Numerical Simulations of Geotechnical Works in Bangkok Subsoil Using Advanced Soil Models Available in Plaxis and Through User-Defined Model

PLAXIS as a Tool for Soil-Structure Interaction Modelling in Performance-Based Seismic Jetty Design

Cyclic Loading of Suction Caissons
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Colophon

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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.
Looking back at 2012, at the beginning of this year the Russian Plaxis site was successfully re-launched following the renewed Plaxis brand which was launched in September 2009. Just before the summer construction of our new head office started in Delft. Plaxis will be moving to this new office next year. Furthermore throughout the year several updates and new versions of our software have been released; in July PLAXIS 3D 2012, in September a new version of PLAXIS Connect, and towards the end of the year PLAXIS 2D 2012.

Keep an eye on www.plaxis.nl for more news and information on software releases.

In this 32nd issue of the Plaxis Bulletin we have again tried to compose a nice collection of interesting articles and useful information for you. In the New Developments column we take a closer look at the new PLAXIS 2D 2012 program and specifically the new feature for embedded piles.

The third user’s article covers the development of a new gas process and production platform, on four non-braced legs, each with a suction caisson foundation, in the Dutch sector of the North Sea. A method to incorporate pore pressure build-up under cyclic loading is described. Cyclic simple shear laboratory tests were performed in order to determine the number of cycles required to reach liquefaction.

In addition to the contributions by PLAXIS users, there is again a joint presentation about a project where Plaxis has provided expert services to a client. Plaxis was contracted for a 3D simulation of a proposed two-storey detached dwelling house founded on a concrete raft. Modelling results in terms of bearing capacity and settlement predictions have been provided enabling the client to safely suggest a more cost-effective solution.

We wish you an interesting reading experience and look forward to receiving your comments on this issue of the Plaxis Bulletin.

The Editors
New Developments

Ronald Brinkgreve, Plaxis bv

With the success of the embedded pile element in PLAXIS 3D, users started to ask for a similar facility in PLAXIS 2D. However, the stress state and deformation pattern around piles is fully three-dimensional, so at first sight it seemed very difficult to develop an element in 2D that can realistically model piles. Nevertheless, we have succeeded to do so. The new 2D embedded pile element is available in PLAXIS 2D 2012.

In the past, plate elements as well as node-to-node anchors (with some sort of foot) have been used to model piles in 2D. Both have their disadvantages:

- Plate elements in 2D completely separate the soil at both sides of the element, whereas in reality, the soil is continuous and can ‘flow’ in between the pile in the out-of-plane direction. When using interface elements to model soil-structure interaction, the separation of the soil at both sides is even more pronounced and the interface does not resemble pile-soil interaction in reality. This way of modelling may only be applicable when piles have a low out-of-plane spacing \( l_s \) compared to their diameter \( d \) (e.g. \( l_s/d < 3 \)).
- Node-to-node anchors, on the other hand, do not have any interaction with the soil at all, except for the top and foot. Although the soil is continuous and can ‘flow’ independent from the pile, reality always shows some sort of interaction. Moreover, some sort of pile foot modelling is needed in order to sustain axial forces in the pile, since a single node at the foot is generally insufficient and leads to mesh dependent results.

The idea behind the 2D embedded pile is that the pile (represented by a Mindlin beam element) is not ‘in’ the 2D mesh, but superimposed ‘on’ the mesh, while the soil element mesh itself is continuous (Figure 1, after [1]). A special out-of-plane interface connects the beam with the underlying soil elements. The beam is supposed to represent the deformations of an out-of-plane row of individual piles, whereas displacements of the soil elements are supposed to represent the ‘average’ soil displacement in the out-of-plane direction. The interface stiffness should be chosen such that it accounts for the difference between the (average) soil displacement and the pile displacement while transferring loads from the pile onto the soil and vice versa. This requires at least the out-of-plane spacing of the piles to be taken into account in relation to the pile diameter. PLAXIS has default values for this interface stiffness, but users may change these values if necessary.

The new 2D embedded pile element has been tested and validated in various situations and loading conditions [1]. Tutorial examples have been updated using the new pile element to better model the situations as considered.

A particular application of the 2D embedded pile element is to use it as the grouted part of ground anchors, in combination with a node-to-node anchor representing the free length. This is a better way of modelling ground anchors than using geogrids as suggested previously, since it overcomes the force drop in the transition from the node-to-node anchor to the embedded pile.

We are confident that the new 2D embedded pile element is a valuable extension to the existing PLAXIS 2D capabilities and we are very interested to hear about your experiences with this new element.

References


![Figure 1: Schematic representation of embedded pile (after [1])](image)
TJ Consultants was looking for an alternative solution besides the use of the popular but costlier conventional pile foundation for low-rise buildings. Considering the structure load from the two-storey building is moderate, raft foundation was chosen in the design phase as it is relatively cheap and fast to implement.

However, there were concerns over fulfilling the settlements requirement as the ground exist a thin layer of soft and compressible material. The need to understand the load distribution and settlement behavior that includes differential settlement, coupled with the irregular loading distribution attributed from the superstructure, were crucial to their design decision.

In the framework of this project:
- The problem was modeled using PLAXIS 3D to account for the geometry and load distribution of the proposed raft. Hence, it is capable of computing more realistic settlement profiles as compared to two dimensional analyses.
- The performance of the raft was evaluated in short-term and long-term conditions.

Main Results
The FE analyses that have been carried out have demonstrated that the raft foundation generates fairly uniform settlements despite the irregular load distribution and varying raft thickness.

Although, the influence of settlement to the adjacent structure was small, it was found that possible excessive deformation may occur beneath the raft especially during the construction phases, partly attributed by the thin soft strata.

Revisions were made to the original design by enlarging the raft to cope with the excessive settlements. The final layout of the raft foundation was also checked to satisfy the limits of the allowable bearing capacity.

Customer Quotes
“Engaging PES services benefitted us by tapping into the computational geotechnics knowledge from PLAXIS. This enabled us to provide a cost effective and reliable solution. Most importantly, valuable time was saved and it minimized our heavy workflow.” Sarah Liew

The Company
TJ Consultants is a local consultancy in Singapore consulting mainly for local civil and structural engineering projects.

PLAXIS Expert Services Update
Sarah Liew, TJ Consultants

Plaxis was contracted by TJ Consultants, a local civil and structural consulting firm in Singapore, for a 3D simulation of a proposed two-storey detached dwelling house founded on a concrete raft. In the framework of PLAXIS Expert Services, valuable modelling results in terms of bearing capacity and settlement predictions have been provided enabling the client to safely suggest a more cost-effective solution.

