BULLETIN

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IN THIS ISSUE:

Editorial

It has been a year since we started to publish the international Plaxis bulletin. Since the first issue we have had many positive reactions. Some people also asked us for the address of the Plaxis Internet site, which did not exist at that time. Since a few weeks the Plaxis web-site is available and can be visited at http://www.plaxis.nl. In addition to the articles printed in the bulletin the web- site will contain more up-to-date information. Besides providing information the electronic highway offers extended ways for user support. For now the web-site will only contain information, however possibilities for interactive services are being explored for the benefit of users in the future.

Figure 1
Plaxis Home-page http://www.plaxis.nl

Looking back at the past few months we have had several courses. Two short courses, one in Grenoble (France) and another short course in Kuala Lumpur (Malaysia). In addition two new courses were launched. On June 2-4 we have had the "International course for experienced Plaxis users" held in the Leeuwenhorst conference center in Noordwijk (Netherlands). In this new course attention was focused on the use of advanced soil models in practical applications. More than 30 participants from 10 different countries attended this course.

The second new course was the first German course on backgrounds and application of the finite element method for geotechnical engineering problems. This course was held June 16-18 at the Technische Akademie Esslingen (Germany) and attracted 30 participants. We hope you enjoy this extra thick Bulletin.

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Column Vermeer

In the previous bulletin Kenneth E. Tand published on the behaviour of cemented sands, giving data on the initial Young's modulus E_i (or E_0) as obtained from triaxial stress strain curves. In fact this initial modulus is often used in the USA and we have also considered the idea of using it as an input parameter for the Hard Soil model. Finally we decided not to do so, because E_i is very sensitive with regard to testing techniques used in the lab. Conventional triaxial equipment tends to give relatively low values, whereas special measurement techniques will produce extremely high values. Therefore the literature gives an extremely large range of values for E_i. Hence taking E_i values from the literature has to be done with some care. To avoid any possible confusion we have chosen to use E_{50}.

As all soil stiffnesses are stress-level dependent, the true input parameter for the HS model is normalised for a confining pressure of one bar and it is denoted as E_{50}^{ref}. An intensive literature survey by Schanz (1997) indicates for remoulded quarz sands:

$$E_{50}^{ref} = 15 \text{ to } 75 \text{ MPa}$$

where the lower values tend to be found for very loose silty sands and the higher values for very dense gravelly sands. Schanz's study on the stiffness of remoulded quartz sands is especially interesting as it shows that E_{50}^{ref} is correlated to E_{oed}^{ref}, i.e. the stiffness modulus for one-dimensional compression. Hence, if one has data on the oedometer modulus, it can be used to estimate the triaxial modulus.

The behaviour of the undisturbed in-situ soil will of course be stiffer than the behaviour of remoulded samples. For non-cemented sands, the in-situ stiffness will only slightly exceed the remoulded stiffness, but depending on the degree of cementation the difference will increase as reported by Kenneth E. Tand in the previous bulletin.

Let us now further concentrate on stiffnesses of remoulded sand samples. In triaxial testing samples can also be unloaded and reloaded to measure the modulus E_{ur}^{ref}, another input parameter of the HS model. For remoulded quartz sands, one tends to find a very high unloading-reloading modulus so that $E_{ur}^{ref} \approx 4E_{50}^{ref}$. I would advise you to use this correlation when applying the HS model to sands.

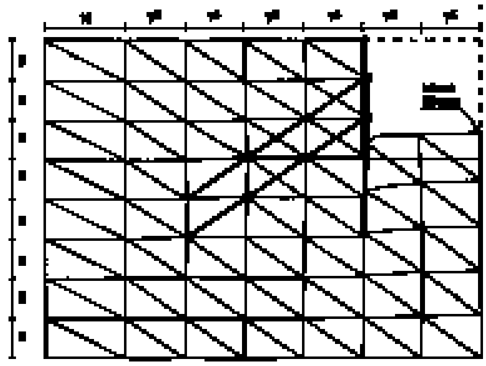

Figure 1 Deformed mesh with HS-model.

In the first two international bulletins I have demonstrated the power of the HS model with reference to two problems: a tunnel and an excavation with slopes. This time I would like to consider an excavation with a bored-pile wall, as shown in Figure 1. This excavation has a depth of 15 m and also a half width of this length. The anchored wall has a total length of 25 m. The groundwater level is considered to be very deep, further relevant input data are:

Table 1: Parameters for wall and anchor.

	EI [GNm²/m]	EA [GN/m]	ν [-]	w [kN/m²]	pre-load [kN/m]
wall	1.5	80	0	8	N/A
anchor	N/A	0.2	N/A		300

Table 2: Soil parameters used for the Hard Soil model.

	γ_{dry} / γ_{wet} [kN/m³]	E_{ur} [MPa]	ν_{ur} [-]	E_{50} [MPa]	φ [°]	Ψ [°]	c' [kPa]	R_{inter} [-]	K_0 [-]
Soil	18	80	0.1	20	35	5	1.0	0.67	0.3

The unloading-reloading Poission s ratio of ϑ_{ur} = 0.1 conforms to the standard setting of the new Version 6.31, but previous releases of the code had other standard settings. We also adopted the standard setting for the power law for stress level dependency of E_{ur} and E_{50}, i.e. m = 0.5. The computed deformation field after excavation is shown in Figure 1. Both the heave of 56 mm for the excavation floor and the maximum horizontal wall displacement of about 10 mm look reasonable, at least for a deeply embedded stiff wall. It should be noted that we completed the HS-analysis with a plastic-nil-step. To achieve good accuracy we simply entered an extremely low non-standard value for the tolerated error. In this manner a plastic-nil-step was achieved.

