PLAXIS FINITE ELEMENT CODE FOR SOIL AND ROCK ANALYSES

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Capacity Analysis of Suction Anchors in Clay by Plaxis 3D Foundation

On Stability Analysis of Slurry-Wall Trenches

Seabed instability and 3D FE jack-up soil-structure interaction analysis

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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Editorial

Ronald Brinkgreve

A new Plaxis Bulletin is in front of you, with interesting articles about Plaxis applications, new developments and a full agenda of activities. We are also pleased to announce that new Plaxis products have been released, which are the 2D version 9.0, 3D Foundation 2.2 and Plaxis-GiD. The latter is a general CAD-like 3D pre-processor which has been configured to address the Plaxis calculation kernel.

Regarding 'Plaxis Practice', it is interesting to see that all three articles involve 3D calculations. Hence, there is a clear trend to perform 3D calculations, at least for complex geo-engineering projects. At the same time, the use of 2D FEM is still increasing, since Plaxis 2D is used more and more for daily geotechnical design.

The first article describes an evaluation of suction anchor bearing capacity with the 3D Foundation program. In general, 3D models are not as accurate as 2D models. As a result, ultimate limit states (such as safety factors or bearing capacities) may be overestimated. It is demonstrated that interface elements play a crucial role in the accurate prediction of the suction anchor bearing capacity.

The second article involves the stability analysis of slurry wall trenches. The situation is during construction is clearly three-dimensional. In addition to safety factor analysis, the authors describe a probabilistic design method. The results seem to be in good agreement with the geo-engineering practice.

In the third article a 3D analysis of a complex offshore foundation is described. Instability of the seabed is an important issue here. Gravel banks were proposed to stabilize the foundation. In the calculation different loading situations were considered. A good and stable solution could be obtained for the designed foundation. The structure has been build successfully and behaved well according to the predictions.

Hereby we trust to have compiled again an interesting Plaxis bulletin for you. Do not hesitate to contact us with your response on one of the published articles, or with new articles for future Bulletins. We are looking forward to receive many contributions.

The Editors

New Developments

Ronald Brinkgreve

In the previous Bulletin information was given about the Plaxis 3D developments. It is a pleasure to mention now that the first general 3D program (Plaxis-GiD) has been released. This program is available as a service to those who feel restricted by the geometrical limitations of 3D Tunnel or 3D Foundation. More information can be obtained from the Plaxis sales department.

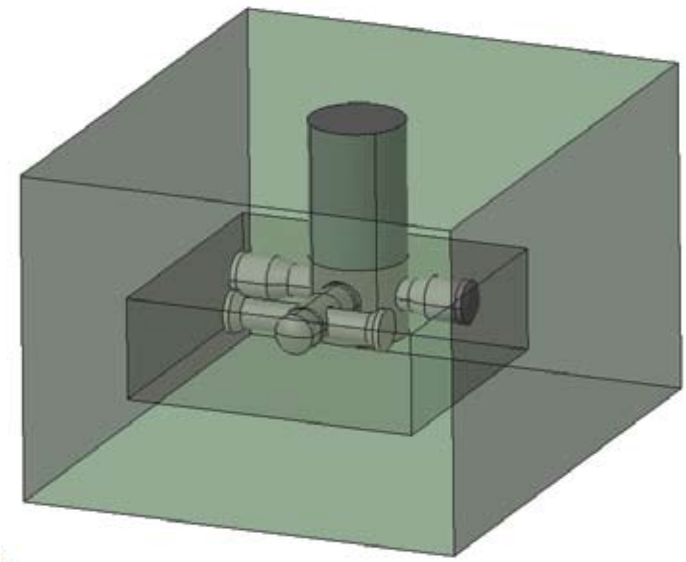
In this Bulletin I like to mention another new development that is currently in progress: Fully coupled flow-deformation analysis.

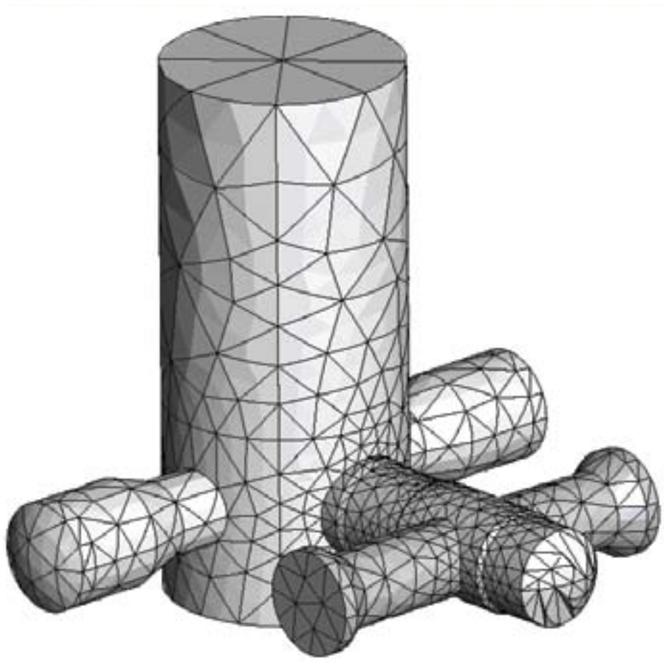
Most Plaxis users are familiar with the Consolidation option in Plaxis 2D and 3D. So far, Plaxis has only considered Biot coupled consolidation under saturated conditions, forming a coupling between deformation and excess pore pressures. This works well for cases with constant hydraulic conditions, where the time interval or loading rate is such that the situation is neither fully drained nor fully undrained. For cases with changing hydraulic conditions, a simplified solution is available by combining the standard Plaxis program with the transient flow module PlaxFlow. However, if pore pressure is influenced by loading of (partially) undrained soil as well as changing hydraulic conditions, there is a need for consolidation based on total pore pressures, i.e. fully coupled flow-deformation analysis. Examples where this type of analyses is required are clay embankments in tidal areas or excavations with dewatering in medium soft soils.

With the change of consolidation based on excess pore pressure to total pore pressure it becomes important to consider the phreatic surface and the unsaturated zone above. As a result of loading or changing hydraulic conditions, ground water flow may occur, and the position of the phreatic surface may change. Soil that has been fully saturated may become unsaturated or vice versa. Hence, together with the implementation of fully coupled flow-deformation analysis, there is also need for models that can describe unsaturated soil behaviour in more detail. First of all, there are the Van Genuchten relationships between suction, relative permeability and degree of saturation, which are also used in PlaxFlow. Secondly, there is the well-known Barcelona Basic Model that deals with suction and swelling in the unsaturated zone. All this is implemented in the Plaxis calculation kernel to complete the fully coupled flow-deformation analysis feature.

These new features will be available in Plaxis 2D version 9.1, which is planned for release mid 2009. When the implementation is ready, we can start beta-testing with a selected group of users. After implementation in Plaxis 2D we will proceed with the implementation in the 3D calculation kernel. We are confident that the new features will help many users in analysing their coupled and unsaturated soil problems.

Ronald Brinkgreve
Plaxis bv





Capacity Analysis of Suction Anchors in Clay by Plaxis 3D Foundation

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Introduction

This article describes the use of Plaxis 3D Foundation v. 2.1 (Plaxis, 2008) to compute the undrained capacity of a suction anchor in clay. The objective of this study was to evaluate the performance of Plaxis 3D Foundation for analyzing this particular problem by comparing the Plaxis 3D Foundation results with results from other software including Plaxis 2D and NGI in-house codes. The effects of mesh fineness, use of interface elements and the wall roughness on the calculated capacity were also studied. There are several other aspects in the design of skirted anchors in clay which is not covered in this article. The reader is referred to Andersen and Jostad (1999). A particular issue that this study focused on was use of interface elements adjacent to cylindrical suction anchors. The lack of isoparametric interface elements in the 2.1 version of Plaxis 3D Foundation is known to introduce some error to problems where curved soil-structure interfaces are defined by the volume pile generator. This issue is described in the "Known issue Plaxis 3D Foundation version 2.1" (www.plaxis.nl).

Description of the Problem Considered

Figure 1 illustrates the cylindrical suction anchor analyzed in this study. It is one of the four hypothetical capacity cases presented by Andersen et al (2005) in an industry sponsored study on the design and analyses of suction anchors in soft clays. The anchor was assumed to have a closed top, no tension crack on the active (windward) side and to be very stiff compared to the soil. The load was attached at the optimal load attachment point at depth z_p to produce a failure corresponding to pure translation, i.e. maximum capacity is obtained when there is no rotation of the anchor.

The soil was assumed to be a normally consolidated clay with an average undrained strength increasing linearly with depth as follows:

$$s_u \text{ (kPa)} = 1.25 \cdot z \text{ (m)}$$

A strength intercept at the surface of 0.1 kPa was used. The soil was modeled as an undrained, cohesive linear elastic- perfectly plastic (Tresca) material. In Plaxis, we used the Mohr-Coulomb strength model with the friction and dilatancy angles equal to zero ($\phi = \psi = 0$), cohesion equal to the undrained strength ($c = s_u$), and no tensile cut-off strength.

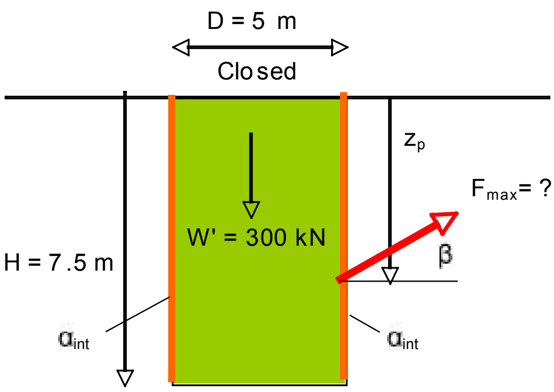

Figure 1: Description of the Suction Anchor Problem

The anchor was modeled by linear elastic wall elements with a high stiffness making them virtually rigid. Because the governing failure mechanisms do not involve the soil plug inside the anchor, this soil plug was modeled as a stiff, elastic material. For all the FE-models in this study we have used interface elements along the outside skirt walls. These elements are used to improve the results by allowing for slip between the anchor wall and the soil, and to model a possibly reduced strength $s_{u,int} = \alpha_{int} \cdot s_u$ along the outside skirt walls to account for reduced soil strength due to effects of the anchor installation. Recommended values of α_{int} for design situations are given in Andersen and Jostad (2002) and results from centrifuge testing are presented in Chen and Randolph (2006).

Plane Strain Analyses

The suction anchor on Figure 1 was first analyzed as a plane strain problem using both Plaxis 2D and Plaxis 3D Foundation. The objective was to compare results from Plaxis 3D Foundation with the well established 2D code and to the readily available hand calculated capacity. An extensive study of the discretization error was also performed. Computations were made with both the 6- and 15-noded elements available in Plaxis 2D.

Horizontal interface elements were used along the soil-soil contact underneath the anchor tip in addition to along the outside skirt wall. The vertical and horizontal interfaces were extended 0.2·D outside the anchor. This was to allow possibly full slip around the bottom corners of the anchor. A wall interface factor α_{int} of 0.65 was used along the outside skirt while full interface strength ($\alpha_{int} = 1.0$) was used under the anchor tip and for the interface extensions. The load was applied horizontally at a depth (z_p) of 5 m. The in-plane width D of the anchor was 5 m.

