ROCK-FILL DIMENSIONING, CONSTRUCTION AND TESTING FOR FOUNDATION OF A JACK UP ON A SLOPING SEABED

S. Rességuier* and L. Hamre,
Marine Structures Department, Foundations Section,
DNV GL, OSLO

** corresponding author: sebastien.resseguier@dnvgl.com*

ABSTRACT

A rock-fill for support of Jack-up foundations have been constructed at the Nymo Yard in Eydehavn in Southern Norway to enable stationing of Jack-ups close to the quayside during top-side installation and completion work. The main challenges for dimensioning of the fill were the seabed bathymetry with a sloping seabed, soil properties with soft clays and outcropping rock. In addition, to get sufficient bearing capacity, a consolidation load exceeding the weight of the planned rock fill was required. This was not possible to achieve before first jack-up arrival and jacking system testing. Therefore, the jack-up weight was used as consolidation load, and an appropriate construction sequence and load stepping of the Jack-up spud-can loads was determined. The rock-fill stability during construction of the fill and under load application from the jack-up was evaluated with 2D and 3D FEM analyses. The analyses focused on stability and excess pore pressure generation throughout the whole process.

The soil investigations were rather limited; therefore, the excess pore pressure build-up was closely monitored by use of piezometers. The piezometer monitoring together with soil settlements measurements were used to validate the seabed stability before each next construction or jack-up testing stages.

A comparison between the excess pore pressures predicted with the FEM analyses and the real measured values showed good agreement and gave a good confidence in the rock-fill design.

KEY WORDS: Jack-ups, Geotechnics, Spud-can, FEM analyses, Monitoring, Slope stability, Rockfill

INTRODUCTION

The Nymo Yard in Eydehavn, Norway, has been hosting a large 4-legged accommodation jack-up for its top-side installation and completion as well as spud-can add-ons installation and jack-up leg testing. The yard is located in a strait, close to shore, with sloping seabed consisting of soft clay of variable thickness overlying bedrock. To be able to install the spud-can add-ons and later test the jacking system, a level and stable seabed was necessary. A rock-fill consisting of 4 gravel pads of varying thickness together with counter fills to ensure stability was therefore constructed.

Several conditions needed to be satisfied by the gravel pads. It needed to be able to support up to 8000t leg loads, have a sufficiently stiff response to leg loading to avoid differential settlements and subsequently excessive RPDs (Rack Phase Difference) and have a fine enough top layer to ease skirt penetration and avoid damage to the spud-can skirts. In addition, due to time constraints, short time was allowed for the soft clay below the rock-fill to consolidate and the jacking system testing needed to be done in a short enough time frame to allow for on-time delivery of the jack-up. To comply with these demanding conditions, a continuous monitoring of the excess pore pressure development and dissipation during construction and jack-up loading was required. Several piezometers were therefore installed in the soil below the fill. The measurements from these where then used to calibrate the soil consolidation parameters and monitor that sufficient soil consolidation had taken place before each increase in leg load on the gravel pads was decided on.

LOCATION AND SOIL DATA

The Nymo Yard is located in Eydehavn on the North-Sea coast in southern Norway. The yard is placed in the sheltered Tromøy Strait. The seabed at the site is significantly sloping towards the center of the strait with large variations in soil cover over the bedrock. An overview map of the location is shown in Figure 1. The original water depth at the planned jack-up location varies from around 32m to around 38m. The seabed slopes over the planned jack-up locations are steep and the bedrock below the soil cover has a very variable topography with some locations where it is even outcropping.

