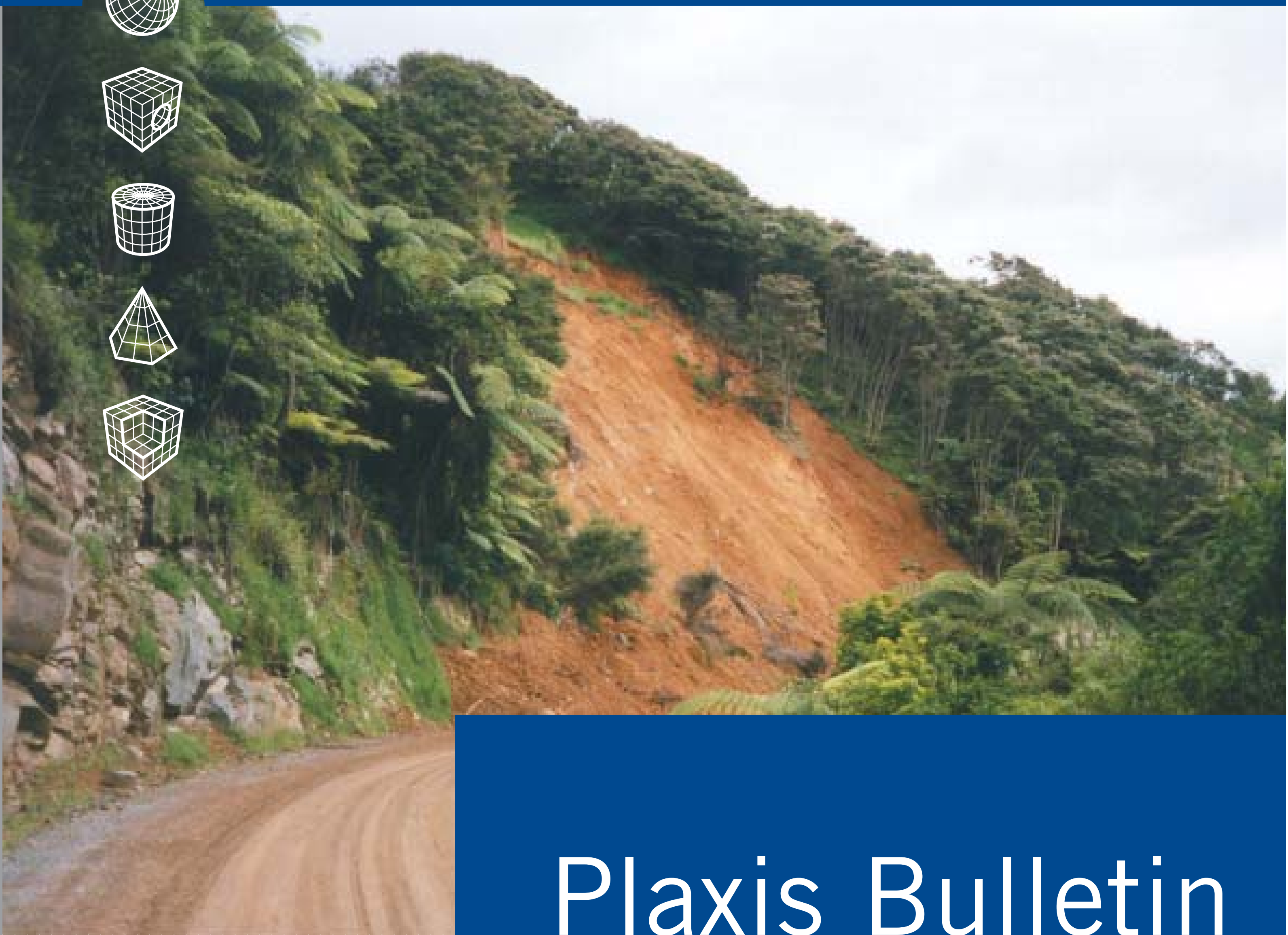
PLAXIS FINITE ELEMENT CODE FOR SOIL AND ROCK ANALYSES



Plaxis Bulletin

issue 16 / october 2004

SPRING CONSTANT AND SOIL-STRUCTURE INTERACTION PROBLEMS

Use of Interface Element for Simulation of Breccia Resliding on Claystone

PLAXIS

Colophon

The Plaxis Bulletin is the combined magazine of Plaxis B.V. and the Plaxis Users Association (NL). The Bulletin focuses on the use of the finite element method in geotechnical engineering practise and includes articles on the practical application of the Plaxis programs, case studies and backgrounds on the models implemented in Plaxis. The Bulletin offers a platform where users of Plaxis can share ideas and experiences with each other. The editors welcome submission of papers for the Plaxis Bulletin that fall in any of these categories.

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

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

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Cover photo: Hill slide near Kawhia, NZ

SINCE THE PREVIOUS BULLETIN WAS PUBLISHED, A LOT HAS BEEN HAPPENING FOR USERS OF THE THREE-DIMENSIONAL PLAXIS PROGRAMS: THE FIRST VERSION OF 3D FOUNDATION SAW THE LIGHT IN MARCH AND A NEW VERSION OF 3D TUNNEL (VERSION 2) WAS RELEASED JUST LAST MONTH. AIMED AT DIFFERENT APPLICATIONS, THESE PROGRAMS OFFER A USER-FRIENDLY ENVIRONMENT, EASY INPUT OF 3D PROBLEM GEOMETRIES AND A ROBUST AND STATE-OF–THE-ART CALCULATION KERNEL. MORE INFORMATION CAN BE FOUND IN THE RECENT ACTIVITIES COLUMN AND AN OUT-LOOK ON UPCOMING DEVELOPMENTS IS, AS ALWAYS, GIVEN IN THE NEW DEVELOPMENTS COLUMN.

Why such 3D finite element programs are useful is discussed in one of the Plaxis Practise articles. It shows different approaches for calculating raft foundations and concludes that for the complex foundation constructions that are needed more often nowadays, a simple spring constant approach may no longer be a safe and valid design method. Another article looks into the formation of a land-slide in Indonesia and uses interface elements to capture the local softening of the rock-soil boundary that leads to the failure.

This issue of the Bulletin also contains the long-awaited results of the Plaxis Benchmark No.3, an embankment on soft soil. Helmut Schweiger discusses the main results and differences sent in by various Plaxis users for this exercise in the Plaxis Benchmarking column on page 5. As not all results could be discussed in detail, some more results from this Benchmark can be found on the Plaxis website. Also, a new benchmark case, this time simulating the behaviour of a single axially loaded pile, is introduced. I encourage everyone to take part in this informal exercise and to send your results for Benchmark No. 4 to Helmut Schweiger at Helmut.Schweiger@tugraz.at before December 15.

New Developments

Ronald Brinkgreve, Plaxis BV

In March this year, the Plaxis 3D Foundation program was released. Since then we have received a lot of feedback on the performance and functionality of the product. Therefore our programmers have worked hard to improve and extend the user-interface. Also the efficiency and speed of the calculation kernel has been improved. The improvements are now available as Update Pack 3.

Moreover, an upgrade of the 3D Tunnel program (Version 2) was recently released. In this release missing features as compared to the Plaxis 2D Version 8 have been included, such as steady-state groundwater flow, consolidation and updated mesh calculations. Moreover, to facilitate the sometimes labourous input of settings for construction stages a convenient copy option has been implemented.

Regarding Plaxis 2D a start has been made with some new developments. Some of these will be made available for Version 8, whereas others will become available in Version 9 (2006):

A start has been made with the implementation of the Sekiguchi-Ohta model. This has been done in cooperation with a co-worker of Professor Ohta from the Tokyo Institute of Technology. The Sekiguchi-Ohta model is a soil model that has been developed in Japan. It has some similarities with the well-known Cam-Clay model, but it has an anisotropic hardening rule. With the implementation of this model an attempt is made to adapt Plaxis to users in the far east.

In addition, the Hardening Soil model was extended with a formulation for small-strain stiffness. This development has been performed in cooperation with the German BundesAnstalt für Wasserbau. The extension makes the HS model even more suitable to model soil in general. Examples where small-strain stiffness is particularly important are tunnelling problems and excavations. The existing models usually overpredict the width of the settlement trough above tunnels and behind excavations. Small-strain stiffness can improve this situation. Last but not least, small-strain stiffness is also important for dynamic applications. However, a further development to account for cyclic loading effects still needs to be considered.

