BULLETIN

Bulletin of the
PLAXIS
Users Association (NL)

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IN THIS ISSUE:

PLAXIS

Nº 11 - SEPTEMBER 2001

Editorial

Since the release of the 3D Tunnel program in April this year, users have been investigating the new possibilities this program offers. Results of research by Prof. Vermeer and Prof. Schweiger are published in Column Vermeer and PLAXIS Practice. Besides the well known topics as 'Column Vermeer', 'New Developments' and the 'Users-Forum' you will find a new item in this bulletin, 'Benchmarking'.

Benchmarking is a new and regular item for the PLAXIS bulletin, which will be presented by Prof. Helmut Schweiger. Prof. Schweiger has a strong belief in the PLAXIS product(s) and has stated that 'benchmarking will provide awareness for the sensitivity of results on particular assumptions which have to be made in numerical modelling and additional support to the PLAXIS community in order to improve the reliability of computational models and increase the confidence in numerical predictions'. In this edition he challenges the PLAXIS users to participate in the "first" PLAXIS benchmark (see Benchmarking).

In the previous bulletin the 3D Tunnel version release was announced and some practical case histories from the beta testers were presented. Now research has been done on the smart and practical use of the 3D Tunnel program (Column Vermeer, Plaxis Practice). In 'On a smart use of 3D-FEM in tunnelling' Prof. Vermeer gives an overview of the necessity of 3D calculations in some parts of the design and hints how to optimise 3D analysis to judge 2D calculations. Prof. Schweiger places some remarks on modelling NATM-tunnels with the 3D Tunnel program and additionally gives a suggestion for the modelling of face reinforcement.

The second article in PLAXIS Practice handles about the installation of ground anchors in saturated and fine sands. The solution to use sheet pile walls as anchor walls in Switzerland is described by H. Gysi.

To train users in the new and existing PLAXIS programs courses were given worldwide. The PLAXIS Advanced course Computational Geomechanics held at Noordwijkerhout was again a succes. In this course not only the second order models like the Harding Soil and the Soft-Soil creep models were taught, but also background information was given on the new 3D Tunnel program. Next to this event PLAXIS was involved in courses in Malaysia, Indonesia, the United Kingdom and the United States.

We hope that the research on the 3D Tunnel program published in this bulletin will open new possibilities for you and we are looking forward to the results of the first Benchmark test.

Column Vermeer

ON A SMART USE OF 3D-FEM IN TUNNELLING

At present, tunnels tend to be analysed on the basis of 2D finite element computations, because 3D analyses are considered to be extremely time consuming. As a result, 3D analyses are presently the domain of researchers. Consulting engineers will only perform 3D-FEM analyses when facing complex geometries, e.g. tunnel joints or connections to underground stations, but not for straight-ahead tunnelling. Indeed, 3D analysis can be very cumbersome and we better retain existing 2D approaches. However, it is smart to supplement existing 2D approaches by some 3D calculations. For judging possibilities, we should distinguish between the 3 main focuses of tunnel analyses, i.e.

A tunnel heading stability
B surface settlements
C loads on lining

Tunnel heading stability: I consider tunnel heading stability as the most important issue of tunnelling. For sandy soils and soft clays with low effective cohesion, this is obvious and one needs a shield with a face pressure to support the excavation front. On the other hand stiff clays and cemented sands are cohesive and it might seem that tunnel heading stability is not so much an issue. Indeed, highly cohesive soils do not need any face support, but economic NATM tunnelling requires a large unsupported excavation length. To reduce costs one thus maximizes the unsupported length. In this way one obviously increases the risk of a cave-in. Tunnel heading stability is thus a main issue for NATM tunnelling as well as shield tunnelling.

A 3D analysis of tunnel heading stability is extremely easy. One needs a simple block-type mesh as indicated in Fig. 1. At the boundaries the mesh may be coarse, but relatively small elements are needed at the tunnel face. The tunnel lining consists of stiff shell elements and may have any possible shape. All different soil layers can be modelled by the simple Mohr-Coulomb model. For settlement analyses, I prefer the Hardening Soil model, but for tunnel heading stability the focus is on soil strength and not on soil stiffness.

For NATM tunnelling, the entire analysis consists of one or two phases. In the first phase the lining is switched on and interior soil elements are switched off. PLAXIS will mostly need several computational steps, as we perform a staged construction analysis in which the supporting pressure within the tunnel is stepwise reduced. For small cohesion numbers ($c/\gamma D$ or $c_u/\gamma D$), this phase will lead to collapse, as indicated in Fig. 2. For sufficiently large cohesion numbers, the supporting pressure can be completely removed. In the latter case it is useful to perform an additional computational phase with φ-c-reduction to compute the factor of safety.

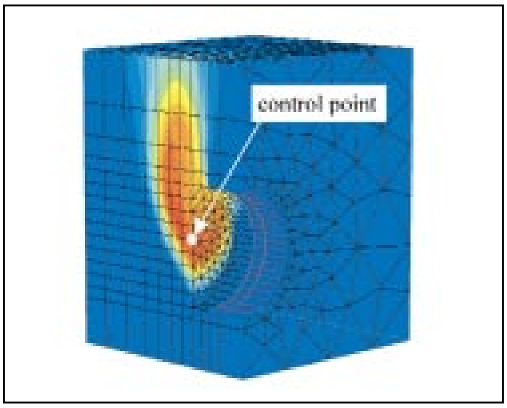

Fig. 1: Computed failure mechanism for a very shallow tunnel

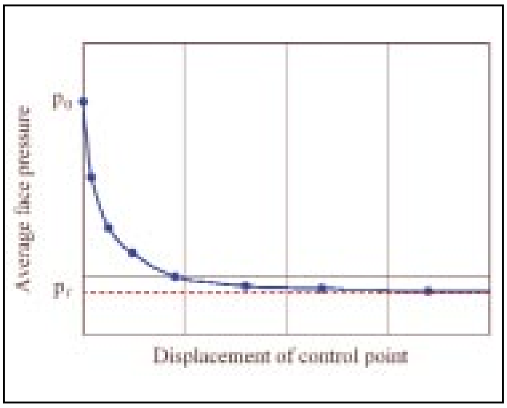

Fig. 2: Computed load-displacement curve on reducing face pressure. This is achieved by switching-off all soil elements inside the tunnel of Fig. 1

Shield tunnelling tends to be done in soils with little cohesion and the first computational phase will usually lead to collapse (a cave-in) as indicated in Figs 1 and 2. In case of an EPB-shield, one might perform an extra analysis for computing the ultimate collapse pressure.

As yet we have not had the slightest difficulty when performing stability calculations. Just like PLAXIS 2D, the 3D version appears to be a highly efficient tool for the computation of failure loads. In fact, it is a nearly tailor-made tool for analysing the stability of a tunnel heading. It is not yet completely tailor-made as we miss the option of 3D groundwater flow. This option is not needed for undrained soil layers, but drained layers may involve a destabilizing groundwater flow towards the tunnel face, at least in NATM tunnelling when the tunnel face is not sealed off by a filter cake. This groundwater option is considered for the next version of PLAXIS 3D Tunnel.

Surface settlements: Another important concern of tunnelling is the development of surface settlements. As for the tunnel heading stability a 3D-analysis is needed for a proper prediction of the settlement trough. To investigate its development we divided a block of 100 x 40 x 28 m in 3400 volume elements with a total of 10409 nodes (Fig. 3). For the parameters of the MC-model, we took E = 42 MPa, $\nu = 0.25$, c = 20 kPa, $\varphi = 20°$, $\psi = 0$ and $K_0 = 1 - \sin\varphi$. The NATM tunnel with a diameter of 8 m and a cover of 16 m was modelled with an unsupported excavation length of 2 m. Each computational phase consists thus of 2 m of excavation, in which one slice of soil elements is switched off. Within the same phase a ring of lining elements is switched on to support the previous excavation. The shotcrete lining has a thickness of 30 cm, a Young's Modulus of 20 Gpa and a Poisson's ratio of $\nu = 0$. This way of modelling a NATM tunnel will be referred to as 'step-by-step installation'. Figs 3 and 4 show the computed settlement trough after 30 excavation phases. The cross section of this trough compares well to the Gaussian shape as measured in practice, but the longitudinal shape is somewhat peculiar. In fact, a steady-state solution with a constant shape of the trough is only reached after about 35 m of excavation.

To investigate the reason of the peculiar longitudinal settlement distribution, we varied the initial conditions at the very beginning of excavation. Please note that Fig. 4 had been obtained by performing an unsupported excavation for 2 m of tunnel for the first computational phase. This analysis also leads to the lower curve in Fig. 5. In another analysis the first phase of excavation was changed into a supported excavation, but all further steps were unsupported excavations, as also considered in the analysis of Figs 3 and 4. The tremendous influence of the first excavation phase is demonstrated in Fig. 5. Depending on the starting conditions, initial settlements are either below or above the stationary solution. In both cases the disturbance extends over a considerable length of about 35 m.

