PLAXIS

essential for geotechnical professionals

Plaxis bulletin

Issue 28 / Autumn 2010

Swell in building pits using 3DFoundation

3D finite element analyses of deep soil improvement

PLAXIS analysis of a basement excavation in central London

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The Plaxis bulletin is the combined magazine of Plaxis bv 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 e-mail) 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 vector based format (eps,ai). If photographs or 'scanned' figures are used the author should ensure that they have a resolution of at least 300 dpi or a minimum of 3 mega pixels. The use of colour in figures and photographs is encouraged, as the Plaxis bulletin is printed in full-colour.

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Colophon

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Editorial board:
Wout Broere
Ronald Brinkgreve
Erwin Beernink
Arny Lengkeek

Design:
Blemmodesign

Any correspondence regarding the Plaxis bulletin can be sent by e-mail to:

bulletin@plaxis.nl

or by regular mail to:

Plaxis bulletin
c/o Erwin Beernink
PO Box 572
2600 AN Delft
The Netherlands

For information about PLAXIS software contact your local agent or Plaxis main office:

Plaxis bv
P.O. Box 572
2600 AN Delft
The Netherlands

info@plaxis.nl
www.plaxis.nl

Tel: +31 (0)15 251 7720
Fax: +31 (0)15 257 3107

Editorial

This year has been an exciting year, with the release of two new PLAXIS versions, and the celebration of this achievement during a surprise programme at the European Plaxis Users Meeting in Karlsruhe. This event was visited by around 100 participants from all over Europe.

During the EPUM many Plaxis users filled out the survey on the quality of PLAXIS products and services. The survey is available until the end of 2010. Your input is not only very useful to help us to improve our products and services, but also to help children in Bangladesh with safe drinking water, since Plaxis supports the Cordaid charity project. For more information see the Recent Activities column in this Bulletin, or the Plaxis website www.plaxis.nl/page/home3/.

In this 28th issue of the Plaxis bulletin we have again assembled a nice collection of interesting articles and useful information for you. Do not hesitate to send us your comments or contact the author in case you like to discuss about some items.

The first user's contribution involves a study on swell in building pits using 3DFoundation. The study was based on the entrance shaft of the Sophia Railway tunnel near Rotterdam, The Netherlands. The article shows the deformation and stress developments underneath the concrete floor for the situation without tension piles.

The second user's article involves the analysis of a deep soil improvement, also analysed using the 3DFoundation program. A high number of rigid inclusions was used to model the soil improvement, leading to a impressive 3D model. The article shows the settlement distribution as a result of the different loaded areas.

The third user's contribution involves the analysis of a basement excavation in London and the consequences for adjacent structures. Different scenarios were analysed using PLAXIS 2D V9. It was concluded that PLAXIS provided the necessary confidence to undertake this deep excavation close to sensitive buildings in the city centre.

In addition to the contributions by Plaxis users, the New Developments column continues on the modelling of undrained soil behaviour by presenting an anisotropic undrained strength model, the NGI-ADP model. Then, there is again a joint presentation about a project where Plaxis has provided expert services to a client. More and more clients use to this service and find out that it is an efficient way to perform advanced numerical modelling in collaboration with Plaxis experts. Finally, at the end of the bulletin, you find an overview of our activities in the first half of 2011.

We wish you an interesting reading experience and look forward to receive your contributions for future Plaxis bulletins.

The Editors

New developments

Ronald Brinkgreve, Plaxis bv

In this bulletin I would like to discuss the modelling of undrained shear strength in PLAXIS. Soil behaves undrained if it saturated and if the pore water cannot flow freely in the soil skeleton. As a consequence, the incompressibility of the pore water prevents the soil to change volume. Changes in total mean stress will be carried by the pore water, resulting in excess pore pressures, whereas changes in deviatoric stress will be carried by the soil skeleton, resulting in effective stress change.

In the previous bulletin we described the different methods in the new PLAXIS version to deal with undrained soil behaviour: Undrained A (effective stiffness + bulk stiffness of water; effective strength), Undrained B (effective stiffness + bulk stiffness of water; undrained strength), Undrained C (undrained stiffness; undrained strength). In this bulletin a new model in PLAXIS is described, which is the NGI-ADP model with anisotropic undrained shear strength.

When effective stress models, in which the shear strength is mainly described by friction properties, are subjected to undrained conditions, the generation of pore pressure and the resulting effective stress path become essential in predicting the right shear strength. Most effective stress models, including advanced models, are not capable of predicting appropriate shear strengths under undrained conditions. In this respect, it is also good to mention that the actual shear strength also depends on the specific loading or unloading conditions. Starting from an initial K_0 stress state, different (anisotropic) undrained shear strengths are observed for stress paths leading to active, passive or pure shear failure states, as already reported by Bjerrum in 1973 . Instead of trying to predict the right pore pressure and effective stress path, models have been proposed that enable a direct input of undrained strength (according to the above Undrained B or Undrained C methods).

The NGI-ADP model is such a model, with direct input of undrained shear strength for Active, Direct simple shear and Passive stress states. In addition, three corresponding strain levels need to be defined, which determine (together with the shear strength) the (non-linear) stiffness in those stress paths. A consistent interpolation is used to deal with intermediate stress and strain conditions. The model also allows for a convenient increase of shear strength with depth. This model is now available as a standard model in PLAXIS 2D. For more details on this model the reader is referred to the literature (e.g. Andresen & Jostad, 1999) and the new PLAXIS 2D Material Models manual.

With this new model we trust to have fulfilled the requirements of those who frequently deal with the difficulties of defining the right undrained shear strength.

Ronald Brinkgreve

References

1. Bjerrum L., Aitchison G.D. (1973). Problems of Soil Mechanics and Construction on Soft Clays and Structurally Unstable Soils. Oslo: NGI.

2. Andresen L., Jostad H.P. (1999). Application of an Anisotropic Hardening Model for Undrained Response of Saturated Clay. Proc. NUMOG VII, Graz, Austria, pp. 581-585.

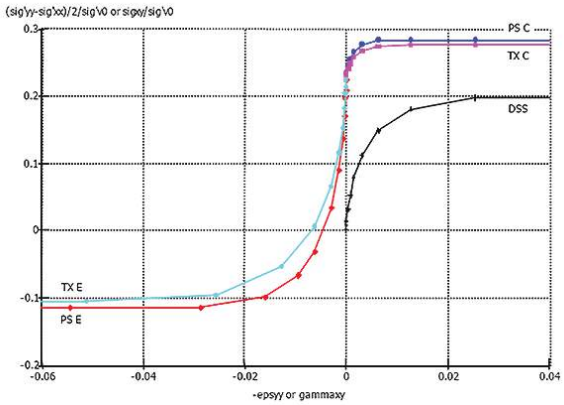

Figure 1: Simulated results of five different lab tests, plotted in the plane strain deviatoric stress space, and shear stress-strain plots

PLAXIS Expert Services update

Tony Ruban, EBA, A Tetra Tech Company

Plaxis was contracted by EBA, A Tetra Tech Company, to provide assistance in setting-up a finite element model for the serviceability analysis of a micropile group to be constructed close to a existing belled pile foundation. Thanks to PLAXIS Expert Services (PES), valuable results in terms of settlement predictions under service loading have been obtained which safely validate the proposed micropile group foundation.

The current project primarily aimed at analyzing the interaction between an existing belled pile and a micropile raft foundation. The micropile group that needed to be analyzed will be built next to an existing 500 mm shaft with a 1500 mm diameter belled pile supporting the column load. As the analysis was meant to be a deformation analysis, only service loads have been considered in the current project. To better understand the behaviour of the belled pile, it was decided to perform a back-analysis of a belled pile load test. This analysis provided a good understanding of how the existing belled pile would behave and enabled PES to calibrate the model against available factual data from a load pile test carried out near the site. A second stage of analysis was carried out including the existing belled pile and the construction of the micropiles where the interaction between the belled pile and the micropile raft foundation could be investigated.

Main results

The FE analyses that have been carried out on the single belled pile model have enabled a calibration of the model parameters to reproduce the pile load test data with a good match of load-displacement curves in both loading and unloading situations, as well as reasonable agreement of forces with available values at strain gauge locations. In the second model, the behaviour of the loaded micropile group was investigated with and without taking into account the interaction from the existing belled pile foundation. Considering different soil-structure interaction coefficients (Rinter) it has been concluded that there was minimal effect of the micropiles on the existing belled pile in terms on load-displacement in the framework of a serviceability analysis.

The company

EBA, A Tetra Tech Company (EBA) provides a broad range of engineering and scientific consulting services to clients in domestic and foreign markets. EBA has progressively expanded over the last 40 years and today clients are served by more than 600 engineers, scientists, technologists, and support staff from 10 offices located in Western and Northern Canada. EBA provides a full range of services including engineering, scientific, technical, planning, prime consulting, and project management on projects within different areas of practice such as geotechnics, mining, energy, transportation and waste management.

> "PLAXIS Expert Services was able to provide the results for our problem in a timely manner."