Besides constitutive model developments, the Plaxis 2D user-interface has been modified internally to be able to deal with different languages. In addition to the standard english language a first translation has been made in Indonesian. The Indonesian Plaxis version will be released soon, and some other languages will be considered in the near future. Please note that a language will only be considered if a particular region or language requires special attention to enable the use of Plaxis and there is a good incentive to make the translation.

Plaxis and EuroCode 7

Ronald Brinkgreve, Plaxis BV

Plaxis is often used for geotechnical design. In this respect, many Plaxis users have to deal with Plaxis results in relation to geotechnical design codes, like Eurocode 7, for ultimate limit states (ULS) as well as Serviceability Limit States (SLS). Eurocode 7 is based on the philosophy that the design value of resistance, R_d, must be larger or equal to the design value of action, E_d: $R_d \geq E_d$ where $R_d = R_c / \gamma_R$ and $E_d = E_c\,\gamma_E$. γ_R and γ_E are partial factors on the resistance and action respectively. Considering the safety of slopes and excavations, distinction is made in EC7 in three different design approaches: DA1, DA2 and DA3, whereas in DA1 two sets of partial factors have to be considered (DA1/1 and DA1/2). Moreover, distinction is made between Actions, Soil properties and Resistances *(see Table 1 and Reference [1])*.

Table 1: Partial factors for Actions, Soil properties and Resistances according to Eurocode 7

Design approach	Actions γ_F		Soil properties γ_M				Resistances	
	Permanent unfavourable [1]	Variable [2]	tanφ	c	c_u	Unit weight	Passive	Anchor
	γ_G	γ_Q	γ_φ	γ_c	γ_{cu}	γ_γ	$\gamma_{R;e}$	γ_a
DA1/1	1.00	1.50	1.00	1.00	1.00	1.00	1.00	1.10
DA1/2	1.00	1.30	1.25	1.25	1.40	1.00	1.00	1.10
DA2	1.35	1.50	1.00	1.00	1.00	1.00	1.40	1.10
DA3	Geot.[3]: 1.00 Struct.[4]: 1.35	1.30 1.50	1.25	1.25	1.40	1.00	1.00	1.00

[1] Favourable permanent action: $\gamma_G = 1.00$
[2] When unfavourable; favourable action should not be considered
[3] Geotechnical action: action by the ground on the wall
[4] Structural action: action from a supported structure applied directly to the wall

With the current option of phi-c-reduction in Plaxis it is, to a certain extend, possible to prove that situations comply with DA1 or DA3. DA2 involves an increase of unfavourable permanent action. This means for a situation of an excavation that the active soil pressure behind a wall (= unfavourable permanent action) needs to be increased by a factor 1.35. However, in finite element calculations the soil pressure behind a wall is a result of the calculation and cannot be directly controlled without influencing the passive pressure (= favourable permanent action). Moreover, there are some other limitations that complicate a Plaxis calculation according to Eurocode 7:

- Partial factors on c' and $\tan_\varphi$' cannot be accomodated for.
- Factoring of load and strength properties cannot be done simultaneously.
- Structural safety is not properly considered.

In order to improve this situation and to enable Plaxis calculations to comply with Eurocode 7 (at least DA1 and DA3), an extension is planned with the following provisional procedure:

- An extended type of safety analysis (in addition to the existing phi-c reduction procedure) will be created that is based on the 'total multiplier' approach in Plaxis, such that the design value of resistance and the design value of action will both be reached precisely at the end of the phase. This calculation will also provide the design values of forces in structural elements.
- Distinction is made between partial factors for tanφ, c (or c_u), structural strength, load system A, load system B and soil weight, such that diffent partial factors can be applied simultaneously. It may be needed to include a better description of non-linear behaviour in structural elements.
- The extended safety analysis is a separate calculation phase that can start from any serviceability state where characteristic values of strength and load have been utilized. If geometrical safety factors are required (for example considering over-digging), then such a situation should be created first as the starting point for the safety analysis.
- In the safety phase the partial factors are applied step-wise and linearly, such that at the end of the calculation the partial factors have been reached precisely (provided that soil failure does not occur).
- If the calculation finishes succesfully and the partial factors have been reached, it can be concluded that there is sufficient safety against geotechnical failure according to the corresponding Design Approach in Eurocode 7. Structural safety still has to be checked on the basis of the structural forces resulting from this calculation. These can be interpreted as the design values of structural forces.
- If soil failure occurs before the factors have been reached, it can be concluded that the design does not comply with the safety requirements of Eurocode 7.

The above procedure does not solve the problem of increased unfavourable permanent action in DA2. Nevertheless, it, it will definitely be an improvement compared to the current method of phi-c reduction.

In the coming months details of this procedure will be discussed with experts on geotechnical design codes. We also welcome comments from Plaxis users (from you!) to make sure that the new procedure is clear and user-friendly. In addition to the extended safety analysis procedure we also plan to implement a new procedure for parameter variation. This will bring Plaxis a step further in geotechnical design. As usual, the procedures will first be implemented in Plaxis 2D (Version 9) before the implementation in 3D versions is considered.

[1] European Committee for Standardization (1997). EN 1997-1, 2002. Eurocode 7 – Geotechnical design, part 1: General rules. Final Draft CEN/TC 250/SC7.
[2] Bauduin C., De Vos M., Frank R. (2003), ULS and SLS design of embedded walls according to Eurocode 7. Proc. XIII Conf. on Soil Mech. and Geotech. Eng., Prague. Balkema, Lisse.

Plaxis Benchmark No. 4:

Helmut F. Schweiger, Graz University of Technology, Austria

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

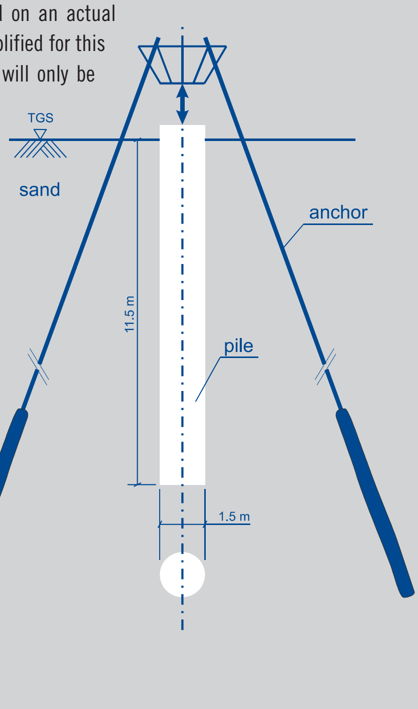


Figure 1: Geometry of the pile load test

Plaxis Benchmark No. 3: Embankment 1 - Results

Helmut F. Schweiger, Graz University of Technology, Austria

INTRODUCTION

The Benchmark Problem No. 3 has been published in Bulletin No. 13 in January 2003 but due to a misunderstanding the specification was not complete. The correct specification has been printed in Bulletin No. 14 but here the submission deadline has been set too tight because of some delay in printing the bulletin. Therefore the deadline for submission has been extended as announced in Bulletin No. 15. Probably due to these unfortunate circumstances the number of submissions was not overwhelming. However, some Plaxis users sent results which are presented in the following.

In the specification for the problem the constitutive model (Soft Soil model) and the parameters to be used have been given together with the thickness of the soft soil layer on which the embankment layer is placed. The initial stresses have been specified assuming a pre-overburden pressure.

Single Pile 1

The following computational steps have to be performed:

- Initial stresses with $\sigma'_v = \gamma h$ and $\sigma'_h = \sigma'_v K_0$
- Installation of pile
- Loading of pile

Results to be presented:

- Load-settlement curve of pile (total resistance)
- Load-settlement curve separating tip and shaft resistance

Material	Properties	Sand	Pile
γ	[kN/m^3]	21.0	25.0
c'	[kPa]	1.0	
φ'	[°]	35	
ψ'	[°]	5	
K_0^{nc}	[-]	0.426	
ν_{ur} / ν	[-]	0.2	0.2
$E50^{ref}$ / E	[kPa]	4.5E4	3.0E7
E_{oed}^{ref}	[kPa]	4.5E4	
E_{ur}^{ref}	[kPa]	1.35E5	
m	[-]	0.5	
p^{ref}	[kPa]	100	
R_{inter}	[-]	0.7	

Table 1: Material parameters for soil layer and pile

All results have to be presented in Excel-sheets (Plaxis-project files optional) and mailed to: helmut.schweiger@tugraz.at **Deadline: December 15th, 2004**

According to the rules set when this section of the Bulletin was introduced no names of authors will be disclosed and all entries are labelled Emb_1 to Emb_9. If two different analyses have been submitted by one institution the second one is named e.g. Emb_7a. The specification of the Benchmark example is not repeated here, please refer to the Bulletin No.14 for details.