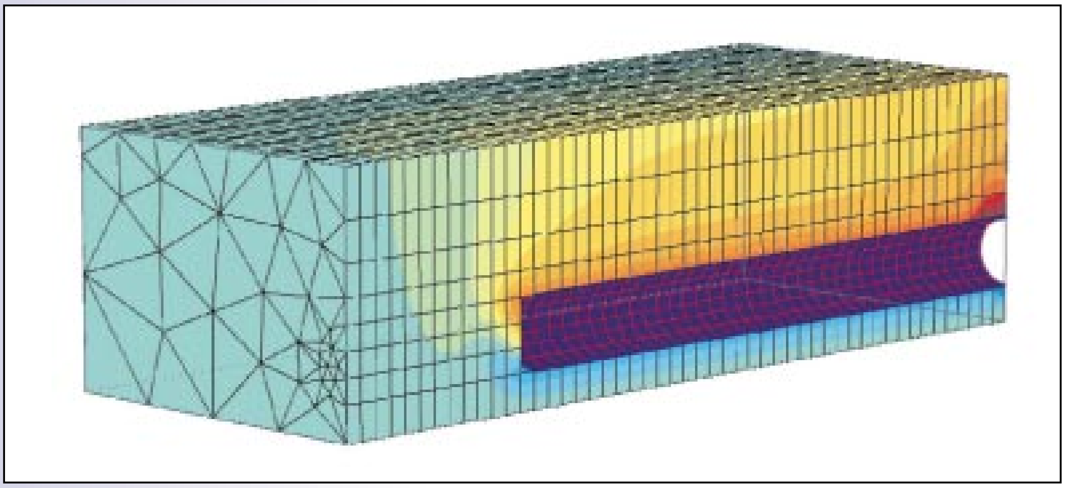

Fig. 3: Shading of vertical displacements after 60 m of stepwise excavation

PLAXIS

The considerable initial disturbance implies that one has to simulate tunnel excavations over a large length with many excavation phases, in order to arrive at a reliable steady state solution. For the present analysis, we needed about 4 hours computer run time on a fast 933 MHz PC with 512 MB of internal memory. For more complex NATM tunnels with staged construction sequences, much longer mesh lengths would have to be analysed and computer run times will become excessive.

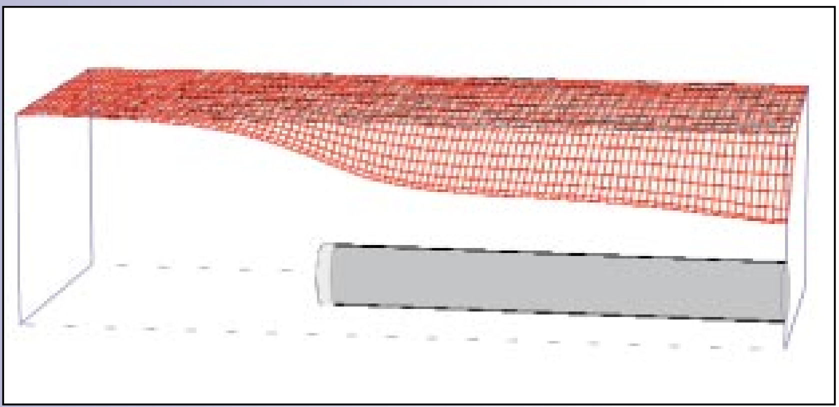

Fig. 4: Settlement trough after 60 m of stepwise excavation

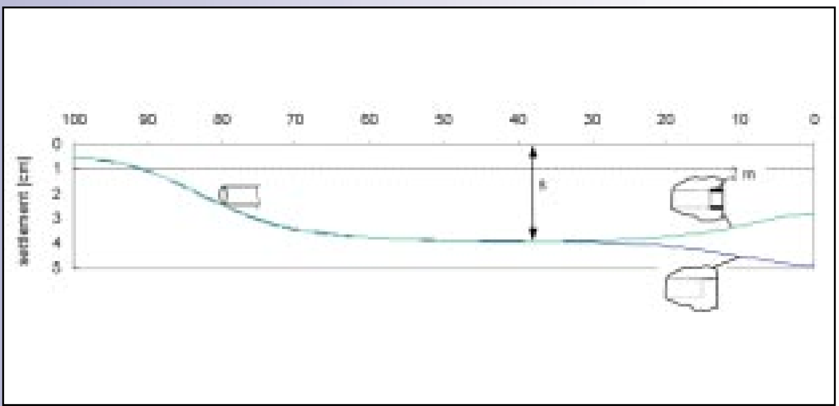

Fig. 5: Settlements above tunnel axis after 80 m of stepwise excavation. Upper curve for start with lining and lower curve for initial phase without lining

To avoid long lasting computations, we developed a fast way of settlement analysis. Instead of performing the time consuming method of step-by-step installation we perform only two phases. The first phase is used to install a more or less complete tunnel. To this end soil elements are switched off and lining elements are switched on over a considerable tunnel length. The second phase is used to model a single excavation with an unsupported length, i.e. 2 m for the present example, and all previous displacements are reset to zero. This method will be referred to as 'all-in-once installation'. One will now compute a settlement crater as indicated in Fig. 6. Its volume ΔV represents the volume loss for a single excavation. The relative volume loss can be computed as $\Delta V/V$ where V is the excavated volume in a single excavation. For the present example we found $\Delta V = 1.001\ m^3$. Together with $V = d\pi D^2/4$, where d = 2 m is the unsupported excavation length and D is the tunnel diameter, this gives a relative volume loss of $\Delta V/V = 0.020$. It can now be argued that this ratio should be equal to the steady–state ratio of $\Delta A/A$, where A is the tunnel area and ΔA the area of the steady-state settlement trough, as computed in a 3D calculation. Indeed, for the present example a direct computation yields $\Delta A/A = 0.022$, which compares well to the finding of $\Delta V/V = 0.020$. Having obtained the amount of relative volume loss, it is easy to predict the precise settlement distribution. To this end one may combine the computed volume loss with the empirical Gaussian curve (Mair, 1997). On using $\Delta V/V \approx \Delta A/A$ this yields a settlement of

$$S = \frac{\Delta A}{i \cdot \sqrt{2\pi}} \approx \frac{\Delta V/V}{i \cdot \sqrt{2\pi}} A$$

where i is the distance between the middle and the inflection point of the settlement trough. According to Mair (1997) it yields $i = k \cdot z_0$ where k is a constant and z_0 is the depth of the tunnel as indicated in Fig. 6. On using the empirical values of k = 0.5 for clays and k = 0.35 for sands (Mair,1997) a realistic depth of the settlement trough is obtained. Considering a tunnel in clay we obtain S = 3.86 cm for the case considered. This value agrees well with the settlement of 3.9 cm as obtained by the step-by-step installation.

Finally it should be noted that a 3D analysis of volume loss does not require the relatively fine mesh of Fig. 3. On computing the relative volume loss $\Delta V/V$ in two computational phases as described above, the fine mesh is only needed at the tunnel face in the middle of the mesh block. Towards the back and beyond the tunnel face the element size can be gradually increased. Moreover one does not need the long mesh block of Fig. 3, as it should only accommodate the settlement crater of a single excavation. On extending the simple 3D volume-loss analysis with a φ-c-reduction phase, both settlements and safety factors for tunnel heading stability are easily computed. The combined analysis of volume loss (ΔV) and tunnel heading stability took about 15 minutes of computer run time on our fast PC, whereas we needed about 4 hours to get Figs 3 to 5. As yet we did not consider volume loss computations for shield tunnelling, but similar to NATM tunnelling a relative simple procedure would seem to be feasible for shield tunnelling. It is concluded that the cost of a 3D analysis

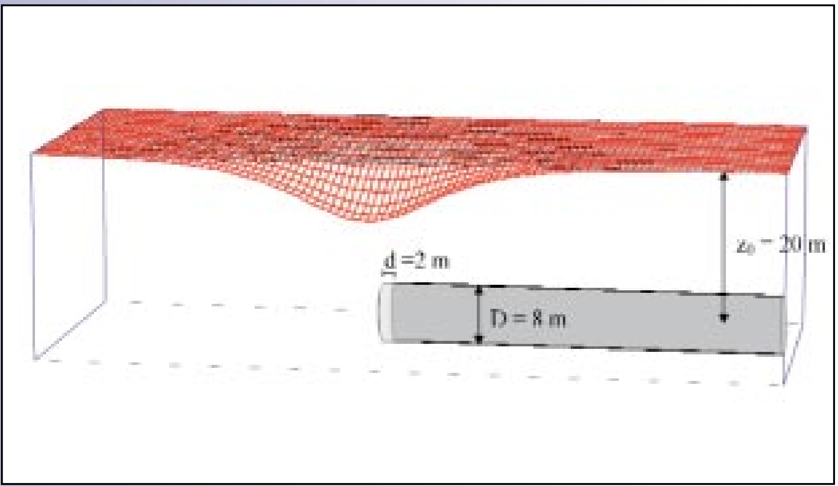

Fig. 6: Settlement crater from all-in-once installation

with 'step-by-step installation' is substantial and difficult to justify for many tunnel applications. However, we developed a smart procedure of 'all-in-once installation' that makes a 3D analysis fully feasible, both for tunnel heading stability and surface settlements. At present, research at Stuttgart University is aimed at a validation of this method for various different tunnel situations.

Bending moments and normal forces: The all-in-once installation will lead to extremely high incorrect loads on the lining, as it does not model any arching in the soil around the tunnel. For the present case we found M_{max} = 105 kNm/m and N_{max} = 2890 kN/m at the front of the lining. The real step-by-step extension of the tunnel yields a significant contraction of the unsupported head with associated arching around the tunnel heading. After the installation of the lining this arching will largely remain and the lining will hardly be loaded. Hence, in reality bending moments and normal forces are relatively small as also illustrated in Figs 7 and 8.