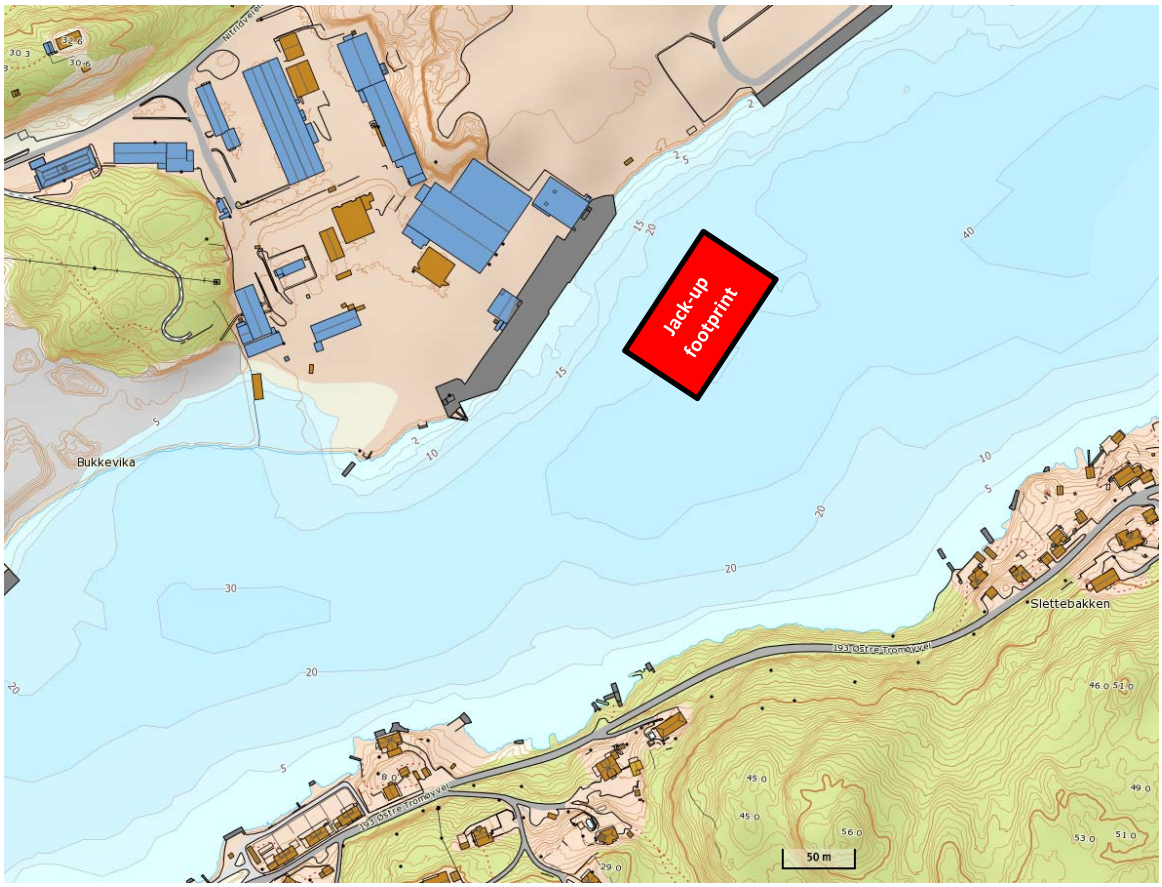


Figure 1 Overview map of the Tromøy Sound with approximate planned Jack-up location

The soil investigation consisted of 14 total soundings, 8 boreholes with piston core sampling in 5 of the boreholes and grab sampling in the remaining. The soil conditions at the location consist of a layer of very soft to soft silty clay with thickness varying from 1 to 5m overlaying a soft to medium stiff silty clay layer with thickness ranging from 1 to 15m. Organic material was found at seabed at some of the test locations. The depth to bedrock was found to range between 3 and 19m and a layer of till with varying thickness was found to be present between the bedrock and the soft to medium silty clay.

The soil layering and depth to bedrock is found to be very variable across the site. A summary of the soil layer properties is shown in Table 1.