Figure 2 presents the deformed mesh (displacements scaled up 5 times) at the end of one analysis i.e. at ultimate capacity, from a Plaxis 2D plane strain computation. A well defined failure surface forms on both the active and passive sides and the suction anchor translates horizontally.

This mesh with approximately ~5000 15-noded elements (~40 000 nodes) illustrates the degree of mesh refinement necessary for accurate computations although many fewer elements could have been used within the suction anchor. The effect of mesh fineness and element type on the computed suction anchor capacity is further illustrated by Figure 3. More than 40 000 nodes are required for convergence to a capacity of 228 kN/m. However, a mesh with only about 10 000 nodes (15-noded elements) produces an ultimate capacity of 230 kN/m, only 1 % higher than the more accurate value. The discretization error increases dramatically for meshes with less than 5000 nodes (2500 elements). Figure 3 also illustrates that the 6-noded elements produced suction anchor capacities very close to those with the 15-noded elements provided the mesh is refined to have approximately the same number of nodes.

Capacity Analysis of Suction Anchors in Clay by Plaxis 3D Foundation

Continuation

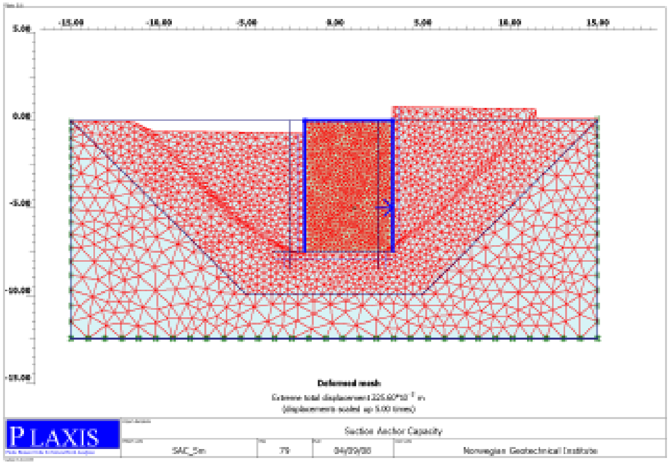

Figure 2: Plaxis 2D Plane Strain Deformed Mesh at the End of the Analysis (α_{int} = 0.65)

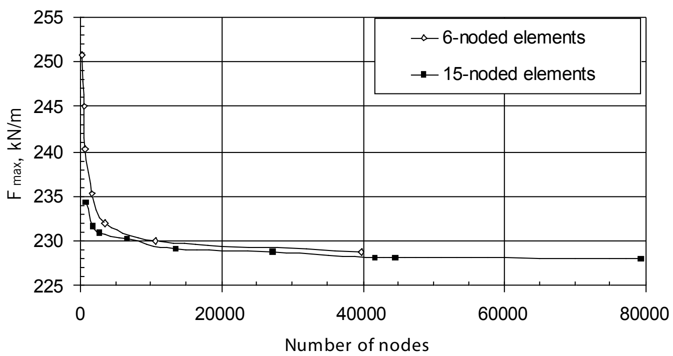

Figure 3: The Effects of Mesh Fineness and Element Type on Computed Suction Anchor Capacity – Plaxis 2D Plane Strain Analyses

The next series of computations utilized Plaxis 3D Foundation to analyze the plane strain problem discussed above as a first step in comparing its performance with Plaxis 2D. Only one element was used in the out-of-plane direction. This was obtained by using a small thickness of 0.25 m in that direction. The 3D mesh has vertical interfaces along the outside walls with extensions underneath the anchor tip but no horizontal interfaces at the anchor tip level.

Interface extension can be provided by deactivated wall extension. Figure 4 shows a deformed mesh (displacements scaled up 5 times) at the end of the analysis i.e. at ultimate capacity from a Plaxis 3D Foundation plane strain computation. A well defined failure surface, similar to the failure surface in Figure 2 for the 2D run, forms and the suction anchor translates horizontally. The mesh shown has ~6700 15-noded wedge elements (~28 000 nodes) and provides a capacity of 233 kN/m for α_{int} = 0.65. Increasing the number of nodes to 80 000 gave nearly the same capacity, while decreasing the number of nodes to less than 10 000 dramatically increased capacity and thus the discretization error. The results from the mesh sensitivity study are shown in Figure 5. As for the 2D calculation the failure mechanism involves a cut-off (thin shear band) at the anchor tip level. It is therefore important to use a thin row of elements at this level to avoid an artificially deeper failure mechanism. This can be enforced by using additional work planes at this depth.

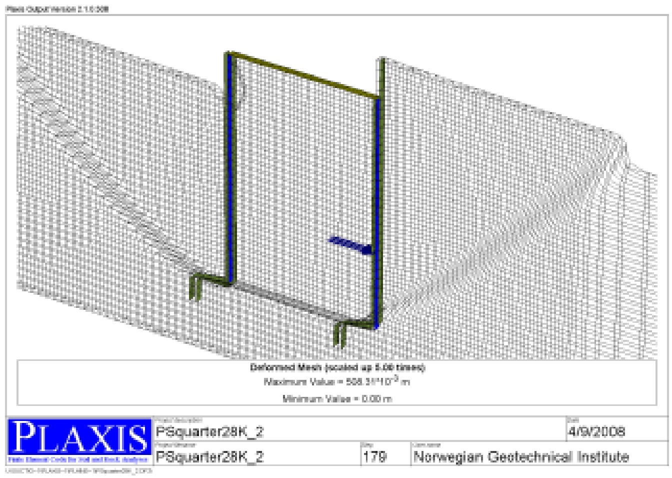

Figure 4: Plaxis 3D Foundation Plane Strain Deformed Mesh at the End of the Analysis (α_{int} = 0.65)

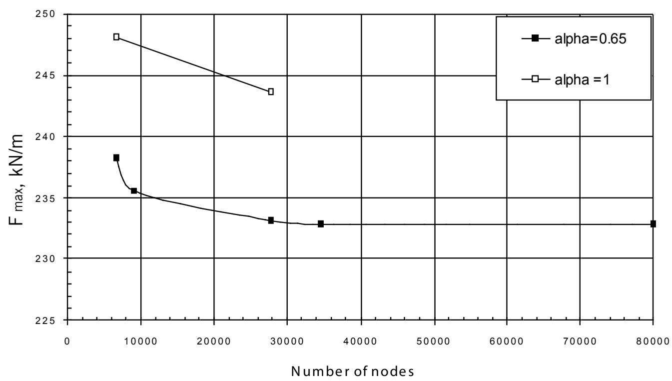

Figure 5: The Effects of Mesh Fineness on Plane Strain Suction Anchor Capacity – Plaxis 3D Foundation

Discussion of the Plane Strain Analyses

Table 1 compares the plane strain suction anchor capacities computed by Plaxis 2D and 3D as well as the capacities estimated by a hand-calculation based on classical earth pressure theory. The capacities of Table 1 are all for the runs where the discretization error is negligible (> 30 000 nodes) and are all in reasonable agreement. The hand-calculation may have some small error because the earth pressure coefficient used is developed for a constant strength profile while the case studied has a linearly increasing strength.

The Plaxis 3D Foundation capacities are about 2 % higher than the Plaxis 2D capacities, probably because of the lack of horizontal interface elements at the bottom of the suction anchor or because of the different element type. The higher wall interface factor (α_{int} = 1.0) increases the capacities by about 5%.

	α_{int} = 0.65	α_{int} = 1.0
Hand calculation	224	232
Plaxis 2D	228	239
Plaxis 3D Foundation	233	244

Table 1: Horizontal Plane Strain Suction Anchor Capacities (kN/m)

Three Dimensional Analyses

Plaxis 3D Foundation was then used to analyze a 5 m diameter cylindrical suction anchor. Only half of the problem was represented in the FE model because of symmetry about the vertical plane in the direction of loading. This feature was important in creating a fine mesh and in reducing computation time. The half cylinder was generated with the volume pile generator. Three rows of elements with thickness 0.1 m were generated beneath the anchor tip by using additional working planes. The mesh refinement studies with strategic refinement led to a mesh of ~26 600 elements and ~76 000 nodes. By plotting the capacity versus the number of nodes as for the 2D calculations it was found that the capacity nearly had converged to a constant value for a mesh with about 76 000 nodes, i.e. this mesh gave only a small discretization error. The load was applied at the optimal load attachment point which was found to be at a depth of approximately 5 m.

As discussed in the “Known issues” section of Plaxis 3D Foundation 2.1, when using the Pile Designer to generate circular piles, the resulting elements (volume elements, plate elements and interface elements) are not curved (isoparametric), but they have straight sides. The ultimate capacity may then be overestimated due to:

- Any given reduced (α_{int} < 1.0) interface shear strength is not taken into effect because horizontal slip in the soil-structure contact is prevented.

- The earliest possibility to yield is in the stress points of the adjacent soil volume elements outside the pile, which increases the effective pile diameter.

Therefore, full roughness (α_{int} = 1.0) was used along the outside skirt walls and a fine discretization was used along the perimeter of the cylinder to reduce the “effective” pile diameter. Figure 6 illustrates the geometry that was used for these analyses and the deformed mesh from one of the computations. The computed ultimate holding capacity for α_{int} = 1.0 was 1870 kN for pure horizontal loading.

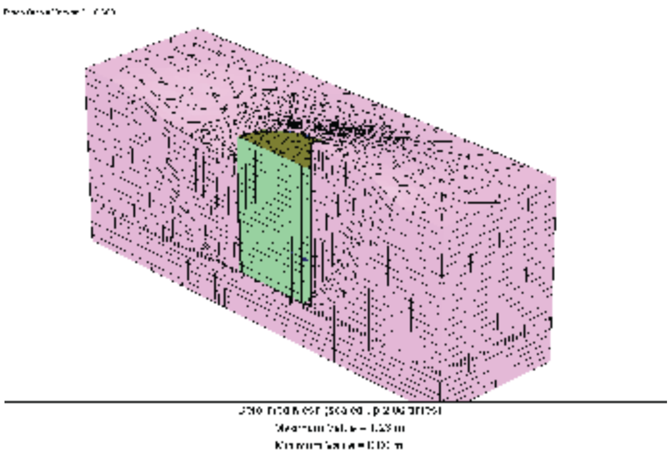

Figure 6: Plaxis 3D Foundation Geometry Model and Deformed Mesh at the End of the Analysis - 5 m diameter Suction Anchor

This computed capacity was compared with the capacity computed by HVMCap (NGI, 2000) and the NGI in-house program BIFURC 3D (NGI, 1999). BIFURC 3D is a general purpose FE program, while HVMCap is a specially made windows program for design analyses of suction anchors, including the effects of reduced interface strength, anchor tilt, tension crack development at the active side, and shear strength anisotropy. HVMCap uses the BIFURC FE program as a calculation kernel. It is a plane model with the three dimensional effects modeled by displacement compatible shear stress factors (side shear) calibrated from full three dimensional finite element studies. The capacity computed by HVMCap for the same case as shown in Figure 1 with α_{int} = 1.0 was 1578 to 1775 kN depending upon the range of values (between 0.5 and 1.0) assumed for the three dimensional side shear factors. The capacity computed by BIFURC 3D was 1780 kN.