Let us now consider analyses using the Mohr-Coulomb model instead of the HS model. In such cases one should first decide whether this problem can be analysed on the basis of the first loading modulus, E_{50}, the unloading/ reloading one, E_{ur}, or some modulus in between. I consider this a difficult question and I would not like to give a general answer for the problem of anchored walls. However, it is interesting to perform a Mohr-Coulomb trial computation with the low E_{50} modulus and another one with the high E_{ur} modulus. Let us first consider the use of E_{50}.

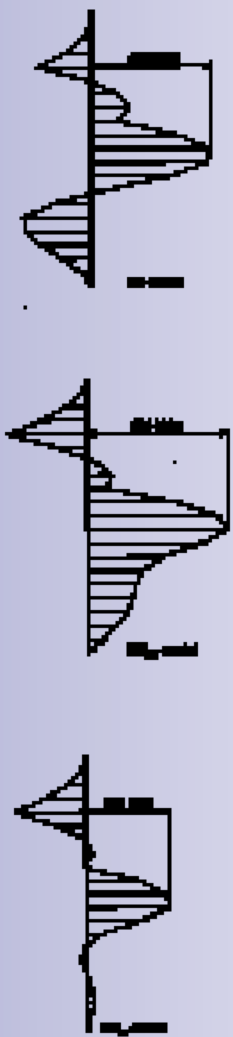

Figure 3 Bending moments from 3 analyses.

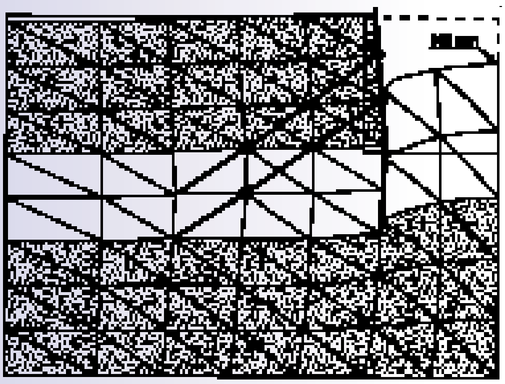

Figure 2 Deformed mesh with MC_{50}-model.

As a deep excavation involves large variations of stress level, we should not use a constant stiffness for the entire depth. Instead we need to use sub-layers with different values of E_{50}. The more or less arbitrarily selected sub-layers are indicated in Figure 2. The top layer has an average depth of 7.5 m, an average vertical stress of 135 kPa and an average horizontal stress of $\sigma_h = K_0 \sigma_v$ = 58kPa. Following procedures as suggested among others by Clough et al (1981), we derive $E_{50} = E_{50}^{ref} (\sigma_h/\sigma_{ref})^{0.5}$. With E_{50}^{ref}=20 MPa and σ_{ref}=100 kPa, this yields $E_{50} \approx$ 15MPa for the top layer. For the middle layer, this procedure yields a higher modulus of $E_{50} \approx$ 2MPa and for the bottom layer we derive $E_{50} \approx$ 32MPa Hence the stress-level dependency of soil stiffness is accounted for by choosing the above moduli for the imaginary soil layers. Naturally, one could adopt more sub-layers, but from a practical engineering point of view three layers will do.

The results of the Mohr-Coulomb analysis are depicted in Figure 2. It is a displacement field that departs completely from the one in Figure 1. Now we have a heave of 165 mm of the excavation floor and a maximum horizontal wall displacement of 38 mm, whereas the HS model yields much smaller displacements. The difference between the two analyses is particularly clear if one considers the bending moments in Figure 3. On the left it is seen that the HS- analysis yields both a field moment and a clamping moment, whereas the MC- analysis shows no clamping moment at all. Clearly the MC-analysis is far from realistic. Again we added a plastic-nil-step with an extremely low tolerated error to ensure good accuracy.

Finally an extra MC-analysis was performed with increased Young s moduli. We simply multiplied the previous E_{50} values by a factor of 4 to obtain E_{ur} values. The result of this final analysis is depicted in Figure 4: the excavation shows an upheave of 41 mm and the wall has a maximum horizontal displacement of 12 mm.

These displacements actually do resemble the HS-results, but the bending moments still differ. Whereas the HS-analysis yields a field moment of 450 kNm/m, the final MC-analysis yields only 315 kNm/m (see Figure 3). Hence it would seem to be impossible to match HS-results with MC-results when one wants to fit both deformations and stresses, i.e. bending moments. For the present problem it appears to be virtually impossible to obtain proper bending moments with a MC-analysis, whereas the HS-results are very realistic.

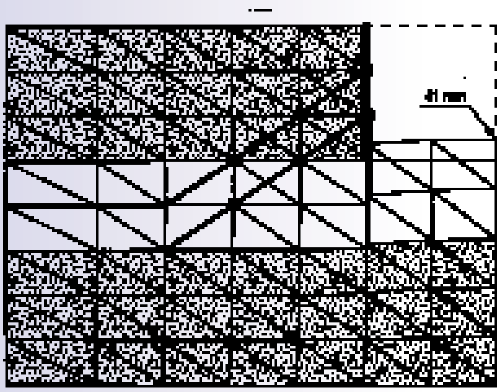

Figure 4 Deformed mesh with MC_{ur}-model.