Due to the little time available for the soil investigation and the difficult weather conditions during the SI campaign, it was not possible to obtain a sufficient number of good quality samples to be able to establish with good confidence the soil permeability. In addition, little time was available between the rock fill construction and jack-up testing and the jack-up test needed to be carried out over a short time. The soil permeability was therefore an important parameter to assess the necessary consolidation time for the rock-fill and consolidation time between each load test steps for the jack-up. It was decided to install piezometers at the location so that the permeability measurements could be calibrated and the excess pore pressure

generation during rock-fill loading monitored. In a first installation campaign, 6 piezometers were installed before rock dumping started. These first 6 piezometers were pushed into the soil by help of a grab. One of the piezometers was found to not work properly after installation and was therefore further disregarded. Eight additional piezometers were conventionally installed after rock fill construction. The piezometers were spread around all the planned jack-up leg locations and distributed at different depths.

In addition to the 14 piezometers, 2 piezometers were installed at seabed some distance away from the rock fill to be able to correct the readings for tide and atmospheric pressure variations. All piezometers were connected to a remote reading system accessible through the internet to allow for continuous monitoring.

An overview plan of the piezometers is shown in Figure 2.

Table 1 Summary of soil layer properties

Layer		I	II
Description		Soft to very soft silty clay with some shell rests, sand grains	Medium stiff silty clay with some pockets of silt and silty sand
Thickness	[m]	1 -5	1 - 15
Unit weight	[kN/m3]	17 -20	20
Undrained Shear Strength	[kPa]	6 - 20	20 -35
Sensitivity	[-]	3 - 9	2 - 3
Friction angle	[°]	20	25
Cohesion	[kPa]	5	5
Permeability	[m/day]	1.5 e-04	0.6 e-04

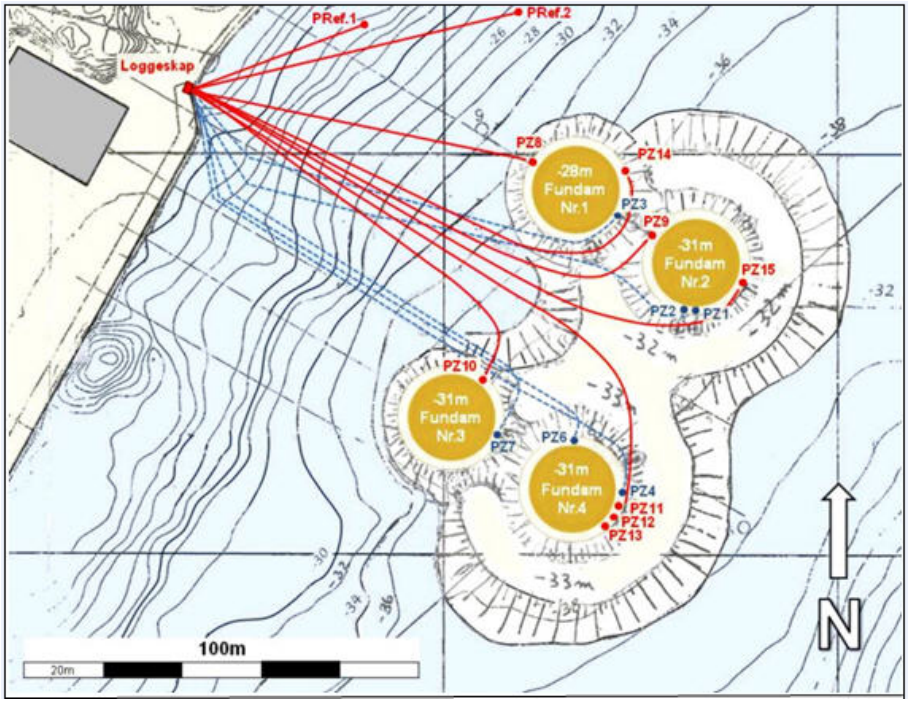


Figure 2 Overview of the rock fill location with piezometer locations

ROCK FILL GEOMETRY, CONSTRUCTION AND LOCATION

The rock fill was constructed over a period of two month out of coarse crushed rock. An overview of its geometry is presented in Figure 2.