To avoid the issue with the non-isoparametric elements for the cylindrical anchor, capacities were calculated also for a rectangular anchor having a cross-sectional area equivalent to a 5m diameter circle (3.93 m x 5 m with the 5m width normal to the loading direction). This is believed to be a very good approximation to a cylindrical anchor. Vertical interfaces were used along the outside walls and extended horizontally as shown in Figure 7 to allow full slip around the anchor edges. Thin rows of elements were also used underneath the anchor tip. The computed ultimate holding capacities for α_{int} = 1.0 was 1895 kN for pure horizontal loading.

Capacity Analysis of Suction Anchors in Clay by Plaxis 3D Foundation

Continuation

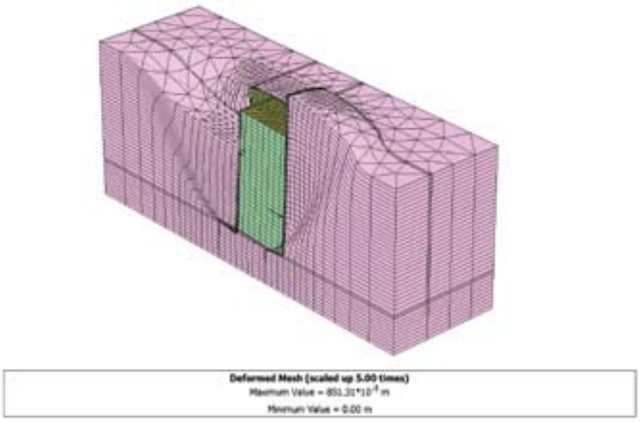

Figure 7: Plaxis 3D Foundation Geometry Model and Deformed Mesh at the End of the Analysis - Rectangular Suction Anchor

Discussion of Three Dimensional Analyses

Table 2 presents the suction anchor capacities computed by Plaxis 3D Foundation for the cylindrical and rectangular suction anchors and the capacities computed by HVMCap and BIFURC 3D. Results for wall interface factor α_{int} =0.65 and 1.0 are given, even if, as noted, it is known that for $\alpha < 1.0$ Plaxis 3D Foundation overestimates the capacity for the cylindrical anchor.

The Plaxis 3D Foundation capacity of 1870 kN for the 5 m diameter cylindrical anchor and 1895 kN for the area equivalent rectangular anchor, both with α_{int} = 1.0, seem reasonable. The minor difference between the rectangular and the circular cross section anchors indicate that the area equivalent rectangle is a good approximation. However, the BIFURC3D results of 1780 kN and the upper bound value of 1775 kN from HVMCap is 5 % less than the Plaxis 3D Foundation result of 1870 kN. As there is no reason to believe that the FEM produce capacities that are too low, this indicates that Plaxis 3D Foundation slightly overestimates the capacity.

Computation	α_{int} =0.65	α_{int} = 1.0
PLX 3DF Circle 5 m diameter	1820[(1)]	1870
NGI BIFURC3D FEM Circle 5 m diameter	1665	1780
PLX 3DF Eqv. area rectangle 5 m x 3.93 m	1715	1895
NGI HVMCap FEM "2D+Side shear"	1463-1723	1578-1775

[(1)]Capacity is too high because of non-isoparametric formulation

Table 2: Horizontal Suction Anchor Capacities (kN)

The recent update Plaxis 3DF version 2.2 includes curved interfaces.

Despite a thorough investigation of the Plaxis 3D Foundation results it has not been possible to identify with certainty what is the cause for the 5 % overshoot. It may be the lack of horizontal interfaces at the anchor tip level that prevents full slip underneath the skirts. For the cylindrical anchor the slightly increased "effective" radius, caused by the non-isoparametric interface elements may also contribute to a small overshoot, although a very fine mesh was used outside the skirt wall.

The Plaxis 3D Foundation result for α_{int} = 0.65 of 1820 kN for the cylindrical anchor is significantly higher than for the equivalent area rectangular anchor and also significantly higher than the BIFURC 3D and HVMCap results. These results confirm that the linear Plaxis 3D Foundation interface elements are too inflexible to model the soil-pile lateral slip along curved surfaces. Later versions of Plaxis 3D Foundation are expected to provide isoparametric, or curved interface elements, for more accurate modeling of curved interfaces.

Non-Horizontal Loadings

Andersen et al (2005) compared calculation procedures for the undrained capacity for varying loading angles β. Figure 8 summarizes results from the independent capacity calculations by three different organizations. The comparison of results from 3D finite element calculations carried out by Norwegian Geotechnical Institute (NGI), Offshore Technology Research Center (OTRC) and the University of Western Australia (UWA) serves as an excellent benchmark for evaluating the performance of Plaxis 3D Foundation.

A series of computations were made to evaluate the performance of Plaxis 3D Foundation when the applied loads are not horizontal. These computations were made for the capacity of the 5 m diameter cylindrical suction anchor. However, an interface factor α_{int} of 1.0 was used for these computations to minimize the effects of non-isoparametric interface issues. All loadings were applied at the optimal loading point to produce a failure corresponding to pure translation.
Figure 8 compares the results of these Plaxis 3D Foundation computations (α_{int} = 1) with the benchmark 3D finite element results (α_{int} = 0.65). Plaxis 3D Foundation shows the same trends with varying load inclination as the other programs but as expected because of the higher interface factor computes higher capacities.

Note that it is only in the lateral direction (z-x plane) that the non-isoparametric elements prevent slip. The interface elements should work well in the vertical direction, thus the capacity for pure vertical loading should not be overestimated. A Plaxis 3D Foundation computation for α_{int} = 0.65 and pure vertical loading produced a capacity of 2570 kN, completely consistent with the benchmark finite element analyses of Figure 13. This agreement occurs because the interface issue described above has little or no effect for vertical suction anchor translation.

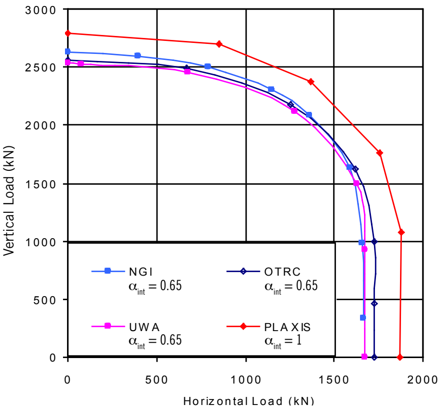

Figure 8:

Comparison of Plaxis 3D Foundation and Benchmark Suction Anchor Computations for Non-horizontal Loadings after Andersen et al (2005) - 5 m diameter Suction Anchor.

Conclusions

For the plane strain computations

- The Plaxis 2D and Plaxis 3D Foundation capacities agree within about 2 % and the Plaxis FE results also agree well with the hand calculation.
- The discretization error always contributes to an overshoot for FE capacity analyses. It was demonstrated how this overshoot can be quantified by plotting the capacity versus the number of nodes. The error was made negligible by the use of interface elements and strategically refining the mesh.
- The 6-noded elements of Plaxis 2D computed the same capacity as the 15-noded elements. However, the 6-noded elements require more mesh refinement so that there is at least an equal number of nodes.

For the three-dimensional computations

- Plaxis 3D Foundation provided a capacity for the 5 m diameter cylindrical suction anchor that is about 5 % higher than the capacities obtained from BIFURC 3D and NGI HVMCap for a wall roughness α_{int} = 1.0 and pure horizontal loading.
- The Plaxis 3D Foundation results for inclined loading and α_{int} = 1.0 seems reasonable and compares well with the Andersen et al (2005) benchmark results.
- The Plaxis 3D Foundation capacity for a wall roughness σ_{int} = 0.65 is clearly too high, confirming the expected overestimation from the issue with the non-isoparametric interface elements. We recommend that the Plaxis 3D Foundation program should not be used as the only tool for design of suction anchors until this issue is resolved and correct performance verified.
- Ultimate capacity calculations by FEA are sensitive to discretization error, and in particular 3D problems. Insight in the geometry of the governing failure mechanism and the use of interface elements, symmetry, reduced model dimensions and strategic mesh refinement greatly reduces this error.
- By running a series of calculations for the same problem with varying mesh fineness and plotting the obtained capacities against number of nodes, number of elements or the average element size it is possible to quantify the discretization error and possibly also making it negligible.

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On Stability Analysis of Slurry–Wall Trenches

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1. Introduction

The Plaxis users at Wroclaw University focus on soil–structure interaction research, which also covers vertical excavations supported by either steel or reinforced–concrete retaining walls. The wall-construction process uses deep vertical trenches that are filled up with a bentonite suspension (Xanthakos, Hanjal). Displacement and stability analyses of the anchored walls belong to standard calculations and they are reported in many places, including the Plaxis Bulletin. In contrast, the stability analysis of the tentative trench itself, supported by the bentonite liquid, is less popular. Therefore, these aspects are the objective of this article.

The technological phase of a bentonite supported trench is – to a certain degree – a critical moment in the construction process. This is so, because the next phase, i.e. the successive replacement of the bentonite suspension with the fresh concrete, improves the stability, due to an increase of the stabilizing horizontal pressure applied to trench faces.

Geotechnical engineers have coped with the trench–stability problems for years using simple design methods (Piaskowski, Morgenstern, Washbourne, Fox, Tsai, Ng) or recently FEM–supported calculations Ng (Oblozinski). However, some questions still remain open. First of all, the slurry–wall trenches consist of sections L×B×H (say, the length L ~ 2÷8m, width B ~ 0.6÷1.2m, depth H ~ 10÷15m or more), so a true 3D stability analysis is required. Indeed, it is a well–established fact that the horizontal ground pressure is usually much less then the 2D active earth pressure yielding from the Coulomb theory. Some authors explain this behaviour making use of the silo–pressure analogy, recalling the Janssen–Terzaghi solution. Other approaches make use of more or less sophisticated limit equilibrium methods and there exists a great variety of sliding wedges of soil mass taken arbitrarily by many authors.

Clearly, layered soils can be analyzed only with difficulty within the limit equilibrium calculations. The same is true for local loads distributed on the ground surface in the trench vicinity. Eventually, no prediction of the ground surface deformation is possible if using statically determinate calculation methods. The advantages of FEM modelling become obvious here.

We used Plaxis 3D Foundation to test a very simply design method. In this context, the simplest elastic–plastic Mohr–Coulomb model seems to be relevant.

2. Deterministic methods

Stability evaluations of slurry–supported trenches use generally 3D models in two versions which are based on:

- the force equilibrium for the sliding soil mass (wedge),
- simulations of developing displacements of one (or a few) points selected on the trench face.

2.1. Limit equilibrium methods

As a first approximation within the limit equilibrium analysis, the 2D solution for triangular wedge and the infinite trench length can be applied [Nash and Johns], in particular using the Coulomb critical angle of sliding $\theta_{cr} = \pi/4 + \phi/2$, Fig.1a. This way, the earth pressure is overestimated and more realistic shapes of the wedge are of interest, Fig.1c–e.

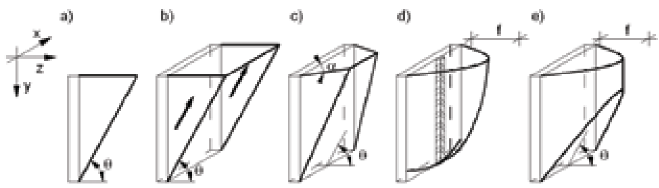

Figure 1: Shapes of the sliding wedges studied by: a) Nash and Johns (2D); b) Morgernstern and Amir–Tahmasseb; c) Washbourne; d) Tsai and Chang; e) Piaskowski and Kowalewski.