Fig. 7 presents normal forces after about 80 m of step-by-step installation. Within a single shotcrete ring of width d = 2 m there is a sharp drop of the normal force from about 1000 N/m at the front of the ring down to virtually zero at the back of the ring. At first glance I did not believe these zigzagging results and I asked Paul Bonnier of PLAXIS B.V. wether or not he had programmed the 3D shell-elements with sufficient accuracy. However, his programming proved to be excellent and now I consider the zigzagging data as logical. Indeed, the unsupported tunnel head is arching on the front and not on the back of a tunnel segment. The average normal force appears to have a magnitude of about 600 kN per metre of tunnel length. At the tunnel heading the normal force has not yet reached the average value of about 600 kN. Instead of a lower value of about 460 kN is obtained.

Just like the normal forces the bending moments show a zigzagging pattern that matches the step-by-step installation with d = 2 m. For convenience we will focus on the average value. Near the tunnel heading vanishing small bending moments of about -4 kN/m are found. However with the advance of the tunnel face the bending moment in Fig. 8 reaches an average steady state value of about –17 kNm/m. Beyond the steady state part on the extreme right in Fig.8 the lining is more heavily loaded up to –30 kNm/m. However, this is a numerical effect that relates to the use of smooth roller boundaries to the sides of the meshblock. No doubt, the zigzagging steady-state bending after some 20 m of tunnel excavation is realistic, but the cost of a step-by-step simulation is difficult to justify for many practical tunnel applications.

For analysing bending moments and normal forces I would advise to use a 2D FEM analysis, in which the lining is installed after a prescribed amount of unloading, i.e. the so-called β-method or λ-method. In this case an appropriate β-value will follow from the depth of the settlement trough, as computed from the smart 3D analysis.

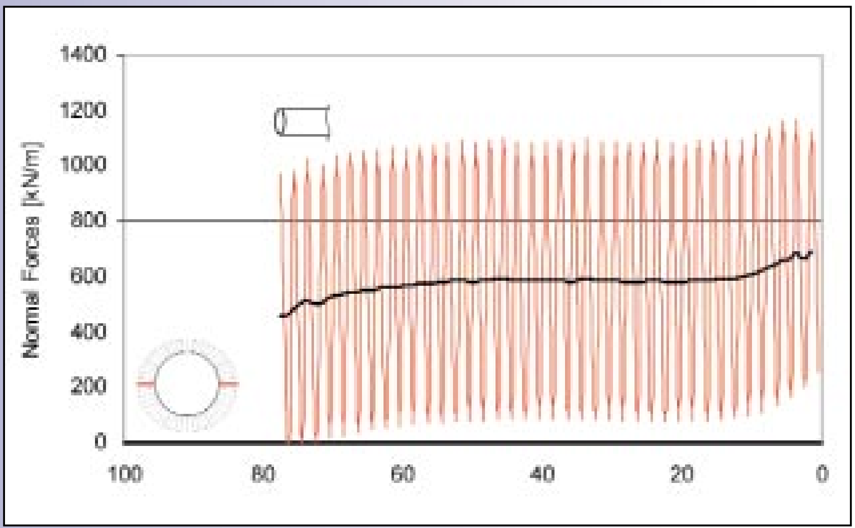

Fig. 7: The step-by-step installation of the tunnel leads to zigzagging normal forces in the ring direction of the lining

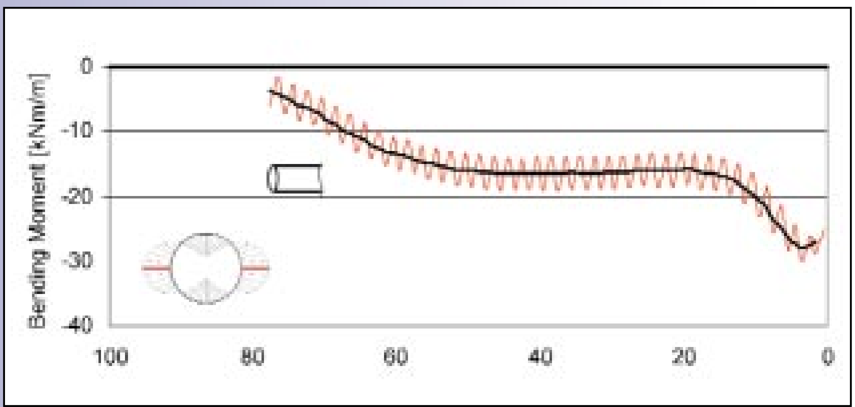

Fig. 8: The step-by-step installation of the tunnel leads to realistic zigzagging bending moments in the ring direction of the lining

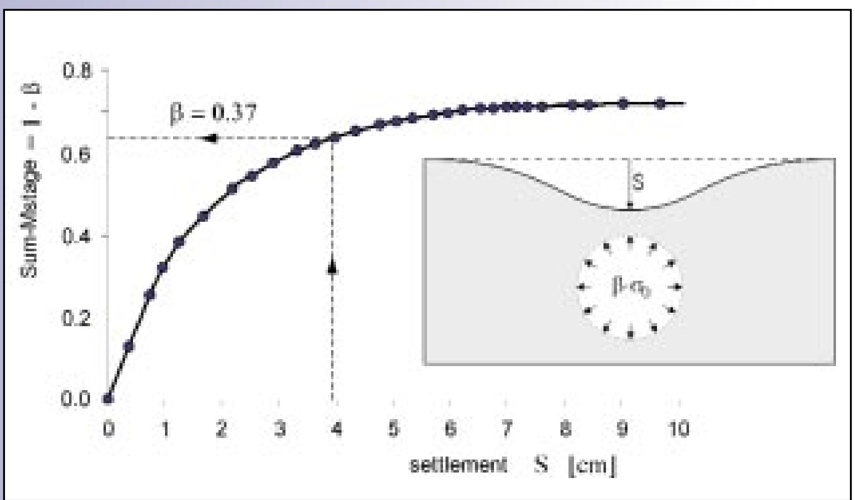

Fig. 9: 2D-tunnel analysis in which soil elements are gradually removed

From that analysis we obtained a depth of S = 3.9 cm for the settlement trough and this value can be used to select the appropriate β-factor. One may simply perform a 2D analysis to obtain the load-settlement curve of Fig. 9. For the present case it appears that a settlement of 3.9 cm corresponds to β = 0.37, i.e. 37% of the initial supporting pressure should be retained before installing the '2D' tunnel lining. After lining installation this 37% is also taken away and the lining will be loaded. For flexible linings an additional settlement will occur, but as a rule there will be little additional settlement due to the loading of the lining.

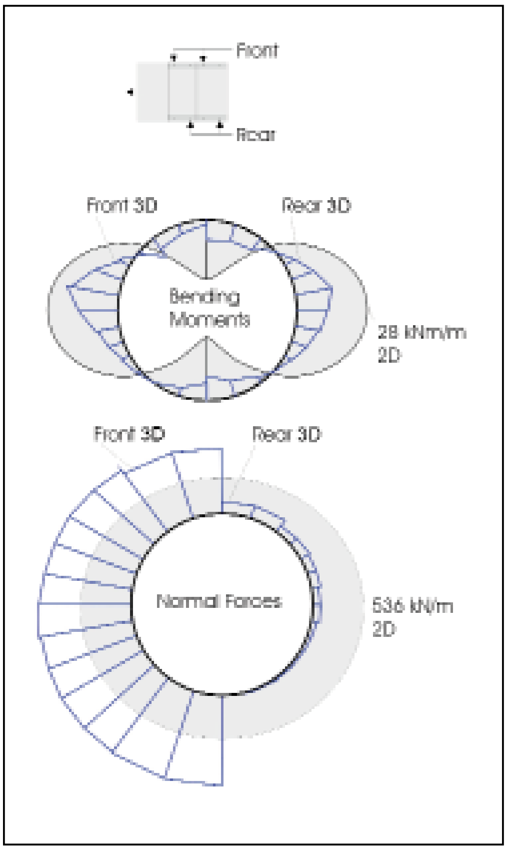

Fig. 10: Bending moments and normal forces

For the present case, we find the bending moments and normal forces as illustrated in Fig. 10. This figure can be used to compare the 2D-data to the zigzagging 3D-data of Figs. 7 and 8. It appears that 2D bending moments are slightly larger than the ones from the 3D step-by-step installation, but differences are modest. 2D normal forces appear to match the average ones from a 3D-analysis quite well. This average value would seem to be a highly realistic value, as the zigzagging in Figs 7 and 8 is most probably excessively large. Shotcrete shows substantial creep and stress-relaxation so that one may expect a considerable damping of all oscillations around the average values.

It is concluded that a very simple 3D-analysis yields proper informations on tunnel heading stability as well as the β-factor for further 2D analysis. As yet we did not validate this idea for many different situations, including staged construction, but we hope to obtain a full validation within the scope of current research project at Stuttgart university.

P.A. Vermeer, Stuttgart University

Literature: R.J. Mair and R.M. Taylor (1997). Bored tunnelling in the urban environment. Proceedings of the 14th Int. Conf. on Soil Mech. and Found. Eng., Vol 4 pp 2353-2385, Hamburg.

New Developments

Scientific PLAXIS developments are organised in bi-annual projects. At the moment we are finalising the activities in the current project. The 3D Tunnel program is one of the results of the current project. In the coming period, the remaining scientific developments of this project will be made operational for PLAXIS Version 8. The latter version will be released in 2002.

