening Soil model with undrained strength parameters it seems that there is no stress dependent stiffness anymore. Is this true?
A: The Hardening Soil model is defined in effective parameters and although it is possible to use the model with undrained strength parameters, the functionality of the Hardening Soil model will then be reduced. If a friction angle equal to zero is used, the model has actually no stress dependent stiffness. Compression hardening does not occur either in this case, as the compression hardening cap is not defined for the case that the friction angle is zero. However, shear hardening is still available, as is the separate unloading/reloading stiffness.
Activities

2005

4 - 6 January, 2005
Short course on Computational Geotechnics + Dynamics
(English)
New York, USA

7 January, 2005
1st North American Plaxis User Meeting

17 - 19 January 2005
International course Computational Geotechnics (English)
Nordwijkerhout, The Netherlands

24 - 26 January 2005
Geo-Frontiers
Austin, Texas

1 - 2 March 2005
Plaxis seminars in London and Manchester
United Kingdom

7 - 9 March 2005
Short course on Computational Geotechnics (German)
Stuttgart, Germany

21 - 24 March 2005
International course for Experienced Plaxis Users (English)
Nordwijkerhout, The Netherlands

May 2005
Short course on Computational Geotechnics (Italian)
Napoli, Italy

7 - 12 May 2005
ITA-AITES 2005
Istanbul, Turkey

17 - 20 May 2005
Short course on Computational Geotechnics (Spanish)
Bogotá, Colombia

19 May 2005
French User Meeting
Paris, France

2005

15 - 17 June 2005
5th International Symposium on Geotechnical Aspects of
Underground Construction in Soft Ground
Amsterdam, The Netherlands

19 - 24 June 2005
11th IACMAG
Turin, Italy

August 2005
Short course on Computational Geotechnics (English)
Houston, USA

August 2005
Short course on Computational Geotechnics (English)
Sydney, Australia

12 - 16 September 2005
12th International Conference on Soil Mechanics and
Geotechnical Engineering
Osaka, Japan

24 - 27 October 2005
International course for Experienced Plaxis Users (English)
Santiago de Querétaro, Qro. México

9 – 11 November 2005
12th European Plaxis User Meeting
Karlsruhe, Germany

November 2005
Short course on Computational Geotechnics (French)
Paris, France

CONTROL OF GROUND MOVEMENTS FOR A MULTI-LEVEL-ANCHORED
DIAPHRAGM WALL DURING EXCAVATION

Deep excavation in soft soils and complex groundwater conditions in Bogotá
Application of the Random Set Finite Element Method (RS-FEM) in Geotechnics