REMARKS ON SUBMITTED ANALYSES

The specification given did not leave much room for personal interpretation other than lateral extension of the mesh, element type and number of elements. However two participants decided to perform an "updated mesh" analysis and one participant submitted two analyses with different K_0-values, because of the slight overconsolidation given via the POP-value. Another matter, which leads to significant differences in results, was the following: in the specification λ, κ and e_0 have been given and not λ^* and κ^* as required for the Soft Soil model. One participant was not sure about this and submitted two analyses which enabled identification of other solutions presumably also based on taking λ and κ for λ^* and κ^*.

COMPARISON OF RESULTS

Figure 1 plots the time - settlement curve for the point x=0 and y=0, i.e. the point on the surface at the centre of the embankment. Two groups evolve, namely the ones who interpreted λ and κ for λ^* and κ^* (Emb_2,3,4,5,6) and the ones who worked out λ^* and κ^* from λ , κ and e_0 (Emb_1,7,8,9). Emb_7 provided both results. Emb_2 and Emb_3 performed an "updated mesh" analysis.

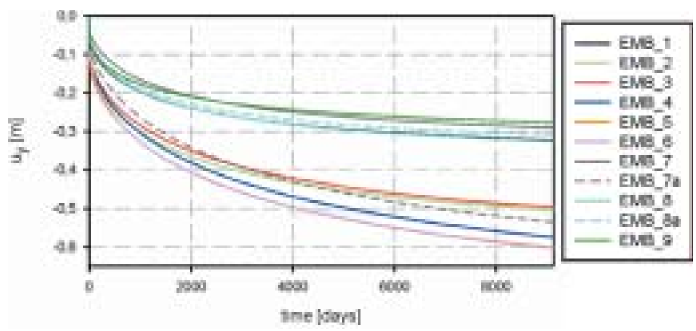


Figure 1: Time vs settlement for point x=0, y=0

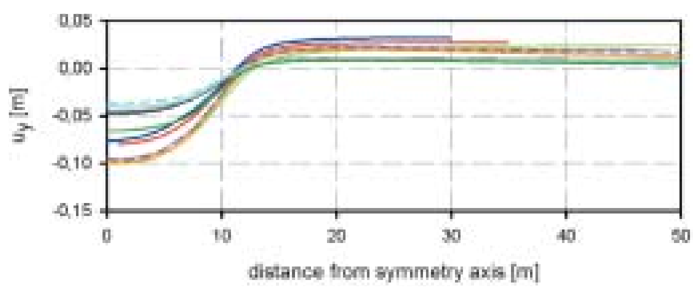


Figure 2: Surface settlement trough after construction

Within the group using the correct λ^* and κ^* only minor differences in results are observed, a slightly wider scatter is noticed in the "other" group. Figure 2 shows the surface settlement trough after construction and Figure 3 after 25 years of consolidation. Figure 4 indicate horizontal displacements at a vertical profile at x=12 m which is at the toe of the embankment. It follows again that in the first group of results only minor differences are visible, whereas in the second group larger differences are observed, the reason for which could not be identified from the information provided with the submitted analyses. They are however in general not larger than 20%. A very similar picture is obtained for time - excess pore pressure curves (Figures 5 and 6) and stress paths in p'-q-space (Figure 7). The trend is very similar for all analyses and most of these differences can be attributed to the fact that depending on the finite element mesh used, the integration points will not be located exactly at the same point for all analyses. The different stress path of Emb_8a is because of the variation in K_0.

Finally it is emphasized that stress distributions along various sections are almost exactly the same for all analyses indicating the negligible influence of mesh coarseness and other modelling details for calculating stresses at working load conditions. These diagrams are therefore not reproduced here but can be found on the Plaxis website for the sake of completeness.

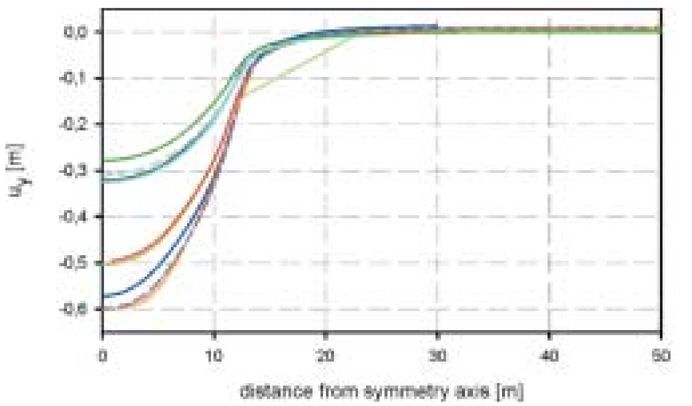


Figure 3: Surface settlement trough after consolidation

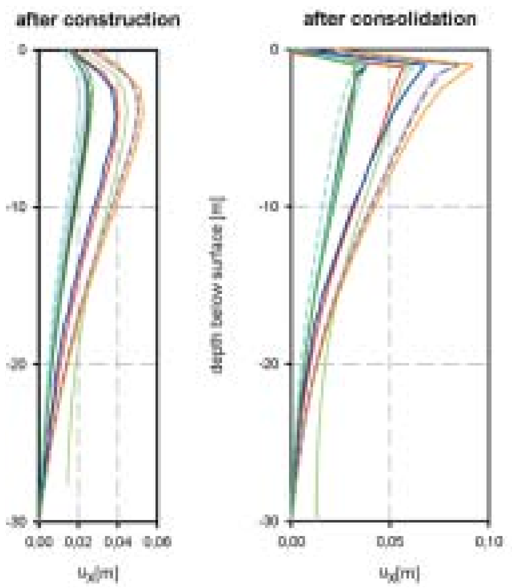


Figure 4: Horizontal displacement at vertical profile x=12 m

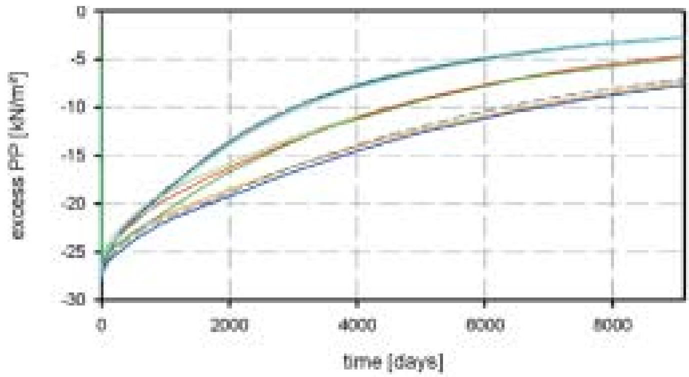


Figure 5: Time vs excess pore pressure for point x=0, y=10

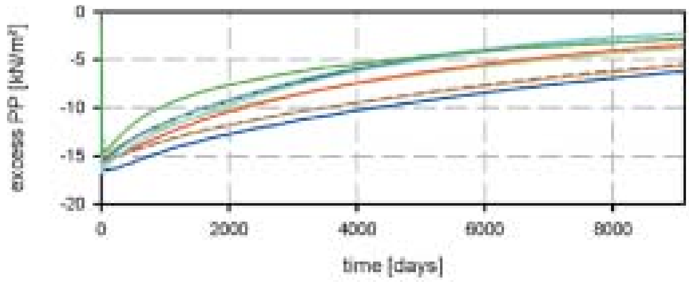


Figure 6: Time vs excess pore pressure for point x=12, y=10

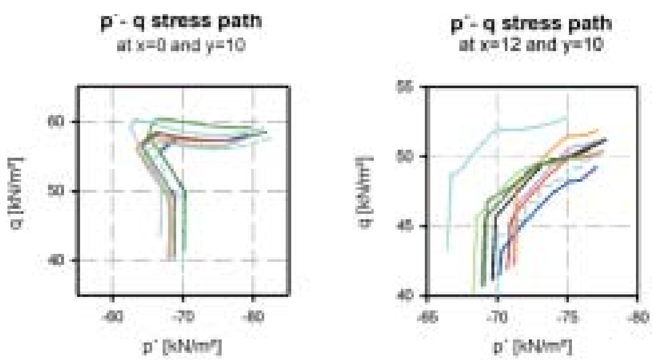


Figure 7: Selected stress paths in p'-q-space

CONCLUSION

Compared to previous benchmark exercises the results submitted for the consolidation analysis of an embankment constructed on a soft soil layer showed quite consistent results. The only major discrepancies can be attributed to the fact that a number of participants of this benchmark exercise took the given parameters λ and κ for λ^* and κ^* as required for the Soft Soil model and thus calculated significantly larger displacements.