Literature:

Clough G. et al. 'Cemented sands under static loading', ASCE Geotechnical Engineering Division, June 1981, Vol. 107, No. GT6, pp. 799-817.

Schanz T., Vermeer P.A. 'On the stiffness of sands', Geotechnique Symposium in Print: Pre-failure Deformation Behaviour of Geomaterials, London, September 1997.

P.A. Vermeer, Stuttgart University

PLAXIS Practice I

THE LEANING TOWER OF ST. MORITZ

The leaning tower of St. Moritz (1139), a landmark of the famous skiing resort in the Swiss Alps, is often referred to as the little brother of the famous Italian one. Nevertheless studying the stirring history of the Swiss one offers the insight into a lot of interesting geotechnical aspects and related questions. The numerical study of this historical monument is one of the activities at the Stuttgart Institute of Geotechnical Engineering. The object of this paper is to explain geotechnical phenomena related to this tower. Because we started this project just some time ago only preliminary results will be presented, but interested readers are encouraged to contact the author directly.

Historical overview:

Before presenting results of the numerical study I like to give a short overview of the troubled times of the tower, see Figure 1, which remainders can still be visited today.

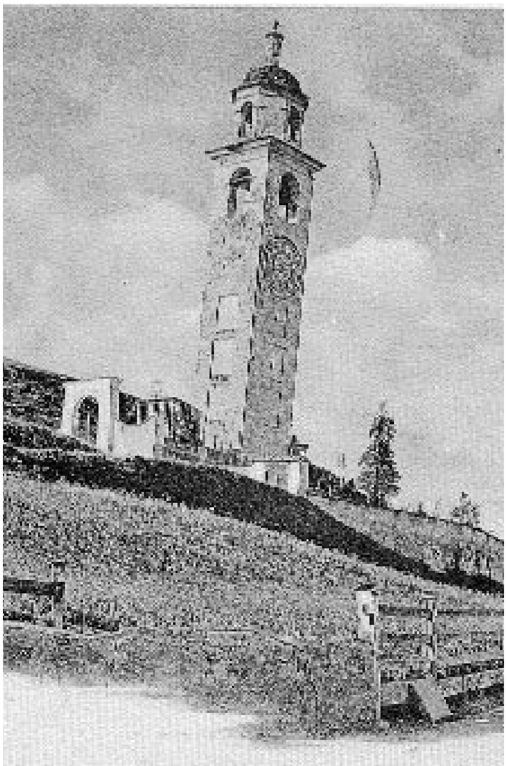

Figure 1: Historical view of the leaning tower of St. Moritz.

During the Middle Ages the original roman tower of the 12th century was changed in its outward appearance many times. Already in 1797 after a recorded earthquake the stability of the tower reached a critical situation. There followed a removal of the bells and the accompanying nave. People early recognised that the local instability of the tower is deeply related to the overall instability of the slope, which is known as a slip area for centuries. Experts suggested several combined activities in order to increase both stability on the local and the global level. During this century drainage of the whole slope, the installation of different anchors and a relaxation of the upper earth pressure were carried out. See Figure 2 for the different stages of construction and improvement, which is also the starting point for the numerical simulations.

Because field measurements showed an acceleration of the downhill deformation rate, the officials decided in the 60ties to enlarge the different observation programs and to further try to relax the situation. Today the tower is moving with 10-30 mm/year, showing an inclination of 5.5 degrees.

Geotechnical situation:

The St. Moritz area is part of a large scale tectonic instability. This geological situation is very complex and will be skipped here. Interested readers are referred to Schlüchter (1988) as he gives a detailed overview. In this short note we concentrate on the local site, but we keep in mind that a proper description of the actual situation should also take the global situation into account. In order to gain a geotechnical soil profile as displayed in Figure 3, an intensive soil investigation program was carried out. Along with this program came the installation of several slope indicators combined with the assembly of a geodetic precision measurement system.

Figure 2: Different stages of construction and improvement of the tower.

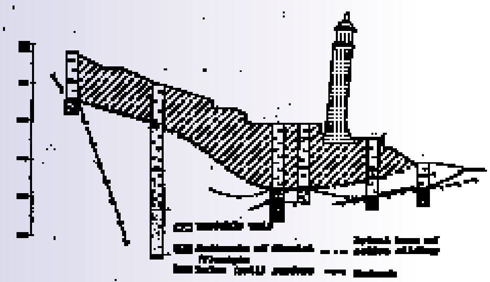

Figure 3: Geotechnical soil model of the St. Moritz site.

The geological profile of today consists of the slide mass, some transition layers and the underlying bedrock ridge. The sediments affected by the instability are of complex lithological properties with an observed vertical and horizontal change from extremely silty gravel to a sandy silt with gravelly components. Deeper layers in the area of the leaning tower are made of poorly sorted fluvial sediments. Given by the analysis of the slope indicators we find an actual thickness of the instability between 20-30 m, which is less than the thickness of the sediment cover. One unknown is the end of this sliding mass when going down the slope to the village. Most of the experts agree that the tower is located at the stagnation point of the slip.