Meanwhile we are planning the next project on PLAXIS developments. This project is mainly based on modelling wishes from existing PLAXIS users. In the project attention is focused on the determination and variability of model parameters and on specific 3D modelling aspects in various applications. The project will include the following subjects:

Subjects:

1. Parameter determination and variation
2. Soil nailing and reinforcement with respect to tunneling
3. Structural behaviour and 3D aspects of excavations
4. Anisotropy and safety aspects of embankments
5. Dynamics
6. Thermal flow

Ad 1:

One of the key issues of finite element modelling is the proper selection of model parameters. There are several methods to determine model parameters, either directly from (in-situ or lab) soil testing data or indirectly from correlations with other soil data. The purpose of this subject is to provide a facility in the PLAXIS material data base to enhance the parameter determination by including formulas, correlations, rules of thumb or design charts for parameter determination. Further more, we will create a possibility to quickly simulate common lab tests with created data sets and enable a comparison with real soil testing data in order to optimise the parameters.

When model parameters are correctly determined and optimised from soil teststing data, it should be realised that the parameters may be different and variable in the soil layer. Therefore a facility will be created in PLAXIS to enable an easy repetition of calculations with a variation of parameters. In this way the sensitivity of parameters on the computational results can be evaluated. Parameter variations can be seen as a simple range values or as a stochastic distribution. Using the latter approach, an extension can be made towards a probabilistic analysis. When taking measured data into account, a next step can be made in the direction of inverse analysis.

Ad 2:

After the release of the 3D Tunnel program, there is a need for an improved modelling of soil nailing and reinforcements. The interaction between nails and the surrounding soil is highly non-linear. This also applies to other types of reinforcements used in tunneling projects. Instead of modelling the reinforcement and the soil-structure interaction in detail, special types of elements will be developed in which the non-linear behaviour is implicitly included.

Ad 3:

In addition to the 3D Tunnel program there is a need for a 3D program that is suitable for the analysis of deep excavations. In contrast to the 3D Tunnel program, a 3D program for excavations can be based on a top view modelling, using 'boreholes' to define the layering in the subsoil. The behaviour of structural elements and staged construction facilities are of major importance in such a program.

To properly take into account pore pressure distributions, a 3D groundwater flow calculation program will be developed. Groundwater flow calculations will be made available within all 3D PLAXIS programs.

Ad 4:

The development of constitutive models has always been a major issue in PLAXIS developments. In the current project, attention is focused on anisotropy of soft soils, which is particularly important for embankment types of applications. It is the intention to improve the capabilities of existing models without making them more difficult to use.

Another issue that is particularly relevant to embankments concerns stability. Stability of embankments and other soil structures, including structural elements and loads, will be studied in the framework of European regulations (EC7). This should lead to an improvement of the method of phi-c reduction for the calculation of safety factors in which loads and structural elements are explicitly taken into account.

Ad 5:

A first version of the dynamic module for 2D applications was released in April 2000. This version uses the existing models to describe the behaviour of the soil. However, an improvement is needed in terms of small strain stiffness, cyclic loading effects, hysteretic damping and liquefaction. Moreover, there is a strong need for a 3D dynamic calculations program, since the evolution of compression waves and shear waves in the soil is highly three-dimensional.

Ad. 6:

For some special applications in geotechnical engineering it is necessary to perform thermal calculations in order to determine the temperature distribution in the soil. Heat mining, i.e. the use of increased temperatures in the subsoil for house heating purposes is one of these applications. Other applications may be found in soil freezing near excavations to avoid inflow of groundwater. A new module will be developed in which thermal flow will first be considered as a steady state flow process (similar to groundwater flow). Future extensions may also include time-dependent effects and eventually an interaction with deformation calculations using temperature-dependent soil models.

Apart from scientific developments as described in the subjects 1 to 6, we will spend quite some time on the robustness of the models and a userfriendly implementation, which is vital to keep the PLAXIS programs convenient to use. It is an ambitious project, which will be executed in cooperation with different universities. We will do our utmost to meet your current and future modelling requirements.

Ronald Brinkgreve
PLAXIS BV

Benchmarking

A NEW REGULAR SECTION IN THE BULLETIN

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 at 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. The first problem specification is given in the following, it addresses the simplified analysis of a shield tunnel in PLAXIS version 7.

PLAXIS Benchmark No.1: 2D-Shield Tunnel 1

We consider undrained conditions and 3 analyses in terms of total stresses should be performed in plane strain conditions:

A) elastic analysis, no lining, uniform initial stress state

B) elastic-perfectly plastic analysis, no lining, K_o = 1.0

C) elastic-perfectly plastic analysis, segmental lining, K_o = 1.0, given ground loss

The tunnel diameter is given as 10 m (corresponds to centre line of lining in case C) and the overburden (measured from crown to surface) is assumed to be 15 m. At a depth of 45 m below surface bedrock can be assumed (see Figure 1).

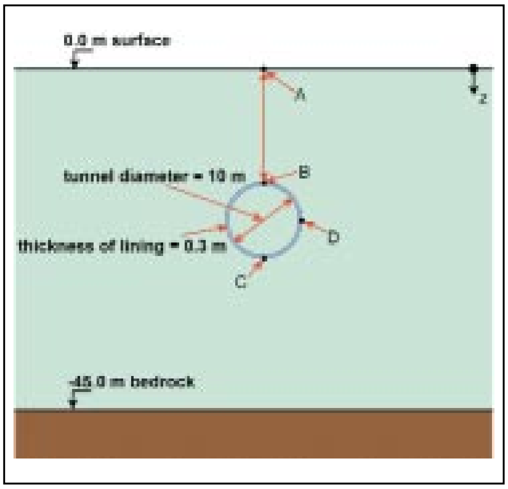

Figure 1 Geometric data for benchmark Shield Tunnel 1

Specification for analysis A)

Soil parameters (elastic):

G = 12 000 kPa

ν = 0.495

uniform initial stress state: $\sigma_v = \sigma_h$ = 400 kPa

no lining to be considered

Computational step to be performed:
full excavation

Specification for analysis B)
Soil parameters (elastic-perfectly plastic):
G = 12 000 kPa
ν = 0.495
γ = 20 kN/m^3
undrained shear strength c_u (s_u) = 130 kPa
initial stress state: $\sigma_v = \gamma z$, $\sigma_h = K_o \gamma z$
K_o = 1.0
no lining to be considered

Computational step to be performed:
full excavation

Specification for analysis C)
Soil parameters (elastic-perfectly plastic):
G = 12 000 kPa
ν = 0.495
γ = 20 kN/m^3
undrained shear strength c_u (s_u) = 60 kPa
initial stress state: $\sigma_v = \gamma z$, $\sigma_h = K_o \gamma z$
K_o = 1.0

Parameters for segmental lining:
$E = 2.1 \times 10^7$ kPa (value assumed to cover possible reduction due to hinges)
Lining thickness, d = 0.3 m
ν = 0.18
γ = 24 kN/m^3

Computational step to be performed:
full excavation with assumed ground loss of 2%
Note: for simplicity it is assumed here that no ground loss occurs at the tunnel face (approximately justified for an EPB-shield).

REQUIRED RESULTS FOR ALL ANALYSES
- surface settlement profile
- horizontal settlements at surface
- vertical displacement at surface, crown and invert (points A, B and C in Figure 1)
- vertical and horizontal displacement at sidewall (point D in Figure 1)

ADDITIONAL RESULTS FOR ANALYSIS C
- bending moments in lining
- normal forces in lining
- normal pressure acting on lining

Note: As far as possible results should be provided not only in print but also on disk (preferably EXCEL) or in ASCII-format respectively. Results may also be submitted via e-mail to the address given below.

Results should be sent before 30.11.2001 to:

Prof. H.F. Schweiger

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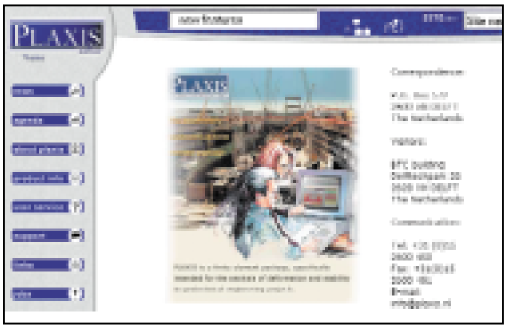
http://www.tu-graz.ac.at/geotechnical_group/

NEW PLAXIS web-site

A few weeks ago, the new PLAXIS website became active. It has not become a flashy site, but we decided to focus on it's informative character. The entire website has been restructured, so that information can be located more easily.



Besides the contents of current and previous PLAXIS bulletins, we plan to include many other publications. Some of our users have published papers for conferences, related to the use of PLAXIS. We would like to encourage these people to allow us to include their reference, or even better, to include the actual publication on our website. Please send your publication (in word or pdf format) to bulletin@plaxis.nl.

Also included on the website are on-line manuals. Both manuals for the Professional Version and the 3D Tunnel program are available. This will allow non-users to evaluate PLAXIS software in more detail.

A new powerful addition is the users-forum. This forum is the place for users of PLAXIS software to come together and help each other with tips, tricks and other kind of help. You can choose any of the subjects and browse through reactions or even post a message yourself.

We look forward to seeing you at www.plaxis.nl

PLAXIS Practice I

CUT AND COVER TUNNEL WITH SHEET PILE WALLS USED AS ANCHOR WALLS IN SWITZERLAND

1. Introduction

The plain between Solothurn and Biel, specially the so called Grenchner Witi, is one of the most important swamp areas of Switzerland and is therefore protected by law. For this reason the new motorway between Solothurn and Biel crosses the central part of this protected area in a tunnel.