The fourth PLAXIS Benchmark, published in this Bulletin (see page 4), is a load test of a single pile and I hope that a number of Plaxis users take the opportunity to join the benchmark community.

Recent activities

GEOTECHNICAL INNOVATIONS

Symposium and 60th Birthday of Professor Pieter Vermeer.

On June 25, a Symposium on Innovative Developments in Geotechnical engineering was held in Stuttgart, which is a regularly organized event. This time the event was dedicated to the handover of a “Festschrift”, (Honorary Book) to Professor Vermeer on the occasion of his 60th birthday. Such a “Festschrift”, is customary in Germany at the occasion of the 60th birthday of a university professor.
For the compilation of the “Festschrift”, the colleagues of Prof Vermeer were invited to contribute with a paper that reflects the common interest in their and Prof Vermeer’s work.

This question was well responded on and more than 80 (co)-authors in total contributed to a total of 45 papers, which where assembled in the “Festschrift”
The symposium itself on the 25th of June 2004 was held at the Hotel Fontana in Stuttgart and was well attended by over 130 persons. During the symposium different speakers presented their contribution on topics, such as Tunneling, Excavations, Soil improvements and Foundations.
Finally afterwards, Prof Vermeer held a short summary of the different topics that had been discussed and thanked the speakers for the symposium and the contributors for their part in the “Festschrift”.

The role of Prof Vermeer in the development of Plaxis is well known, which is reflected in the fact that the Plaxis company maintains a close cooperation with the Geotechnical Institute at the University of Stuttgart ever since he was appointed there as head of the Institute. Therefore we participated with pleasure in his anniversary and congratulate him and his team with this successful event.

NEW PRODUCTS

Since bulletin 15 Plaxis B.V. released 2 new products; 3DFoundation version 1 and 3DTunnel version 2. Although we got very positive response on the concept of the new 3DFoundation program we had to improve the robustness and efficiency up to a level as you may expect from Plaxis products. See also new developments for more details.

Now we are in the process to built introductory versions of both 3D products. Later this year we will come up with a new Demo CD that includes besides Plaxis V8 also introductory versions of 3DFoundation v1 and 3DTunnel v2.

In corporation with RIB Software AG we finalized the English version of FEwalls – a new software tool for checking serviceability requirements of retaining walls using PLAXIS-technology. In the field of geotechnical engineering, the new generation of Euro Code-based national application documents will introduce new requirements for high-tech and economical engineering solutions on an international standard. On this background, several questions arise with respect to the application of standard software solutions for the analysis and design of retaining wall structures. Fewalls is especially designed to meet the requirements of the new Euro Codes in easy to use geotechnical software solutions.

Furthermore we will release an Indonesian version of Plaxis V8 and we are in the process of finishing a Spanish, Portuguese, Russian and Japanese Manual. These new manuals (and online help) will become available through our website or our local agent.

COURSES AND USER MEETINGS

Until the printing of bulletin 16 already more than 300 participants learned something new or improved their skill in 2004 at one of our courses. Besides a global geographical coverage we extended our course program also in number of languages.

“Course participants of the 1st Italian Plaxis course”.

If you did not attend one of our courses you could also have met Plaxis people at Geotechnical Conferences or User Meetings. On November 11-12, 2004 in Karlsruhe, Germany we scheduled the 11th European Plaxis User Meeting. In addition to this 2-day session we organize a special day on the applications of User-Defined Soil Models. This meeting is meant for anyone who is interested in the application of user-defined soil models. We hope to meet you at the European User Meeting, one of our courses or on Geotechnical Conferences. Please check out our presences at the Agenda.

NOTES ON THE APLICATION OF THE SPRING

GOUW Tjie-Liong, PT Limara, Indonesia

AGENTS

Plaxis B.V. appointed a third agent in the U.S.A. From May 1st the company GEMSoft (Geotechnical Engineering Modeling Software) will be an official Plaxis agent for the U.S.A. GEMSoft is located in the Central part of the U.S.A. with offices in Chicago, IL and Houston, TX.

The Houston office is headed up by Kenneth E. Tand, and the Chicago office by Erik G. Funegard. Both Erik and Kenneth have long backgrounds in geotechnical engineering with a strong focus on the petrochemical industry.

Kenneth holds a Master's degree in Civil Engineering from the University of Houston and has been a practicing geotechnical engineer for over 35 years. Ken has been a Plaxis user for over 10 years and has published several papers on the use of Plaxis to solve difficult geotechnical problems.

Erik holds a M.Sc. in Civil Engineering from the Royal Institute of Technology in Stockholm Sweden, as well as an MBA from the University of Chicago. Erik was the chief geotechnical engineer for a major international oil company for over 10 years with first-hand experience of the use of advanced analysis techniques to reduce construction costs.

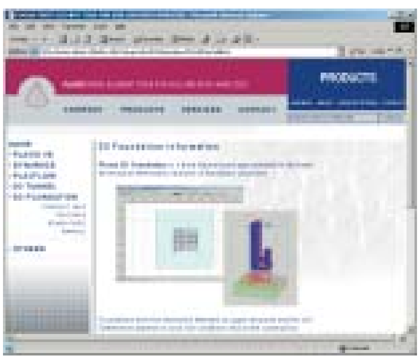
All three agents can act, as agent for the whole U.S.A. but GemSoft will primarily work in the central U.S.A. The Plaxis agent for mainly the West part of the U.S.A., C. Felice & Company, LLC with its headquarter in Kirkland, Washington has been merged with LACHEL & Associates, Inc. The new firm will be known as LACHEL FELICE & Associates, Inc.
For contact details of GemSoft, Lachel Felice and the long-term Plaxis agent in the East of U.S.A., GeoComp see our website.

TERRASOL celebrates its 25th anniversary! This event will take place in Paris-La Défense on September 28th 2004. TERRASOL was founded in 1979, and has regularly grown up since (more than 30 people today) as a leading geotechnical consulting company, working in fields like foundations, tunneling, maritime works, excavations, earthworks, infrastructures, etc.

TERRASOL has always used its geotechnical know-how and expertise to develop and sell its own software. It became PLAXIS' agent for France in 1998, and now participates in the PDC program and Plaxis Advisory Board. Its software department also provides services like technical support and continuing education.

REVISED WEBSITE

A revised website is launched to disseminate information more transparently. We hope we have created an informative Website that makes life easier for the Plaxis users and others. News, product and course information is easy to find and updates are easy to access.
Any additional suggestions are welcome; please send them to a.bregman@plaxis.nl.



INTRODUCTION

The use of a spring constant for the design and analysis of raft and pile-raft foundations has many limitations, related to the proper estimation of the spring constant magnitude and the soil structure interaction. Spring constants have been used for example in the design of a Mass Rapid Transit railway station in an oversea project. The overview of soil condition on that particular site is as shown in Fig. 1 below.

At this project the spring constant concept was adopted for designing the station raft foundation. The structural engineer asked for the magnitude of the spring constant from a young geotechnical engineer, who then gave a coefficient of subgrade reaction (in kN/m^3) derived from a plate-loading test. This parameter was later converted into a foundation coefficient of subgrade reaction, k_s, by using the following equation:

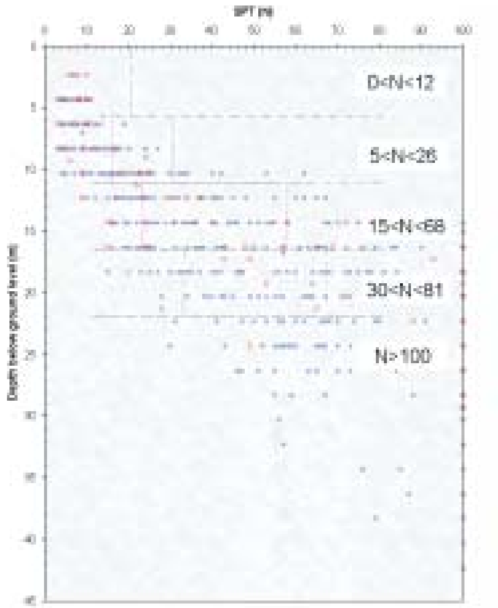


Figure 1: SPT vs Depth

$$k_s = k\left[\frac{B+0.3}{2B}\right]^2 \qquad (1)$$

where B is the width of the raft foundation.