The top 2-3 m of the fill consists of a layer of finer gravel with a diameter of less than 32 mm in order to avoid damages on the spud-can add-ons skirts during penetration and ensure that full penetration and

mudmat contact can be achieved. The fill consists of a pad at each of the four planned touch-down locations of the spud-cans and counterfills around the pads in order to ensure its stability. Due to sloping seabed, three of the pads has a top level at 31 m below sea level, while the fourth one is at 28 m. The thickness of the rock fill varies between 6 and 10 m at the pad locations. In total, the rock-fill covers an area of approximately 9000 m^2 and the total volume of rock used to build it is around 80 000 m^3.

CALIBRATION OF SOIL PERMEABILITIES

The soil permeability is necessary to predict the dissipation of excess pore pressure after a given load is applied on the seabed. This parameter is therefore important to predict how long the soil below the rock fill should be allowed to consolidate before a load can be applied and how fast the jack-up leg loads can be applied. In order to calibrate the permeability measurements, the construction of the rock fill was modelled in a 2D FEM modeling program (PLAXIS 2D, [1]) allowing for comparison of the readings from the piezometers to the results from the FEM analysis. The permeabilities were then adjusted in the program until the same excess pore pressure generation and rate of dissipation was achieved.

Piezometers 1 and 2 (PZ1 and PZ2 in Figure 2) were selected for the calibration of the permeabilities as they are both located under the same rock pad at the approximate same location but in two different soil layers, Layer I and Layer II. This allows for calibration of the permeabilities in both soil layers. In addition, the piezometers were installed sufficiently early to get a stable reading of the pore pressure before the start of the rock fill construction and had a reliable response to the rock fill construction stages. The two piezometers are located under rock pad 2 (Fundam. Nr. 2 in Figure 2).

An axi-symmetric model of rock pad 2 was set up in PLAXIS 2D. The soil layers were modelled as Hardening Soil material with parameters as described in Table 1. The rock fill height was then iteratively increased to simulate the rock fill construction sequence. The excess pore pressure generated during rock fill construction as well as its dissipation during the consolidation time was then extracted from the results at the PZ1 and PZ2 depths. Those results were then compared to the actual piezometer readings and the analyses were repeated with varying permeabilities until a reasonable match was obtained between the calculations and the piezometers measurements. A plot of the PLAXIS 2D model used is shown in Figure 3 and a plot of the comparison between the measured and predicted excess pore pressures with time at the two piezometer depths is shown in Figure 4. It was found that the permeabilities needed to be increased compared to the values obtained from the oedometer tests. This is due to the presence of pockets of silts and sands in the clay layers that were not represented in the oedometer tests that are usually carried out on homogeneous samples. These more permeable inclusions in the two clay layers increase the rate of water drainage and therefore the permeability of the soil. By doing this calibration exercise, a more realistic representation of the soil permeability was therefore obtained for further analysis of the jack-up leg test-load sequence.