Numerical simulation:

Because of the heterogenous nature of the underlying soil it is very difficult to decide about precise values to describe the geological situation. The geological situation shown in Figure 3 was modelled by using two different soil layers with parameters as given in Table 1. For all soil layers we used the Hard Soil model, because we had additional laboratory tests (Triaxial tests, permeability tests, Lang 1990) to determine the parameters used with that model.

For all other parameters the default values of Plaxis 6.31 were used. The distinction between the upper and the lower layer of sediments was made because of the results of the drilling campaign. In brackets we find the original results from the triaxial tests for these two layers. I reduced the shear strength considerably because all tests were done on remoulded samples and from experience with this kind of soils I know there is a rather large influence of (negative) pore pressures resulting from this kind of preparation, especially when choosing reasonable values for the cohesion. A friction angle of about 32 degrees follows from the grain size distribution, so the original value from the triaxial tests is not modified. The lower boundary of the mesh was taken as the upper level of the bedrock, the right and left ones resulted from the number of 15-noded elements available. Because of the

Table 1: Soil parameters used for the Hard Soil model

	γ_{dry} / γ_{wet} [kN/m³]	E_{ur} [MPa]	V_{ur} [-]	E_{50} [MPa]	φ [°]	Ψ [°]	C [kPa]
Sliding mass	18.2 / 21.1	40.0	0.1	10.0	35.0	5.0	35.0 [50.0]
Lower sediments	17.6 / 21.1	40.0	0.1	10.0	28.0	0.0	35.0 [50.0]

boundaries between the different layers and the structure and the wish to model the different stages of construction we ended up with a rather high number of elements as shown in Figure 4.

Both the soil layers were modelled using the ***undrained*** option. So between the different stages of construction several consolidation calculations were added using coefficients of permeability from the lab tests of k_x=0.0865 m/d for the top layer and k_x=0.864 m/s for the lower sediment. In the vertical direction we used only 50% of these values for both layers. All the structural elements were simulated as ideal elastic (V =0.3) with γ_{dry} =19.5 kN/m³ / E=2.9*10⁷ kN/m² for the tower's concrete foundation, γ_{dry} =20.7 kN/m³ / E=6*10⁶ kN/m² for the tower itself, γ_{dry} =25 kN/m³ / E=2.9*10⁷ kN/m², for the additional foundation and EA=3.4*10⁶ kN/m for the anchor, which was modelled using both a node-to-node anchor and a geotextil element. All of these structural elements were modelled as *non-porous*.

With default values for all error ranges and calculation parameters it took about 250 loading steps totally to recalculate the almost 800 years of existence of the tower. Some results from this calculation are given in the following section.

Results:

From the large amount of data resulting from a calculation as described above we will only comment on the most evident numbers. Here fore Figure 4 shows the deformation field for the final stage of the calculation referring to the situation today.

The left corner at the bottom of the tower follows a total horizontal deformation of u_x=0.016 m and a vertical deformation of u_y= -0.166 m. The corresponding values for the right edge are u_x=0.056 m and u_y=-0.313 m. So we end up with a differential settlement of 147 mm of the tower compared with the actually observed deformation rate of 10-20 mm/year. A direct comparison of deformations is not possible because it is not clear to what fixed point field measurements are related to. The inclination of the tower resulting from these differential settlements is about 1.5 degree compared to 5.3 degree in reality. The reason for this remarkable difference between these numbers will be discussed in the following section.

The calculation shown here was supplied by a kind of manual safety analysis described in the

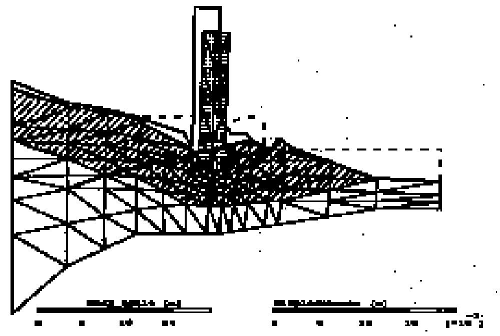

Figure 4: Deformation field for the final stage of the calculation with calculated sliding mass.

following. Because of the water circulation taking place and the additional large deformations a total reduction of the cohesion in the slip mass was assumed. Therefor the cohesion of the top soil layer was reduced from 35 kPa to 1 kPa after the last stage of construction and a new equilibrium state was calculated just keeping all the other parameters according to Table 1. Doing so we find a limit value of the friction of about 19 degree for the top layer. Performing a safety analysis of the slope with Plaxis, using the MC model for the two soil layers with the same parameters as just said, we find a global factor of safety of about 1.23. In literature (Lang, 1978) we can find factors of safety according to Janbu between 1.14 and 1.57 depending on the assumptions about the level of the watertable.

The shaded area in Figure 4 is a region of soil where the mobilisation of the shear strength of the soil, given directly by the program, is above 0.85. This area corresponds well with the range of the sliding mass as we find in Figure 3.

Summary/Discussion:

The positive result is that we end up with some inclination of the tower of the same direction as observed in the field. The slope is in a critical situation with a kind of sliding mass of a dimension as observed in the field measurements. But both the quantitative amount of differential settlements and their development over the time is still rather different from the field observations.