“PES not only benefitted us by tapping into the computational geotechnics knowledge from PLAXIS. It also enabled us to provide a cost effective and reliable solution”

Mesh set-up seen with permanent and imposed loadings for the raft foundation
Settlement prediction
In this article, three constitutive models with enhancing levels of complexity are used to simulate three types of geotechnical works (embankment construction, deep excavation and tunneling) on/in Bangkok subsoil conditions. In dense urban environments where land is scarce and buildings are closely spaced, one of the main design constraints in these projects is to prevent or minimize damage to adjacent structures due to excessive movements induced from the construction. To eliminate or reduce the possibility of such damage, an effective method is needed to accurately “predict” the construction-induced movement for such complex condition.

To eliminate or reduce the possibility of such damage, an effective method is needed to accurately “predict” the construction-induced movement for such complex condition. The magnitude of the settlement and lateral movement and their distributions depend on a number of factors, such as soil profile and its geotechnical engineering properties, stiffness of structure and support system. The finite element method (FEM) is often used to predict ground movements induced by such soil–structure interaction problems. The interaction between existing structures and underground activities is a complex phenomenon in which the behavior of the surrounding ground is one of the main aspects to be taken into account. Consequently, a reasonable soil model is crucial in order to estimate the magnitudes and distribution of the strains. The constitutive model frequently used in numerical simulation of an underground work in current practice is linear elastic perfectly plastic with a Mohr–Coulomb (MC) yield criterion. The greatest advantage of MC is that only five parameters, which includes two elastic parameters (i.e., Young’s modulus $E$ and Poisson’s ratio $\nu$) and three plastic parameters (i.e., friction angle $\phi$, cohesion $c$ and dilatancy angle $\psi$), are sufficient in describing the behavior. Moreover, the parameters can be easily determined. However, the model does not take into account the fundamental aspects of soil behavior, such as variation of modulus according to different stress state and modulus in loading and unloading conditions or stress path dependency. Therefore, in general, the numerical results by MC are only in good agreement with particular field observation at a certain strain range for each case. To achieve good agreement with the measured values, the parameters are usually obtained from back calculation by curve fitting with history records. Although this technique could improve the accuracy of prediction in practice, it is only applicable for a specific range of each work. To overcome such shortages, it is necessary to consider at least an elasto-plastic model with isotropic hardening. In addition to previously mentioned aspects, the non-linear pre-failure initial stiffness of soil at small strain range is necessary for analysis of some geotechnical earthworks (Burland, 1989).

Recently, with the advancement in constitutive model development and implementation in Plaxis, the Hardening Soil model (HS) (Schanz et al., 1999) with small strain stiffness (HS-small) has been proven to be effective in analysis of geotechnical works. Moreover, with the advanced feature in later versions of Plaxis, the implementation of any desired constitutive model via user define-model subroutine is possible. This facilitates the engineers to utilize more advanced soil models. In this article, three constitutive models with enhancing levels of complexity are used to simulate three types of geotechnical works.
(embankment construction, deep excavation and tunneling) on/in Bangkok subsoil conditions. These include MC, HS-small and a hypoplastic (HP) model (Masin 2005). The first two models are available in the Plaxis model library, whereas, the last one was a user-defined model subroutine. First, the calibration of selected models on the basis of the extensive in situ test results for stiff clay and laboratory test results for soft clay is carried out. The analysis results are compared with triaxial test results. Then analyses of three kinds of geotechnical works are carried out with those three soil models with a single set of parameters for each model. The impact of selecting a soil model for geotechnical work analysis on accuracy of the predictions of soil displacements is highlighted.

Subsoil Condition and Soil Parameters
Bangkok Soft Clay was deposited in marine conditions at the delta of the rivers in the Chao Phraya Plain. The typical Bangkok subsoil is shown in Figure 1. It consists of made ground with a thickness of 2.0 m over a 13.0 m thick soft clay layer. The 12.0 m thick first stiff clay layer is encountered below the soft clay layer at the depth of 15.0 m to 27.0 m. Beneath the first stiff clay layer is the 8.0 m thick dense sand layer and the 6.0 m thick second stiff clay layer at the depths of 35.0 m to 41.0 m respectively. The 19.0 m thick second dense sand layer at the depth of 60.0 m is underneath these layers. The soil properties used in the analyses are mainly determined from local investigated data correlation from comprehensive in-situ tests of previous mass transit projects (Prust et al., 2005) and previous laboratory tests (Shibuya et al., 1997; Theramast, 1998; Uchaipichat, 1998).

Table 1: Soil parameters of Bangkok soft clay for MC model

<table>
<thead>
<tr>
<th>Soil parameter</th>
<th>$\gamma$ [kN/m$^3$]</th>
<th>$'c'$ [kPa]</th>
<th>$'\phi'$ [°]</th>
<th>$E'$ [kPa]</th>
<th>$R_{\text{in}}$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>MC model</td>
<td>16</td>
<td>0.33</td>
<td>22</td>
<td>5</td>
<td>10000</td>
</tr>
</tbody>
</table>

Table 2: Soil parameters of Bangkok soft clay for HS-small model

<table>
<thead>
<tr>
<th>Soil parameter</th>
<th>$E_{\text{sat}}$ [kPa]</th>
<th>$E_{\text{sat}}^*$ [kPa]</th>
<th>$E_{\text{ur}}$ [kPa]</th>
<th>$G_{\text{ur}}$ [kPa]</th>
<th>$\gamma$ [kPa]</th>
<th>$\nu$ [-]</th>
<th>$R_{\text{in}}$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>HSsmall model</td>
<td>10000</td>
<td>10000</td>
<td>30000</td>
<td>22560</td>
<td>$1\times10^{-4}$</td>
<td>1</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 3: Soil parameters of Bangkok soft clay for Masin’s hypoplastic model for clay

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>Weather Crust</th>
<th>Soft Clay</th>
<th>Medium Clay</th>
<th>Stiff Clay</th>
<th>Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma$ [kN/m$^3$]</td>
<td>17</td>
<td>16</td>
<td>18</td>
<td>18</td>
<td>20</td>
</tr>
<tr>
<td>$\nu$ [-]</td>
<td>0.32</td>
<td>0.32</td>
<td>0.33</td>
<td>0.33</td>
<td>0.33</td>
</tr>
<tr>
<td>$\phi$ [°]</td>
<td>22</td>
<td>22</td>
<td>22</td>
<td>22</td>
<td>36</td>
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<tr>
<td>$C$ [kPa]</td>
<td>8</td>
<td>5</td>
<td>10</td>
<td>18</td>
<td>0</td>
</tr>
<tr>
<td>$E'$ [kPa]</td>
<td>6000</td>
<td>5000</td>
<td>20000</td>
<td>60000</td>
<td>80000</td>
</tr>
<tr>
<td>$\psi$ [°]</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>$R_{\text{in}}$</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0.7</td>
</tr>
</tbody>
</table>

Table 4: General MC soil model parameters for Bangkok subsoil (Wonglert et al., 2008)

Note that the various soil models are applied to the only soft clay layer, whereas, other layers are assumed to behave as MC model in order to highlight the influence of soft clay model. Tables 1-3 tabulate the soil parameters of soft clay for MC, HS-small and HP model, respectively. The values of other soil layers are listed in Table 4. Figure 1 also shows the typical section and position of geotechnical works on/in Bangkok subsoil.
Numerical Analysis and Results

All problems which are from well-documented case histories having reliable monitored data are analyzed by PLAXIS 2D assuming plane strain condition with the appropriate analysis condition. These include the embankment construction (Bergado et al., 1994), deep excavation (Teparaksa et al., 1999) and tunneling (Suwansawat, 2002). The consolidation analysis is carried out for embankment construction since the monitoring data was obtained at 300 days after construction. For analyses of deep excavation and tunneling, undrained conditions are assumed since the measured data were obtained during and a few days after construction. The extension of the finite mesh is wide enough and suitable boundary conditions are assumed for all model boundaries to ensure the accuracy of the analysis.