Customer quotes

"Our requirements for this project included an analysis to assess the impact of new micropiles being installed adjacent to an existing belled cast-in-place concrete pile foundation. We felt that in order to provide a reliable result with a reasonably high confidence level that a 3D finite element analysis was required".

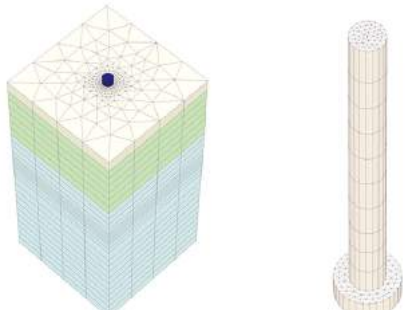

Figure 1: Single pile calibaration model

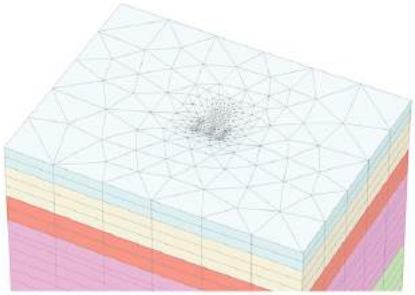

Figure 2: Micropile-belled pile 3D interaction model: settlement prediction

Swell in building pits, using 3DFoundation

Rogier Schippers, Volker Wessels Stevin Geotechniek B.V., rschippers@vwsg.nl

During the design phase and the realization of building pits in the saturated soft soils of the Netherlands, often questions show up with respect to the effects of unloading the subsoil. Due to the reduced pressure, the soil will expand which we call "bottom heave" or "swell". Especially for cohesive soils with a low permeability, it is hard to predict how fast the swell will develop and whether it may result into additional loads on underwater concrete floors and piles.

An MSc thesis project at Delft University of Technology has been performed, in cooperation with the engineering consultant. Movares, to increase the knowledge on this topic. While both the effects on underwater concrete floors and piles were investigated, this article only deals with swell in building pits with an underwater concrete floor without tension piles.

Background

Like elastic materials, soil layers will compress or expand in case of changing pressures. With respect to swell it is important to realize that soft soils generally react differently to loading and unloading. As Figure 1 shows, one may expect that soil layers will not return to the original thickness when loaded for the first time and unloaded afterwards. So the stiffness of the soil is higher for unloading and reloading compared to the primary loading stiffness.

Besides the magnitude of volume change, the time is important. Part of the deformation will take place directly while a more important part will be time dependent in case of cohesive soils. After unloading, groundwater has to flow to the pores as the soil wants to expand. Pouring an underwater concrete floor after excavating a building pit may hinder the expansion of the subsoil (Figure 2). This would cause an upward effective pressure on the floor with a theoretical maximum equal to the effective weight of the excavated soil.

Recommendation 77, drawn up by CUR [1], gives a useful and commonly used method to deal with the load on underwater concrete floors without piles as a result of obstructing swell. Unfortunately, the method presumes a one-dimensional situation and a floor which is fixed in place. A simulation of the problem using 3DFoundation gives more insight.

Working method

During the realization of the starting shaft of the Sophia railway tunnel, upon the instructions of COB committee F210 [2], one has measured the reaction of the subsoil and the construction. Special attention was paid to the thick clay layer of Kedichem. The measurements were used during the MSc thesis project for an inverse analysis to investigate the behaviour of the soil and the construction. Next, a more general model was made to be able to draw conclusions for building pits without the specific constructing method of the Sophia railway tunnel. Although the problem could have been modelled with PLAXIS 2D as well, 3DFoundation was chosen to deal with piles as well during the project later on. As mentioned before, that part will not be discussed here.

Case: Starting shaft Sophia railway tunnel description of the building pit

The building pit at the Sophia railway tunnel was approximately 26 m wide and 64 m long. While several measurements were carried out, the measurements at the clay layer of Kedichem are the most interesting with respect to swell. They were executed in a specific part of the building pit. Figure 3 gives the corresponding intersection and shows the excavation depth of more than 20 m. The clay layer of Kedichem with a thickness of approximately 8 m starts at a depth of -29 m. The

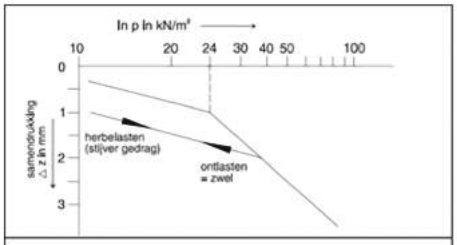

Figure 1: Stress-strain relationships

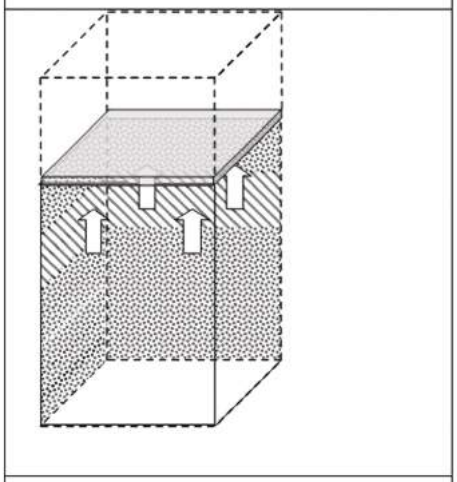

Figure 2: Hindering swell

walls are a combination of steel tubes and sheet piles. In horizontal direction the building pit is stabilized with steel and concrete struts during different stages. After realizing the walls and starting the excavation, the walls were stiffened by filling the tubes with concrete. To reduce the upward pressure on the underwater concrete floor, the water level beneath the floor was lowered during the de-watering process. As a result, a difference in water pressure occurred over the clay layer. This makes the interpretation of the reaction on the unloading more complex

Description of the model

One may assume that the building pit behaves nearly equal in the longitudinal direction, so a semi-2D approach has been chosen. The model in 3DFoundation has a width of 1 m and the boundaries are fixed perpendicular to their planes. Besides, the vertical model boundaries with their normal in z-direction are closed (impermeable) so no excess pore pressures dissipate through these planes. The model is made 140 m wide and 65 m deep so the spreading of loading and unloading will take place without significant influences of the boundaries.

Figures 4 and 5 show the geometry of the model and the different soil layers. The underwater concrete floor is modelled as a floor with linear elastic properties. The hardening process of the floor is neglected so it has an instant isotropic stiffness of 28,500 N/mm^2. The bending stiffness of the walls is determined by the steel tubes filled with concrete. The stiffness of the sheet piles is much less and therefore neglected. The struts are modelled by springs what implies neglecting the weight of the struts. To prevent unrealistic large point loads on the wall, stiff beams are modelled between the springs and the walls.
To deal with swell, a more accurate stress-strain relationship is required than Hooke's law of linear, isotropic elasticity. As noted above, the stiffness of cohesive soil is non-linear and depends on previous stress states. Here, the Hardening Soil model is used which is capable of dealing with these features. Based on CPT's, soil samples and laboratory tests, a parameter set was made. Underneath the concrete floor there are several sandy layers and the clay layer of Kedichem. This important layer with respect to swell contains

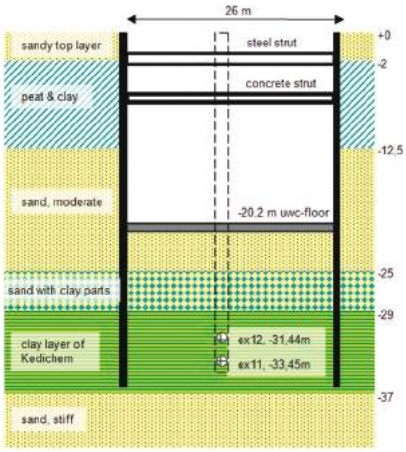

Figure 3: Cross section starting shaft with extensometers

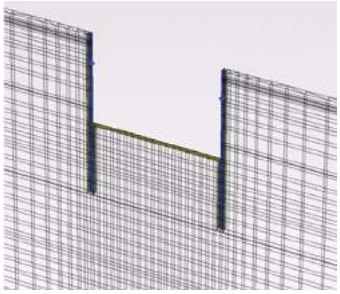

Figure 4: Model of building pit

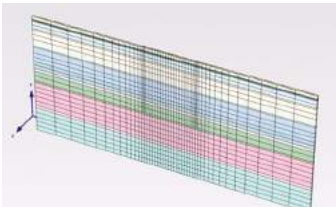

Figure 5: Soil layers in model

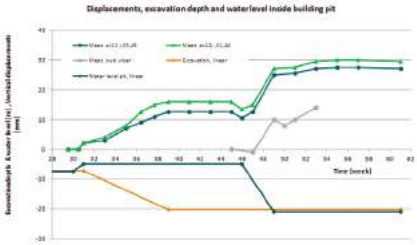

Figure 6: Displacements, excavation depth and water level inside building pit

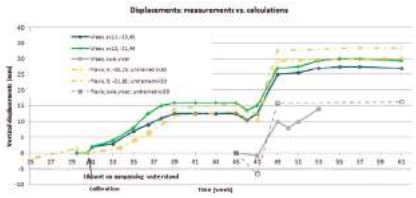

Figure 7: Displacements: measurements vs. calculations

a small layer of peat as well and is, due to ice loads in the past, overconsolidated. Table 1 shows the soil parameters. As one can see, there is a factor up to 7 between $E_{oed,ref}$, and $E_{ur,ref}$. Due to arrangement of clay particles in time, it was also considered to be realistic to increase the horizontal permeability up to 4 times the vertical permeability. In table 2 the phases of the construction and the numerical model are showed. The steps with respect to changing the construction, the water levels or the excavation depth are undrained plastic calculations. The time between steps are taken into account by means of consolidation analyses.