This last parameter was then applied as a spring constant by multiplying it with the unit area under the raft foundation (the unit dimension became kN/m). A certified Professional Engineer then approved the outcome of the raft foundation design for construction. Without prejudice to blame others, it is obviously a mistake! Why it is so? For B greater than 0.3 m, equation 1 clearly shows that the greater the value of B the smaller the value of ks. While it is structurally correct that the wider the foundation the more flexible the foundation is. It does not equally right for the foundation soil. The engineers had missed the fact that the soil at that area was far from homogeneous.

The soil condition shows that, within the influence of the raft foundation, the deeper the foundation soils the harder they are. This means the deeper soils have greater rigidity as compared to the layer right below the raft foundation (note: the width of the raft is around 35 m).

The inappropriate spring constant led to an excessive settlement of the raft. As a result, in order to reduce the settlement, the center of the raft was strengthened with more than 20 number of bored piles. Upon reviewing the design, the author proved that the bored piles were excessive and unnecessary. However, by the time it was found, it was too late.

The above case shows the application of spring constant without considering the characteristics and the behavior of the underlying soils. And it is also an example of the existence of ignorance, gaps and weakness in the relation among the structural and geotechnical engineers. This papers tries to elaborate the underlying principle the spring constant theory, its limitation and the application of specially made geotechnical software to solve the problem of soil structure interaction.

SPRING CONSTANT - THE THEORETICAL BACKGROUND AND THE LIMITATION

"What is the spring constant at this particular site?" or "What is the modulus of subgrade reaction at this location?" is a common question asked by a structural engineer to a geotechnical engineer. It is a straightforward question. Unfortunately, it has no direct, let alone a simple answer.

The concept of spring constant was first introduced by Winckler in 1867. He modeled flexible foundation, such as raft, to stand on an independent discreet spring elements or supports. In 1955, Karl Terzaghi, in his paper *'Evaluation of coefficients of subgrade reaction'* proposed a method to estimate the magnitude of the spring constants. His approach, also known as subgrade reaction model, was then became popular and commonly used in the design of raft foundation.

Looking back into the origin of this concept (see Fig.2), one can see that the modulus or the coefficient of subgrade reaction, $k_s(x)$, is defined as the foundation pressure, p(x), divided by the corresponding settlement of the underlying soil, d(x), i.e.:

$$k_s\,(x) = \frac{p(x)}{d(x)} \qquad (2)$$

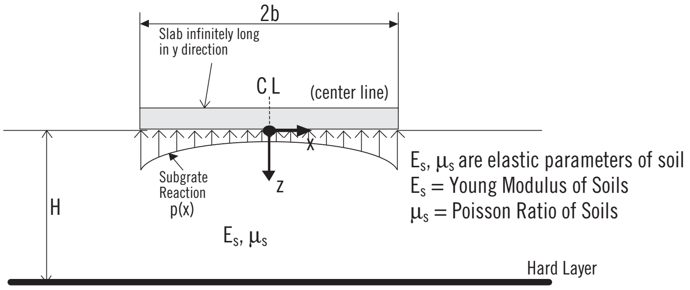


Figure 2: Subgrade Reaction under a Flexible Foundation

In other words, the subgrade reaction is no other than the distribution of soil reaction, p(x), beneath the raft foundation structure against the foundation load. The distribution of the soil reaction is not linear in shape. This is particularly true when the foundation is subjected to uniform load. In this case, generally, the distribution of the soil reaction in clayey soils is curving upward, as shown in Fig. 2, with the largest reaction around the edges of the foundation and the smallest reaction around the center. In sandy soils, the reverse reaction is seen, i.e. zero on the edges and maximum at the center point. In principle, the distribution of the soil reactions right beneath the raft foundation depend on the position of the point under consideration (i.e., the distance of x), the shape of the loading and the relative rigidity (EI) of the raft foundation structure against the underlying soils.

The Winckler model is a simplified mathematical formulation of an elastic soil model. This concept does not take into account the fact that the foundation reaction or the soil stresses is distributed to the deeper soil layer and forming the so called 'bulb pressure'. The soil settlement beneath the foundation is the accumulation of interactions between the soil stresses and the elastic parameters of the soils at each point inside the bulb pressure zone. Assuming the soils inside the bulb pressure zone posses are homogeneous, Vesic (1961) expanded the Winckler model into elastic model and developed the following equation:

$$k_s = \frac{E_s}{B.I_p.\,(1-v_s^{2})} \qquad (3)$$

The above Vesic's equation clearly shows that the modulus of subgrade reaction depends not only on the width of the foundation, B, but also on the elastic parameters of soils, E_s and μ_s, and on the shape factor of the foundation, I_p.

In the earlier days, for the sake of mathematical simplicity, it is generally simplified that the spring constant is not a function of the position x (see Fig.2), hence a single value of spring constant is applied. However, the non-linearity distribution of the soil reactions right beneath the foundation structure suggests that the so-called modulus of subgrade reaction, hence the spring constant, is not a unique value. Terzaghi himself recognized the limitation of this assumption. Bowles (1997) suggested providing higher ks at the edges of the raft and smaller k_s at the center position.

The above explanations show that there is no discrete value of modulus of subgrade reaction for a given type of soil *Therefore, it does not realistic to ask for a spring constant value without the information on the type and the size of the foundation structure.*

In layered soils with different elastic parameters, an equivalent model must be developed in order to derive a representative modulus of subgrade reaction. To do this the elastic settlement of the layered soils induced by the foundation pressure must first be calculated. Poulos and Davis, 1974, mathematical formulation can be used to calculate the elastic settlement of the foundation soils. In a pile raft foundation, to answer the question on the magnitude of the spring constant, the geotechnical engineer also has either to calculate the settlement of the pile foundation or derives it from a pile load test result.

Since the modulus of subgrade reaction (spring constant) is needed to calculate the settlement of the foundation soils, why should one goes to the trouble in providing the spring constant? The structural engineers asked the spring constant because they want to feed in the parameter into their computer software. To the author knowledge, as it is not developed to handle geotechnical problems, the structural engineering software used in analyzing raft or pile raft foundation cannot handle geotechnical parameters.

Another limitation of the spring constant model is the assumption that the foundation soil has linear or elastic behavior. In reality, since Winckler introduced his theory (1867) 133 years have lapsed, and the geotechnical engineering has kept on advancing. It has been known that soil behavior does not elastic.
It is an elastoplastic material with different behavior within each classification, and many soil models have been developed.

PROPER SOIL MODEL AND SOIL STRUCTURE INTERACTION

In order to provide a relatively simple and quick solution for the analysis of raft foundation, Winckler followed by Terzaghi, simplified the mathematical formulation into the spring constant or modulus of subgrade reaction model. Over the time, many geotechnical experts had gained better and better understanding on soil behavior and many soil models has been developed. Many of them come with complex mathematical equations, which needs more advanced computer technology and special finite element software to solve.

Until late 1980s where computer hardware, software and run time cost was still very expensive, the spring constant model was indeed one of a good tool for engineers. However, since mid of 1990s and especially as we enter this new millenium, advanced Personal Computer and the relevant geotechnical engineering software has become available and affordable for most firm. So why don't we use a specific finite element method to solve a soil structure interaction problem?
Nowadays, finite element software, such as PLAXIS, CRISP, SIGMA, etc., which is specially developed to solve geotechnical problems has been available. T

CASE STUDY ON SOIL STRUCTURE INTERACTION

In a densely populated city, it is not uncommon that a subway tunnel must be constructed underneath an existing building foundation or the reverse, that is to construct a building on top of an existing tunnels. In 1998, the author had a chance to evaluate such a problem. At that time a twin tunnel subway project was on its way. These 6.3 m diameter twin tunnels shall cross some 30 m underneath a land where a condominium building was planned. The landlord was wondering when to construct his building, before or after the tunneling?