Embankment Construction

Figure 2 depicts the finite element mesh, geometry and dimension of the problem for the embankment construction case. The case is the construction of 4.0 m high test embankment at AIT (Asian Institute of Technology) in 1992. The data to be compared is the settlement of the embankment measured at the level of the original ground using surface settlement plates for 300 days after the construction.

The comparison between measured data (red dot) and analysis results by three different soil models (color lines) as embankment loading increases is illustrated in Figure 3. It is seen that, for this problem in which the soil behavior is mainly governed by compression loading, the results from HP are best comparable with measured data. While those of HS-small and MC have acceptable agreement with measured data in low to moderate loading range.

Deep Excavation

The finite element mesh, geometry and dimension of the problem for underground construction of Bangkok Metropolitan Hospital (BMH) which is a 14 m deep excavation using 20 m high diaphragm wall as retaining structure, is depicted in Figure 4. Figure 5 shows the comparison between the FEM analysis results and observed values of the excavation works at the excavation depth of 2.0, 5.7, and 11.0 m. For this problem which the soil behaviors are mainly governed by compression unloading and extension unloading paths, it shows that the tendencies of analytical results with HP and HS-small models are satisfactory in terms of magnitude and the shape of the wall movement while those of MC model are over-estimated.

Figure 6 shows the maximum lateral wall movement from analyses compared with the observation results of all cases (Oriflame Building, MRT Bang Sue, China tower and TPI Building). It was shown that the predicted results by HP and HS-small models give a satisfactory accuracy with the observed data. Especially, when the maximum lateral wall movements are lower than 20.0 mm. It can be seen that, for the constitutive soil models which take the small-strain stiffness into the account, the lateral wall movement is accurately predicted. However, the simple constitutive soil model such as elastic-perfectly plastic (MC) is still able to accurately predict the lateral deformation only for specific strain range of each problem.

Tunneling

For tunneling work, the ground surface settlements were measured by settlement marker. The case study is construction of the Mass Rapid Transit (MRT) tunnel having an inner diameter of 5.8 m at three sections including CS-8E, CS-8C and CS-9A with the depth from ground surface at 21.0, 19.0 and 17.0 m, respectively. As an example, the boundary condition and mesh of FEM analysis for MRT tunnel at point CS-8E is shown in Figure 7. The results of ground surface settlement are shown in Figure 8 for comparisons of the FE analysis results and the observed values. It shows that the analytical results from all models give satisfactory tendencies in terms of the magnitude and the shape of the settlement profile. However, the results from HP are closed to measured values at near-centerline (0.0 - 18.0 m), whereas, those of MC are in good agreement with measured ones in the distance of 20.0 - 40.0 m from centerline.

The results of surface settlement at the tunnel center (maximum), 5.5 and 11.0 m from the tunnel center are compared with measurement data at point CS-8E, CS-8C and CS-9A as shown in Figure 9. From FE analyses, HSsmall and HS model give a highly accurate prediction for a wide range of observed settlement. The results from MC model seem to be more scattered.
Discussion
The analyses of three kinds of geotechnical works carried out in this article shows the impact of soil model on the simulations. Using more sophisticated soil models considerably improves the prediction of movements. The HP and HSmall models which include non-linearity at pre-failure and high stiffness under very small strain, particularly the HP which has stress-path dependent stiffness, give satisfactory accuracy of movement prediction for all three types of work covering the wide range of observed data. MC model over-predicts the deformation for analysis of embankment and excavation work, particularly, at range of small movement. However, the advanced models are applied to only the soft clay layer in this study. By applying to the other clay layers, the analysis results can be further improved (Rukdeechai et al., 2009).

References
• Teparaka, W., Thasnanipan, N., Tanseng, P. (1999), Analysis of lateral wall movement for deep excavation in Bangkok subsoils, Proceeding of Civil and Environmental engineering conference "New Frontiers and Challenges", Bangkok, pp. 67-76

Figure 6: Comparison between computed maximum wall displacement by different soil models and measured data of all case studies

Figure 7: FE mesh for tunneling case

Figure 8: Comparison of surface settlement at point CS-8E between measured data and analysis results by three different soil models

Figure 9: Comparison between computed settlements by different soil models and measured data of all case studies
PLAXIS as a Tool for Soil-Structure Interaction Modelling in Performance-Based Seismic Jetty Design

Floris Besseling MSc & Arny Lengkeek MSc, Witteveen+Bos, The Netherlands

The importance of soil structure interaction in seismic design of structures is recognized by the seismic design community, which is very much moving towards performance based design principles. Particularly for structures with deep foundations in soft soil conditions, soil-structure interaction is a key factor that has to be considered in design. Jetty structures can be such structures. However, seismic design standards do hardly provide any straightforward tools for engineers to account for soil-structure interaction in design. It is clear that a challenge exists, which has initiated this study. This paper addresses how Plaxis can efficiently be applied as a tool in seismic soil-structure interaction problems, which has been an important part of the entire MSc thesis study (TU Delft), that has covered a wider range of related aspects.

Jetty structures are, like many other port and offshore structures, at the interface of structural and geotechnical engineering. Traditionally the structural engineering community has relied on pseudo-static response spectrum or linear dynamic modal techniques for seismic design. Accuracy assessment of these techniques has been considering regular multi storey buildings with a fixed base. For geotechnical structures (i.e. jetties, quay walls), however these techniques are less common and their performance is less clear. Past post-earthquake surveys show typical failure modes that are often strongly dominated by differential and permanent deformations in soils. Since these effects are vital for post-earthquake performance and possibly induce global failure, they should in a way be accounted for in performance-based design.