Output and measurements

With a model of the complete realization of the building pit in time as a simplification of the real pit, the behaviour of the soil and the construction can be studied. Two extensometers were placed in the clay layer of Kedichem to measure displacements at depths of N.A.P 31.44 m and N.A.P 33.45 m. The measurements started after excavation to a depth of N.A.P 7.3 m. Figure 6 shows the displacements, the excavation depth and the artificial groundwater level in time.

The first evaluation of the measured and calculated displacements showed that the results where comparable, especially when time increases. The stiffness parameters may therefore be considered as sufficient. Nevertheless the reaction of the soils seemed to be too slow. Several practical reasons may be given but the most important factor is the permeability. After increasing the permeabilities of the clay layer of Kedichem the results are satisfying. Figure 7 shows the results. Besides the displacements and without going in detail, also the behaviour of the modelled construction appears realistic.

A closer look at the measurements and the calculations gives more insight in the behaviour of the soils and the construction. Stress changes and displacements are caused by excavating the building pit and changing the water levels inside the pit, above the clay layer of Kedichem. In case of the starting shaft of the Sophia railway tunnel, heave of the subsoil was clearly visible: during the last phases vertical displacements were measured up to 30 mm. While the clay layer of Kedichem causes the mayor part, also the deep sand layers cause a significant part. Furthermore the clay layer of Kedichem appears to be able to follow the man-made changes in the building pit. A time dependent behaviour is not clearly noticeable which is an important conclusion. This means no upward effective pressures acting on the underwater concrete floor have developed in the starting shaft of the Sophia railway tunnel.

General analyses of swell and upward effective pressures on underwater concrete floors

Based on the soil profile at the location of starting shaft of the Sophia railway tunnel, a more general study has been executed without the specific characteristics of the starting shaft. For example, the drainage inside the building pit during de-watering and the phases of excavation had a mayor influence on the potential upward effective pressure on the concrete floor due to preventing the swelling process. Several aspects have been analyzed like time, the speed of constructing, wall friction, spreading of loads, and the behaviour of the construction itself.

Name	*level* [m + N.A.P]	*thickness* [m]	*Type*	*γ_unsat* [kN/m³]	*γ_sat* [kN/m³]	*k_x* [m/day]	*k_y* [m/day]	*k_z* [m/day]	*E_50ref* [kN/m²]
38D (Kedichem)	-29	1.5	Undrained	18	18	8.00E-05	2.00E-05	8.00E-05	5.60E+03
38B (Kedichem)	-30.5	6.50	Undrained	20.5	20.5	8.00E-06	2.00E-06	8.00E-06	8.50E+03

Name	*E_oedref* [kN/m²]	*E_urref* [kN/m²]	*c_ref* [kN/m²]	*ϕ* [°]	*ψ* [°]	*v_ur* [–]	*p_ref* [kN/m²]	*power (m)* [–]	*OCR* [–]
38D (Kedichem)	5.40E+03	3.20E+04	13	32	2	0.2	100	0.8	1.6
38B (Kedichem)	5.50E+03	3.80E+04	13	29	0	0.2	100	0.8	1.6

Table 1: Hardening Soil parameters for layer of Kedichem

Phase	Description	*Time 1999* [week]	*Duration in model* [days]	*Excavation depth* [m + N.A.P]	*Water level inside pit* [m + N.A.P]	*Groundwater table 1st aquifer* [m + N.A.P]
0	Initial situation		0	0.0	-2.25	-3.0
1	Inst. combined wall		0	0.0	-2.25	-3.0
2	Reset of displace-ments		14	0.0	-2.25	-3.0
3	Change water level	9	0	0.0	-3.0	-3.0
4	Excav. to N.A.P. -2.7	10-11	0	-2.7	-3.0	-3.0
5	Inst. steel struts	12	0	-2.7	-3.0	-3.0
6	Consolidation		42	-2.7	-3.0	-3.0
7	Filling steel tubes	16-17	0	-2.7	-3.0	-3.0
8	Consolidation		35	-2.7	-3.0	-3.0
9	Change water level	20	0	-2.7	-7.5	-7.5
10	Excav. to N.A.P.-7.3	21-22	0	-7.3	-7.5	-7.5
11	Consolidation		72	-7.3	-7.5	-7.5
12	Inst. concrete struts	28	0	-7.3	-7.5	-7.5
13	Remove steel struts	29	0	-7.3	-7.5	-7.5
14	Change water levels	30	0	-7.3	-5.0	-5.0
15	Excav. to N.A.P.-12.5	31-33	0	-12.5	-5.0	-5.0
16	Consolidation		19	-12.5	-5.0	-5.0
17	Excav. to N.A.P.-16.35	33-36	0	-16.35	-5.0	-5.0
18	Consolidation		19	-16.35	-5.0	-5.0
19	Excav. to N.A.P.-20.2	36-38	0	-20.2	-5.0	-5.0
20	Consolidation		51	-20.2	-5.0	-5.0
21	Pour uwc-floor	45	0	-20.2	-5.0	-5.0
22	Consolidation		14	-20.2	-5.0	-5.0
23	Dewatering pit and drainage	46-48	0	-20.2	-21.0	-21.0
24	Consolidation until reloading		98	-20.2	-21.0	-21.0
25	Consolidation. extra		∞	-20.2	-21.0	-21.0

Table 2: Phases in Plaxis model

Swell inside a building pit

The simplified model makes is easier to see that vertical expansion is caused by a direct expansion of sand layer and a time-dependent expansion of cohesive soil with a low permeability. After an unloading by excavating, the induced low water pressures will disappear by a groundwater flow towards the expanding layers. A comparison of this consolidation process is made between the Plaxis model and the theory of Terzaghi. As the unload/reload stiffness Eur is stress dependent, the result of Plaxis correspond with the consolidation process of Terzaghi in case one calculates the coefficient of consolidation c_v based on the unloaded stress situation (Figure 8).

While Terzaghi assumes a one dimensional situation, Plaxis shows this is not the case for swell inside building pits. Like loading, the effect of locally unloading will diminish with an increasing depth. Therefore deep soil layers will contribute less to the total vertical heave. Another important aspect is the interaction between soil and vertical structural elements. Friction along sheet piles or diaphragm walls, limits the vertical displacements close to these boundaries. In Plaxis this friction is modelled by means of interfaces and the roughness of the interfaces is determined by the soil layers and the factor Rinter. Here, the friction between steel and soil is assumed to be significant so $R_{inter} = 1.0$. Figure 9 gives a plot of the effective stress and makes the effect of spreading and wall friction theoretically clear.

Besides friction, the deformation of walls may also influence the development of swell. When excavating a building pit, the walls usually will bend and undergo a horizontal displacement towards the excavated area. In theory, this might load the soil, partly undo negative water pressures and make the soil deform with only little change of volume. In this case, the time-dependent behaviour would be less compared to a one dimensional situation.

Upward effective pressures on underwater concrete floor due to obstructing the swelling process

When the swell is hindered by the presence of an underwater concrete floor, an upward effective pressure may develop. The theoretical maximum equals the effective weight of the excavated soil (the initial effective stress at the excavation depth). Of course, this maximum is partly counterbalanced by the weight of the underwater concrete floor itself. More interesting is the time spent between unloading and pouring the concrete floor. The grade of consolidation determines the potential upward effective pressure. So for the one-dimensional case see Figure 13.

Figure 10 shows the development of the swell and the relation with the potential upward effective pressure. It makes clear that the potential upward effective pressure is large in case excavation takes place fast and the underwater concrete floor is poured directly when the final depth is reached. In common practice the excavation speed is in the order of 1 m/week and some time is required before the floor can be made. This leads normally to a negligible potential upward effective pressure.

With the potential upward effective pressure at a certain time, the interaction with the construction may start. Analyses with 3DFoundation shed light on the effects of the behaviour of the construction. The stiffness of the underwater concrete floor turns out to be important, so is the vertical translation of the construction in total. First of all, when the floor is made of concrete with a small stiffness, then the floor will bend because of upward pressures. Especially when the building pit is de-watered, the curvature is such that swell will be less hindered in the centre of the pit. Figure 11 and 12 show the deformations and the effective stresses.

In the other case, when the stiffness of the underwater concrete floor is large, the upward effective pressure will be of a higher level. Still, the hindering of the development of swell will be restricted in case the total construction will move upwards. This will also especially be the case when dewatering the building pit and large water pressure working on the floor. The starting shaft of the Sophia railway tunnel made clear building pits and its structural elements may be translated by a few centimeters in upward direction.