When interpreting the calculations presented above in detail one should remember the major restrictions of the presented FEM discretisation:

- The calculation is done as a plane strain analysis even the problem is of a 3D type because the cross section of the tower is almost of a quadratic shape. Doing so the results of the numerical simulation are expected to decrease even more.
- The calculation does not take into account a kind of creep of the whole slope resulting from the global geological instability mentioned in the beginning. Even simulating the different stages of construction correctly, some of them are documented rather poorly, gives no guarantee we describe the primary state (initial conditions) for example at the beginning of the 20th century precisely.
- The influence of the groundwater regime on the slope stability is not definitely modelled at the moment. A more realistic catch of the water movement and the permeability distribution is desirable.

The efforts described up to here are a first step in the direction to model the actual situation of the leaning tower of St. Moritz as realistic enough to judge any suggestion of its improvement in advance. Our further activities will try to take into account the understanding summarized above.

Literature:
Lang H.J. 'Several reports on the leaning tower of St. Moritz', IGB Bericht 3676/29, 1978-1990. Institut für Geotechnik, ETH Zürich.
Schlüchter Ch. 'Instabilities in the area of St. Moritz, Switzerland', Proc. of the 5th Int. Symp. On landslides, Lausanne 10-15 July 1988, A.A. Balkema Publishers.

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New developments

In particular for soft soils, the time dependent deformation is a very important property. In addition to consolidation, i.e. the compaction of soil due to the outflow of overstressed pore water, soft soils show a continuing deformation in time when they are subjected to load or stress. The latter process is referred to as 'secondary compression' or 'creep'. In this contribution a new 'creep model' for Plaxis is discussed.

The current Soft Soil model is a time independent model, formulated as a plasticity model with cap hardening. In this model strains are composed of elastic (reversible) strains and plastic (irreversible) strains. The Soft Soil model is very useful to simulate the behaviour of normally consolidated clays and peat in primary loading, as long as time aspects are of minor importance. Although the model can be used in a consolidation analysis, it is not suitable to deal with secondary compression (creep) in detail.

A new creep model for soft soils was formulated by professor Vermeer of the University of Stuttgart [1]. In contrast to some existing models, the new creep model is formulated in an incremental form, such that it can generally be used in finite element programs. The model was robustly implemented in Plaxis [2] and tested for some

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simplified applications. Although the current experiences with the creep model are quite good, it will take a while before the model is officially released. In the mean time the model will be extensively tested in several practical applications.

In primary loading or unloading / reloading at a certain loading rate, the creep model behaves similar as the Soft Soil model. The model shows logarithmic compression with a clear distinction between primary loading and unloading / reloading. The amount of creep or the creep rate depends on the stress level in relation to the preconsolidation stress or to be precise: on the ratio of overconsolidation, OCR. For (near) normally consolidated soils the creep rate is very high and for overconsolidated soils the creep rate is very small. As a result, when the soil is loaded beyond the preconsolidation stress, creep develops quickly and causes the preconsolidation stress to increase. When the load is kept constant, the preconsolidation stress will still increase in time, but as the soil becomes 'more overconsolidated' the creep strain rate will decrease. As a result, the additional compaction evolves logarithmicly in time, which is typical for secondary compression behaviour.

In addition to the known parameters λ^* and κ^*, as used in the Soft Soil model, the creep model has the parameter μ^* (secondary compression index). The meaning of these parameters is depicted in figures 1 and 2. The parameters λ^*, κ^* and μ^* can easily be related to the well known compression test parameters C_c, C_s and C_α respectively, which are generally used in soft soil engineering.

$$\kappa^* \approx \frac{2\,C_r}{2.3\,(1+e_0)} \quad ;\lambda^* = \frac{C_c}{2.3\,(1+e_0)} \quad ;\mu^* = \frac{C_\alpha}{2.3}$$

Note that the creep model is not only meant for situations of one dimensional compression, but it is capable of describing the full three-dimensional time dependent behaviour of soft soils in any stress path. As a result, the creep model is suitable to utilize in practical applications, provided that proper data from compression tests are available.

For a detailed description and theoretical formulation of the creep model reference is made to the previously mentioned publications [1] and [2].

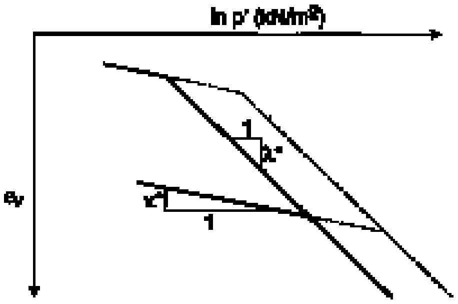

Figure 1 Meaning of the parameters λ^* and κ^* in isotropic compression at a constant rate of loading. For higher loading rates, the primary compression line (λ^*) will move parallel to the right.

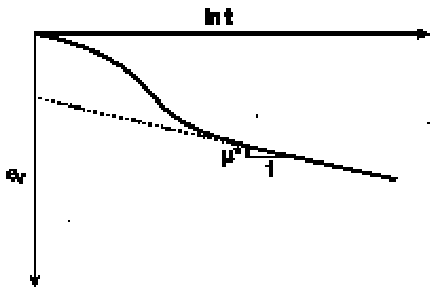

Figure 2 Meaning of the parameter μ^* in isotropic compression. The figure emphasizes on the secondary compression at a constant load.