Problem definition

The study has concentrated at performance-based seismic design of jetty structures and aspects related to numerical soil-structure interaction modelling. A Witteveen+Bos case of a jetty design project located in a high seismicity area in Turkey was taken as a starting point. The following study however has considered soil-structure interaction for piles in general, in order to be able to extend the results into a wider range of future seismic design projects and other projects relating to the dynamics of foundations. It is noted that in this study seismicity is assumed to be represented by vertically propagating shear waves, which is a common simplification adopted in the seismic engineering community. It is realized that this assumption is not generally justified, in particular for very near-fault projects.

Proposed Method

In the first stage of the study a comprehensive literature study was performed, aiming at the definition of possible performance based design strategies for jetty type of structures. Based on this literature study [references can be found in the thesis report] a flowchart has been constructed [Figure 1], that describes proposed steps to be taken by engineers to efficiently account for soil-structure interaction in design and obeying performance based code requirement in final design stages. According to literature jetty type

![Flowchart of proposed seismic design analysis methods for jetty structures](image-url)
of structures may be analyzed by means of the following procedures:

- Simplified dynamic analysis (pushover + response spectrum method)
- Uncoupled dynamic analysis of site and structure
- Coupled dynamic analysis of site and structure

For all three methodologies pushover analysis of the jetty is an important sub step that determines the nonlinear lateral load-deformation characteristics and capacity of the structure.

Towards uncoupled or coupled dynamic analysis of site and structure, site response analysis is an important preliminary step, for which both equivalent linear and nonlinear methods were addressed and compared in this study. The different steps as outlined above will subsequently be discussed in this paper.

**Pushover Analysis**

According to literature and seismic design standards, pushover analysis combined with response spectrum procedures is the most common approach for seismic design of regular structures (often referred to as simplified dynamic analysis). Since this approach relies on pre-assumed failure modes it requires caution when applied to irregular structures or other structures for which the dominant dynamic modes cannot easily be identified. Structural engineering community often accounts for soil in their models by simplified Winkler p-y springs. The characteristics of these springs are commonly based on decades old codes, which are based on limited experimental data. In this study the common code-based p-y expressions were verified by PLAXIS 3D numerical analysis of single piles embedded in layered soil.

In pushover-analysis the hardening soil (HS) constitutive model was adopted in order to account for hardening plasticity and stress dependent stiffness characteristics, as are observed for real soils. In order to identify the most likely HS input parameters based on limited soil survey, a large number of correlations from literature where included in parameter selection for both sand and clay materials. The pile was modelled by means of a combined plate and solid in order to be able to easily identify pile bending moments and also properly account for geometric aspects of the circular shaped pile interacting with the soil. Point of attention in the modelling of these circular shaped elements has been the locking of interfaces, which may result in overestimation of pile stiffness.

According to performance based design principles, most codes allow pile deck systems to develop limited plastic hinging at the fixed pile heads during high intensity earthquakes to dissipate energy and prevent global collapse. In the recent release of PLAXIS 3D however plasticity of plates is not included. A workaround is found by an artificial plastic hinge by means of elastic-plastic anchors to include the important local nonlinear response at the pile connection to the deck.

Based on a large series of pushover analysis, including a parametric study of variations in HS input parameters, it was concluded that the commonly applied Matlock p-y expressions for soft clays have a too low initial stiffness and ultimate capacity. This conclusion corresponds to the conclusion drawn by Jeanjean [1], who recently has proposed alternative p-y expressions for soft clays. The Jeanjean p-y expression where compared to the results from PLAXIS 3D pushover analysis. An almost perfect fit on both global pile and local pile-soil level was obtained, as is indicated by figure 2 that shows the bending moment distributions along
The verification of p-y correlations by PLAXIS 3D analysis has been identified as an important and efficient calibrated sub step towards the application of these Winkler p-y springs in simplified soil-structure dynamic analysis. Additionally, also group effects were studied. Group efficiency factors for both transversely spaced and shadowing piles proposed in literature where assessed by PLAXIS 3D pile group analysis. While group efficiency reductions proposed in literature were found to be strongly varying, Plaxis 3D pushover analysis was identified as a very efficient and useful tool to find case specific group efficiency reductions, taking into account various contributing factors like e.g. pile diameter, spacing and stiffness and soil types and layering.

**Response Spectrum Analysis**

The jetty transverse lateral load-deformation characteristics resulting from pushover analysis on both PLAXIS 3D and Winkler p-y jettys-soft models were included in a performance-based response spectrum procedure as proposed by Fajfar (N2-method) [2]. Although the Fajfar method is in itself purely analytical, the relation of pushover capacity and spectral demand is nicely represented in a purely analytical, the relation of pushover capacity method) [2]. Although the Fajfar method is in itself purely analytical, the relation of pushover capacity and spectral demand is nicely represented in a graphical acceleration-displacement response spectral format, by utilizing the acceleration – pseudo displacement relationship:

\[
\frac{\ddot{u}}{\ddot{u}_w} = \frac{1}{\omega^2} = \frac{\gamma_o}{4x^2}
\]

The response spectrum demand was based on ISO 19901-2 regulations, as a function of the bedrock acceleration level, the upper 30 m soil deposit characteristics and the structure estimated fundamental period, as is common in most seismic design codes. It is noted here that the Fajfar N2-method relies on the equal-displacement rule for elastic response spectrum reduction beyond the elastic range. Alternatively, the equal-potential-energy criterion can be applied, as was also considered in this study. For the jetty structure (with fundamental period T0 = 1.25s) no significant difference in results was obtained, as was to be expected based on the fundamental frequency according to Miranda & Berteno [3].

In the present study response spectrum analysis was included as a reference solution and it was decided to stick to a practical single-mode response spectrum procedure, where more advanced multi-mode (adaptive) methods are available.

**Site Response Analysis**

As was shown in figure 1, in this study finally two jetty dynamic analysis methods were studied and compared, being uncoupled and coupled dynamic analysis. For both methods preliminary site response analysis is an important sub step.

For the uncoupled variant the soil deposit dynamic responses to an applied bedrock signal are determined at different depths, which subsequently are applied to Winkler support nodes of the nonlinear p-y springs that account for near field pile-soil interaction in the structural dynamic model.

Towards coupled dynamic analysis of site and structure in PLAXIS 3D, the site response analysis forms an important calibration step of the finite element model.

Different recorded bedrock horizontal motion signals were selected, filtered and scaled before being applied to the bottom of the finite element model. In a large series of dynamic site response analyses first the model was calibrated with respect to element size, dynamic time stepping the time integration scheme and boundary effects. To this extend initially a linear elastic soil constitutive model was applied, for which the soil deposit response obtained from the numerical 2D PLAXIS model should converge to the frequency domain solution for 1D shear wave propagation problem through layered soil, provided that similar dynamic characteristics are assigned to the soil.

Adopting a damped Newmark time integration scheme was found to be essential for a stable solution were it only has a very limited effect on the calculated response amplitude, as was also concluded by Sigaran de Loria and Jaspers-Focks [4]. Boundary effects were studied and compared with the 1D frequency domain analysis solution and a PLAXIS 2D model with tied boundaries. Also a comparison of responses obtained from a PLAXIS 2D plane strain model and a PLAXIS 3D soil slice model was made, in order to verify the PLAXIS 3D dynamics module performance.