Some final words

The analyses give insight in the development of the swell and interaction with a building pit. In the analyzed Dutch situations, several effects prove to limit the possible upward effective stresses on underwater concrete floors. This article only discusses analyses with 3DFoundation in case of building pits with no tension piles. For questions and remarks about this topic and the interaction with piles, please contact the author.

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- COB committee F210 Swell, Meten en interpreteren van zwel in een bouwput, startschacht Oost Sophiaspoortunnel, final report F210-E-02-083, 2002.

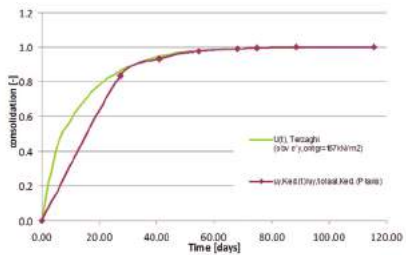

Figure 8: Development of swell

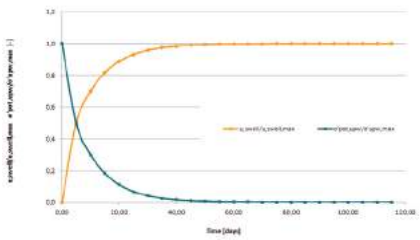

Figure 10: Development swell and the potential upward effective pressure

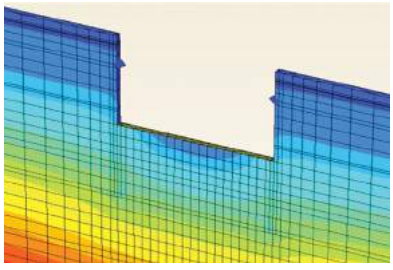

Figure 12: Effective stresses underneath concrete floor

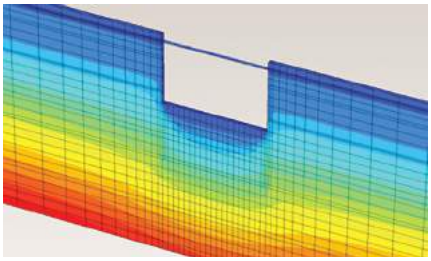

Figure 9: Effective stresses after excavation

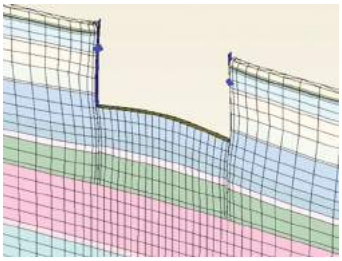

Figure 11: Deformation of soil and construction

$$\sigma'_{pot,upw} = \sigma'_{max,upw} \cdot (1 - U(t_{pour.})) = \sigma'_{in,z} \cdot (1 - U(t_{pour}))$$

in which:

$\sigma'_{pot,upw}$ = the potential upward effective pressure by hindering swell

$\sigma'_{max,upw}$ = the theoretical maximum upward effective pressure by hindering swell

$\sigma'_{in,z}$ = initial effect vertical stress

$U(t_{pour.})$ = degree of consolidation when the underwater concrete floor is poured

Figure 13

3D finite element analyses of deep soil improvement

Florian Scharinger, Roland Lüftenegger, GDP ZT-OG civil engineers, Graz/Klagenfurt/Oberalm, Austria

Due to the fact that deep soil improvement by rigid inclusions is common practice when soft underground conditions are explored, a study including analytical and numerical analyses was performed. For the full study, a large field with different commercial constructions was investigated, whereas a local part with which was classified as very sensitive relating to settlements and/or differential settlements was analyzed in detail.

Furthermore, the constructions in this area cause high loads on the foundation slabs. Since the expected subsoil conditions in the concerned area have to be classified as soft to very soft in most of the soil layers, a deep soil improvement by rigid inclusions was planned and investigated by different analyses. In order to estimate the prospective settlements of the different components of the construction, settlement analyses with analytical methods were performed in advance and more complex finite-element-analyses (3DFoundation) were carried out in order to obtain more detailed information.

The performed analyses were initiated by similar conditions for a real project, but it is clearly stated, that the presented analyses do not represent direct input from the real project and results cannot be compared to monitoring data. The aim of the numerical analyses is to show how the estimation of the final settlements and of the required consolidation time can be performed.

Underground conditions and modelling

Based on the results from underground investigations in the vicinity of the investigated area by cone penetration tests and core drillings an underground model consisting of five layers was developed for the 3D finite element analysis. Generally, the subsoil conditions in the first 35 metres below the top ground surface (approximately flat) are described as an alternating layering of sandy, silty clay and gravel with different thickness:

0 m to 6 m, 8 m to 18 m below surface and below 21 m: sandy, silty, clay consistency: soft to very soft $E_{oed}^{ref} = E_{50}^{ref} = 4{,}000$ kN/m², $E_{ur}^{ref} = 12{,}000$ kN/m², $p^{ref} = 100$ kN/m², m=0,85 OCR=1.0, POP = 0	6 m to 8 m and 18 m to 21 m below surface: gravel consistency: loose $E_{oed}^{ref} = E50^{ref} = 32{,}000$ kN/m², $E_{ur}^{ref} = 96{,}000$ kN/m², $p^{ref} = 100$ kN/m², m=0,5 OCR=1.0, POP = 0

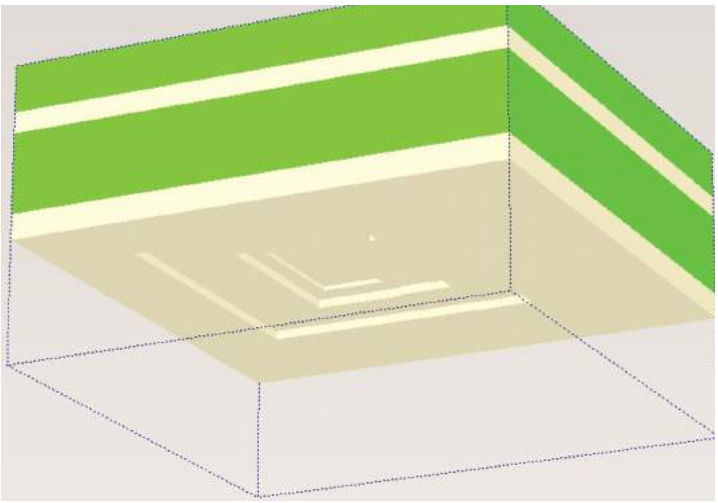

Figure 1: Bottom view of deep gravel layer below 18 m, visualisation of the varying layer thickness (calculation model without lowest clay layer)

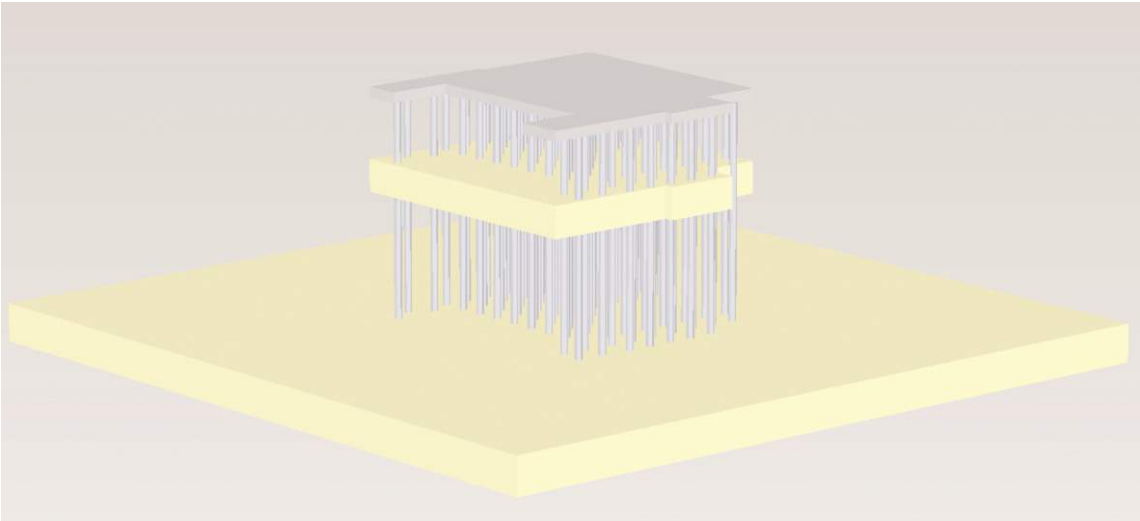

Figure 2: Partial view on the structural elements

For the second gravel layer in between of 18 and 21 metres below surface a very local changing of the thickness was adopted in order to model a local weakness directly below the foundation slab. The shape of the area with reduced thickness of the gravel layer was estimated by almost concentrical rectangles, whereas the thickness of the layer reduces from 3 metres to 2, 1 and 0.5 meter(s) (Figure 1). In reality, the shape would show a more continuous decrease of thickness. For the sake of a simple finite-element-mesh this type of modelling was chosen to allow for more complexity in the area of the rigid inclusions.
A smooth shape was modelled in the beginning (first trials) by using 18 "instant" boreholes, but the entire mesh with the rigid inclusions (volume elements) resulted in a six-digit number of finite-elements and was therefore not followed.
The chosen model for calculation has dimensions of 60 by 60 metres in plan view and reaches to a maximum depth of 35 metres. The groundwater table was explored in a depth of approximately 1 metre below the to ground surface. For the 3D finite-element-analyses the Hardening Soil Model was used for modelling the deformation behavior of the soil layers.