2. Project

Length of tunnel	1760 m
Length of west ramp	285 m
Length of east ramp	359 m
Total length of construction	2404 m
Overall width of construction	30 m
Deepest excavation	10.70 m
Begin of construction works	autumn 1998
End of construction works	summer 2000
Overall costs	180 million CHF

3. Geotechnical Conditions

After the retreat of the Rhone glacier sandy sediments of more than 40 m thickness were deposited at the bottom of an ancient lake. On their top follows a layer of young lake sediments, consisting mainly of clay or clayey silt with organic matter. The groundwater table coincides almost with the ground surface, therefore flooding is quite frequent.

4. Construction Procedure

The motorway section was built as a cut and cover tunnel. Because the installation of ground anchors in the saturated, loose to medium dense fine sands was a mayor risk with regard to the time schedule, the contractor made an alternative proposal with two sheet pile walls used as anchor walls. These anchor walls had a length of 12 m and were driven at 14 m distance from the main sheet pile walls. From the main sheet pile wall anchor rods were drilled to the anchor wall. The distance between the single anchor rods was 4 m.

As a first step the 12 m long anchor walls were driven into the soil in two rows at a distance of 58 m. A first excavation step with slopes on each side was cut down to a depth of -4.10 m within the anchor walls (see fig. 1). From this level the 18 m long main sheet pile walls were driven at a distance of approximately 30 m from each other respectively at 14 m from the anchor walls (see fig. 1). The next excavation phase within the main sheet pile walls reached the level of -7.00 m. Now the 14 m long achor rods were drilled from the main sheet pile wall to the anchor wall at a distance of 4 m. After their prestressing to 1000 kN the excavation could proceed to the final depth of -10.70 m. The lowering of the groundwater table was performed by deep wells located in the centre of the excavation and along the inner sides of the main sheet pile walls. The depth level of the wells was 2 m less than the tip of the main sheet pile walls. Thanks to the safe and quick anchoring system two tunnel sections of 12.5 m length could be constructed each week.

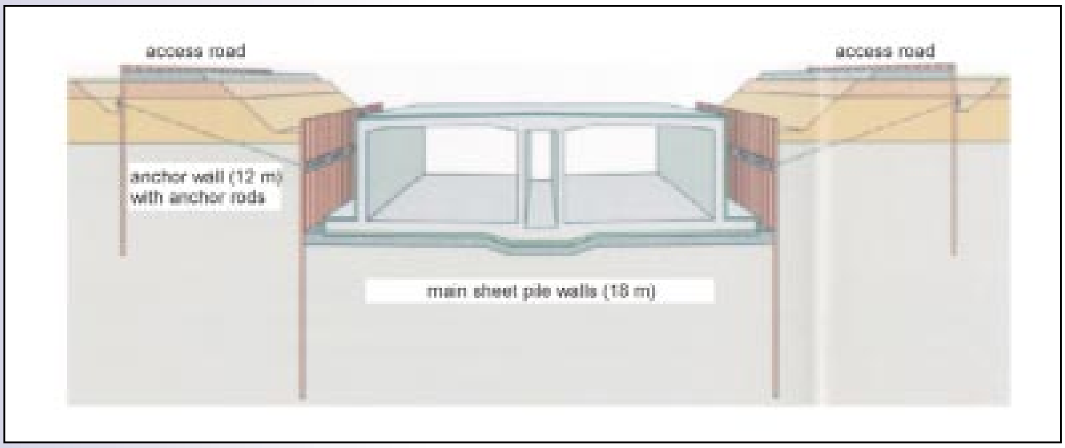


Figure 1
General cross section

5. Calculations

The initial calculations were performed with the usual statical programs based on beam theory and limit equilibrium loading. Using the contractors alternative proposal, the combined behaviour of the main sheet pile wall, the anchor rod, the anchor wall and the soil between the walls was of greatest interest, specially with regard to deformations and bending moments of the sheet pile walls. The calculations were made with the PLAXIS program version 7. The finite element mesh is given in fig. 2.

- hardening soil model
- plane strain 15 node elements
- 550 elements

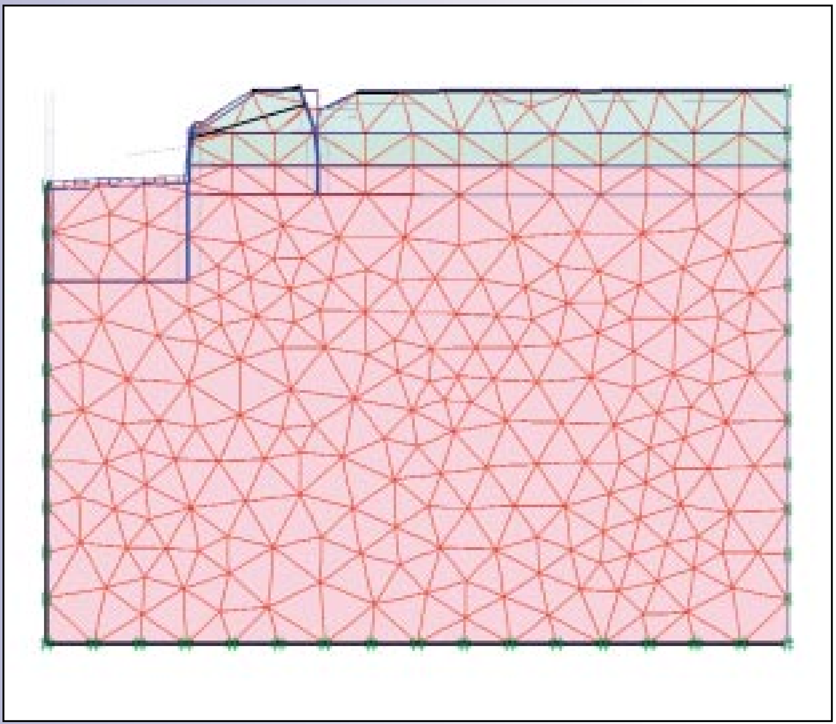


Figure 2 Deformed mesh, max. displacement 173.3 mm

The following layers of soil are modelled:

layer 1: recent lake sediments, clayey silt or silt with large amounts of organic matter

layer 2: transitional zone, fine sand and silt with some organic matter

layer 3: fine to medium sand

The soil properties are shown in table 1

- The interfaces were set to "impermeable" for the steady state groundwater flow calculations. The results of the groundwater flow calculations for the final dewatering step are shown in fig. 3.

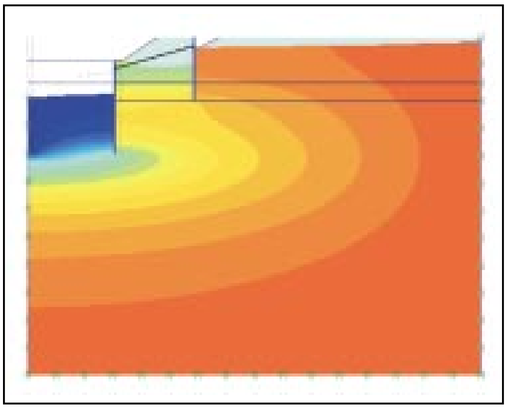


Figure 3 Groundwater head contours, 59.50 – 49.00 m

- sheet pile walls
  main sheet pile wall: Larssen 24
  anchor wall Larssen 23

- anchor rod
  high tensile steel
  tensile force 2000 kN
  prestressing force 1000 kN (250 kN/m)
  distance between anchors 4 m

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Table 1 Soil parameters

Parameter	Symbol	Layer 1	Layer 2	Layer 3	Unit
Thickness		4.6	3.5	>40	m
Material model	Model	Hardening-Soil	Hardening-Soil	Hardening-Soil	-
Type of behaviour	Type	Drained	Drained	Drained	-
Dry weight	γ_{sat}	17.0	18.0	18.0	kN/m^3
Wet weight	γ_{unsat}	19.0	20.0	20.0	kN/m^3
Horizontal permeability	k_h	1.0	1.0	1.0	m/day
Vertical permeability	k_v	0.05	0.05	0.05	m/day
Young's modulus	E_{50}^{ref}	5.0 10^3	$2.0 \cdot 10^4$	$6.0 \cdot 10^4$	kN/m^2
Oedometer modulus	E_{oed}	5.0 10^3	$2.0 \cdot 10^4$	$6.0 \cdot 10^4$	kN/m^2
Power	m	0.5	0.5	0.5	-
Unloading modulus	E_{ur}^{ref}	1.5 10^4	$6.0 \cdot 10^4$	$1.8 \cdot 10^5$	kN/m^2
Poisson's ratio	ν	0.2	0.2	0.2	-
Reference stress	p_{ref}	100.0	100.0	100.0	kN/m^2
Cohesion	c_{ref}	10.0	1.0	1.0	kN/m^2
Friction angle	φ	27.0	33.0	33.0	
Dilatancy angle	ψ	0.0	0.0	4.0	
Interface strength reduction	R_{inter}	1.0 (rigid)	1.0 (rigid)	1.0 (rigid)	-