Frequency domain analysis of shear waves propagating vertically through equivalent linear layered soil was coded in Matlab, based on the theory as outlined in Kramer (1996) [5]. In equivalent linear frequency domain analysis effective modulus reduction and equivalent damping are assumed to be constant over time, corresponding to a shear strain averaged over time. Various expressions for these modulus reduction and damping curves are available in literature, where in this study expressions according to Hardin & Drnevich were applied, after being verified in a comparative study including relationships proposed by Hardin & Drnevich [6], Vucetic & Dobry [7], Ishibashi & Zhang [8] and Santos & Correia [9].

After the PLAXIS 2D plane strain finite element model was calibrated with respect to the model...
basic issues mentioned before, the focus was shifted towards the performance of the various soil constitutive models available in Plaxis when applied in dynamics. Herein the focus was on the performance of the Hardening Soil model with Small strain stiffness (HSsmall) and soil modulus reduction curves and damping characteristics as a function of cyclic shear strain. The HSsmall includes hysteretic damping as a function of strain amplitude and hence is conceptually very attractive to be applied in dynamic problems. However, in this study the HSsmall model was found to have a poor performance in dynamics when applied to shallow soft soil layers having low stiffness due to low local stress levels. More specifically can be stated that the reset of the HSsmall stiffness at deviatoric principal strain rate reversals may for these conditions result in unrealistic development of accelerations as is shown in figures 6 and 7 for a specific dynamic time interval. The suddenly changing stiffness matrix in the equation of motion forces the acceleration vector to undergo sudden changes as well. In reality this behaviour will not be observed since a finite time interval is related to the stiffness development, where in the HSsmall numerical model it is not. Additionally the relatively high $G_v / G_u$ ratio, as typically applies to soft to medium clays, was found to further deteriorate the HSsmall performance for these type of soils.

Remedial measures to improve the HSsmall performance were sought. Removing the stress dependency of shallow layers (by setting HSsmall stress dependency parameter $m = 0$) and assign a constant stiffness to shallow layers was found to be the most effective measure. Doing so allows engineers to benefit from the HSsmall hardening plasticity and hysteretic strain dependent damping features, while minimizing negative consequences of HSsmall performance for low stiffness soil layers.

Compared to the response calculated by equivalent linear frequency domain analysis, Plaxis nonlinear HSsmall site response analysis results in lower peak acceleration levels of the soil deposit at high intensity shaking, but higher response levels at low intensity shaking. This is explained by much higher damping levels at these high intensity motion intervals and effects of plasticity that limit peak acceleration responses. This behaviour may be considered more realistic for real soils that also show failure and permanent deformations during earthquakes.

During the last step of the study presented in this paper the dynamic response of a jetty transverse cross-section was calculated. As explained before, both uncoupled and coupled analysis of site and structure were performed. In the uncoupled approach, the structure response was calculated with Seismostruct, which is a structural finite element code specifically suitable for structural seismic design purposes. A structural pile-deck model supported by Winkler springs was built. The complex Winkler spring characteristics were obtained by combining springs calibrated by static pushover analysis with parallel dashpots according to Gazetas & Dobry [10, 11]. The dynamic response of the structure then was calculated for imposed Winkler support node excitations that were derived from separate site response analysis by either nonlinear PLAXIS 2D site response analysis or equivalent linear frequency domain analysis.

For coupled dynamic analysis of soil deposit and structure, a single PLAXIS 3D finite element model was built including both the soil deposit overlying bedrock and the jetty structure cross-section. With this model the coupled dynamic response was calculated. The geometry of the coupled system is presented in figure 8.

Although the number of elements was minimized as far as possible, accuracy and stability requirements resulted in a finite element mesh consisting of approximately 60000 elements and a maximum allowable time stepping of 0.003 s for a 30 s seismic input signal. On a modern pc a single run of these type of coupled dynamic analysis takes about 3 days and the required model calibration takes weeks. Hence it may be concluded that for general seismic design projects the full coupled analysis computational effort still is a factor limiting its applicability. Additionally it is noted that a strong signal-dependence of response levels was obtained, based on which a larger number of input signal time histories than proposed in seismic design codes is to be recommended, further increasing engineering effort.
Uncoupled structure responses obtained based on equivalent linear site analysis however were found to deviate significantly from responses calculated from coupled analysis. This may be explained by development of extreme acceleration level variations of the soil along the pile, not being limited by global or local soil failure. Besides, permanent deformations calculated from this uncoupled approach differ significantly from the full nonlinear analysis results, at least for the high acceleration levels considered in the present study.

From the results of the dynamic analyses it was concluded that the uncoupled type of dynamic analysis as proposed in this study may be a reasonably accurate and relatively efficient tool in performance based seismic design, especially for irregular pile-deck type of jetty and wharf structures where torsional effects may be dominant for response.

Comparison of peak responses from dynamic analysis and simplified dynamic analysis
Peak displacement demands calculated by simplified dynamic response spectrum analysis were found to be similar to the peak displacement demands calculated by coupled and uncoupled nonlinear dynamic analysis. Therefore it was concluded that simplified dynamic response spectrum analysis for jetty type of structures is the tool to be used in preliminary design stages. It is however noted that additional uncoupled dynamic analysis definitely is to be preferred in final design stages in order to be able to identify unexpected failure modes and estimate permanent displacements, the latter of which is limited by modern performance based seismic code requirements.

Conclusions and Recommendations
The study presented in this paper considers a seismic design problem at the interface of structural engineering and geotechnical engineering, for which straight forward design procedures are very limited. Along modern performance-based design principles a design strategy was defined for jetty type of structures.

Finite element modelling with PLAXIS 2D and 3D and the dynamics modules has been a key aspect in; calibration steps, preliminary static analysis, and dynamic analysis in final design stages. As is often the case for seismic design projects, no verification of models by measured responses during earthquakes was available. Hence it was a key issue in the present study to verify all sub steps in order to prevent black-box finite element analysis. During this study it was obtained that PLAXIS 2D and 3D are very useful tools in different stages of seismic jetty design. Engineers should still be aware of their limitations and the need for verification of results.

PLAXIS 3D was found to be a powerful tool for static verification of equivalent Winkler foundations for pile groups towards dynamic analysis. However, for high intensity earthquake design utilizing nonlinear structural behaviour of plates should preferably be included in new releases of PLAXIS 3D. PLAXIS 2D may be applied to perform nonlinear site response analysis in the proposed uncoupled dynamic analysis approach. PLAXIS 3D coupled dynamic analysis of site and structure resulted in excessive computational demands which still is a limiting factor for general application.