The foundation slab, which has an irregular shape in plan view and the nearly 100 rigid inclusions directly below the slab were modelled as volumetric elements assuming linear elastic behavior. The rigid inclusions were modelled according to an almost regular layout from the bottom of the slab to the top surface of the deep gravel layer. The used finite-element-model consists of 63,750 15-noded elements (prisms and tetrahedrons) and involves approximately 170,000 nodes. Figure 2 gives an impression of the finite-element model incorporating the high number of rigid inclusions.

Loading situation

Based on a possible configuration of the construction the following loading situation was developed for the numerical analysis. Due to the fact that a large number of single loads with minor influence on the overall settlement behavior of the slab would be present in the adopted construction, a simplified pattern with respect to the major loads was incorporated. Therefore, the top surface of the foundation slab was divided into 5 zones for modelling uniform distributed loads in each of them.

The value for each zone represents the mean value of the corresponding loads. The centre zone contains the main part with a high-rise construction. In zone II additional point loads were considered, whereas lower loads are present in zones III, IV and V. Figure 3 shows a sketch of the modelled load pattern in plan view.

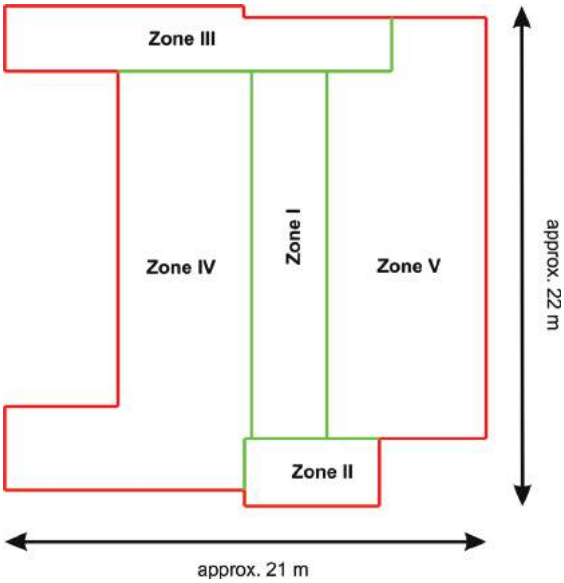

Figure 3: Sketch of the different loading zones

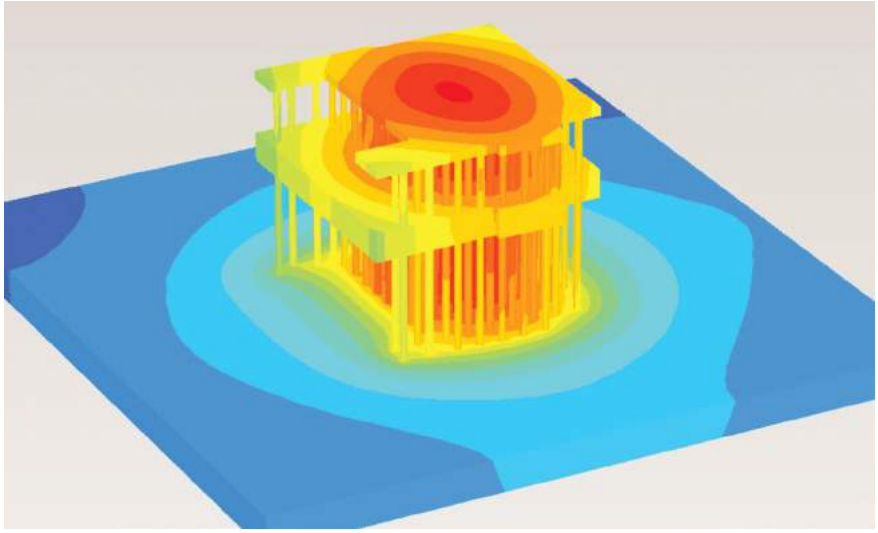

Figure 4: contour plot of settlements, partial geometry

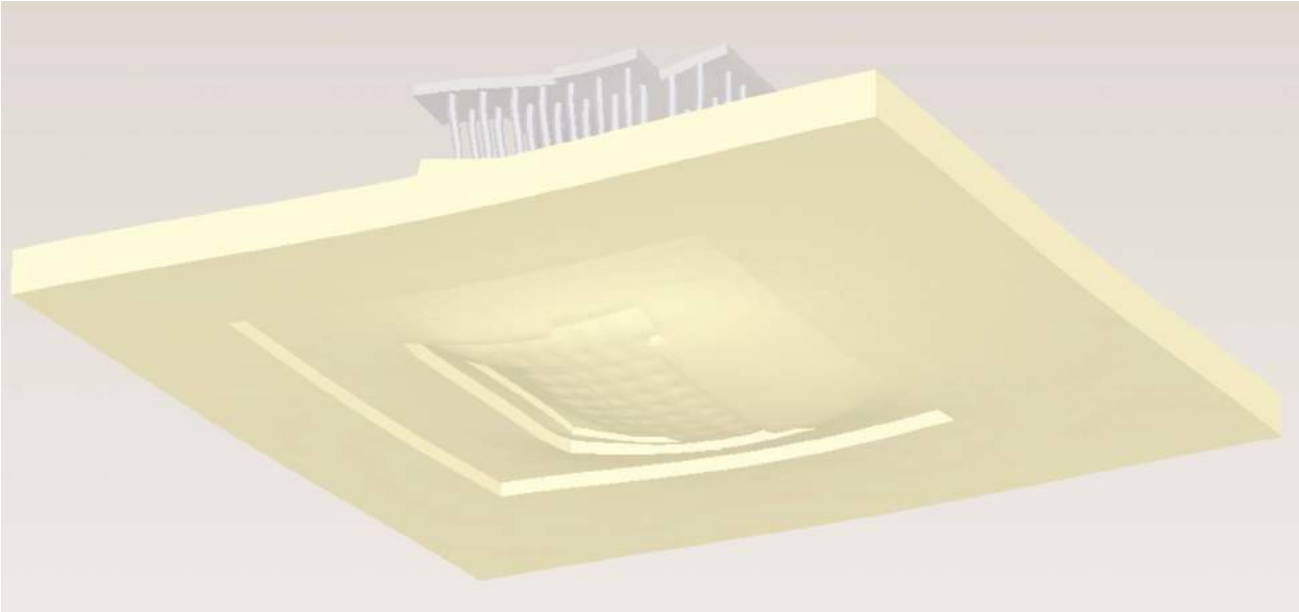

Figure 5: Deformation of deep sand layer, bottom view, partial geometry

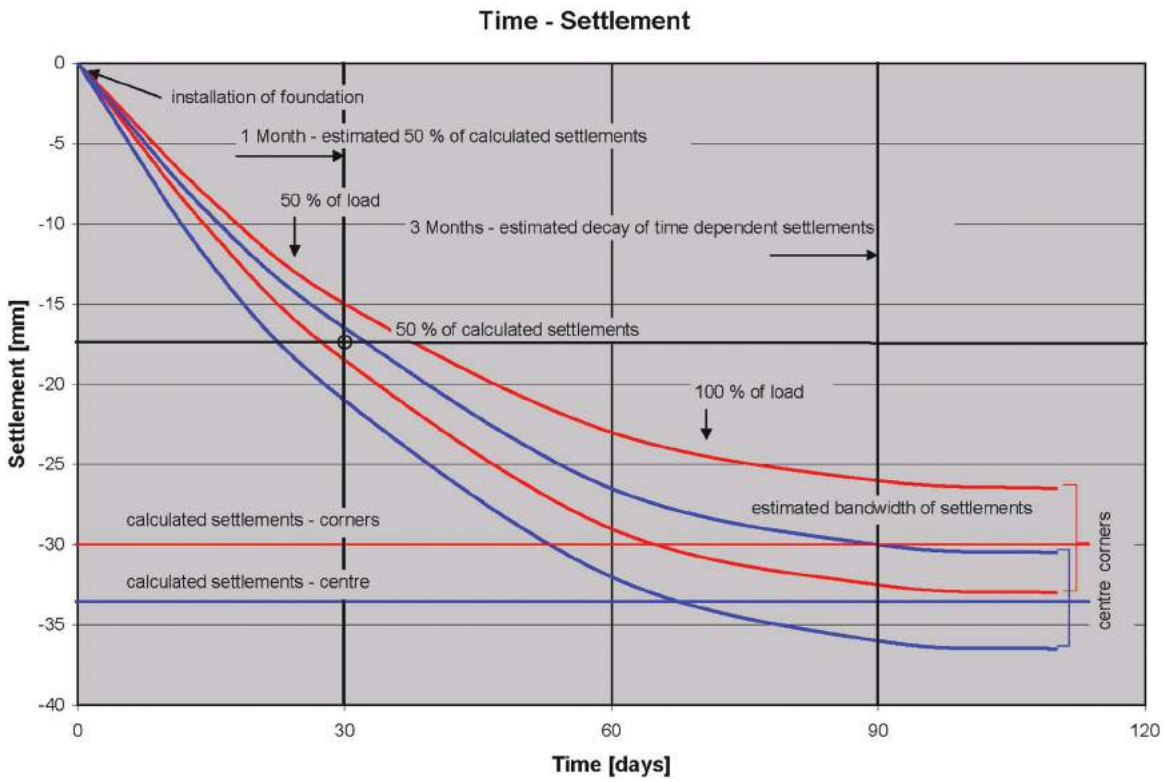

Figure 6: Estimated band width of calculated settlements with consolidation time

For the finite-element-analysis, the following uniform pressures have been considered:
Zone I: ca. 138 kN/m²
Zone II: ca. 37 kN/m²
Zone III: ca. 24 kN/m²
Zone IV: ca. 21 kN/m²
Zone V: ca. 22.5 kN/m²
Due to the fact that the concrete slab was modelled by using volumetric elements, the weight of the concrete slab with a thickness of 0.9 meters is not included in the given values. The additional weight due to the installation of the rigid inclusions was modelled by increasing the unit weight of the corresponding material data set.