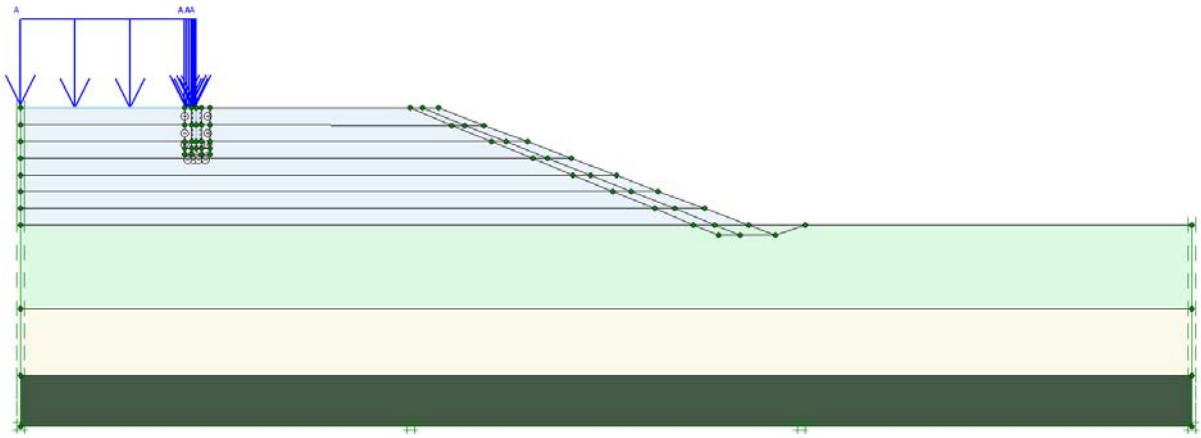


Figure 3 Plot of the PLAXIS 2D axi-symmetric model of the gravel pad no.2

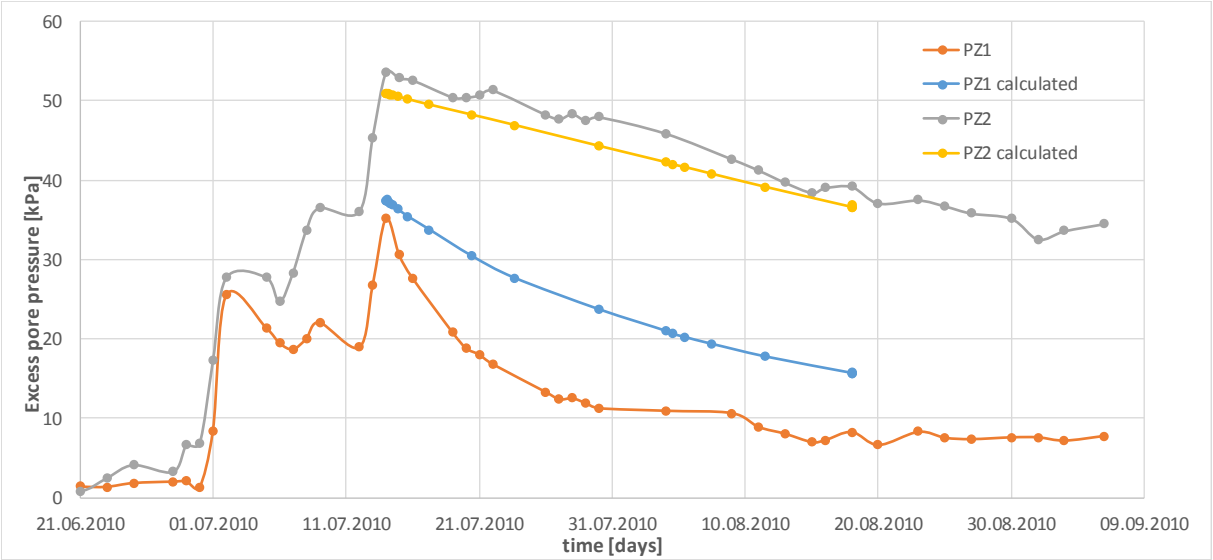


Figure 4 Comparison between piezometer readings and predicted excess pore pressures.

LOADING AND SCHEDULE

At arrival of the jack-up, penetration of the skirts in the fine top layer of the rock fill was done by jacking the structure alternatively on each pair of diagonal legs. After the first load step, where 3000t was applied on each leg, the skirt penetration checks carried out with an ROV, revealed that the penetration was not complete. The skirts were however found to be fully penetrated after the second load step where 4250t was applied on each leg. At the end of installation, only 17000t, i.e. approximately 65% of the total load of the jack up was transmitted to the fill while the rest was compensated by buoyancy.

In the following three weeks the weight applied on the foundation was increased stepwise to 5000t, 5900t and 6500t per foundations. Consolidation time was allowed between each of the load increase. After a period of 2 weeks consolidation at 6500t, the load was reduced to 6100t on the fore legs and 5000t on the aft legs as the jack-up was elevated in order to allow for top side completion work. This load was then kept nearly constant through the whole jack-up building completion phase allowing for a significant consolidation of the soil below the rock fill.

In the final load test of the jack-up, the leg loads where increased to 8000t on each pair of two diagonal legs while the load on the two remaining legs was kept at 5000t.