- calculation procedure

phase 1: soil weight using Mweight = 1

phase 2: installation of anchor wall

phase 3: excavation behind anchor wall, with preceeding groundwater flow calculation of change of groundwater table

phase 4: first excavation within anchor walls down to -4.10, with preceeding groundwater flow calculation of change of groundwater table

phase 5: installation of main sheet pile wall

phase 6: second excavation within main sheet pile walls down to -7.00 m, with preceeding groundwater flow calculation of change of groundwater table

phase 7: installation of tensile rods, prestressing to 300 kN/m (instead of the 250 kN/m in reality)

phase 8: third excavation within main sheet pile walls down to -10.70 m, with preceeding groundwater flow calculation of change of groundwater table

- Results

FINAL EXCAVATION STAGE

main sheet pile wall

max. deformation: 54.7 mm (see fig. 4)

min. bending moment: -413.1 kNm/m (see fig. 5)

max. bending moment: +195.6 kNm/m (see fig. 5)

anchor wall

max. deformation: 171.6 mm (see fig. 6)

max. bending moment: +245.2 kNm/m (see fig. 7)

tensile force in anchor rod

max. force 265.1 kN/m

STAGE OF ANCHOR ROD INSTALLATION

main sheet pile wall

max. deformation 9.9 mm (towards soil)

min. bending moment -148.8 kNm/m

max. bending moment +49.8 kNm/m

anchor wall

max. deformation 112.1 mm

max. bending moment +316.7 kNm/m

tensile force in anchor rod

prestressing force 300.0 kN/m

It is important to note that the main deformations at the anchor wall occur during prestressing of the anchor rod (112.1 mm). The increase of deformation at the anchor wall due to the last excavation step is 59.5 mm, at the main sheet pile wall 50.0 mm. The chart of mobilised shear strength for the final excavation stage is shown in fig. 8.

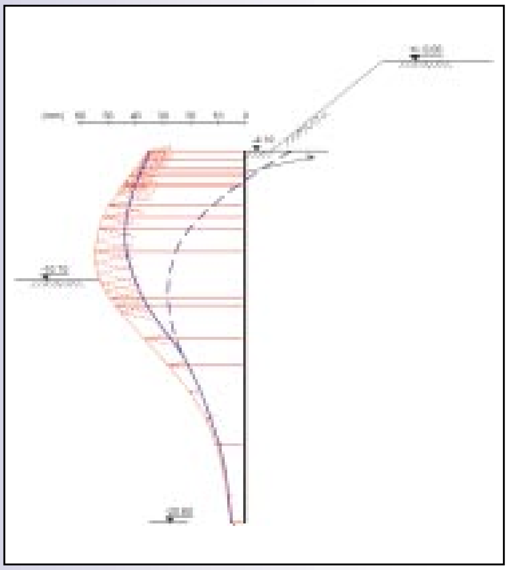

Figure 4 Horizontal displacements of main sheet pile wall

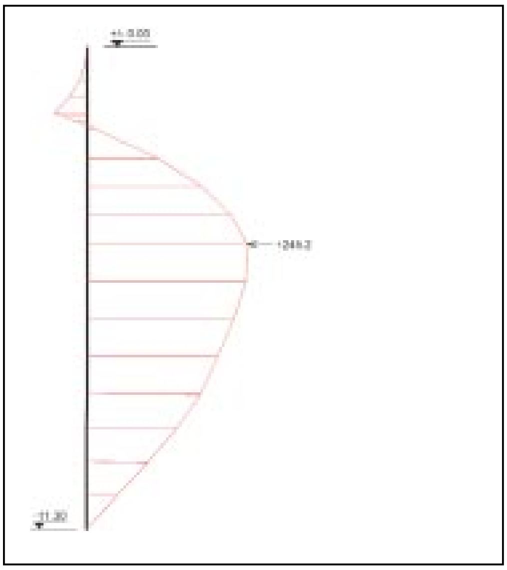

Figure 7 Bending moment anchor wall

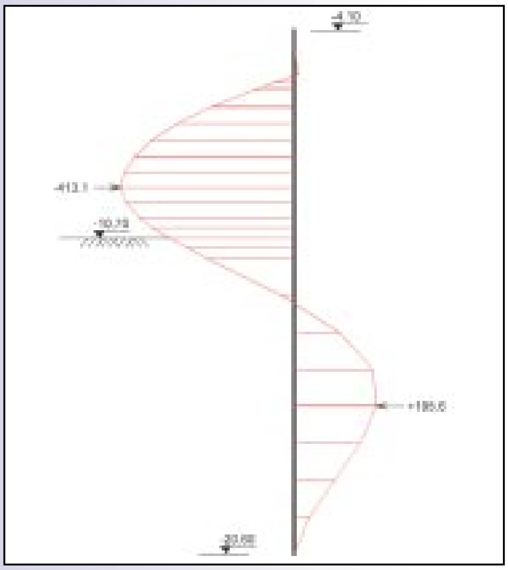

Figure 5 Bending moment main sheet pile wall

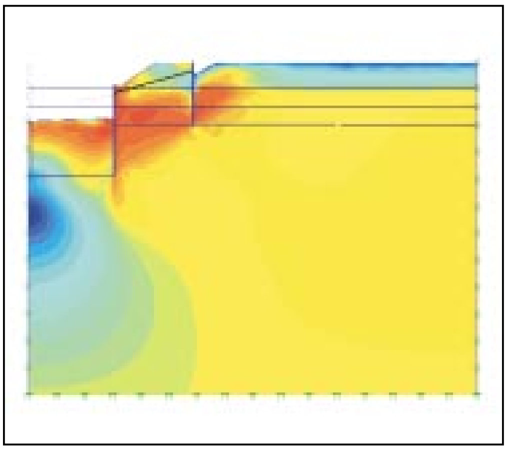

Figure 8 Relative shear stresses

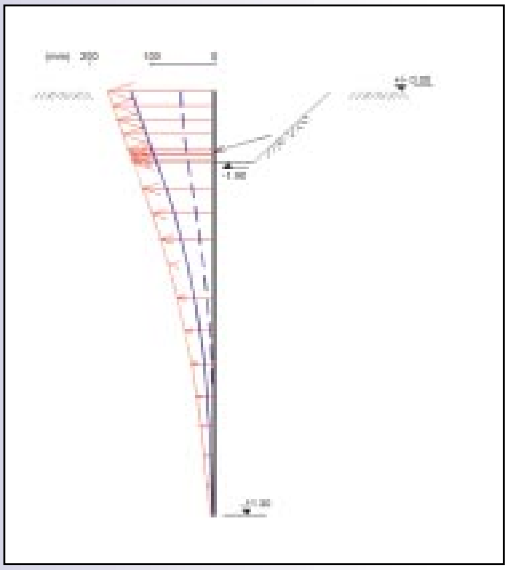

Figure 6 Horizontal displacements of anchor wall

6. Measurements on Site

The following measurements were performed at several typical cross sections.

- inclinometer at main sheet pile wall
- inclinometer at anchor wall
- tensile force in anchor rod
- groundwater table

The measured deformations are shown in fig. 4 for the main sheet pile wall. The full line represents the maximum value, the dashed line the minimum value. The maximum measured deformations reach approximately 80 % of the calculated deformations. Please note that the clinometric line has been shifted to fit the toe of the main sheet pile wall, because the clinometric measurements give only a relative displacement curve. Fig. 6 contains the results of the deformation measurements at the anchor wall. The maximum measured deformations are about 77 % of the calculated ones.

The tensile force in the anchor rod was measured to be 820 kN after the final excavation. Before the last excavation step the prestressing in the anchor rod was 1000 kN. Therefore a drop of 180 kN during excavation was measured.

In the original safety documents for the main sheet pile wall combined with prestressed ground anchors the allowable horizontal deformations were set to 50 mm. For alternative design, the allowable deformations of the main sheet pile wall were kept at 50 mm, the allowable deformations of the anchor wall were limited to 170 mm in accordance with the predicted PLAXIS deformations. Due to the absence of any buildings and other facilities such large deformations could be tolerated.

7. Conclusions

The calculated deformations of the main sheet pile wall and the anchor wall coincide reasonably well with the measured maximum deformations. The smaller measured values (indicated as dashed lines in fig. 4 and 6) can be explained with a deeper actual groundwater table (due to dewatering), better soil properties and a smaller thickness of the weak surface layers. Furthermore the magnitude of prestressing force in the calculation model was assumed to be 1200 kN (300 kN/m) instead of 1000 kN in reality, leading as well to smaller deformations mainly at the anchor wall. Thanks to PLAXIS the rather large deformations of the anchor wall could be predicted in good agreement with the real behaviour. This helped to avoid extensive discussions about permissible deformations during construction.

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PLAXIS Practice II

SOME REMARKS ON MODELLING NATM-TUNNELS WITH PLAXIS 3D

With the release of the PLAXIS 3D Tunnel program it is now possible to model a typical sequential tunnel excavation in accordance with the principles of the New Austrian Tunnelling Method (NATM) more realistically. So far plane strain models have been employed and in order to account for 3D-effects so-called pre-relaxation factors had to be used. This could be achieved in PLAXIS 2D by setting Σ-Mstage – values < 1.0 (β – method) and thus deformations and stress redistributions took place before the shotcrete lining was put in place. However, estimation of β-values is not easy and is based purely on practical experience. With the 3D Tunnel program estimation of β-values is no longer required because the excavation stages can be modelled not only in the cross section but also in the longitudinal section, e.g. excavation of the bench and invert can be modelled in the actual distance behind the excavation of the top heading.