Calculation procedure and results

As already mentioned, the aim of the calculations was an estimation of settlements and consolidation time. In order to fulfil both requirements in one analysis, undrained conditions and corresponding consolidation steps were defined.

Due to the size of the model, calculation time and stability were problematic and therefore, a different way of solving the problem was chosen. This means, that finally an analysis with drained conditions was performed with the presented model for an estimation of the settlements and another model using a less accurate discretisation of the rigid inclusions – blocks with mean material parameter sets – and undrained subsoil conditions was used for estimating the consolidation time.

For the analysis with the presented model, the following calculation steps have been defined:
Initial state (K0-procedure):
K_0 = 1-sin φ, deep sand layer with constant thickness of 3 metres

Modelling of the discontinuity of deep gravel layer:
Reduction of thickness by changing the corresponding material data sets.

Installation of rigid inclusions:
Effects due to the installation process have not been taken into account; the activation of the rigid inclusions happened by changing the material parameters (wished-in-place).

Installation of foundation slab:
Activation of the concrete slab with a thickness of 0.9 metres.

Loading situation:
The load distribution presented before was activated in two calculation phases (50% and 100%).

As already mentioned, the main focus in this drained analysis was the determination of the prospective settlements in the final state (after decisive consolidation phase). Due to the calculation results, the maximum settlement in the centre of the foundation slab is approximately 33 mm, whereas the settlements along the boundary of the slab are in the range of 24 to 30 mm. Differential settlements in the centre part are in the range of 2 to 3 mm.

Figure 4 shows a contour plot of the settlements in the area of the foundation slab and the rigid inclusions. Looking at the columns closely, it can be seen that the compression of the rigid inclusions is very small and that local dents in the deep sandy layer are visible (Fig. 5).

However, failure by punching of the columns in the partly very thin gravel layer below the toes of the rigid inclusions is not indicated by the analysis (plastic points plot, stress distributions – not shown here).

Regarding the consolidation time, the simplified model (no detailed discretisation of columns) resulted in approximately 90 days (3 months) to reach 95% of the final settlements. Approximately 50% of the estimated settlements were predicted after approximately 30 days (1 month).

In order to define the estimated development of settlements with time and to form a basis for comparison with monitoring data, the following diagram was derived from the calculation results. Figure 6 shows the results of the drained and undrained analysis and gives a band width for the estimated settlements in the centre of the slab and points along the boundary, respectively.

Conclusions

The presented boundary value problem could be solved by using the finite-element-method incorporating 3DFoundation. Despite of the problems with too large systems (too many elements) for undrained analysis, the presented boundary value problem could be solved and an estimation of final settlements and consolidation time could be derived. With the presented diagram in Figure 6, the basics for comparison of calculation results and monitoring data were prepared.

PLAXIS analysis of a basement excavation in central London

John Rigby-Jones, RJM Ground Solutions Limited, Chris Milne RJM Ground Solutions Limited

Excavation of a new 6 m deep basement was required as part of the development of a city centre site in London. The proposed excavation was within a very congested former car park area and would take place immediately adjacent to a row of 5 storey Georgian town houses, many of which were founded on shallow strip footings. In order to demonstrate that the proposed method of basement excavation would have a negligible effect on the existing properties a detailed Finite Element (FE) analysis of the various stages of basement construction was undertaken. This allowed action levels to be set for monitoring and provided the necessary assurance to the owners of adjacent properties.

Construction in city centre locations brings with it a host of additional challenges for the geotechnical engineer. Sites are typically compact with little space for operating plant or storing materials and surrounding roads are often congested. In order to maximise developable space new structures often extend to the edge of the development site and in close proximity to existing foundations and services for which little information is often available. New development must not only avoid these existing structures but must also limit ground movements to acceptable levels to prevent damage.

These various constraints to development are nowhere more prevalent than in central London. When a developer decided to construct a new property with a deep basement on a congested site in Victoria all of the above constraints were present (Figures 1 & 2). The site was located to the rear of Georgian properties dating from the early part of the 19th Century. The masonry and brick construction of these properties combined with their generally shallow foundations made them particularly sensitive to ground movement and so it was necessary to develop a robust methodology for the construction of the new basement.

In order to validate the proposed construction methodology FE modelling was undertaken using PLAXIS 2D V9.01 for each construction stage to confirm foundation movements of existing properties would be acceptable and to allow action levels for monitoring to be set.

Construction methodology

In order to minimise ground movements adjacent to deep excavations at sensitive sites it is common to utilise a top down construction process which provides a high level of wall restraint during excavation. Prior to excavation the perimeter wall is constructed and then progressively propped by construction of the permanent internal slabs as excavation progresses. This technique was proposed at the Victoria site using a perimeter contiguous piled wall composed of 450 mm diameter piles at 600 mm centres extending to a depth of 7 m providing a minimum toe beneath the deepest section of excavation of 1.05 m (Figures 3 & 4).

The proposed construction sequence comprised the installation of permanent slabs at three levels; ground floor, basement floor and swimming pool floor, following the sequence below;

1. underpinning of the existing 3rd party boundary wall (Figure 5)
2. installation of permanent and temporary piles
3. excavation to1m below finished ground floor slab level
4. construction of the ground floor slab
5. excavation to 0.2m below the lower ground floor slab level
6. construction of the lower ground floor slab
7. excavation to 0.2m below the swimming pool floor slab (maximum excavation depth of 6.25m)
8. construction of the swimming pool floor slab

Figure 1: Congested central London site

Figure 2: Basement excavation was to take place adjacent to existing Georgian properties

Ground conditions

Available site specific ground investigation indicated the site to be underlain by clayey sand Made Ground overlying the Kempton Park Terrace Gravels (present as a gravelly sand). London Clay was present just below the proposed pile toe level. Groundwater was recorded at the base of the Terrace Gravels. The soil profile is summarized below in Table 1. A hydrostatic ground water distribution was assumed beneath a level of -1.3 mAOD (below base of excavation at 0.25 mAOD) based on the results of long term monitoring in a number of ground water monitoring instruments at the site. In situ testing was limited to the undertaking of Standard Penetration Tests (SPTs) in boreholes with no direct measurement of soil stiffness undertaken. The shear strength properties of soils were estimated from relationships with design SPT profiles, however this crude process was not considered appropriate for the estimation of appropriate soil stiffness parameters. Initial large strain stiffnesses associated with first time loading were estimated using the empirical relationships provided in CIRIA report R143 (ref 1) for both worst credible (WC) and most probable (MP) values of SPT N. These values were then adjusted for the increased

Name	Depth to base of stratum (mbgl)	Level of base of stratum (mAOD)
Fill	2	4.2
Terrace Gravels	8	-1.8
London Clay (assumed in model)	16.2	-10

Ground level = 6.2 mAOD Ground water level = -1.3 mAOD

Table 1: Design soil profile

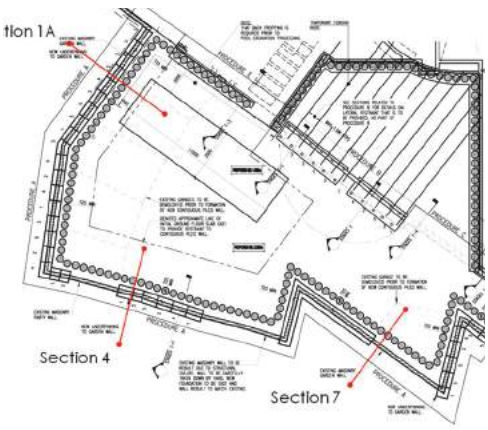

Figure 3: Site layout and cross sections analysed

Parameter	Symbol	Fill	Terrace Gravels	London Clay	Unit
Material Model	Model	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	-
Type of behaviour	-	Drained	Drained	Drained*	-
Coefficient of at rest earth pressure	K_0	0.45	0.37	1.5	-
Unsaturated weight	$\gamma unsat$	18	21	19.8	kN/m^3
Saturated weight	γsat	20.5	23	19.8	kN/m^3
Permeability x direction	k_x	0	0	0	m/d
Permeability y direction	k_y	0	0	0	m/d
Stiffness (moderate strain unloading)	E	WC=8,000 MP=18,000	WC=75,000 MP=140,000	WC=50,000 MP=100,000	kN/m^2
Poisson's ratio (unloading)	υ	0.15	0.15	0.495	-
Cohesion	c	0.2#	0.2#	100	kN/m^2
Friction angle	ϕ	33	39	0	°
Dilatancy angle	ψ	3	9	0	°
Interface strength reduction	R_{inter}	0.8	0.8	0.7	-