The simplest transition from 2D to 3D solution in Fig.1b bases on taking into account shear forces on all sides of the sliding wedge (Morgernstern and Amir–Tahmasseb). Washbourne modified the shape of rigid block assuming the angle $\alpha = \pi/4+\phi/2$ between slide surface and face of the trench, Fig.1c. FEM simulations made by the authors indicate that such a value of the angle α seems to be underestimated.

The latest 3D solutions by Tsai and Chang employ more realistic – smooth and convex – shear surface. The method uses vertical columns as a generalization of standard 2D slices. The Piaskowski and Kowalewski solution, proposed as early as in the mid-sixties, uses a vertical elliptic cylinder cut by a critical plane. The approach has a profound justification in terms of elliptic compression arches observed in rock mechanics (though in vertical planes, not the horizontal one).

From our experience and many tests performed, we could recommend the situation presented in Fig.1b which reconciles simplicity and accuracy.

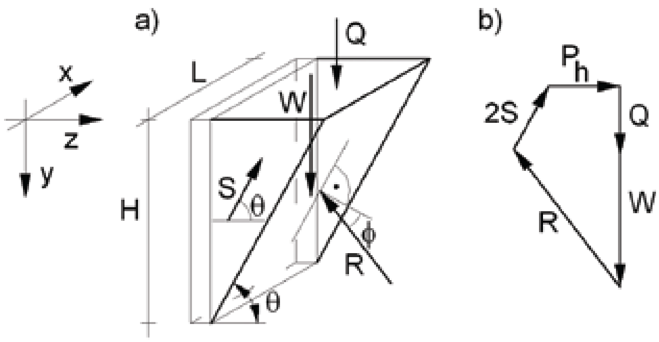

Figure 2a: The 3D-view of the sliding block; b) the polygon of acting forces in the plane of symmetry.

Introduce the acting forces [kN]: W – bulk effective weight of a wedge, R, S – soil reactions, Q active load in line of symmetry (Q = 0 hereafter), P_s – hydrostatic horizontal slurry pressure on the vertical face L × H of the trench, P_h – hydrostatic horizontal ground-water pressure on the face L × (H-h_w) of the trench. Note that the slurry table is kept on the ground level and the water table is situated h_w meters under the ground

level. Both P_s and P_w do not depend on the angle θ which is to be found. The reaction S is calculated by integrating horizontal stresses over the triangle and the horizontal stresses are, by assumption, proportional to effective vertical ones. Testing calculations with Plaxis 3D Foundation did not confirm large values of such coefficient of lateral pressure K which could be expected due to arching effects. The values situated between K_a and K_o were generally observed, so $K = K_a$ can be assumed as a safe approximation, $K_a = tg^2(\pi/4+\phi/2)$.

For simplification, it is also assumed that there is no hydraulic contact between ground water and the slurry - no filtration is considered. To be more realistic, such contacts occur in noncohesive soils but they are of a specific character. The filtration of slurry suspension takes place towards the soil mass thus increases safety margins. It is also reported (Elson, Filz), that the penetration of the slurry suspension has a very limited scale and a skin-contact colmatation is observed – called "filter cake". Such a behavior is not obvious in coarse-grain soils.

The governing equations for cohesionless soils follow the standard Coulomb approach with the discussed modifications, Fig.2:

$$\begin{cases} \sum F_z = 0 \\ \sum F_y = 0 \\ R_z = R_y \cdot tg(\theta - \phi) \end{cases} \Rightarrow \begin{cases} P_h = R_z - 2 \cdot S_z \\ W = R_y + 2 \cdot S_y - Q \\ R_z = R_y \cdot tg(\theta - \phi) \end{cases} \qquad (1)$$

Critical failure plane θ_{cr} can be found such that it maximizes the value of P_h.
Note that in 3D, for realistic values of L/H, the critical angles θ_{cr} are usually some 10% greater than $\pi/4+\phi/2$. Such a behaviour is governed by the stabilizing forces S applied to the lateral triangular surfaces. Clearly, the critical angles θ_c tend to $\pi/4+\phi/2$ for L>>H, i.e. if the relative contribution of the forces S becomes small.

The limit equilibrium in terms of horizontal forces can be expressed as $P_s - P_h - P_w = 0$ thus also as $FS_1 = P_s/(P_h + P_w) = 1$ or as $FS_2 = (P_s - P_w)/P_h = 1$. Due to a lack of uniqueness ($FS_1 \neq FS_2$ for $FS_i > 1$, i=1,2), and bearing in mind a comparison of results with Plaxis calculations, the authors define factor of safety FS in the standard way:

$$FS = \frac{\tan\phi}{\tan\phi_{red}} \qquad (2)$$

where the limit equilibrium $FS_i = 1$ must be reached for ϕ_{red}.
Clearly, the factor of safety has a global character, as the one using resultant forces, so it can be less useful when a local loss of stability can happen.

Example 1.
Consider the depth of the trench H = 10m and the water table which can change: h_w = 1m, 2m, 3m, respectively. The material parameters are presented in Table 1.

	γ	γ'	K_a	ϕ	c
	kN/m³	kN/m³	–	°	kPa
Fine sand	18.5	9.0	0.31	32.0	0

Table 1: Parameters of a homogeneous soil used in (1),(2).

The results in Fig.3a confirm that short sections of the trench are more safe. Therefore, the static analysis in direction perpendicular to the trench width B is out of considerations.

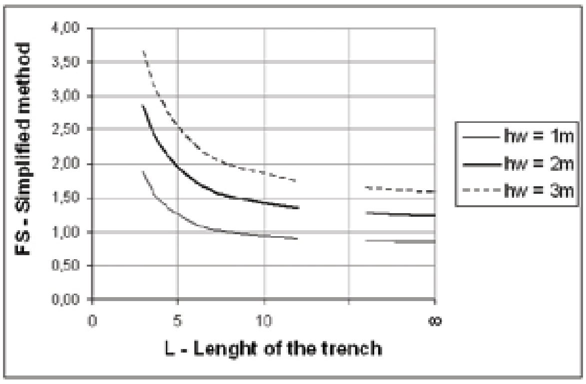

Figure 3a: Plots of FS versus section length L (symbol ∞ stands for the 2D case), H = 10m.

The role of the slurry density can be presented as follows.

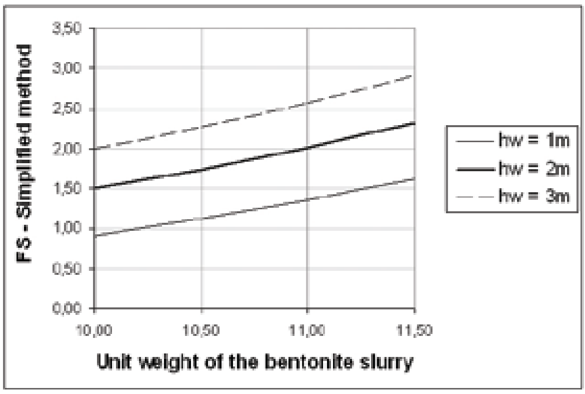

Figure 3b: Plots of FS versus slurry unit weight for L = 6m, H = 10m.

On Stability Analysis of Slurry–Wall Trenches

Continuation

2.2. The FEM–based testing using Plaxis 3DFoundation

The trench dimensions are 6×1×10m (L×B×H) but two axes of symmetry reduce it to a quarter 3×0.5×10m. The soil spreads within a bounded block 12×14×15m which vertical boundaries are fixed for horizontal displacements.

Excavation process was performed by successive removing 1m-thick ground layers at each calculation phase. Also at each phase, the slurry pressure was increased by application of external loads on trench faces (linearly increasing with depths, starting from the ground level) as well as on the bottom of the trench. The slurry unit weight was 10.5kN/m³. For the water table $h_w = 2$m was assumed.

The standard φ-c reduction technique was used to determine values of the factor of safety FS thus the methodology coincides with the one presented by the expression (2).

The material parameters are as follows.

	γ	γ'	K_0	φ	c	ψ	E	ν
	kN/m³	kN/m³	–	°	kPa	°	MPa	–
Fine sand	18.5	9.0	0.47	32.0	0	0	70.0	0.25

Table 2: Parameters of a homogeneous soil analyzed by Plaxis

Example 2.

When the values of FS start to stabilize during the reduction of φ, the maximal 3D displacements are close to 20mm (Fig.4a), on the axis of symmetry the sliding wedge develops almost linearly, the angle θ_{cr} is close to $\pi/4+\phi/2$ and the sliding wedge is relatively large. For engineering purposes, most of the 3D models presented in Fig.1 can be used to model the shape of the wedge.

Clearly, some settlements far from the trench can be also observed – caused by the elastic soil behavior, unloading first of all.

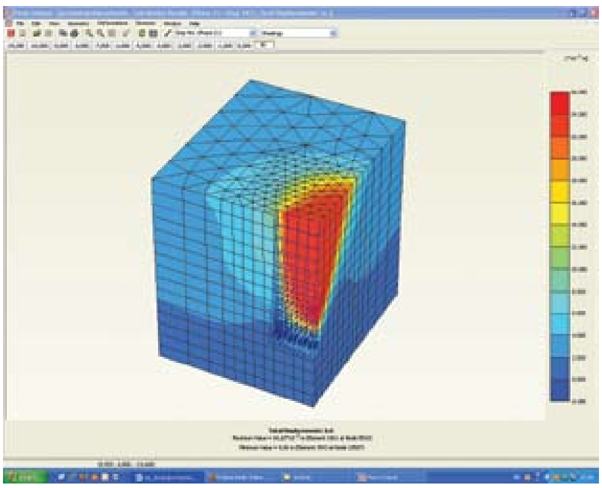

Figure 4a: The 3D total displacements (at failure).

Focusing on horizontal displacements, it can be observed that the failure initiates in the lower part of the trench, Fig.4b. The same conclusion holds for incremental displacements. The uniform red color in Fig.4a confirms an almost vertical kinematics of the wedge.

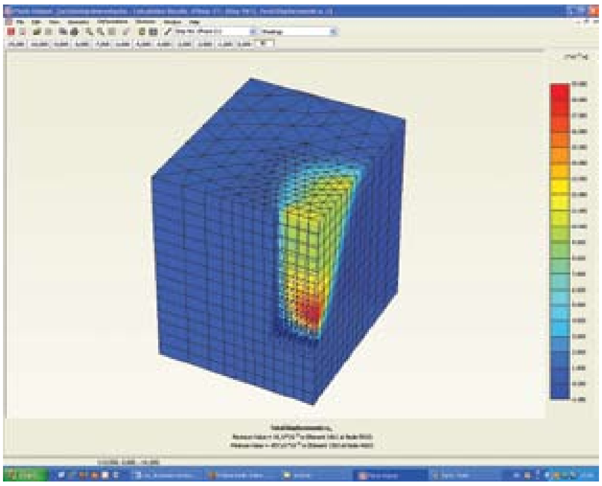

Figure 4b: The horizontal displacements of soil towards the trench (at failure).

Example 3.

Assume the section length of the trench L = 6m and the water table that can change: h_w = 1m, 2m, 3m, respectively. Fig.5 presents the decreasing of the factors of safety FS when the excavation proceeds. Although based on very different assumptions, both methods coincide.