If the building was constructed before the tunnels passed the area, he had no responsibility on the tunnel construction and it would be the tunnel contractor responsibility to take precaution not to induce any negative impact to the building. However, at that time the macro economy situation was not favorable for the sales of the condominium. On the other hand, if the building was constructed later, the impact of the building construction to the twin tunnels had to be studied. And this might lead to a more costly foundation, as there is a requirement that any pile foundation from the ground surfaceto the spring-lines of a subway tunnel must not bear any friction resistance. The other option available is to strengthen the tunnel lining to anticipate the future additional stresses that come from the building foundation. And the building owner would have to contribute on its cost.

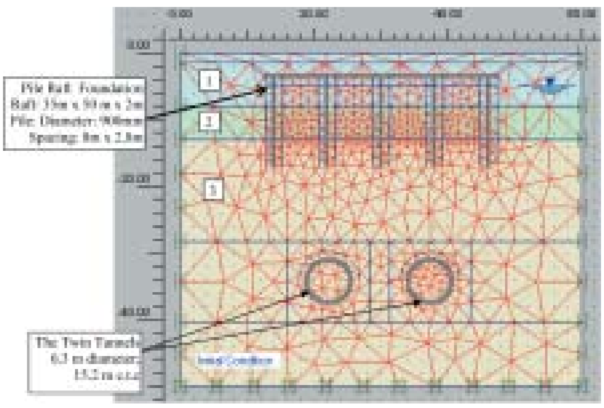


Figure 3: The Finite Element Model of The Initial Condition

Figure 3 shows the initial condition of the site and the subsequent soil parameters. The center of the tunnel lines is 35 m below the ground surface. Landscaping of the site required a 1.5 m excavation and this was done before the tunneling. The base of the raft foundation would be around 3.5 m from the ground surface. The groundwater level was found at about 3.75 m below the ground surface. Table 1 shows the soil data. Mohr-Coulomb soil model was adopted to perform the analysis.

Mohr-Coulomb Number	Identification	Type	γ_{dry} [kN/m³]	γ_{wet} [kN/m³]	k_x [m/day]	k_y [m/day]	ν [-]	E_{ref} [kN/m²]
1	stiff silty clay	Drained	16.0	18.0	1.3000E-3	5.0000E-4	0.33	30000.0
2	hard silty clay	Drained	16.0	20.0	1.3000E-3	5.0000E-4	0.33	60000.0
3	sandstone	Drained	19.0	21.0	0.0130	0.0086	0.33	1E5

Number	Identification	c_{ref} [kN/m²]	ϕ [°]	ψ [°]	R_{inter} [-]	Interface Permeability [-]
1	stiff silty clay	5.0	22.0	0.0	0.70	Neutral
2	hard silty clay	10.0	38.0	0.0	0.70	Neutral
3	sandstone	15.0	42.0	5.0	0.80	Impermeable

Table 1: The Soil Data

Many possible construction sequences were analyzed. The construction sequence presented in this paper is as follows:
- Overall excavation up to 1.5 m deep.
- Bored piles construction
- Tunneling (followed by volume loss)
- 2.0 m excavation for raft construction
- Raft construction
- Building Construction and Load Application

The results of the final stage construction are presented below.

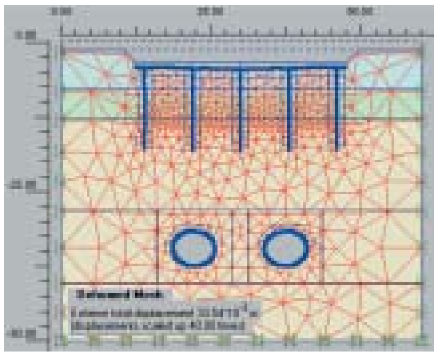


Figure 4: Deformed Mesh

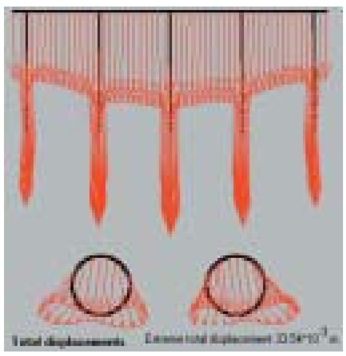


Figure 5: Pile Raft and Tunnels Total Displacement

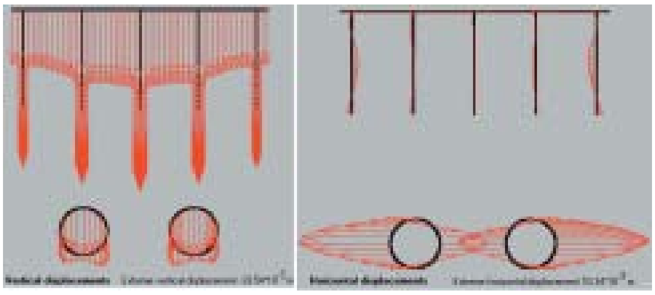


Figure 6: Pile Raft and Tunnels Vertical and Horizontal Displacement

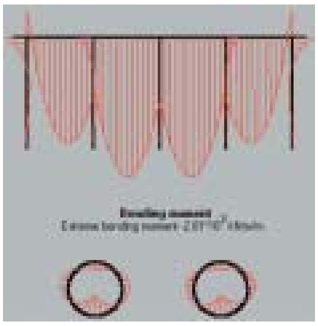


Figure 7: Pile Raft and Tunnels Bending Moment

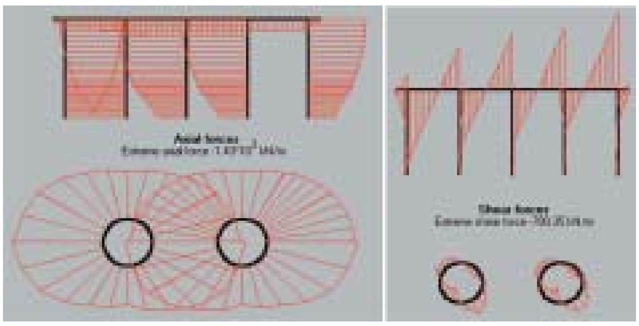


Figure 8: Pile Raft and Tunnels Axial and Shear Forces

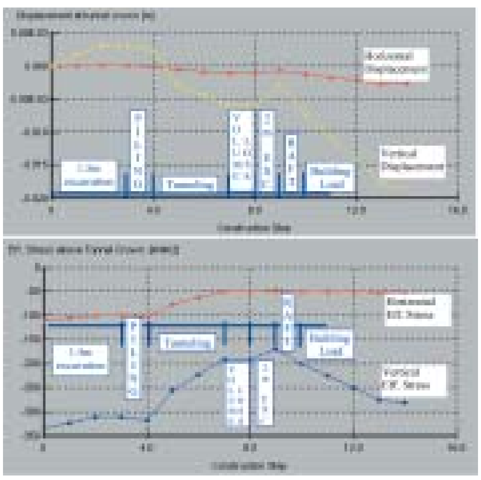


Figure 9: Changes of Stress and Displacement right above Tunnel Crown

With the said construction sequence, the result of the analysis shows that the maximum pile raft settlement would be in the order of 33 mm. The analysis also predicted that the building would exert additional vertical stress of 90 kN/m^2, with a corresponding 12 mm vertical displacement, to the tunnel crown. The above example was one of the input for project evaluation. It shows the importance of the soil structure interaction analysis, which cannot be solved by using the spring constant model.

CONCLUSIONS

The above discussions show that there is no straightforward answer to the question of: "What is the magnitude of the spring constant (or the modulus of subgrade reaction) at this site?" It is inappropriate for a geotechnical engineer to provide the said parameters without knowing the system and the size of the foundation. The non-linearity of soil reaction beneath a footing or raft foundation suggests that the k_s value is not a unique value. Great care must be exercised in deriving the value. It is always important to have a good communication, understanding and cooperation between the structural engineer and the geotechnical engineer in solving a particular foundation problem.

Since the computer technology and the relevant finite element software has become relatively cheap and readily available, whenever possible, it is suggested to perform a soil structure interaction analysis and leave behind the spring constant concept. As demonstrated above, nowadays, the geotechnical finite element software is capable to handle complex soil structure interaction problem, which cannot be solved by the spring constant model.

Last but not least, the derivation of the input soil parameters is very important. As soil is not manmade materials, strong theoretical knowledge and sophisticated engineering software alone is not adequate.
A geotechnical engineer must gain plenty of practical experiences in order to come out with a sound engineering judgment in determining the relevant soil parameters for a particular soil model. It does not mater how sophisticated computer software is, the adage "Garbage in Garbage out" is always prevails.