Literature:
[1]Vermeer P.A., Stolle D.F.E., Bonnier P.G. (1997), From the classical theory of secondary compression to modern creep analysis. To be published in proceedings IACMAG conference 1997, Wohan (China).
[2]Stolle D.F.E., Bonnier P.G., Vermeer P.A. (1997), A soft soil model and experiences with two integration schemes. In: S. Pietruszczak & G.N. Pande (eds), proceedings NUMOG IV symposium, Montreal (Canada). Balkema Publishers.

Ronald Brinkgreve, Plaxis BV

PLAXIS Practice II

SOME COMMENTS ON MODELLING DEEP EXCAVATION PROBLEMS WITH PLAXIS

In this short contribution some comments are made on parameters influencing the results obtained for deep excavation problems using PLAXIS. Based on an example specified by the working group 1.6 (Numerical Methods in Geotechnics) of the German Geotechnical Society, which was used to compare numerical results obtained from different users and codes for a given problem, some additional analyses have been performed. Some results of these studies are briefly presented in the following.

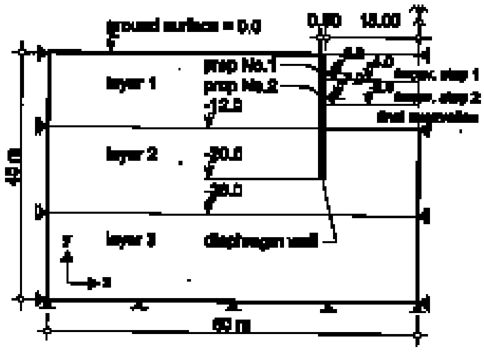

Figure 1 Cross section of excavation.

Description of problem:

The specification of the problem follows from Figure 1. The construction steps, prop levels and a simplified soil profile are also indicated. It follows from this figure that the excavation is 30 m wide and 12 m deep.

Figure 2 shows the mesh which consists of 200 15-noded triangle elements. The wall is assumed to be in place i.e. the construction of the wall has not been modelled. The diaphragm wall was considered as linear elastic material and the relevant parameters are given in Table 1. The diaphragm wall is supported by two props modelled by using fixed-end anchor elements (Figure 2).

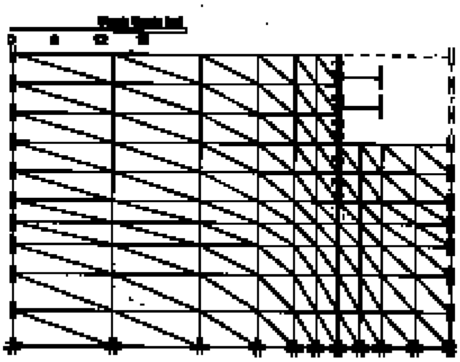

Figure 2 Finite element mesh.

Table 1. Stiffness parameters for diaphragm wall.

diaphragm wall	E [MPa]	V [-]
concrete	21000	15

The strength and stiffness parameters for the soil layers used in the drained, elastic perfectly plastic analysis using a Mohr-Coulomb failure criterion, which serves as reference calculation, are given Table 2.

Table 2. Soil properties.

soil layer	friction angle ϕ [°]	cohesion [kPa]	E [MPa]	V [-]
layer 1	35.0	2.0	20	0.40
layer 2	26.0	10.0	12	0.40
layer3	26.0	10.0	80	0.30

The following computational steps have been performed:

step 1: initial conditions (layer 1: Ko = 0.5, layer 2+3: Ko = 0.65) and self weight of diaphragm wall

step 2: excavation to a depth of 4.0 m

step 3: excavation to a depth of 8.0 m, prop in -3.0 m in place

step 4: excavation to a depth of 12.0 m, props in -3.0 and -7.0 m in place.

A comparison of the results obtained from various analyses from different codes and users according to these specifications is given in [1]. In the following some results from additional investigations which have been made in order to clarify some discrepancies are presented.

Influence of interface elements on the deformation behaviour:

A parametric study was carried out to investigate the influence of interface properties on the deformation behaviour varying the reduction factor R from 0.45 to 1.0. R=1.0

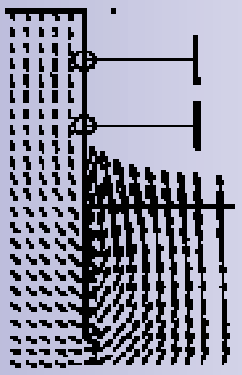

Figure 4 Incremental displacement field for excavation step 3.

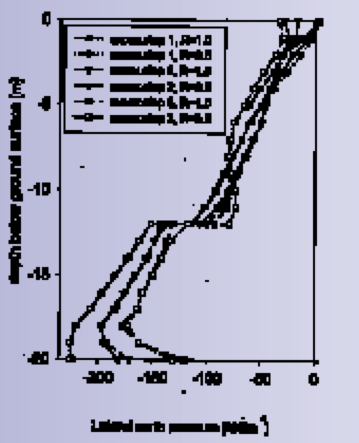

Figure 5 Lateral earth pressure distribution

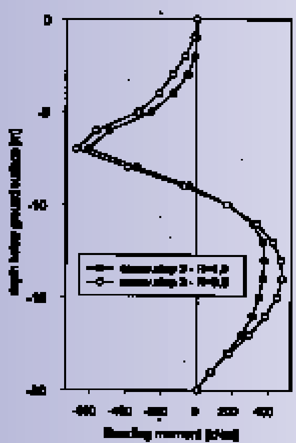

Figure 6 Bending moments after excavation step 3.

serves as reference solution, i.e. no interface behaviour is considered (strictly speaking a minor influence remains compared to a mesh without interface elements because of some prefixed interface properties).