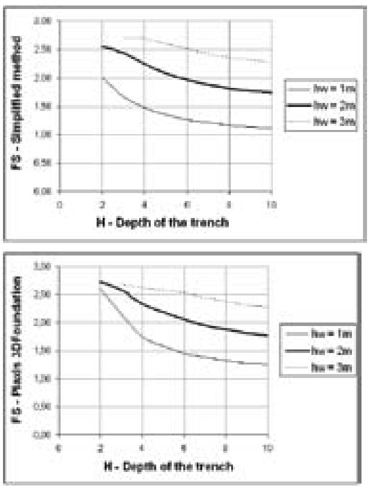

Figure 5: Comparison of two calculation methods in term of the factors of safety FS.

To get a more complete comparison of results, a wider spectrum of numerical examples for H and h_w is presented in Fig.6. Generally, the limit equilibrium method seems to be more conservative. Significant differences, up to 20-25%, can be observed but only for high water table h_w = 1m; the influence of the trench depth H is less evident. On the other hand, the differences are located in the range of small values of FS. In our opinion, just the small values of FS are the general reason of the differences, not the high water table itself. This happens due to the simplified wedge shape that can be more decisive for small values of FS.

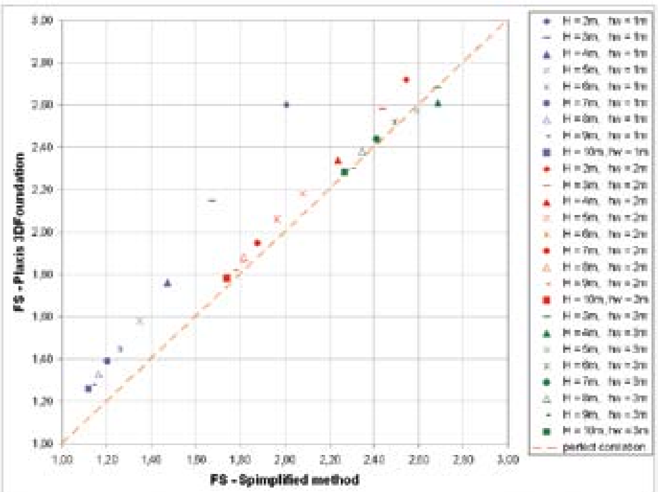

Figure 6a: set of points FS versus FS for the same geoengineering data (the dashed line would mean a perfect correlation of results).

2.3. Further examples calculated using Plaxis 3D Foundation

In addition to the presented material, consider a little weaker 1m-thick sublayer situated at the depth of 4-5m.

	γ	γ'	K_0	φ	c	ψ	E	ν
	kN/m³	kN/m³	–	°	kPa	°	MPa	–
Fine sand	18.5	9.0	0.47	32.0	0	0	70.0	0.25
Weaker layer	22.0	12.0	1.00	0	15.0	0	32.0	0.30

Table 3: Parameters of a layered soil analyzed by Plaxis

Example 4.

Fig.7 correspond to Fig.4, respectively. Note that the differences in kinematics are not so much significant as expected.

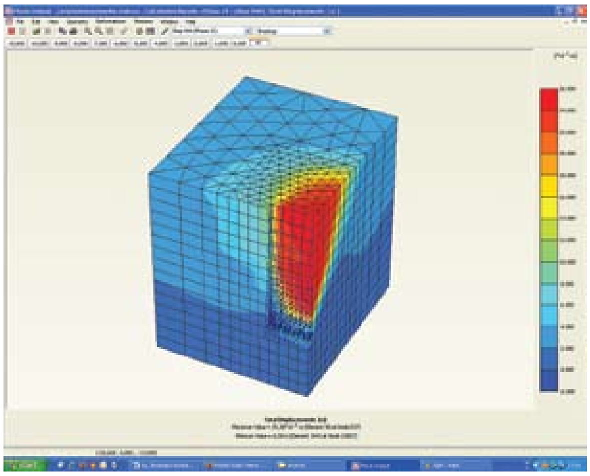

Figure 7a: The 3D soil displacements (at failure).

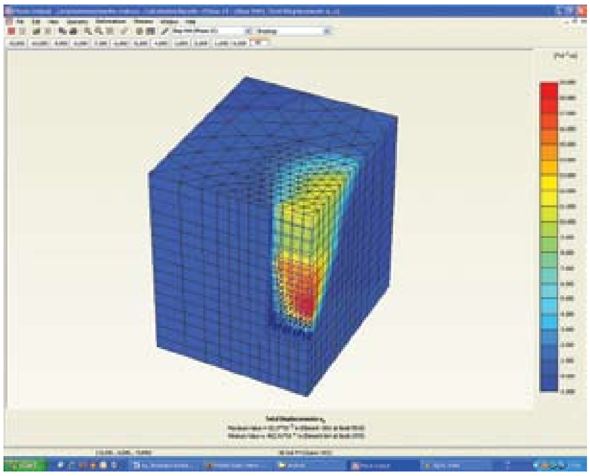

Figure 7b: The horizontal displacements of soil towards the trench (at failure).

3. A probabilistic method

Another safety analysis can be based on a probabilistic methodology (Brzakala and Gorska), following the method of the so-called design point (see Thoft-Christensen and Baker, Baecher and Christian).

Consider two uncorrelated random variables:

- the water table h_w, with the expected value $E\{h_w\}$= 2m and the standard deviation σ_h = 1m,
- the friction angle φ, with the expected value E{φ}= 32°, and the standard deviation σφ = 3.2°.

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Note that only two moments of the random variables are required and the probability distributions are not specified in this method (second-order distribution-free approach). Other deterministic data follow from the previous section (a homogeneous soil).

In terms of the dimensionless coordinates

$$z_1 = \frac{h_w - E\{h_w\}}{\sigma_{h_w}} \quad \text{and} \quad z_2 = \frac{\phi - E\{\phi\}}{\sigma_\phi} , \qquad (3)$$

Hasofer and Lind (see Thoft-Christensen and Baker) introduced a measure of safety – called the safety index $\beta = \min\sqrt{z_1^2 + z_2^2}$ – which means the shortest distance from the beginning of coordinate system (expected values of the considered random variables) to a failure surface.

So, first the failure surface can be found making use of Plaxis 3D Foundation assuming a limit displacement. For two considered random variables, the failure surface reduces to a curve, almost linear one in Fig.8. It is composed of all points (z_1, z_2) for which the displacement limit condition is reached (25mm in this case). In detail, successive values of h_w were fixed and the limit state in terms of the displacement was reached by reducing the angle of friction.

As the second step, the shortest distance β has to be found and the design point for which this distance is reached.

Clearly, less attention is paid to points and the shape of the failure surface in regions situated far from the design point.

Analysis of a greater number of random variables is in principle the same, making use of the same two steps. However, for practical applications, Thoft-Christensen and Baker recommend to focus on the most significant variables. “Significant”means here both a large parameter-sensitivity of the model and large randomness (standard deviaton) of the parameter. Neither slurry density nor soil density fulfil this requirements but the water table and the soil strength do.

Finally, note that the obtained value of $\beta = 1.4$ is relatively low - in random conditions we would recommend a value $\beta > 2$.

The direct comparison with Plaxis safety evaluation is not easy because of completely different background. Assuming the mean values as a reference level, so the deterministic parameters $h_w = 2$m and $\phi = 32°$, the FS yielding from the ϕ-c reduction method in Plaxis is however similar: FS = 1.7.

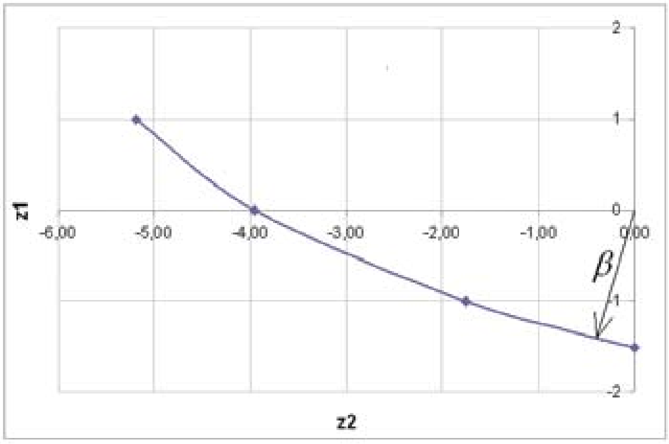

Figure 8: The Hasofer and Lind safety index $\beta = 1.4$.

4. Conclusions

1. Plaxis 3D Foundation appeared to be a useful numerical tool for testing a simplified design method of stability analysis.
2. For 3D analysis of stability, a significant reduction of the resultant soil pressure P_h can be observed, especially for small L/H that can increase the trench safety to required levels.
3. The trench depth H in the numerical examples was limited to 10m but the results can be representative also for deeper trenches. The calculations reveal that the failure initiates mainly within the upper 10m, event for H >> 10m. Such a conclusion is in agreement with other models (Piaskowski and Kowalewski). There is also a coincidence with the geoengineering practice, though probably many other factors support such a practical conclusion (suspension weight increasing with depth, soil orthotropy, soil parameters changing with depth, etc.).
4. In contrast to calculations using Plaxis 3D Foundation, more complex studies (displacements, local inhomogeneities, local loadings, etc.) are far beyond the scope of the limit equilibrium methods.

Acknowledgement

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Seabed instability and 3D FE jack-up soil-structure interaction analysis

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1. Introduction

Seabed instability is an important aspect in the design of different offshore structures. Particularly for a jack-up drilling rig, which is supported by three independents legs, this becomes a crucial issue.

A geotechnical engineering analysis for the installation (preloading) and storm loading of the world's largest jack-up rig, temporarily installed next to a quay of a Norwegian yard, to be upgraded for production work on the North Sea, is given in this article.

From a preliminary site survey the seabed in the considered area was expected to be rock outcrop, undulating across the site. Considering that rig's footings have outer / inner skirts, which could not penetrate the rocky seabed, modification in the seabed conditions, creating flat areas at the footing's locations, through construction of shallow gravel banks, was initially proposed.

A detailed geotechnical investigation was carried out to verify the soil conditions. From the investigation a sediment layer of varying thickness overlying the undulating bedrock was identified.

Several possible rig locations were investigated and discussed to a final one, which was thoroughly assessed. The sediment layer consisted of a very soft to firm silt (mix) layer overlain by a thin layer of seabed sand. Therefore, preliminary engineering analyses, conventional and numerical, with originally low or increased elevations of the gravel banks, indicated instability of the free skirted spudcans under preloading conditions.

The two-dimensional (2D) and three-dimensional (3D) finite element (FE) analyses of the free skirted spudcans, which are usually applied for non-uniform soil conditions, were currently considered conservative. For a more realistic evaluation, jack-up structure - skirted spudcan - gravel bank - soil interaction effects were included in the analyses.

The full 3D structure-foundation model was applied for varying heights of the gravel banks. showing non-uniform skirted spudcan penetrations, rotations and sliding. The FE results from the final location and final heights of the gravel banks, showing that the structure forces are within the expected limits, are presented in the following.

2. Structure - Foundation System

The current jack-up drilling rig, the world's largest, is type independent leg cantilever. It operates in water depths up to 150 m and it has leg lengths of about 205 m.

2.1 Structure elements and stiffness

The considered jack-up rig is a complicated structure to be modelled in details. Therefore, in the 3D FE model calculations only the main structure elements were considered taking into account the interaction with the foundations.