The proposed approach supplies engineers with a design strategy better fitting to modern code requirements compared to traditional methods. When applying nonlinear time domain analysis, one should be aware of the high sensitivity to the selected seismic input signal, and its intensity. Based in the present study the authors recommend to calculate dynamic responses for a higher number of input signals than the relatively limited number ranging from 3 to 7 as typically required by international seismic design codes.

As a last remark it is noted that the focus of this study has been on jetty structures, but the typical dynamics of large end-bearing shafts in soft soils are relevant for different types of onshore and offshore structures as well. Further development of knowledge, concepts and methodologies is being planned by Witteveen+Bos for the near future.
More information, the thesis report, questions or request for corporations with Witteveen+Bos, please contact: f.besseling@witteveenbos.nl or h.lengkeek@witteveenbos.nl

References

Cyclic Loading of Suction Caissons

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Venture North Sea Oil Ltd. (Venture/Centrica) has developed a new gas process and production platform in Block F3 of the Dutch sector of the North Sea, see Figure 1. The platform topside is founded on four non-braced legs, each with a suction caisson foundation. This project was executed by Heerema Vlissingen (HEVL), IV Oil & Gas (IVoG) and SPT Offshore (SPT). The SPT scope included the suction caisson geotechnical and structural design and fabrication, followed by transport and installation of the complete platform. The geotechnical and structural design of the suction caissons have been carried out by SPT and inhouse design department Volker InfraDesign (VID). The platform was successfully installed in September 2010.

Each suction caisson is a 15.0 m diameter, 13.0 m high steel cylinder with a dry mass of approximately 400 metric tonne. The target penetration is 12.0 m below seafloor. The water depth is about 41 m. The soil consists of silty medium to very dense sand. Several metres below target penetration depth stiff clay is found.

Problem Description
Within the offshore industry different foundation concepts are known. A suction caisson foundation is one of them. The application of a suction caisson is based on proven technology. Advantages of suction caissons above other foundation concepts are that no piling hammers or welding is required and easy and complete retrieval or removal after installation and/or use is possible. Venture/Centrica selected suction caissons, as their aim is to deplete marginal fields for as much is economically viable and technically possible. Using suction caissons, a single platform can be re-used for several fields.

A specific item of the overall geotechnical foundation design is the response to cyclic loading. The importance of cyclic loading effects is mentioned in a variety of design guidelines. How to deal with the effects from cyclic lateral loading of open-ended pipe piles or gravity base structures for example is presented in DNV and API design codes. Elaboration on how to properly assess effects from cyclic loading (degradation of strength and stiffness due to both axial, lateral and moment loading, pore pressure build-up) taking
into account the load spectrum and the total number of cyclic loads, however, is not thoroughly specified.

**Cyclic Loading**

For cyclic loading of suction caissons no direct guideline is available. Cyclic loading effects may include:

1. strength and stiffness degradation due to cyclic displacements (axial and lateral movement of the caisson along the soil interface) and
2. excess pore pressure build-up due to cyclic shear.

Cyclic loading may therefore have serious consequences for foundation integrity and should be accounted for in foundation design. This paper describes how the response to cyclic loading of the Centrica F3FA platform suction caissons has been assessed by using cyclic shear test data in conjunction with 3DFoundation finite element calculations. Finite element software is used due to the complex foundation loading, i.e. a combination of vertical, horizontal and moment (VHM) loads. From the resulting stress distribution, excess pore water pressures are determined which form the input for updated capacity calculations in the 3DFoundation model.

**Cyclic Degradation**

Axial and lateral cyclic loads may lead to strength degradation along the soil-pile interface and/or large cumulative displacements. A foundation subjected to cyclic loading should be designed for effects associated with cyclic degradation. Several cyclic loading model tests on suction caissons installed in sand have been conducted by e.g. Byrne (2000), Feld (2001), Watson et al. (2005) and Senders (2009). In general, axial cyclic degradation was only found when the foundation was cyclically loaded close to the maximum soil resistance and proved especially relevant for tension loading (i.e. where the top plate, for drained loading, does not contribute to capacity). All suction caisson loads for the Centrica F3FA project are compressive; no tensile loads are encountered during the operational life of the structure. Limited
Cyclic Loading of Suction Caissons

Axial cyclic loading (compared to the suction caisson capacity) resulted in a cyclic displacement amplitude that does not reach the displacement required to cause static slip. For compressive loading, additional displacement of the suction caisson with its top plate embedded in the seafloor results in a dramatic increase in capacity (due to partly mobilising top plate end bearing). Hence, stiffness degradation effects were found to be not relevant.

Due to the high rotational stiffness of the foundation superstructure combination, limited displacements (both rotational and lateral) are anticipated. Moreover, for lateral loading, the tolerated lateral displacements of the large diameter suction caissons were significantly smaller than the displacements required to mobilise sufficient lateral resistance resulting in degradation.

Pore Pressure Build-up During Cyclic Loading

During cyclic loading, loose saturated non-cohesive soils (predominantly sands, but also silts and some gravels) exhibit contractant behaviour when subjected to shear, resulting in a pore water pressure increase $\Delta u$ (i.e. a reduced effective stress) and consequently, a decreased shear strength. When excess pore pressures equal the vertical effective stress $\sigma'_v = u/s'_n = 1.0$, liquefaction occurs. This results in the saturated soil going from a solid state to a liquefied state. In general, loose to moderate saturated granular soils with poor drainage, such as silty sands or sands containing lenses of impermeable sediments, are more prone to liquefaction than dense sands.

To assess pore pressure build-up, either undrained cyclic triaxial tests or undrained cyclic direct simple shear laboratory tests can be carried out. For the Centrica F3FA project, a series of undrained cyclic simple shear tests were carried out on representative soil samples reconstituted to the appropriate relative density. The large number of cyclic loads on the offshore platform is induced by wave loading. A representative wave period for design conditions is approximately 10 seconds ($f = 0.1$ Hz), and was used for the cyclic shear tests.

The cyclic shear stress ratio (CSSR) for a cyclic simple shear test, which is commonly used in earthquake engineering, is defined as follows:

$$CSSR = \frac{\Delta \tau}{\sigma'_v}$$

In which:

- $\Delta \tau$ = shear stress amplitude [kPa],
- $\sigma'_v$ = initial effective vertical stress [kPa].

The relation between the number of cycles to reach liquefaction and the shear stress amplitude is described by the function:

$$N_{liq} = \frac{CSSR}{a \cdot I_D}$$

In which:

- $N_{liq}$ = number of cycles to reach liquefaction for an undrained condition [-],
- $CSSR$ = cyclic shear stress ratio [-],
- $a$, $b$ = empirical constant [-].