It is well known that for these types of problems finite element calculations usually exhibit somewhat unrealistic vertical displacements of the wall when simple elastic-perfectly plastic constitutive laws are employed. This behaviour is usually not observed in practice.

It follows from Figure 3 however that the use of interface elements improves the numerical predictions significantly. The first two excavation steps still lead to a heave of the wall but as expected the lower the value of the factor R is, the less heave is calculated. In the last excavation step (both props built-in) the wall settles, especially in the case of low wall friction, corresponding to the movement of the surrounding soil as indicated in Figure 4, which compares qualitatively well which observations in the field.

The horizontal displacements of the top of the wall increase with decreasing R. The influence of the factor R on the horizontal displacements

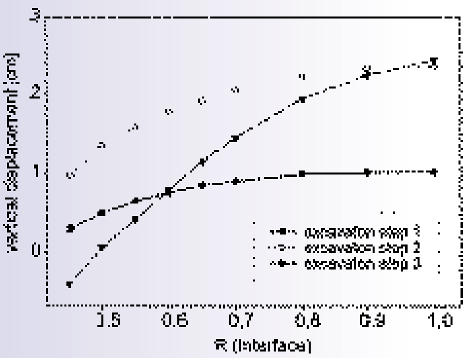

Figure 3 Vertical displacement at top of diaphragm wall.

of the toe of the wall is not as pronounced. As expected the lateral earth pressure distribution is only slightly influenced by R (Figure 5). The effect on the bending moments is pointed out in Figure 6. If wall friction is reduced bending moments increase, the maximum difference being less than 30% for the final excavation step for R ranging from 0.5 to 1.0.

Beam-elements vs continuum elements for modelling of the diaphragm wall:

When comparing PLAXIS results for the problem given by the AK 1.6 as described in section 2 some differences which could not be easily explained became apparent. It was suggested that these differences are caused by the element type used for modelling the wall (beam or continuum).

Figure 7 shows that this is indeed correct.

Beam elements lead to less (unrealistic) upwards movements of the wall. This may be partly due to the stresses acting on the bottom cross section of the wall pushing the wall upwards, which is not the case for beam elements where stress transfer is possible only via interface friction. It is also interesting to see that the interface properties have a more pronounced effect in combination with beam elements.

In addition horizontal displacements of the top of the wall (+ve denotes displacement towards the excavation) are also very much influenced whether beam or continuum elements are used. The influence of the interface properties is also obvious (Figure 8).

Influence of prop stiffness:

Finally the influence of the prop stiffness is briefly addressed. Figures 9 and 10 show the expected behaviour as far as maximum bending moments and prop forces are concerned. The irregularities in Figure 9 are due th the change of sign of the maximum value for very soft props. Considering prop stiffnesses of 1E+4 to 1E+5 as common in practical applications it follows that it is worthwile optimizing prop levels and stiffnesses according to the designed excavation sequence. It is important to note that the overal deformation behaviour is not very much influenced by the prop stiffness which follows e.g. from Figure 11 where the horizontal displacement of the top of the wall is seen to increase significantly only in the case of soft props.

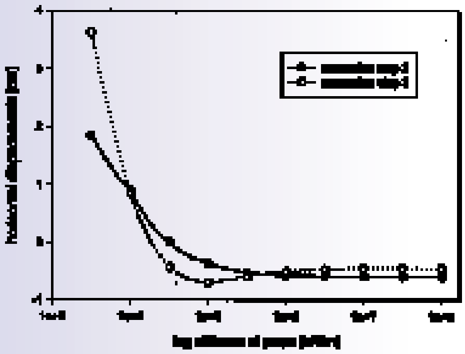

Figure 11 Horizontal displacement of the top of the diaphragm wall.

Conclusion:

The influence of some parameters important in the analysis of deep excavation problems using PLAXIS has been addressed. It can be concluded that the use of interface elements in connection with beam elements for modelling the retaining wall is to be preferred over continuum elements. The stiffness of the props has significant influence on bending moments and prop forces (as expected) but relatively little influence on the overall deformation behaviour, at least for prop stiffnesses in the range commonly used in practice. Further studies in particular with respect to constitutive modelling are presently under progress.

Literature:

[1]H.F.Schweiger (1997). Berechnungsbeispiele des AK 1.6 der DGGT - Vergleich der Ergebnisse für Beispiel 1 (Tunnel) und 2 (Baugrube). Tagungsband Workshop "Numerik in der Geotechnik", DGGT/AK 1.6, 1-29.

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Users Forum

Question:

I get several error messages when running Plaxis on a fast Pentium Pro computer. What is wrong ?