Only the three legs and the hull were included in the FE model. The legs were simplified to 3D beam elements, and the hull to plate or floor elements with the equivalent thickness / area. The rig designer provided the geometry data for the legs and the hull.

2.2 Footing geometry

The considered jack-up footings have a diameter D = 22 m and are fitted with outer and internal skirts, which divide the spudcan into 6 compartments. Figure 1 show a photo view of the spudcan.

The vertical geometry of the spudcan structure is mainly given by: Distance from spudcan base to tip of outer skirts 2.3 m; Distance from spudcan base to tip of internal skirts 1.1 m;

Figure 1: Skirted spudcan view

The spudcan itself is almost a flat rigid plate. The transverse stiffnesses of the skirts are derived from the structural FE model of the spudcan. These thicknesses are applied in the 2D and 3D FE analyses employing beam and wall structural elements, respectively.

2.3 Soil conditions

To identify the seabed / soil conditions at the considered locations a new site survey, seismic, (sparker and pinger) and bathymetry was carried out.

From the survey, generally sediments of varying thickness overlaying hard ground / bedrock were found. The largest sediment thicknesses were seen at the largest water depth.

Gravity vibrocore samples taken from seabed could not reach the bedrock and showed mostly sediments of clayey, gravely sand. At the shallow water depths the bedrock outcrops the seabed.

After the interpretation of the seismic survey (sparker) geotechnical investigation including 5 piezo-cone penetration tests (PCPTs) and one vibrocore for each spudcan location were carried out. Good definition of the seabed level and the bedrock was found. However, discrepancies were recognized at some PCPT locations. The inconsistency was explained by the fact that the PCPTs were not carried out on the seismic lines.

Considering the limitations of the sparker survey, a pinger survey was carried out. With a less penetrating, but a smaller opening angle seismic source, the pinger survey was applied to better identify the slope of the bedrock and supplement the previous investigations in the area. Based on the pinger data combined with the existing soil information, a re-interpreted model of the sediment and bedrock surface was produced.

As a result, the original proposed locations were reduced to a final one. At each leg position four vertical cross sections showing the seabed and top bedrock profiles from centre of the spudcans out to a distance of 50 m, are presented in Figure 2.

To identify the soil conditions and the soil parameters applicable to the design of the gravel banks at the final location a new geotechnical investigation consisting of 5 boreholes, about 70 PCPTs and laboratory tests were carried out.
On the basis of all the geotechnical data it was evaluated that the soil conditions consist of overall quaternary marine sediments, mainly deposits consisting of a seabed layer of sand overlying clayey, sandy silt with variable thickness (0 - 9) m overlying crystalline bedrock.

2.4 Water depth
The water depth or the seabed elevations at the centre of the three spudcan locations are as seen from Figure 2, approximately –23 m at spudcan S1, -19 m at S2 and –26.5 m at S3.

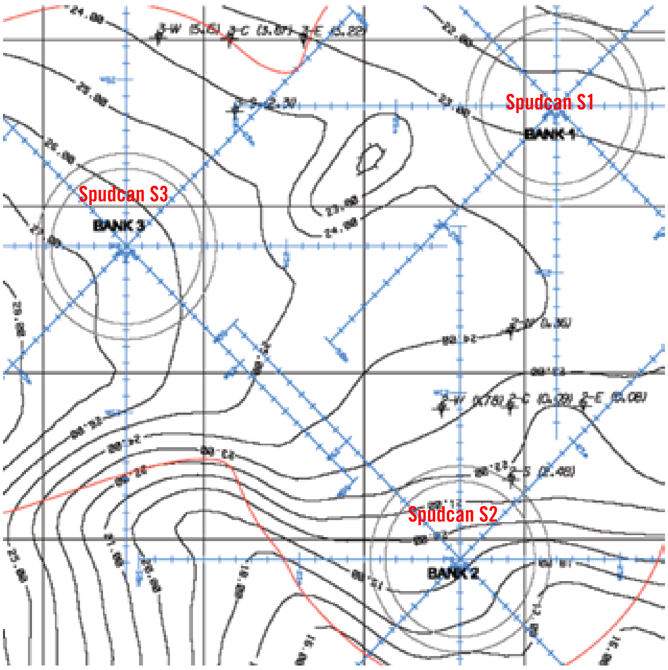

Figure 2: Seabed bedrock profile, final rig location

2.5 Design soil profiles and parameters
On the basis of the seismic surveys, PCPTs / boreholes and laboratory test results (classification and triaxial, unconsolidated undrained (UU) and consolidated isotropic drained (CID)) performed for the final location, the soil profiles and soil parameters applicable to the engineering assessment are derived. The soil parameters for the bedrock are evaluated based on the engineering experience.
There seems to be a good correlation with the PCPT data for the depths where samples were taken and laboratory tests performed. From the UU triaxial tests undrained shear strengths of value minimum c_u = 33 kPa are measured for the extracted samples. However, at the depths where lower cone strength as shown in Figure 3, is recorded from the PCPT, no soil sample could be extracted and no correlation is available. Under these circumstances the correlation $N = q_{net} / c_u = (15 – 20)$ is found applicable.
When applying such a correlation on the PCPT data undrained shear strength for the silt c_u = (15 – 30) kPa is assessed. Based on the test results and the engineering judgement initially c_u = 25 kPa for the silt layer and a friction angle $\varphi = 35°$ for the seabed sand layer were assessed as lower bound values. For the bedrock an undrained shear strength c_u = (1000 – 1500) kPa was assigned to represent the strong subsurface. A summary of the soil parameters applied in the analyses is given in Table 3.
The gravel bank material is modelled applying a unit weight $\gamma' = 11$ kN/m^3 and a friction angle $\varphi = 40°$. The deformation parameter E = 100000 kPa.

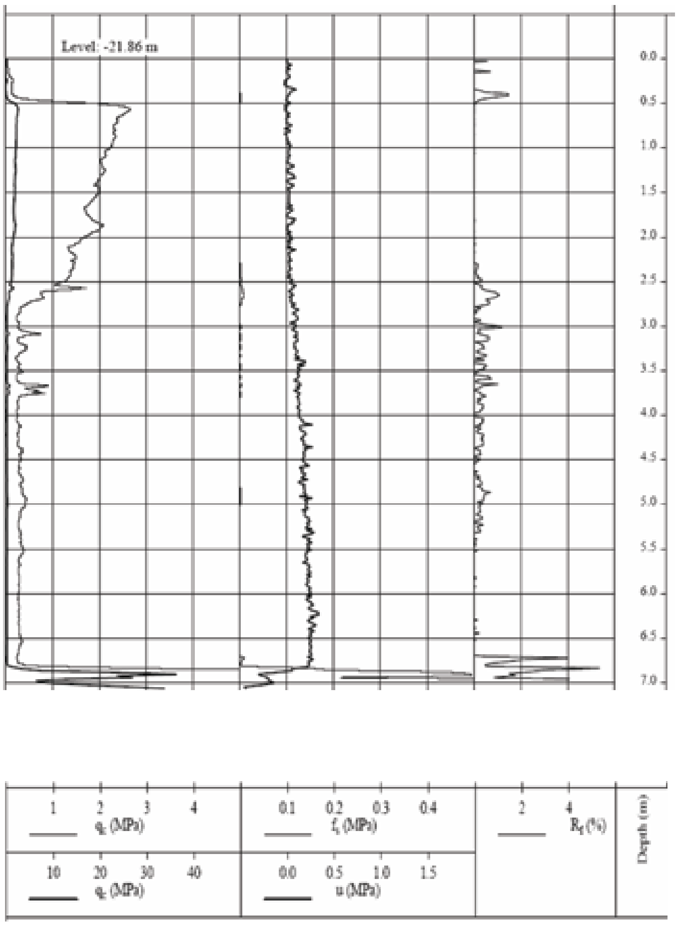

Figure 3: PCPT profile at S1 location

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Soil Type	h (m)	γ' (kN/m³)	φ (°)	cu (kN/m²)	E (kN/m²)
Sand, loose to medium dense	Varying	10.0	35	-	35000
Silt, very soft to firm	Varying	9.0	-	15/25/30	100*cu
Bedrock	Varying	12.0	-	1000/1500	200*cu

Table 1: Soil profile applied in the FE analyses

3. Structure - Foundation Analyses

Different analyses consisting of conventional and 2D / 3D FE modelling are carried out.

3.1 Preliminary 2D and 3D FE modelling, low gravel banks

The 2D FE free skirted spudcan - low gravel bank - soil interaction analyses with Plaxis 2D Version 8 (2002) showed instability of the S1 and S3. However, the 2D analyses were considered very conservative due to 3D soil conditions.

Under these circumstances 3D FE modelling of the free skirted spudcan - gravel bank - soil interaction was performed, The model was built with Plaxis 3DFoundation (2006) assigning boreholes at the location where soil profile changes. An implicit interpolation between the boreholes is carried out during the calculation. By this method the soil conditions at the spudcan area are modelled to the extent the seismic survey and the geotechnical investigation allow.

Mohr Coulomb constitutive soil model for the soil layers in the drained (seabed sand and gravel bank) and undrained (bedrock and clay / silt / mix) conditions are applied. The preliminary analyses applying c_u = 25 kPa for the silt layer showed large rotations and horizontal movement for the free S1.

To take into account the structure foundation interaction it was discussed to apply some stabilizing loads on the spudcan while preloading. After many calculation attempts it was found difficult to assess the limited reaction forces needed to stabilize the spudcan and the procedure was cancelled. The issue of skirted spudcan – structure – skirted spudcan - soil interaction was raised at this time.

The first full 3D model consisted of low gravel banks at elevations –20.0 m, (about 3 m height), -13.5 m, (about 5.5 m height), -24.0 m, (about 2.5 m height), for S1, S2 and S3, respectively. Large penetrations and horizontal movements, particularly for S3 were calculated. The reaction forces in the structure were far beyond the limits. Under these circumstances the effect of the higher gravel banks at S1 and S3 locations were investigated.

3.2 Conventional skirted spudcan differential penetration

Based on SNAME (2002) and Hansen (1970) conventional skirted spudcan penetration analyses were carried out at each spudcan location to get an idea on the effect of the height of the gravel banks on the spudcan differential penetration. These were also compared with some FE axisymmetric analyses of the spudcan penetration.

Such analyses are previously carried out by Kellezi & Stromann (2003), Kellezi et al. (2005a,b),

In the analyses c_u = 25 kPa for the silt was applied. The results for location S1 and gravel bank at elevation –19 m are given for illustration in Figure 4. Two extreme soil profiles within the spudcan area are chosen, which are expected to give max and min penetrations. The differences in penetrations give the expected differential penetration of the free spudcan. The elevation of the gravel bank is moved from -21 m to -19 m to -14 m.. No punch through risk is expected for any of the scenarios.

For S2 the height of the gravel bank is determined from the length of the spudcan skirt/chord, plus some tolerance. The top of the bank will be at -13.5 m and small differential penetrations are expected.

For S3 two extreme soil profiles are chosen as well expected to give max and min penetrations. The elevation of the gravel bank is moved from -25 m to -23 m to -21 m to -18 m to -16 m.. No punch through risk is expected for any of the scenarios.