Figure 4: Residual pore pressure build-up (red line)

Figure 5: Relative pore pressure build-up for different conditions

Figure 6: Back-calculated pore pressure build-up
In order to determine the empirical constants $a$ and $b$, a series of cyclic tests with varying shear stress amplitude and relative density is required. When assessing pore pressure build-up for undrained conditions, one mainly focuses on residual pore pressure build-up, i.e. pore pressure build-up remaining after cyclic loading stops. This is the red line in Figure 4.

The relative excess pore pressure during cyclic loading may be assessed from:

$$ R_u = \frac{2}{\pi} \arctan \left( \frac{N}{N_{\text{liq}}} \right) $$

In which:
- $R_u$ = relative excess pore pressure or pore pressure ratio ($u/s_n'$) [-],
- $N$ = elapsed number of cycles [-],
- $\theta$ = empirical constant [-].

The following variables are important when assessing cyclic soil behaviour at field conditions and should be investigated before cyclic laboratory tests are carried out:
- Expected cyclic shear stress ratio (CSSR [-]): increase in CSSR leads to a decrease in $N_{\text{liq}}$.
- Relative density ($I_D$, $D_R$ or $R_e$ [%]): increase in $I_D$ leads to an increase in $N_{\text{liq}}$.
- Presence of initial shear stresses (contraction) prior to cyclic loading ($\tau_c$ [kPa]): generally some initial contraction due to $\tau_c$ leads to an increase in $N_{\text{liq}}$.

At field conditions the following mechanisms may also be present. These mechanisms, however, prove difficult to implement in standard cyclic laboratory tests:
- Effects from partial drainage during cyclic loading (especially for small sized suction caissons and/or short loading periods).
- Effects from compaction during cyclic loading.

Drainage time (consolidation time) may be relatively long for large diameter suction caissons. Moreover, due to the uncertainty in the permeability of the silty sand layers, a cautious approach for the Centrica F3FA project was adopted, namely a fully undrained soil response. Figure 6 shows the back-calculated pore pressure build-up for some of the cyclic simple shear tests.

From determination of the liquefaction potential (increase of $R_u$ over $N$) one is able to produce a diagram showing the relationship between the CSSR and the total number of load cycles required to achieve a relative pore pressure ratio $R_u = 1.0$.

By means of finite element calculations (in this case using 3DFoundation) it should be verified which CSSR values should be accounted for during the 100 year design storm. This is done on the basis of the stress distribution ($\Delta \tau / \tau_n'$) resulting from the series of cyclic loads; in this case a 6-hour Hansteen wave distribution. Since standard soil models, as available in 3DFoundation, are not capable of predicting excess pore pressure build-up, a method to assess pore pressures is elaborated.

Assessment of pore pressure build-up:
1. per series of wave loads ($F_1$, $F_2$, $F_3$, et cetera), calculate the associated CSSR (in depth) for applicable stress points (Figure 9 shows the distribution of CSSR in depth for 25% - 100%...
Cyclic Loading of Suction Caissons

of the maximum cyclic load resulting from the Plaxis analyses; 2. determine \( N_u \) values for the applicable CSSR values and relative densities for each soil cluster (example Figure 7); 3. calculate the increase of relative pore pressure \( \Delta R_u \) from \( N_u/N_{liq} \), in which \( N_u \) represents the total number of cycles for the first series of wave loads \( F_1 \) (example Figure 6); 4. the new (increased) value of \( R_u \) determine the number of equivalent cycles represented by the CSSR induced from the subsequent wave load series \( F_2 \); i.e. the relative pore pressure ratio for 100 cycles of \( F_1 \) may be equivalent to 60 cycles of \( F_2 \), one starts the series of subsequent wave loads at \( N_2 = 60 \); 5. calculate the relative pore pressure increase \( \Delta R_u \) for the total number of associated \( N_2 \) cycles (according to Figure 6); 6. repeat from step 4, et cetera.

The governing loading situation can either be the foundation subjected to the largest load (in this case \( F_3 \) in Figure 8) or subjected to a smaller load but with generally higher excess pore pressures \( (F_4 \text{ in Figure 8, et cetera}) \). A simple way to model pore pressure build-up (due to \( R_u \)) is to reduce the effective soil strength. Since the submersed unit weight of sand is about the same as the unit weight of water, this reduction can be reasonably accounted for by applying the following formula:

\[
\phi_{\text{reduced}} = \arctan((1 - (R_u)) \cdot \tan \phi_{\text{valid}})
\]

In case \( R_u = 1.0 \) a fully liquefied state of the sand is to be taken into account. On the basis of literature review it was concluded that the shear strength of liquefied sand is approximately 5% of the effective stress, see e.g. Stark and Mesri (1992), Olson (2001).

3DFoundation Capacity Analysis

The finite element ground model has four soil layers. A surface 9 m thick medium dense sand layer is underlain by 6.8 m of very dense slightly silty sand. Below these sand layers, two stiff clay layers are found, see also Table 1. The dilation angle \( \phi \) equals 30° and the \( R_{\text{liq}} \) value equals 0.75. The model is based on Mohr-Coulomb. The total model depth is 30 m. Around the 12.0 m long, 15.0 m diameter 0.055 m wall thickness caisson, the finite element mesh is locally refined. Using 3DFoundation, soil parameter values can be adapted if required, e.g. to accommodate the reduced internal friction angle described above.

Capacity calculations are therefore characterised by pseudo-static loading.

Based on initial analyses, the first metre below seafloor is the only location where considerable relative excess pore water pressures are anticipated. This is the result of high shear stress levels and low overburden pressure. Due to the short drainage path length, however, the effect of this excess pore water pressures is marginal and not taken further into account.

The soil capacity for static loading is compared to the cyclic loading case. For this cyclic loading analysis the soil strength parameter \( \phi' \) is reduced to \( \phi_{\text{reduced}} \) for the applicable layers. Incorporating reduced soil strength in the model effectively reduces the capacity of the soil. Consequently, the factor of safety decreases. Analyses show that, compared to the static case, the factor of safety for the cyclic load case reduces by approximately 10%.

Concluding Remarks

Cyclic axial and lateral loading may lead to a reduction of soil strength. This may adversely affect the suction caisson capacity. This paper describes a method to incorporate pore pressure build-up under cyclic loading. Cyclic simple shear laboratory tests were performed in order to determine the number of cycles required to reach liquefaction. For storm conditions (e.g. Hansteen), excess pore pressures resulting from a series of cyclic loads have been assessed. In conjunction with 3DFoundation finite element analyses a soil strength reduction has been determined which was incorporated in the geotechnical foundation design for the F3PA platform.