Answer:

Recently we have received some similar questions on the use of Plaxis on fast Pentium (Pro) computers. The errors that occur are all system errors, such as: 'Division by zero', 'Stack overflow', etc. These errors are caused by high processor speeds (> 133 Mhz). One of the older Pascal

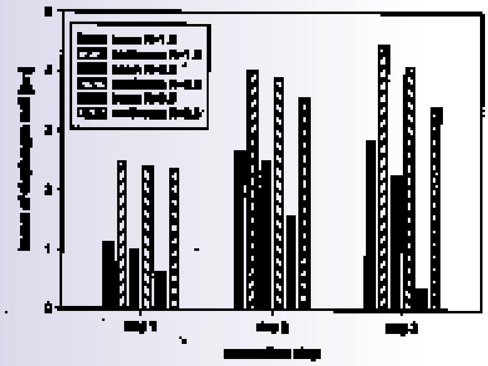

Figure 7 Heave of diaphragm wall.

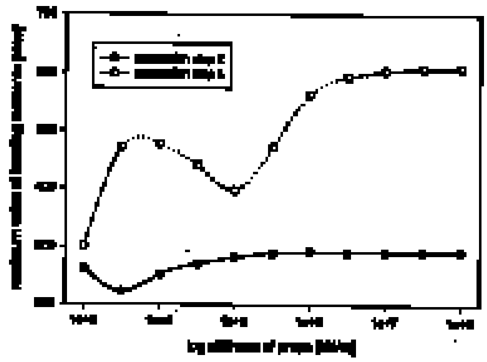

Figure 9 Maximum bending moments vs prop stiffness.

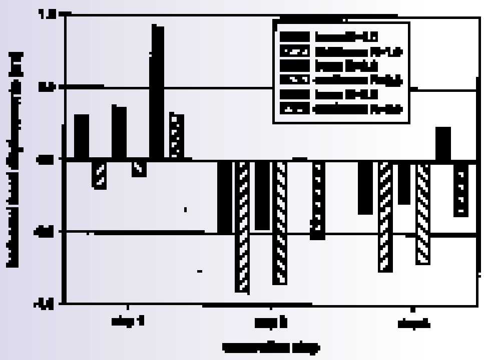

Figure 8 Horizontal displacement of top of wall.

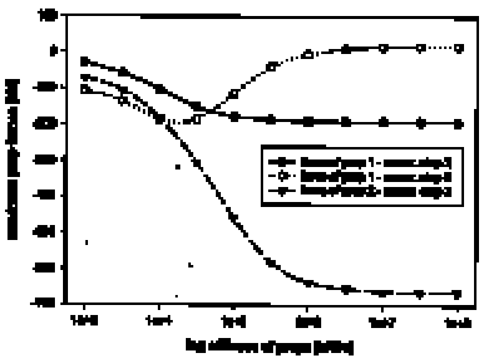

Figure 10 Prop forces vs prop stiffness.

PLAXIS

components was not prepared for the high clock speeds that are available today. This problem has been solved in Version 6.31.

Question:
In some situations I see that the factor of safety decreases to values below 1.0 in a Phi/c reduction calculation. How can I determine a factor of safety for these situations ? Does this mean the factor of safety has been insufficient in previous calculation stages as well ?

Answer:
In situations where the initial unbalance is relatively large, it may be hard to get the Phi/c reduction process started. To this end, Plaxis automatically takes one nil-step prior to the actual start of the Phi/c reduction process and the global error is automatically reduced to 3%. Apparently these precautions are not always sufficient. If you experience a decreasing factor of safety (Σ-Msf) at the start of a Phi/c reduction calculation you should try to run a 'Staged construction nil-step' (Staged Construction calculation without switching on or off elements) in order to reduce the unbalance forces (see also bulletin No. 3) . In some situations the nil-step may give the message 'Soil body collapses'. If this is the case, the stability is already critical (SF<1.0) and the entire analyses should be re-run, using a lower global error, if more insight in the exact point of failure is required. If, on the other hand, the nil-step ends normally ('Ultimate level fully reached'), the unbalanced forces have generally been reduced sufficiently to successfully re-start a Phi/c reduction calculation.

Finally another general remark on Phi/c reduction calculations is made. A Phi/c reduction calculation is a Load advancement - number of steps calculation. Therefor the calculation process will continue until the prescribed number of steps has been reached. As an initial start 30 steps can be chosen. However it may be necessary to continue the calculation for several additional steps in order to obtain a steady state solution. On continuing a number of steps calculation it is advised to use the option 'Extrapolation' rather than prescribing again the increment of the first load step. On using extrapolation the calculation process restarts using the step size information from previous load steps. Using extrapolation ensures a continuation of the calculation process. In Version 7, the program will automatically use the extrapolation procedure if applicable.

ACTIVITIES

9 OCTOBER, 1997

Workshop (Dutch) Plaxis User Association (NL), Delft, the Netherlands

6-7 NOVEMBER 1997

4th European Users meeting (English), Karlsruhe, Germany

20 NOVEMBER, 1998

First Norwegian Users Meeting (Norwegian), Oslo, Norway

26-28 JANUARY, 1998

Standard course on Computational Geotechnics (English), Noordwijkerhout, the Netherlands

16-19 FEBRUARY, 1998

Short course on Computational Geotechnics (English), Bangkok, Thailand

30, 31 MARCH AND 1 APRIL, 1998

Short course on Computational Geotechnics (German), 'Finite Elemente Anwendungen in der Grundbaupraxis', Stuttgart, Germany

25-27 MAY, 1998

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

8-10 JUNE, 1998

Short course on Computational Geotechnics (English), Boston, U.S.A.

For more information on these activities, please contact:

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