* undrained behaviour of the London Clay to represent ground response to short term excavation has been modelled using an undrained total stress analysis (ref. Plaxis 2D materials model manual Section 2.7) with the material behaviour set to drained in conjunction with undrained soil parameters
# a small cohesion applied to avoid complications in obtaining a numerical solution
WC = worst credible
MP=most probable

Table 2: Soil parameters used for analysis

stiffness associated with unloading behavior and reduced strain level (0.01-0.1%) following published guidance (refs 2 & 3) to provide input values for analysis. The proposed soil stiffness values were compared to published values for similar soils at nearby sites in order to confirm their appropriateness. The input values adopted for analysis are presented below in Table 2.

Analysis

Analysis was undertaken using PLAXIS 2D V9.01, modelling soil behavior using the Mohr-Coulomb failure criterion with granular fill and Terrace Gravels considered as drained and the underlying London Clay as undrained (as only the short term displacements during construction were to be determined). Analysis was undertaken as a series of drained construction stages and as such no consolidation analysis was required to calculate intermediate pore water pressures.

The strip foundations of existing properties were modelled as fully flexible to prevent soil from "hanging up" and giving a false reduction in true settlement. Piles were modelled as plate elements and floor slabs as fixed end anchors.

Loadings from 3rd party properties were applied as a combination of floor UDL and wall line load. The problem geometry was meshed using 15 noded triangular elements with local mesh refinement to the piled wall (Figure 6).
Three critical cross sections (Figure 3) through the proposed basement excavation were analyzed using both worst credible (WC) and most probable (MP) soil stiffnesses in order to determine a range of likely settlement magnitudes. Ground movements due to the application of the building loads associated with existing adjacent properties were removed from the output due to the age of the buildings indicating that this settlement would now be complete.

The deformed mesh is shown in Figure 7 with vertical settlements shown in Figure 8 and horizontal displacements in Figure 9. The results of analysis are summarized below in Table 3.

Vertical displacements (mm)	section 1A		Section 4		Section 7	
Location	WC	MP	WC	MP	WC	MP
Underside of underpinned party wall	-3.8	-3.4	-1.8	-1.4	-1.9	-1.4
2m from excavation	-3.5	-2.9	-1.5	-1.1	-2.5	-1.6
5m from excavation	-2.6	-1.4	-1.2	-0.6	-1.6	-0.8
centre of excavation	9.5	4.9	8.5	4.4	8.4	4.4

Vertical displacements (mm)	section 1A		Section 4		Section 7	
Location	WC	MP	WC	MP	WC	MP
Pile head (positive = into excavation)	1.4	1.2	0.4	0.36	0.5	0.4
Mid prop (positive = into excavation)	4.2	3.25	1.8	1.3	1.4	1.3

Table 3: Summary of ground movements

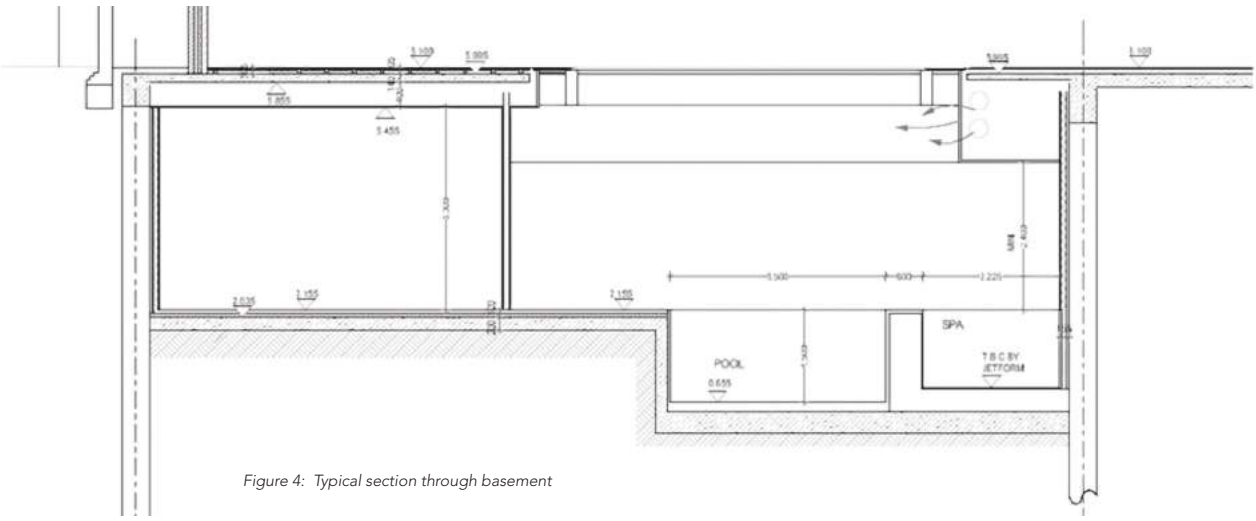

Figure 4: Typical section through basement

Figure 5: Underpinning of existing foundations

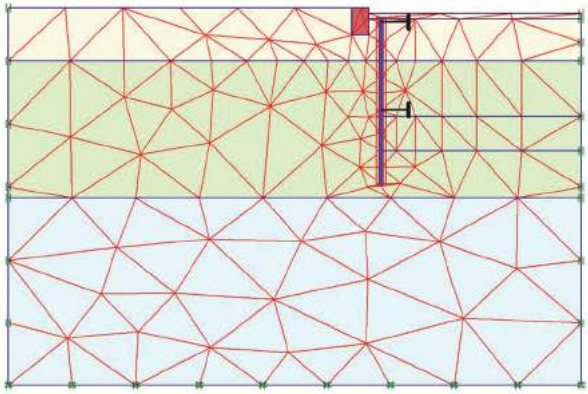

Figure 6: FE mesh

Discussion

There are two subtle features of the FE modelling that are worthy of note. Firstly, the pre-loading effect of imposing the adjacent building loads on the sub-soil results in very slightly reduced settlements beneath the adjacent buildings when compared to the ground immediately beyond them.

Secondly, the settlements immediately behind the wall are slightly reduced due to the combined effect of a slight unloading due to the excavation of ground between the piled wall and the 3rd party wall and the effect of the interface elements modelling the soil structure interaction between the piles and the adjacent soil locally increasing vertical stiffness.

The results of the analyses carried out indicated that in all cases ground settlements adjacent to the basement excavation were less than 5 mm. This confirmation that anticipated settlements were to be very small provided reassurance to the adjacent homeowners. Monitoring of structural movements was undertaken with action levels of 3 mm and 5 mm set corresponding to increased frequency of monitoring and halt of excavation works respectively.

Conclusions

PLAXIS analysis provided the necessary confidence to undertake deep excavation close to sensitive buildings in a city centre location and set monitoring action levels. Following the successful construction of the deep basement at the study site work has commenced nearby on the new North Ticket Hall being constructed as part of improvements to the London Underground Victoria station.

This excavation is also to be constructed using a top down technique, however due to the greater depth and penetration beneath the ground water table a secant wall is to be constructed which will toe into the low permeability underlying London Clay.

The Company

RJM Ground Solutions is a small geotechnical consultancy specializing in high quality geotechnical advice and designs servicing a diverse client base across the UK. www.rjm-ground.co.uk.