Table 1: Soil models parameters

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Layer (m)</th>
<th>( E_{soil} ) (MPa)</th>
<th>( E_{max} ) (MPa/m)</th>
<th>( c' ) or ( c_u ) (kPa)</th>
<th>( \phi' ) (°)</th>
<th>( \phi_{\text{reduced}} ) (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top</td>
<td>Bottom</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Medium dense sand</td>
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<td>9.0</td>
<td>0.4</td>
<td>7</td>
<td>0.01</td>
<td>35</td>
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<tr>
<td>Very dense sand</td>
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<td>15.8</td>
<td>64</td>
<td>3</td>
<td>0.01</td>
<td>42</td>
</tr>
<tr>
<td>Stiff clay</td>
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<td>18.8</td>
<td>26.5</td>
<td>0</td>
<td>105</td>
<td>0</td>
</tr>
<tr>
<td>Stiff clay</td>
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<td>30.0</td>
<td>18.8</td>
<td>0</td>
<td>75</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 1: Soil models parameters

References

• Det Norske Veritas (DNV), DNV Classification Notes no. 30.4, February, 1992.
Figure 10a: Excess pore pressure distribution for 100% of the cyclic load

Figure 10b: Example of deformed mesh for a moment load case
Recent Activities

Plaxis New Headoffice
Plaxis bv will be moving to a new office next year. Just before the summer construction of this new office has started, with the first pile driven on the 11th of May 2012. Of course Plaxis employees, family and friends wanted to be there for this special moment.

The new office will be three storeys high, and the first pile is one of 72 piles in total that were used. The highest point of the building was reached in early July. An interesting fact is that for the new building the heating will be achieved through geothermal energy, for which the special wells have already been dug. We will keep you updated on further developments of our new office as construction continues.

Plaxis USA Update
Spring of 2012 saw many activities in the field of earthquake geotechnics. First of all, two free short seminars were organized on the US West Coast in the last week of March focusing on the state of PLAXIS in the field of earthquake engineering. These short seminars marked the release of the 3D Dynamics module; other topics were soil-structure interaction, material models, and liquefaction risk assessment. A total of over 60 engineers attended these short seminars in San Francisco and Seattle.

Plaxis was present at the exhibitions of ASCE’s Geo-Congress and DFI’s Liquefaction seminar. The latter was held in St. Louis, Missouri and mainly saw attendees coming from the New Madrid Fault Zone, the “other” seismic active US region. There was particular interest in PLAXIS’s capability in the field of material models, eg HSsmall and UBC Sand, and in applying LRFD methodology by using PLAXIS’ Design Approach feature.

A three day advanced course was organized in New York City in June. This was the first US-based advanced course in many years; topics included: advanced soil modeling, pore pressure, groundwater flow, rock mechanics, and 3D modeling. Based on the number of attendees and the positive feedback received we plan to organize another advanced course in the not too far future.

Recently there was a lot of interest in PLAXIS 3D 2012, including many US-based 3DFoundation users upgrading to PLAXIS 3D 2012. Growing number of PLAXIS 3D users can be found in the offshore field (see suction anchor study in the Knowledge Base), and in seismic evaluation of nuclear power plants.

Recent Activities in Asia
In July Plaxis Asiapac has attended the 47th National Conference on Geotechnics at Hachinohe, Amori, Japan. Furthermore several special workshops and courses were organized over the last several months:
- June, a PLAXIS seminar in Jakarta, and a 2 days PLAXIS 3D workshop
July; a 2 days special workshop in Tokyo, Japan.
August; a 3 days Plaxis Standard course in Seoul, South Korea, and a short seminar in University of Busan on the 13th August 2012.
September; a successful 3 days Plaxis advanced course conducted in National University of Singapore, and a 3 days Plaxis Standard course in Mumbai, India.

Another special event Plaxis Asiapac and Taiwan Geotechnical Society have co-hosted was the successful Plaxis seminar in June. The seminar was attended by over 100 ppl.

We look forward to more successful events for the rest of this year and in the future.

Product Updates
Throughout the year several updates and new versions of our software have been released. In July PLAXIS 3D 2012 was released, the new version includes the first update for the 3D Dynamics module that was launched last year. A major addition is the total restructuring of the phases explorer including a new phases window giving a better overview of the calculation phases. Furthermore capabilities to extrude a polycurve along another polycurve has been added allowing you to create curved tunnels and other complex objects. Other new features include support for connections, parallel processing in the mesher, and the new output model explorer to match the input model explorer for easy navigation through the results.

In September a new version of PLAXIS Connect has also been released, making it an even more flexible updating tool. Go to the applications tab within the program to check the availability of updates.

Furthermore PLAXIS 2D 2012 will be released. This new version will have several new features with a major addition being the embedded piles feature, the New Development column gives a more in depth look at this new feature.

Keep an eye on www.plaxis.nl for more news on updates and software releases.

Russian Site
At the beginning of this year the Russian Plaxis site was successfully re-launched following the renewed Plaxis brand which was launched in September 2009.

The new site offers the viewer up to date information on Plaxis products and services, as well as the latest news, an overview of upcoming events in Russia, and the possibility to contact Plaxis representatives in Russia. Furthermore our Plaxis videos will be translated to Russian and also posted on the site regularly.

Check out the new site at www.plaxis.ru
Activities 2012

October 3, 2012
Funderingsdag 2012
Ede, The Netherlands

October 3, 2012
Workshop on the Use of PLAXIS 3D
Delft, The Netherlands

October 10 - 11, 2012
PLAXIS Training - Introduction to PLAXIS 3D
Singapore

October 16 – 19, 2012
37th Annual Conference on Deep Foundations
Houston TX, U.S.A.

October 23 - 26, 2012
Curso de Geotecnia Computacional
Santiago, Chile

October 24 - 25, 2012
PLAXIS Training - Introduction to PLAXIS 2D
Singapore

November 6 – 9, 2012
Pratique Éclairée des Eléments Finis en Geotechnique
Paris, France

November 6 – 9, 2012
Advanced Course on Computational Geotechnics
Hong Kong, China

November 7 – 9, 2012
13th World Conference of ACUUS
Singapore

November 14 – 16, 2012
European Plaxis Users Meeting
Karlsruhe, Germany

November 9, 2012
PLAXIS Workshop on the use of PLAXIS 2D
Earthquake Geotechnical Analysis
Singapore

November 20 – 23, 2012
Standard Course on Computational Geotechnics
Churchill, VIC, Australia

November 26 – 28, 2012
Advanced Course on Computational Geotechnics
Perth, Australia

December 5, 2012
PLAXIS Workshop on the use of PLAXIS 2D
Earthquake Geotechnical Analysis
Oslo, Norway

Activities 2013

January 13 - 17, 2013
TRB 92nd Annual Meeting
Washington, D.C., U.S.A.

January 14 – 18, 2013
Standard Course on Computational Geotechnics
Schiphol, The Netherlands

February 25 – 27, 2013
FE in der Geotechnik - Theorie und Praxis
Ostfildern, Germany

March 3 - 6, 2013
Geo-Congress
San Diego, U.S.A.

March 11 – 14, 2013
Advanced Course on Computational Geotechnics
Schiphol, The Netherlands