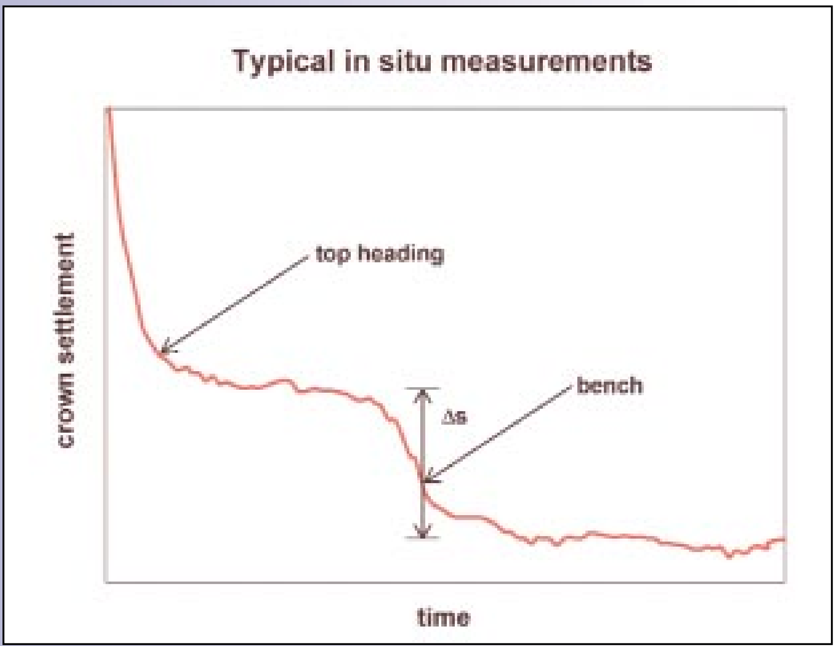
However, what looks straightforward in the first place turns out to be not completely trivial and therefore some remarks on 3D Tunnel modelling simulating sequential excavation will be made in this note. In addition, a possibility of modelling face reinforcement is suggested.

NATM tunnel with sequential excavation

First of all one has to be aware that 3D models easily become very big. What looks a bit crude in 2D can be far too fine for a 3D analysis if one considers a realistic number of slices in the direction of the tunnel axis. So one needs to be careful with the number of elements created in the cross section. The number of slices required depends on the distance between the excavation faces of top heading, bench and invert respectively. It may not be required to model the invert distance in detail (except in very difficult ground conditions and in these cases the invert is usually constructed in a closer distance to the top heading anyway), but it is important to model the distance between top heading and bench because a significant increase of displacements is often observed in practice when the bench passes a particular cross section. This increase in vertical displacement is denoted as Δs in Figure 1, which shows a typical crown displacement vs time curve. It has to be mentioned though that the increase of crown settlements due to bench excavation is usually underestimated in the numerical model (in particular when a Mohr-Coulomb Model is used). Further studies are under progress at the moment, but it seems as one would need a constitutive model incorporating small strain stiffness effects in order to obtain results in better agreement with in situ experience. Whether the "smart use of 3D-FEM" (see Column Vermeer in this issue) is applicable to an excavation sequence top heading/bench/invert under difficult ground conditions with highly nonlinear soil/rock behaviour has to be further investigated.

Furthermore, the placing of the boundary condition needs some attention. Careful studies show that you need about 4-5D (D = diameter of tunnel) of distance from the boundary in order to avoid boundary effects. This leads to quite long models for typical distances of top heading and bench. The next crucial point is the excavation length which governs the maximum thickness of the slices.

Figure 1
Typical development of crown settlements in NATM tunnelling



A round length of 2 m may be a reasonable assumption for many cases, but in very difficult ground conditions, and these are of course the ones where a numerical analyses are essential, the round length may be only 1 m or even below.

As an example of such an analysis the shadings of vertical displacements are shown in Figure 2 at a particular advance of the top heading. The model consists of appr. 5 400 elements and 15 500 nodes. With 256 MB of RAM it cannot be solved in core (if it can be solved in core, calculation times will be reduced considerably), but the complete excavation sequence (roughly 55 load cases) can be calculated in about 24 hours on a standard PC, which is still very reasonable for a 3D calculation of this size (improvements on the solver in progress at the moment will speed up run times significantly in near future). Figure 3 shows the deformed structure at a later excavation stage with bench and invert also advancing.

In Figure 4 the (normalized) settlement trough on the surface perpendicular to the tunnel axis is shown at a particular cross section, again for different stages of tunnel advance. The increase of settlements after passing of the top heading is clearly evident. In addition, two Gaussian curves which represent roughly the bounds observed in situ (K between 0.4 and 0.6, Mair & Taylor, 1997) are included in Figure 4. It can be seen that the shape of the settlement trough obtained from finite elements is wider than the Gaussian distribution observed in practice suggests. However, this is a well known fact and may be even slightly more pronounced when the Mohr-Coulomb model is used and not the Hardening Soil Model as in this case. Again only very advanced constitutive models including small strain stiffness effects may significantly improve the shape of the settlement trough (Addenbrooke, Potts & Puzrin, 1997).

As far as the shotcrete modelling is concerned the increase of stiffness of the lining has been modelled as an approximation of the real time dependent behaviour in such a way that two sets of parameters for the shotcrete have been created ("young" and "old" shotcrete). When placing the lining for the first time the set "young" has been assigned to the lining and for further excavation steps this has been changed to the set "old".

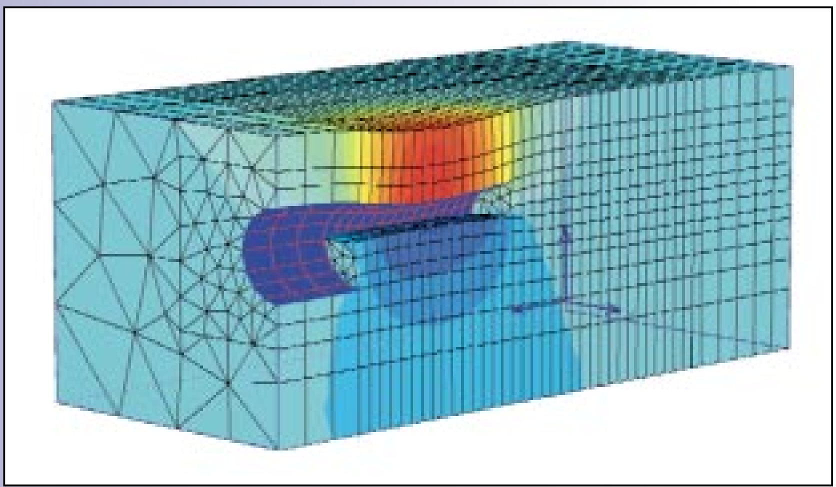


Figure 2 Shadings of vertical displacements for top heading advance

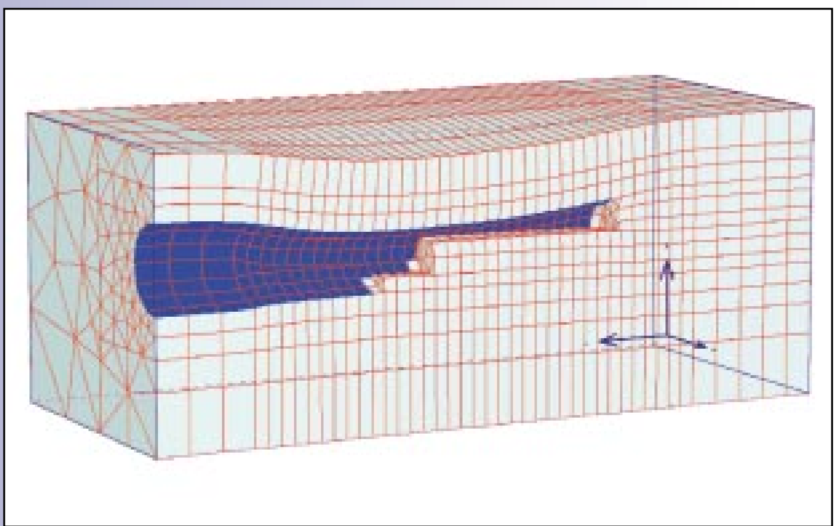


Figure 3 Deformed mesh for excavation step with advancing top heading, bench and invert

Another nice feature of the 3D Tunnel module has not been used here but is worth mentioning. When creating the tunnel with the tunnel designer (Figure 5) it is now possible to specify the thickness of the inner lining (usually cast concrete). Thus the long term situation, where it is assumed that the shotcrete does not carry any load, can be accommodated for in a straightforward manner by eliminating the beam elements for the shotcrete and introducing properties of concrete for the geometrically predefined inner lining.