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USE OF INTERFACE ELEMENT FOR SIMULATION OF BRECCIA RESLIDING ON CLAYSTONE

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1 INTRODUCTION

South Semarang is a hilly and mountainous area where a number of landslides or land movement occur almost yearly. The area is well known as Jatingaleh, which in Javanese means ‘moving trees’ since most of the trees are moving in certain direction mainly during rains.

In the project site where new road was to be constructed, an excavation of 4.0 m depth was conducted. This excavation was initially thought to be the cause of the landslides. Spring water appeared at the toe of the slope and the excavation shows the severely weathered breccia. However, this initial opinion cannot be justified due to the fact that the building on top of the excavation still remains in good condition. No significant damage was noticed and the steep slope looks very stable. Figure 1. shows this phenomenon.

Figure 1: Excavation in the breccia layer

2 RESEARCH OBJECTIVE

This research was intended to study the mechanism of the sliding that occurred and to recommend a solution for the landslide problem. The work was divided into a geological investigation, including the drilling and soil laboratory tests, and the geotechnical work, including analysis and recommendation for future development. The geological work was conducted by Wahjono, et al (2002) and the geotechnical analysis was done by the author and team. The use of interface elements is the main method for simulation of the sliding mechanism.

3 DAMAGES OF INFRASTRUCTURES

Based on information by the residence, the land movement occurred yearly but in small increment so that ordinary paste and rehabilitation is very common. In this particular slide, however, the damage was so severe and large soil movement destroyed the infrastructure as well as housing.

The movement started early in the morning when about 25 cm of sudden drop initialized the whole movement. Afterward, subsidence occurred systematically reaching 3.5 m in about 10 hours. A number of cracks appeared in this area and water sprang out through gaps in the cracking ground. This phenomenon caused several houses tilt and crack and by the end of the day the whole infrastructure was devastated.

The subsidence was localized in the fill area where a number of infrastructure including roads, drainage and electric towers were damaged. Since the subsidence caused 3.5 m difference in elevation, the connecting road tore out in certain patterns either laterally across the road or longitudinally. Figure 2 and Figure 3 shows severe damage of the infrastructure.

Figure 2: Damages due to subsidence

Figure 3: Road damage and gabion heaves

Figure 4: Ground cracking

Figure 4 shows ground cracking parallel to the road and Figure 5 shows how the land elevation moved downwards about 3.5 m in one day. The direction of the cracking indicated that the land experienced lateral spreading. However, at this point the real mechanism was not known until re-measurement of the contour was conducted.

Figure 5: Land subsidence causing different elevation

4 THOUGHT ON THE LANDLSIDES MECHANISM

It is of interest that this phenomena gives thought about a possible sliding mechanism which seems to be movement of the breccia over the impervious layer since a water level rise was detected the day after. Although in the east side, movement and cracks were seen, in the west side no cracking or damages were observed.

The fact that water level increase is so rapid in the breccia layer, means that water accumulated and was retained in this layer. Weathered breccia on the surface shows a lot of cracking that eases water seeping into the ground. Land development hinders water outflow which cause high lateral forces for the whole layer and subsequently moved it in a translational direction.

It was later known that the subsidence has been caused by this lateral spreading of the breccia. The subsiding ground consisted of fill material, which is well compacted in a valley located between 2 hills of breccia. According to the geologist, these valleys were formed by very old slides or volcanic activity.

5 SOIL CONDITION AND GEOLOGY

The geomorphology of the area consisted of a valley within 2 hills and was a low site where sediment was deposited. This area was then filled with local material to increase the elevation and to form a more or less plane area. The stratigraphy shows that 3 kinds of materials dominated the site. The lower one is claystone over the whole area and forming an impervious layer. The upper one is breccia with andesitic fragment and filled with secondary material. The average thickness of the breccia layer in this area is about 15 m and this layer formed two hills as shown in the cross section.

The other material is found as alluvium, which is the minority of the material type. This material was the result of ground erosion over time.

6 MECHANISM OF RE-SLIDING OF BRECCIA LAYER

After the landslides, a contour measurement was conducted as shown in Figure 6.

The topography shows a long steep slope over the whole area except the area where the sliding problem is being investigated. The difference in elevation over a distance of about 500 m is about 100 m, hence on the average the slope is about 20∞. The mechanism of re-sliding was known from visual observation verified by contour measurement. From site survey, the breccia layer is shown to have many cracks to facilitate water seeping into the layer and causing a ground water level increase.

The cross section of this area is shown on Figure 8 where it is clearly shown that the breccia layer is limited and sitting on top of the impervious layer. The fill and the land development area are also shown.

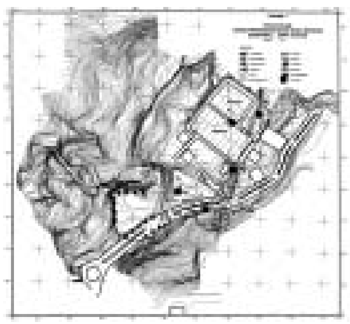

Figure 6: Topography of the study site

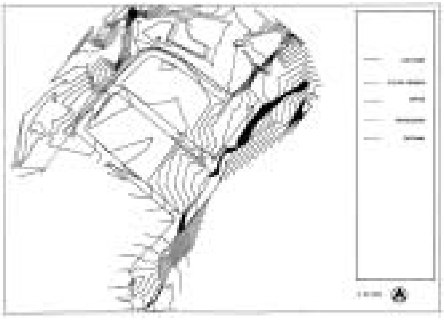

Figure 7: The re-measurement of contour

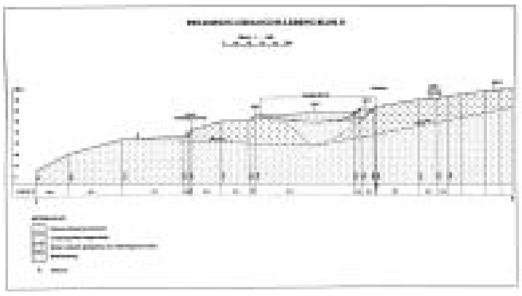

Figure 8: Cross section of the weathered breccia on claystone

7 SIMULATION OF RE-SLIDING

This research was facilitated by use of finite element analysis (computer program PLAX-IS) where stages of land changes are made possible. The analysis is limited to two-dimensional. The model started with initial conditions where the breccia layer was assumed to be stable and hence interface elements were used in the boundary between the breccia and the claystone. The value of the interface strength was exercise by changing the value of R-interface until movement started in the breccia.
The model was continued by placing fill material on top of the sliding area and infiltration of rainwater is made possible. The effect of rain water infiltration is modelled as an increase of the ground water in the breccia layer. Soil model is ordinary Mohr – Coulomb. The soil has low strength of 6 – 24 kPa and a friction angle about 8∞.
The soil system is discretized to clusters of material where geotechnical properties can be input. Modelling the excavation causes some movement of the breccia layer, which is insignificant. However, the main movement occurs when modelling the ground water level increase, which certainly gives lateral forces to the whole mass.
Figure 9 shows the discretization of the mass through the critical section where the boundary of the breccia and the claystone is characterized by a sliping plane having the potential to slip between each other.

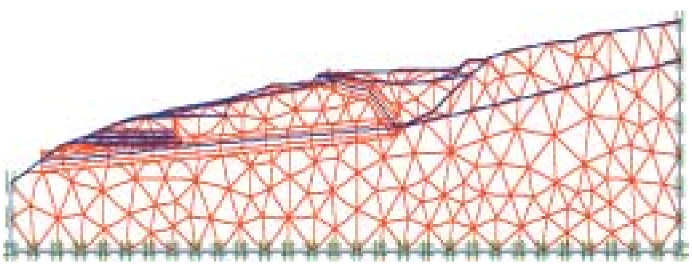

Figure 9: Discretization of the soil mass

Based on the result of the analysis, the effect of fill material is a compression of the sediment to about 27 cm, especially in the mid area. However, this analysis cannot be verified since no measurement was available. The mass failed with huge displacements when the ground water was raised to about ground surface.
Figure 10 shows the localized mass movement resulting from the finite element modelling.