3.3 Preliminary 3D FE modelling, high gravel banks

To make the location applicable for the rig installation based on the conventional and FE axisymmetric results, higher gravel banks were proposed. The soil mechanic principle of load spreading is used. Higher banks will increase the bearing capacity of the silt layer as a result of increasing fictive bearing area.

Except for the preloading phase this model was also calculated for the storm load, wind speed 33 m/s. The storm load may come from any direction so different analyses are needed to define the critical one. The 3D model calculation procedure consists of 3 load stages, which are:

Preloading to max vertical load V =145 MN; Unload to vertical V = 112 MN, V = 100 MN, V = 115 MN for S1, S2, S3, respectively; Apply storm loads, horizontal H = 6.4 MN, moment M = 345.6 MNm at the most critical plane;

The horizontal force is applied at the hull plate pointing towards the critical leg. The moment is implemented as a set of two vertical loads, applied downward at the critical leg-hull connection and upwards at a point in the hull between the other two legs as shown in Figure 6. Except the 3 load phases, an initial phase is calculated consisting of the construction of the gravel banks.

The limited combined loads at the structure, one single leg, are calculated as:

Horizontal shear force Q = 18 MN; Vertical force (at hull) V = 145 MN; Bending moment at hull M = 325 MNm;

Taking into account the limits for the structure reaction forces and the result from the 3D FE structure – foundation models with increased height of the gravel banks at S1 and S3, a reassessment was found necessary.

The soil strength for the silt layer c_u = 25 kPa, as mentioned previously, was evaluated based on the engineering judgement. This is however, not a lower bound assessment based on the PCPT data and usual North Sea (q_{net} - c_u) correlation.

After reviewing the available soil data, to increase to some level safety concerning the soil parameters, DNV (1992) it was decided to reduce the shear strength for the silt layer from c_u = 25 kPa to c_u = 15 kPa. This strength is considered a lower bound design value, when taking into account the consolidation during construction of the banks and two weeks rig location with lightweight.

Investigating different 3D FE models with slightly different heights of the gravel banks, which could indicate less spudcan rotation / sliding, a final model, was constructed and given in detail in the next section.

3.4 Final 3D FE modelling, high gravel banks

The model scenario with gravel bank elevations at –14.5 m, (height about 8.5 m), -13.5 m, (height about 5.5 m), -15.8 m, (height about 10.7 m) for S1, S2 and S3 locations, respectively was chosen as final as the reaction forces and the amount of sliding were within the structure limits. Despite, this is the largest model with respect to mesh size, which could be run from the workstation.

The 2D build-up model geometry is given in Figure 5. The skirted spudcans are simplified by octahedrons. The spudcan is flat and in full contact with the gravel bank soil from the start of the preloading. The 3 chords and the inner skirts are not included. The tip of the outer skirts is from the start of the analyses at elevation calculated from bank elevation minus 2.3 m (the skirt length).

The jack-up structure is modelled in a simple way using vertical 3D beam elements for the 3 legs and plate / floor elements for the hull, as shown in the Figure 6. The leg elements are based on the Mindlin's beam theory. In addition, the elements can change length due to applied axial force. The leg beams and the spudcan plates at the connection points can simulate the 6 degrees of freedoms.

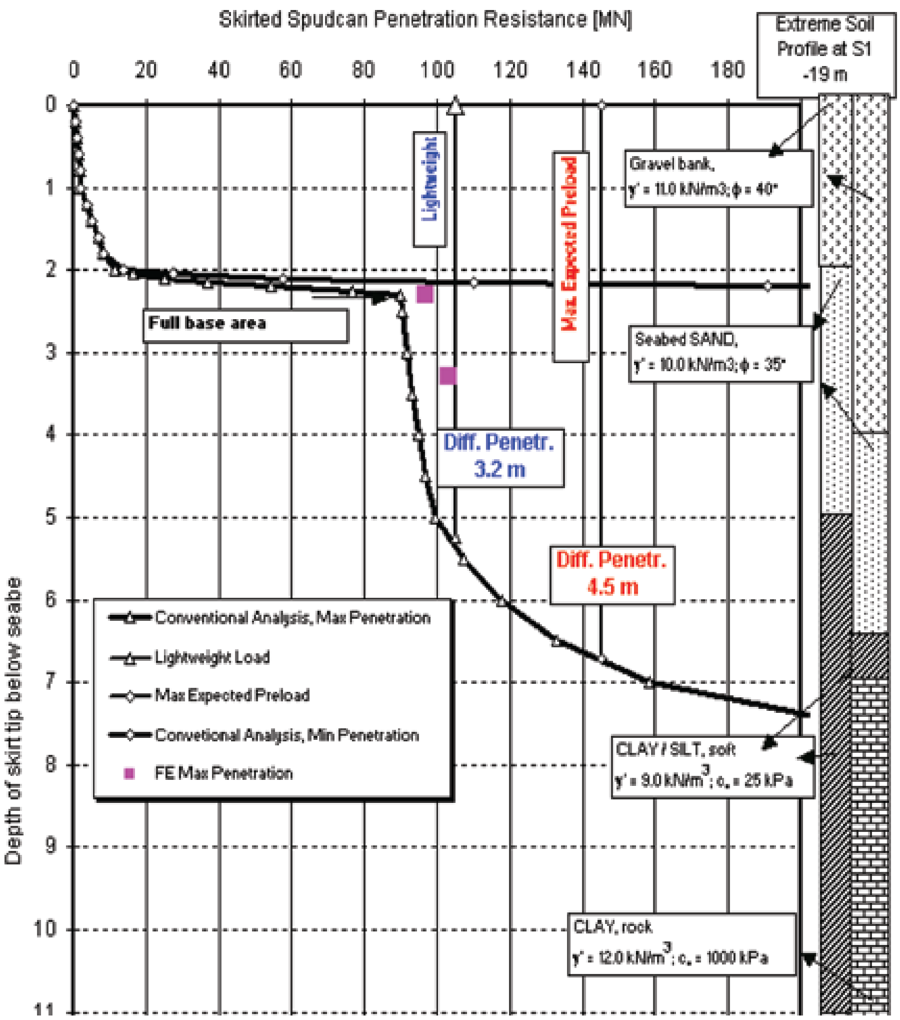

Figure 4: Conventional skirted spudcan differential penetration analysis, S1 extreme soil conditions, and gravel bank at –19 m

Seabed instability and 3D FE jack-up soil-structure interaction analysis

Continuation

The soil conditions, (soil profiles derived from the seismic, PCPT / borehole data at different cross sections), are modelled by implementing boreholes, as seen from the horizontal planes in Figure 5. Some of the soil profiles / sections with final gravel banks designed based on the 3D FE structure - foundation model are given in Figure 7, 8, 9.

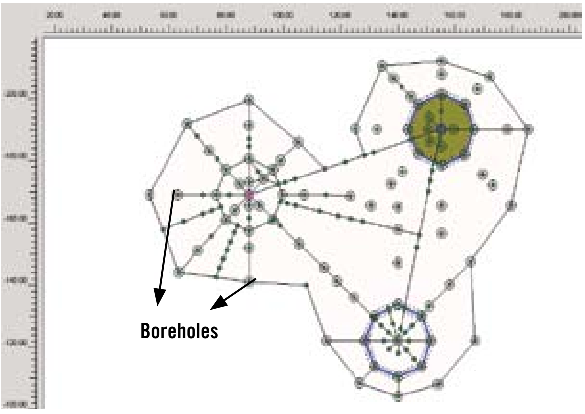

Figure 5: 3D FE structure-foundation model, 2D build-up, horizontal plane at S1 level, -14.5 m

The gravel bank sand material was specified to correspond to the soil strength applied in the analyses. The construction of the banks was performed following a procedure, which gives the possibility for some consolidation or drainage for the silt layer to occur. The total volume of the sand material used was about 60000 m^3.

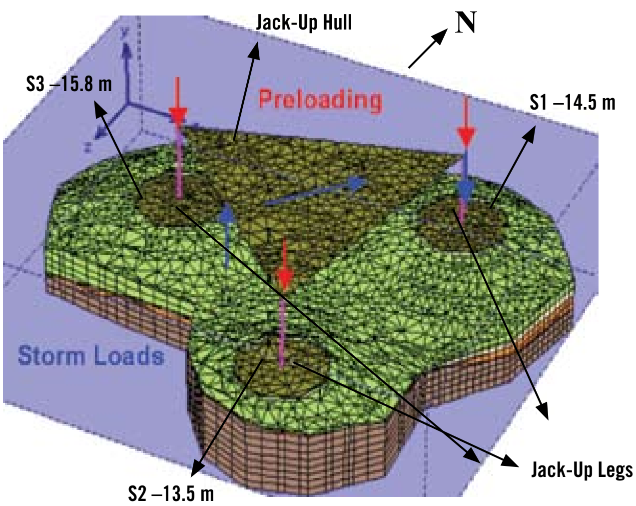

Figure 6: 3D FE structure-foundation model, final gravel banks

The results for the initial phase, including the construction of the gravel banks, are given in Figure 10. Vertical non-uniform settlements of about (20 – 40) cm are expected taken into account in the calculation of the total gravel volume.

The results for the preloading phase as total structure displacements are given in Figure 11. At the end of this phase the maximum calculated reaction forces are M = 211.04 MNm, shear force Q_{max} = 15.13 MN, differential penetration at S1, about 20 cm, sliding of S1, about 12 cm. For these values the structure integrity is found satisfactory.

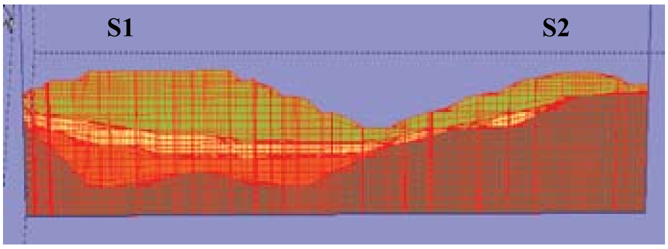

Figure 7: Cross section profile North – South at S1 across the 3D FE model (not to scale)

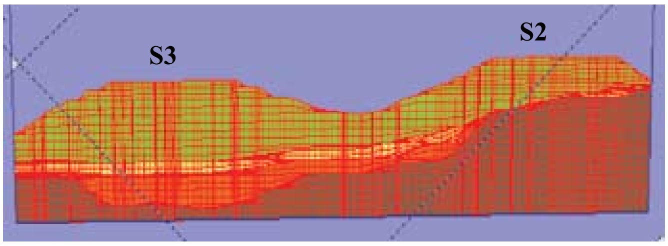

Figure 8: Cross section profile North West – South East at S2 across the 3D FE model (not to scale)

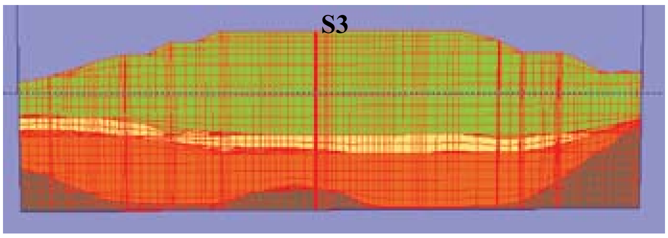

Figure 9: Cross section profile North - South at S3 across the 3D FE model (not to scale)

The results for the unloading phase show slight changes in the deformations and structure reaction forces. The results from the storm analyses show also slight changes in the deformations and structure reaction forces.