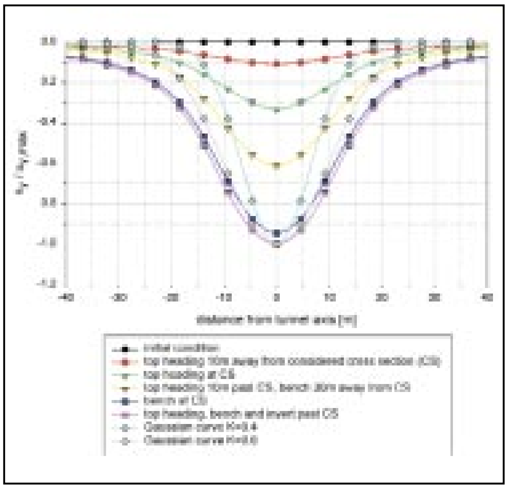


Figure 4 Calculated settlement trough at a particular cross section and comparison with typical Gaussian curves observed in practice

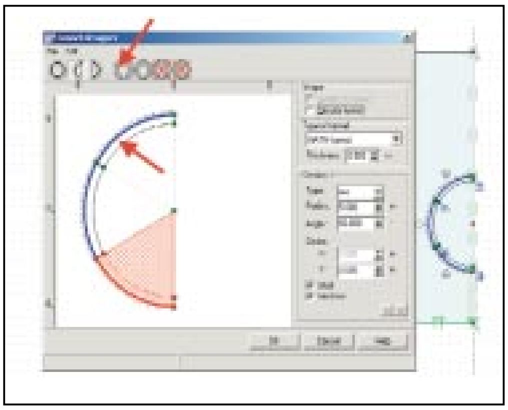


Figure 5 Specification of inner lining in tunnel designer

Face stability with reinforcement

Above arguments concerning the model size do not hold if one is only interested in face stability. In these cases a much shorter model is sufficient (e.g. Vermeer & Ruse, 2000). There are different ways of investigating face stability problems: firstly one can excavate the face in a standard procedure and look at the S-Mstage value when the analysis does not reach equilibrium. Secondly a face pressure can be applied at the face and reduced until failure. Thirdly reinforcement can be provided in longitudinal direction which also creates a support at the face. The last option is the most realistic one from a practical point of view because this is what is done in practice by installing rock bolts (or sometimes fiberglass rods). Figure 6 shows shadings of longitudinal displacements (maximum value 140 mm) for such a model where geotextiles, representing

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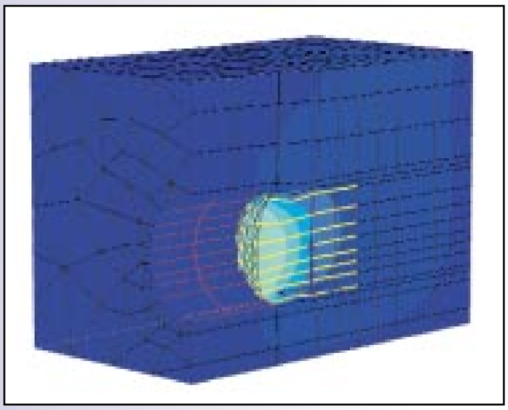

Figure 6
Shadings of longitudinal displacements with reinforcement at tunnel face

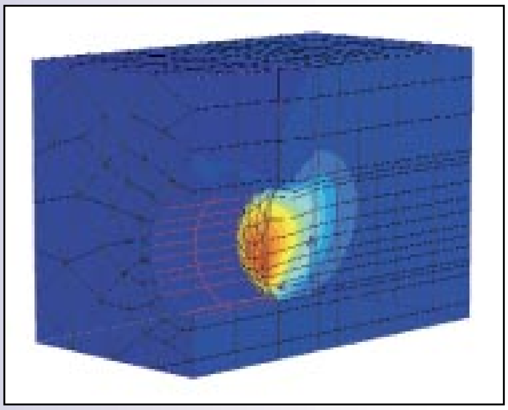

Figure 7
Shadings of longitudinal displacements without reinforcement at tunnel face

the reinforcement along the tunnel axis, have been installed. Figure 7 shows the same picture for the face without reinforcement at an Σ-Mstage value of appr. 0.85 (the face is not stable without support). The shadings are in the same scale and the maximum value is 320 mm at this stage. The stabilising effect of the reinforcement is obvious, in addition they reduce settlements at the surface.

At present some studies are under way whether comparable results are obtained by the different methods described above, i.e. by replacing the geotextiles by a face pressure and work out the required reinforcement from there. A generally adopted procedure to model rock bolts in tunnelling, namely to increase the cohesion, will also be investigated.

References

Mair, R.J., Taylor, R.N.: Theme lecture: Bored tunnelling in the urban environment. Proc. 14th ICSMFE, Hamburg, 1997, 2353-2385.

Addenbrooke, T.I., Potts, D.M., Puzrin, A.M.: The influence of pre-failure soil stiffness on the numerical analysis of tunnel construction. Geotechnique 47, No.3, 1997, 693-712.

Vermeer, P.A., Ruse, N.: Face stability when tunneling in soil and homogeneous rock. Proc. Developments in Theoretical Soil Mechanics - The John Booker Memorial Symposium, Sydney, 2000, 123-138.

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Users Forum

Question:

I have the following problems under Windows 2000:

- The mesh generator fails to generate a mesh though all the possible solutions have been applied.
- The calculation hangs directly after starting. In the bottom left corner of the calculation window the status of the calculation shows "Profile…".

Answer:

Both problems are related to the Windows' temporary directory stored in the TEMP environment variable. By default, under Windows 2000, this TEMP variable contains a rather long path ("C:\Documents and Settings\<username>\Local Settings\Temp" in case Windows has been installed on drive "C") causing the problem.

The solution is to set the TEMP variable to a shorter, existing, path. To do so:

- Go to the Windows Start Menu and successively select "Settings", "Control Panel" and "System".
- In the "System Properties" window that has now appeared choose the last tab sheet called "Advanced".
- From this tabsheet choose the middle option "Environment variables"
- In the "Environment variables" window choose from the uppermost list the variable called TEMP and select the "Edit" button in order to change its value.
- Set the TEMP variable's value to for instance "C:\TEMP".

- Close all windows.
- Make sure the newly defined temporary directory exists. If this is not the case, please create the directory using for instance the Windows Explorer.

Note that possibly the above procedure must be repeated after installing a Windows Service Pack.

Question:

Often, I have to send PLAXIS projects per e-mail. Which files do I have to send and how can I reduce the file size to a minimum?

Answer:

The main file used to store information for a PLAXIS project has a structured format and is named <project>.PLX, where <project> is the project title . Besides this file, additional data is stored in multiple files in the sub-directory <project>.DTA. Both the <project>.DTA sub-directory with its contents and the <project>.PLX file are needed to open and (re-)calculate the problem. In case of the PLAXIS 3D Tunnel program the file is named <project>.PL3 and the sub-directory <project>.DT3.

The size of the files can be reduced to a minimum using the Save as option in the File menu and saving the project under another name. However, calculation steps (<project>.### where ### is a calculation step number) are not copied in this way and the copied project has to be calculated again.

If it is inconvenient to re-calculate the copied project then make sure that for each phase the option 'Delete intermediate steps' is switched on in the parameters tab-sheet. For most calculation types the default option is that PLAXIS only saves the last calculation step per phase, e.g. the option 'Delete intermediate steps' is switched on in the parameters tab-sheet. For the following calculation types it is also interesting to analyse the intermediate calculation steps and therefore the default option is to save the intermediate steps: Plastic: Load adv. nr of steps (Incremental multipliers and Phi-c reduction), Plastic: Manual control (Incremental multipliers and Phi-c reduction), Consolidation and Dynamic analysis. Another exception is ultimate level calculations that end with a warning message like 'soil body collapses'. In these situations, all calculation steps are saved.

The file size can always be reduced by using a compression program to compact the PLAXIS sub-directory and files.

The structure of the PLAXIS files is described in the Reference Manual, Appendix B.2

Question:

The surface of my model is slightly tilted and therefore I use Gravity loading instead of the K0-procedure. I have an interface around my beam elements in which Rinter = 0.7, these beam elements are installed in a later phase and are initially not active. However, after generating the initial stresses I can see in output that the initial stresses are not correctly generated. Are the interfaces already activated, and if so, how can I switch them off?

Answer:

Interfaces are always activated and deactivated together with the adjacent soil clusters and cannot be deactivated separately. This feature in version 7 can cause unexpected results when using soil-interface interaction values (Rinter) lower than 1.0 (Rigid) and a sloping surface of the model.

If a Rinter value of 1.0 is used then it will have little or no consequence for the initial stresses. Lower values than 1.0 of Rinter and a sloping surface of the model do influence the initial stresses.

A solution to this problem is the creation of additional material data sets with a Rinter of 1.0 for the clusters in which the interface lies. Use the data sets with a Rinter of 1.0 in the initial stresses calculations and following calculations in which the interfaces should not be active. In the phase in which the beam is activated use the material data set with the correct and lower Rinter.

We are investigating the option to switch interfaces on and off for future versions.

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ACTIVITIES

27-31 AUGUST, 2001

XVth ISSMGE International Conference on Soil Mechanics and Geotechnical Engineering (XV ICSMGE)
Istanbul, Turkey

01-03 SEPTEMBER, 2001

Post-conference event (XV ICSMGE) Short course on Computational Geotechnics (English)
Istanbul, Turkey

17-19 OCTOBER, 2001

Short course on Computational Geotechnics (French)
'Pratique des éléments finis en Géotechnique'
Paris, France

21-24 OCTOBER, 2001

Short course on Computational Geotechnics (English)
Santiago de Querétaro, Mexico

15-16 NOVEMBER, 2001

10th European Plaxis Users meeting (English)
Karlsruhe, Germany

20-23 JANUARY, 2002

Short course on Computational Geotechnics (English)
Noordwijkerhout, The Netherlands

04-06 MARCH, 2002

Course on Computational Geotechnics (German)
'Finite Elementen in der Geotechnik - Theorie und Praxis'
Stuttgart, Germany

24-27 MARCH, 2002

International course for experienced Plaxis users (English)
Noordwijkerhout, The Netherlands

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