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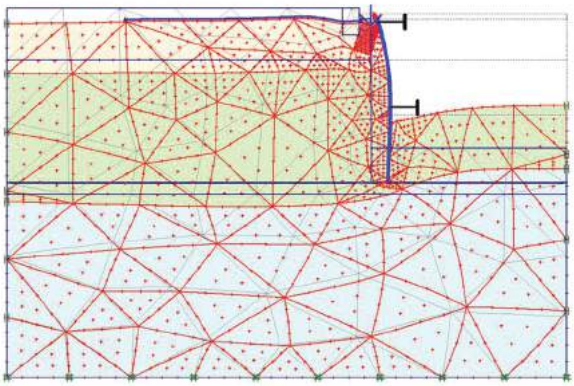

Figure 7: Deformed mesh at section 1A Stage 7

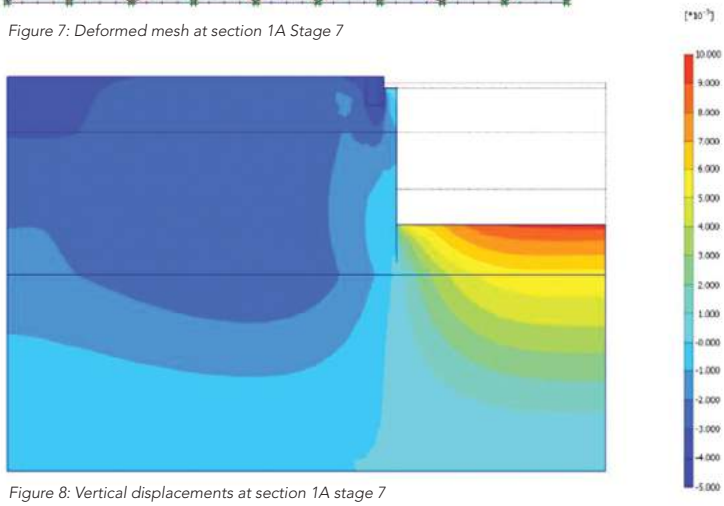

Figure 8: Vertical displacements at section 1A stage 7

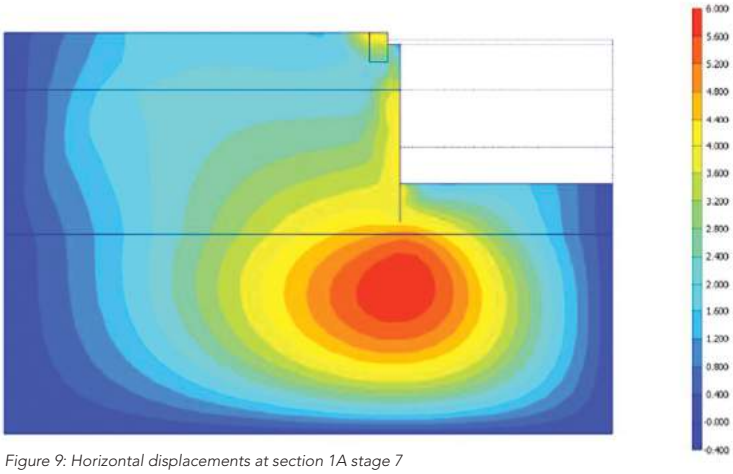

Figure 9: Horizontal displacements at section 1A stage 7

Recent activities

Filling in the survey is worth safe drinking water

After the successful introduction of PLAXIS 3D last summer we have also released PLAXIS 2D 2010 at the Plaxis User Meeting in Karlsruhe on November 11, 2011. The new PLAXIS 2D 2010 is equipped with a new fully coupled flow deformation kernel. PlaxFlow users will have access to a more advanced mode of a transient groundwater flow deformation analysis with additional features for suction, retention curves and unsaturated soil modelling. PLAXIS VIP members have access to the renewed material database (including Hoek-Brown and NGI-ADP) and they can use the possibility to import CAD-files. In the Dynamic Module independent horizontal and vertical acceleration components for earthquake loading can be applied. Also eigen frequencies of soil-structure systems can be analyzed via a free vibration implementation. The output program is one program, used for both PLAXIS 3D and PLAXIS 2D, which has new or improved features in Report Generator, Animations, Cross sections and many more. In Figure 1 you see a new possibility of the output program.

At the moment we are already working on new developments for both programs. For this purpose, your opinion is very important and we are curious about your feedback on our products and services. Therefore, we will ask you to fill out our customer satisfaction survey. The outcome of the questionnaire will help us to improve our products and services according to your wishes. In return for every completed and returned questionnaire Plaxis will support safe drinking water for one child in Bangladesh. The chosen charity project is part of the 'Disaster Friendly Water & Sanitation' program in Bangladesh. In this project schools are helped to provide safe drinking water and good sanitation services. So every filled in survey gives one child the opportunity to receive safe drinking water at school. The program is not only for providing safe drinking water, but also to protect drinking water and sanitation structures against flooding. This theme is close to our own history, developments to support the construction of the

One Survey One Child Safe Drinking Water

Oosterschelde barrier, which was built to protect people against flooding. The schools of this project are located in the Ganges-Brahmaputra Delta in Bangladesh. This delta covers a large area of Bangladesh and monsoon floods and cyclones are a constant threat for this delta. The enormous and severe floods are called 'bonnas' and have disastrous consequences.

In 1999 a 'bonna' claimed more than a thousand lives, one million people lost their homes, a large part of the harvest was lost and thousands of kilometres of roads and 4,500 km of dikes disappeared under water. Sanitary services and safe drinking water are destroyed and water reserves become polluted by these 'bonnas'. As a result, the population is exposed to unnecessarily high health risks during periods of flooding. The goal of this project is to build drinking water facilities and latrines for 100 schools, which stands for 30,000 students. During times of disaster the total community uses these facilities, because the schools are used as a shelter during evacuations, which makes the capacity of this project for more than 200,000 people. In order to ensure sustainability, the local communities are trained in subjects of water, sanitation and hygiene.

The structures, which protect the safe drinking water access and sanitation from flooding, are simple. In the past two years several structures have been realised. In figure 2 you see that the pump cannot be swept away or stand below the water level.

We look forward to receive your input via this survey so that we can fulfill your wishes and we also support this fantastic charity project.

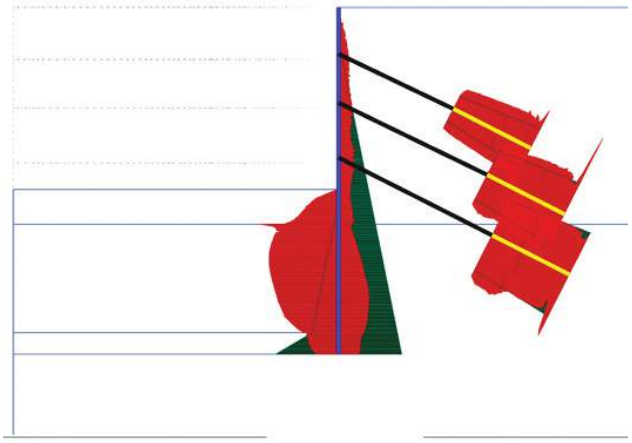

Figure 1: Force view of a sheet pile with three anchors

Figure 2: The concrete levelled water pump from the TW Platform at Modhyo Para in the district Bhola

Cordaid
mensen in nood

Plaxis researcher wins prestigious award

Dr. Vahid Galavi, researcher at Plaxis bv, has won the prestigious George Stephenson Medal, issued by the Institution of Civil Engineers in London for his paper entitled "A multilaminate framework for modelling induced and inherent anisotropy of soils".

The research work, which resulted in the prizewinning paper, has been performed for his PhD project at the Technical University of Graz under supervision of Prof. Helmut Schweiger. Meanwhile, Vahid has worked as a researcher at Plaxis on various interesting projects, among which groundwater flow, fully coupled flow-deformation analysis, unsaturated soil behaviour and dynamic analysis.

The prize was handed over to Vahid in a special ceremony at the Institution of Civil Engineers in London on Friday 8th October 2010.

We like to congratulate Vahid with this achievement and we are happy that he has chosen to continue his excellent research work as part of the Plaxis team.

PLAXIS

essential for geotechnical professionals

Activities 2010

December 1 – 3, 2010
Standard Course on Computational Geotechnics
Newcastle, Australia

December 6 – 7, 2010
Short Advanced Course on Computational Geotechnics
Perth, Australia

Activities 2011

January 10 – 13, 2011
5th IC EGE
Santiago, Chile

January 12 – 15, 2011
Standard course on Computational Geotechnics and Dynamics
New York, Canada

January 23 – 27, 2011
Transportation Research Board
90th annual meeting
Washington DC, USA

January 24 – 27, 2011
Standard course on Computational Geotechnics
Schiphol, The Netherlands

February 9 – 11, 2011
Curso de Geotecnia Computacional
Barcelona, Spain

February 14 – 16, 2011
Finite Elemente in der Geotechnik
Stuttgart-Ostfildern, Germany

March 1 – 4, 2011
Standard course on Computational Geotechnics
Bogotá, Colombia

March 13 – 16, 2011
Geo-Frontiers 2011
Dallas Texas, USA

March 21 – 24, 2011
Advanced course on Computational Geotechnics
Schiphol, The Netherlands

April 4 – 7, 2011
Standard Course on Computational Geotechnics
São Paulo, Brazil

April 13 – 15, 2011
Standard Course on Computational Geotechnics
Genova, Italy

May 16 – 18, 2011
7th International Symposium Roma 2011
Rome, Italy

May 23 – 27, 2011
The 14th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering
Hong Kong, China

July 18 – 21, 2011
5th African Regional Conference on Soil Mechanics and Geotechnical Engineering (15th ARCSMGE)
Maputo, Mozambique

Plaxis bv
Delftechpark 53
2628 XJ Delft

P.O. Box 572
2600 AN Delft
The Netherlands

www.plaxis.nl
Tel +31 (0)15 2517 720
Fax +31 (0)15 2573 107

Plaxis Asia
Singapore
Tel +65 6325 4191

16 Jalan Kilang Timor
#05-08 Redhill Forum
159308 Singapore