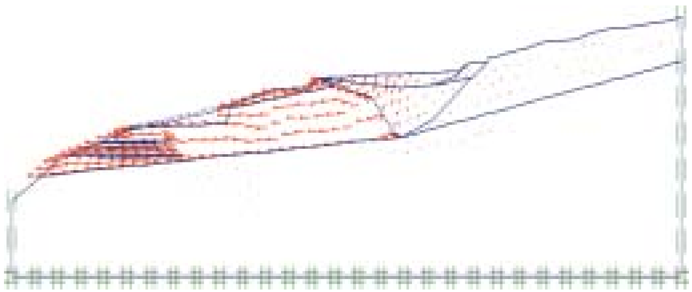

Figure 10: Result of computer simulation

The movement was predicted to reach about 1.3 m but this is lower than measured data, since after failure the computer simulation cannot be continued. Back analysis for the value of R-interface yields a friction coefficient as low as 0.25.
The recommendation, which resulted from this study, is a water level decrease by use of pumping in several areas. For the long term, several horizontal drains were installed. Rows of drilled shafts were installed to protect the rest of the building.

8 CONCLUSION SUMMARY

- The landslides, which occurred on February 8, 2002, were basically re-sliding in a big scale on the breccia layer on top of the claystone due to ground water rise.

- Fill placement and limited excavation was not the cause of the landslides, however, small movement were detected.

- Finite element modelling is very useful in simulating the mechanism of the slides and can also be used to back analyse the interface value between the breccia and the claystone.

- The fact that water level increase significantly affects the landslides tells the developer to be cautious and minimizing rain water seeping in into the breccia.

- Lesson learned from this landslide occurrence is that the landslide is transtational and in such rock stratification, landslide could occur as a sudden mechanism due to the increase of ground water level alone.

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PRACTICAL APPLICATION OF THE SOFT SOIL CREEP MODEL – PART II

D. Waterman, Plaxis BV & W. Broere, Delft University of Technology/Plaxis BV

UNDRAINED BEHAVIOUR

In the previous issue of the Plaxis Bulletin some basic aspects of the Soft Soil Creep model have been discussed. For simplicity, the soil behaviour was limited to drained behaviour, so that the influence of the overconsolidation ratio (OCR) and the initial creep velocity could be shown clearly. However, in the real world soils that exhibit creep behaviour generally have a low permeability. These soft soils almost always show undrained behaviour under short term loading. This combination of undrained behaviour and creep raises some additional issues that will be discussed here.

In the previous Plaxis Tutorial, a square block of clay was modelled using Soft Soil Creep (SSC) material, with standard boundary conditions, initial stresses due to its own weight and no initial excess pore pressures. For the current example, we will change the material behaviour from drained to undrained, and re-examine its behaviour.

Assume first that it is possible to seal all sides of this block of soil, so that any excess pore pressures that develop can not drain off and effectively consolidation cannot occur, but deformation is not hindered. Common clingfoil will do this nicely, as will the closed consolidation boundaries that are available in Plaxis. Now leave the block of soil undisturbed for a considerable time period. After for example 10 years, the material will still be almost undeformed, but has developed significant excess pore pressures inside, even though no external load was applied.

This development of excess pore pressures in the absence of external loads or deformation is completely logical for a SSC material. Normally, creep behaviour of the soil causes plastic deformation and, as a result, a decrease in volume. However, as the material is undrained and there is no possibility to consolidate, volume strains are not allowed in the model. Therefore, the plastic volume strain that is calculated due to creep has to be compensated by an elastic volume strain of equal magnitude but opposite direction. As the total volume strain is the sum of the elastic and plastic volume strains, this results in a nett volume strain equal to zero.

According to Hooke's law, elastic expansion, a negative elastic compression, leads to a decrease of the effective stresses. But, since no external load was applied, the total stresses must remain the same, and a decrease of the effective stresses is only allowable if the excess pore pressures increase. It is this mechanism that causes the observed excess pore pressures. Of course, this mechanism will continuously increase the excess pore pressures over time, but the creep that drives it will diminish rapidly. As the creep rate depends on the effective stress level, and the excess pore pressures decrease the effective stresses in the sample, the creep rate will decrease at an even higher rate than in the case of a drained material.

The shift of stress from the effective stress part of the total stress to the (excess) pore pressures is also the cause for the very small deformation that is observed in the Plaxis model. Even though water is often assumed to be totally incompressible, it is not, and the increased pore pressures cause a very small volume strain of the pore water, and thereby of the entire model.

This is, of course, a rather theoretical example. Soil is hardly ever left completely undisturbed for 10 years in a situation where consolidation cannot occur. But this example still shows an important feature of the SSC model: creep behaviour of undrained soils leads to an increase of excess pore pressures.

Consolidation, on the other hand, deals with the dissipation of excess pore pressures over time in soils with low permeability and has the opposite effect. The combination of consolidation and creep in a creep sensitive soil can therefore cause a wide range of behaviour. The precise behaviour depends on whether creep or consolidation is dominant.

Over time, creep would tend to increase the pore pressures, causing the creep velocity to decrease and the consolidation velocity to increase. Due to consolidation the pore pressures will drop again, which causes the consolidation rate to decrease and the creep rate to increase. This combination of effects is illustrated below.

A simple 1D consolidation test has been performed on four different soil data sets, each with the same strength and stiffness parameters as listed in the previous Plaxis Bulletin, but with permeabilities varying between 0.1 m/day and 10_{-4} m/day. For this exercise closed consolidation boundaries have been added to the left and right side of the square soil block, as shown in Figure 1.

Figure 2 shows the excess pore pressure in the middle of the soil sample as a function of time. After first loading the sample with 100 kPa over a period of 1 day, the excess pore pressure is 109 kPa in all cases (100 kPa due to the load and 9 kPa due to creep during the first day). For the highest permeability (0.1 m/day) the excess pore pressures immediately drop when consolidation starts, whereas for lower permeabilities the excess pore pressures first increase for a while, until consolidation really becomes the dominant effect and the excess pore pressures finally decrease. Note that for a permeability of 10_{-4} m/day the excess pore pressures even rise to a peak value of 130 kPa after 63 days!

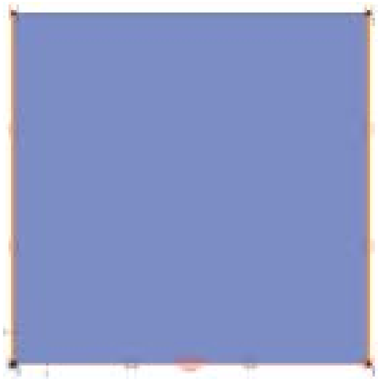

Figure 1: Geometry with closed consolidation boundaries

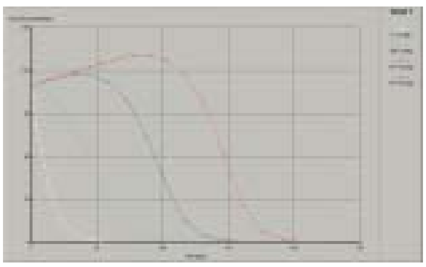

Figure 2: Development of excess pore pressures over time for the various cases

Activities

2004

29 - 30 September 2004
Plaxis 2-Day Course for Intermediate users (English)
Petaling Jaya, Selangor Malaysia

5 October 2004
Funderingsdag
Ede, The Netherlands

18 - 20 October 2004
International Course for Experienced Plaxis Users (English)
Trondheim, Norway

19 October 2004
Norwegian Plaxis users meeting
Trondheim, Norway

11 - 12 November 2004
European Plaxis users meeting,
Karlsruhe, Germany

13 November 2004
Special users meeting on applications of User Defined Soil Models
Karlsruhe, Germany

17 - 19 November 2004
Pratique éclairée des éléments finis en Géotechnique (French)
Paris, France

22 - 26 November 2004
15th Southeast Asian Geotechnical Conference (English)
Bangkok, Thailand

2005

4 - 7 January, 2005
Short course on Computational Geotechnics + Dynamics (English) - New York, USA

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

24 - 26 January 2005
Geo-Frontiers - Austin, Texas

February 2005
2-Day Course for Intermediate users (English) - Bali, Indonesia

March 2005
Finite Elementen in der Geotechnik (German)
Stuttgart, Germany

21 - 25 March 2005
International course for Experienced Plaxis Users (English)
Noordwijkerhout, The Netherlands

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

13 – 15 May 2005
Short Course for Experienced Plaxis Users (English)
Istanbul, Turkey

15 - 17 June 2005
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