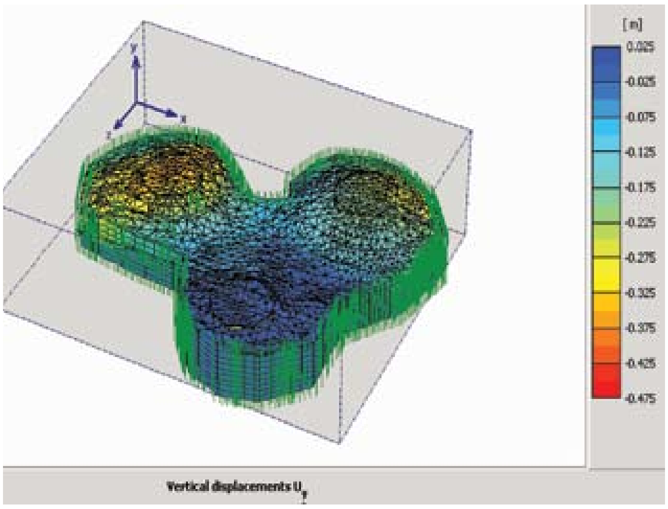

Figure 10: Initial phase, construction of the final gravel banks

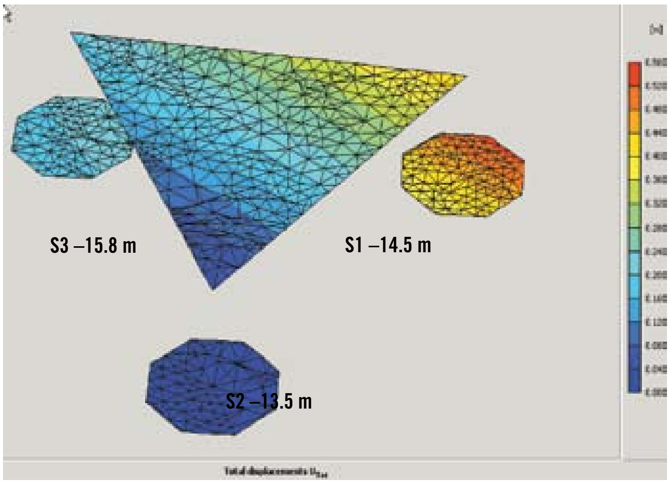

Figure 11: Preloading phase, structure total displacements (only plate elements shown)

Conclusions

3D FE structure - foundation interaction analyses are carried out for the installation of a jack-up rig, offshore, Norway, where seabed instability was a concern.

Gravel banks were designed at the skirted spudcan locations with different heights, ensuring that the structure reaction forces developed due to footings rotation / sliding, do not exceed the calculated limits.

The jack-up rig was successfully installed at the location and spudcan penetrations / displacements similar to the predicted values were recorded.

Acknowledgement

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Recent Activities

"Are you a Plaxis V8 user and not yet a V.I.Plaxis Service Member?

➔ Sign Up Now and get immediate access to Plaxis 2D V9.0"

We proudly present Plaxis 2D V9.0. In september we released this new version. Close to 4,000 Plaxis 2D programs are used in the world. By becoming a V.I.Plaxis Service program member users can take advantage of the latest developments like;

1. Hardening Soil Small Strain Stiffness model
2. Soil Test Facilty
3. Parameter Variation/ Sensitivity Analysis
4. Automatic regeneration of stage settings

The first 3 items are also discussed in previous bulletins and for item 4 you can find some technical information on page 23. For more information please send your request to info@plaxis.nl.

Furthermore we released update pack 2.2 of 3DFoundation. Besides fixing some known issues also a pile group wizard has been implemented. With the availability of defining pile groups instantly as a grid we respond to a frequently asked request after the release of 3DFoundation version 2.1.

In July our French Agent Terrasol sold 2 extra 3DFoundation licenses to Keller Foundations Special (Keller France). Keller France has now 5 copies of the Plaxis 3DFoundation program. *"With the Plaxis 3DFoundation program we are able to model geotechnical structures of which complexity and sensibility require that all geometrical constraints be taken into account"* said Mr Lambert of Keller France. With this order we welcomed our 1,000th 3D Plaxis user.

Plaxis America

In the Americas we participated in 2 major geotechnical events. The first one was GeoCongress 2008 in New Orleans, USA. The theme, a spin-off of the effects of the hurricane Katrina, gave a lot of extra exposure in the geotechnical sustainability simulations of Plaxis programs both 2D and 3D. The second conference was Cobramseg 2008 in Buzios Brasil. Because of the increasing demand on exchange of geotechnical experiences in Brasil this conference will be organised every 2 years in stead of 4. Courses were held in Buenos Aires - Argentina and Houston - U.S.A.

Plaxis Courses

The requests for courses has significantly increased in 2008, resulting in fully booked courses sometimes weeks before the actual course date. In that respect the very successful course in Argentina was the most outspoken example of this with over 70 registrations for only 45 available course seats. It is currently under investigation whether we can soon organize a second course in Argentina to accommodate the remaining registrants. Until now it was impossible to increase the amount of courses per year as the available staff for lecturing at courses was limited. However, with our recent increase of capacity in the field of courses we can now work on an increase of the amount of courses in the next year.

In the last quarter of 2008 the first Plaxis course in Spain has been scheduled, and is by now already fully booked. After the success of the standard course at Griffith University last February we continue our scheme of courses in Australia with another standard course this November in close cooperation with and held at the University of Newcastle. For details on this course please contact our local agent in Australia.

Plaxis Asia

In Asia we participated in the 6th International Symposium Geotechnical Aspects of Underground Construction in Soft Ground in Shanghai, China in April. We later went on a Plaxis roadshow in major cities like Beijing, Wuhan, Xian, Chengdu and Guangzhou to promote Plaxis. In May, we exhibited at a 2 days International Conference on Geotechnical and Highway Engineering (Geotropika 2008) in Kuala Lumpur, Malaysia. During the 2 days there, we met many existing Plaxis users as well as potential customers in highway engineering who express interest in Plaxis. On the last week of June, we organised a one day Plaxis seminar in cities of Ho Chi Min and Hanoi. Both of the seminars attracted more than 70 participants and many of them are from local engineering consulting and contractor firms. Plaxis Asia has also made her presence in the 10th International Symposium on Landslides and Engineered Slopes held in Xi'an, China from 30th June to 4th July 2008. The symposium is one of the most important activities of the Joint Technical Committee on Landslides and Engineered Slopes (JTC1) under the ISSMGE, ISRM and IAEG. On the 25th July, Plaxis Asia jointly organised a one day 3D Plaxis hands-on workshop with our Hong Kong Agent (see photo). There is an increasing demand on 3D FEM analysis in geotechnical application from the Plaxis users in Hong Kong. We intend to organise more of this type of courses in the near future. Please visit the agenda on our website on regular basis for such upcoming events.

We hope to see you soon at the 15th European Plaxis User Meeting in November, the 3rd Asian Advanced Plaxis User Course in December or one of our other events.

New managing director for Plaxis bv

After having moved to a brand new office at Delftechpark in Delft (NL) earlier this year, PLAXIS has now appointed a new managing director. Jan-Willem Koutstaal joined Plaxis on April 1st, 2008. This gives PLAXIS a solid foundation for further growth and professionalisation of the organisation, its products and services. Mr. Koutstaal has held several (international) management positions in ICT. He will employ the extensive experience gained to enforce and expand the position of PLAXIS as top player in the area of advanced software for geotechnical applications and services for geotechnical experts.

"PLAXIS is a company with in-depth knowledge and high quality products which, also through relationships with various national and international scientific institutes, is capable to play a world class role in its field. Definitely something to be very proud of!" according to Mr. Koutstaal.

Plaxis 2D Version 9 stage regeneration

One of the practical inconveniences of Plaxis V8 is the fact that staged construction phases have to be redefined whenever the mesh is regenerated, even if the geometry of the project has not been changed. In Plaxis V9 this inconvenience is overcome by a new functionality called "stage regeneration'. This stage regeneration, that is performed automatically every time the mesh is regenerated, applies the already defined stage settings to the newly created mesh. This includes per phase:

- Material assignment to clusters
- Load values
- Definition of phreatic levels and user-defined pore pressure settings including the regeneration of the pore pressures for phreatic line calculations. The settings for a groundwater flow are regenerated, but the user should recalculate the groundwater flow: this is not done automatically due to the duration of the groundwater flow calculation.
- If the K_0 procedure is used, the previous settings of K_0, OCR and POP are applied to the new mesh. Note that material changes do not have any effect on the values of K_0, OCR and POP: the absolute values of the previous K_0-procedure are used to regenerate the initial stresses.

This means that any change in the material assignments and load definition in the geometry will not be applied to the regenerated phases, including the initial conditions phase: the original phase settings of material assignment and load definitions are kept.

Material replacement in Plaxis 2D V8 and Plaxis 2D V9.0

One important topic in the stage regeneration is the replacement of soil materials. A lot of users do their first calculations with material datasets that do not have very detailed parameter information. Then, in a later stage of the project, more detailed information is available, and the same Plaxis project will be recalculated with changed materials.

Changing materials

There are three ways to change your materials:

A Change the parameters of the current material dataset

B Add a new material dataset, and replace the material in the geometry

C Add a new material dataset, remove the old dataset, and apply the new datasets to the appropriate clusters

In Plaxis 2D Version 8, methods A and B could be used. With method A, the material dataset was changed, which would effect all new calculations. With method B, the user still had to change all material assignments for the appropriate clusters in each phase.

Method C lead to a warning that material sets had not been assigned to all clusters. In that case, all phases had to be redefined.

In Plaxis 2D Version 9, method A and B still behave the same as in Plaxis 2D Version 8. Method C, however, does no longer lead to undefined clusters. When a stage is regenerated the regeneration process will reassign the material that was already applied in that phase. If that material data set has been removed from the material database, the regeneration process will assign the material from the geometry definition.

Note: After changing the material assignment, please check the values for the K_0-procedure: the regeneration process will use the values from the previous K_0-procedure.

Activities 2008 - 2009

October 1 - 6, 2008
12th IACMAG
Goa, India

October 6 - 8, 2008
Congres International de l'AFTES
Monaco

October 8, 2008
Funderingsdag
Ede, The Netherlands

October 15 - 17, 2008
Course on Computational Geotechnics
Paris, France

October 27 - 29, 2008
NUCGE 2008
Skikda, Algeria

October 27 - 29, 2008
Course on Computational Geotechnics
Barcelona, Spain

November 5 - 7, 2008
15th European Plaxis Users Meeting
Karlsruhe, Germany

November 10, 2008
Course on Computational Geotechnics
Newcastle, Australia

November 14, 2008
Plaxis Introduction Seminar
Auckland, New Zealand

December 17 - 19, 2008
Indian Geotechnical Conference IGC 2008
Bangalore, India

December 15 - 18, 2008
3rd Asian Advanced Plaxis Users Course
Chiang Mai, Thailand

January 13, 2009
Course on Computational Geotechnics
Berkeley, U.S.A.

January 26 - 28, 2009
International Course on Computational Geotechnics
Schiphol, The Netherlands

February 9, 2009
TAE course
Germany

March 15 - 19, 2009
IFCEE 09
Orlando, Florida, USA

March 23 - 26, 2009
International Course on Computational Geotechnics
Schiphol, The Netherlands

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