GEOTECHNICAL ANALYSIS WORK

In order to ensure the stability and integrity of the structure during the installation and the test operation, the following aspects have been studied:

- bearing capacity of the sea bed soil during construction of the fill
- add-on penetration analyses
- bearing capacity of the soil and rock fill under the step-wise applied foundation loads
- settlement magnitude and development with time during consolidation process
- differential settlement and rotations of the four legs.
- structural analyses of the jack-up.
- soil structure interaction

ADD ON PENETRATION ANALYSES

The penetration resistance of the add-ons has been calculated assuming 2 different scenarios, upper bound and best estimate gravel resistance. This was reflected through the generic q_c profiles used for the

calculations. The q_c profile was taken as a constant value of 5 MPa for the best estimate calculations and was established through bearing capacity equations for wedge shaped foundations [4] for a gravel with 40° friction angle for the upper bound resistance (i.e. q_c=7.8 z MPa).

It was conservatively chosen to use the highest expected coefficients (kp and kf) as described in DNV Classification note 30.4 [2] for both best estimate and upper bound cases.

The penetration analysis conducted for BE parameters for the gravel resistance showed that the spudcan add-ons would not penetrate under their own weight when parked on the top of the fill, but would penetrate while installing the structure.

The results of the penetration analyses have been validated by measurements and observations during the installation of the jack-up and found to be slightly higher than the best estimate gravel q_c profile and well below the upper bound q_c profile. It was found through the jack-up installation measurements that the add-on skirt penetration resistance was around 33MN for each foundation.

PLAXIS ANALYSES

As the dissipation of the excess pore water pressure is vital for the stability of the seabed under the loads from the rock fill and the jack-up, the finite element programs PLAXIS 2D [1] and PLAXIS 3D Foundations [3], which can model the consolidation process, have been used for geotechnical analyses. As a result of these studies, the geometry of the rock fill, fill construction procedure and the test procedure of the jack-up were proposed.

Within the rock fill area, the sea bed and the bed rock is sloping, and the thickness of the different soil layers is variable. In order to ensure the stability of the foundations and at the same time minimize the volume of the fill, the rock fill has different heights throughout the jack-up area. This geometry combined with the loads applied under each foundation makes the problem highly three dimensional. On the other hand it is difficult to model the details in PLAXIS 3D. Therefore it was decided to use the following combination of analyses:

- 2D axi-symmetrical consolidation analyses are performed to calibrate the permeability and compressibility parameters and the stability of the fill,
- 2D plane strain consolidation analyses are performed in order to design the construction sequences of the fill and to study the stability of the sea bed and the bearing capacity of the foundations during the test operation,
- 3D analyses on simplified geometry model are performed to study the 3D effects.

The variable soil stratigraphy is modelled by two layers using Hardening Soil Model (Isotropic hardening) as material model. The model is an advanced model for simulating the behaviour of different types of soil, both soft soils and stiff soils. The model realistically simulates the effective stress path during undrained loading. It is therefore used to predict the excess pore water pressures induced by the weight of the fill and the loads applied by the structure.

Several analyses with different geometries of the fill were performed both in axi-symmetric analyses and plane strain analyses in order to determine the optimum fill height and minimum extension of the fill under the four foundations. An example of such analysis is shown in Figure 5 where a calculation of the safety factor with regards to rock fill stability is done for a given leg load. Such analyses where used to adjust the rock-fill geometry in order to achieve a sufficient capacity for the fill.

An example of the 2 D plane strain model is shown in Figure 6. Two profiles were studied, one for the profile going through rock pads 1 and 2 and one going through rock pads 3 and 4. All the different possible leg load combinations during jack-up testing were checked. These analyses were important in order to model properly the interaction between the loads from two neighboring legs and include the effects of accurate soil layering and seabed and bedrock topography. They were used first to establish the rock-fill geometry and then to determine the adequate jack-up leg loading sequence, investigating how long consolidation time should was necessary between each increase in leg load. Finally, these analyses were used to determine the excess pore pressure generated in the soil for each load step. The excess pore pressure values were extracted from the results for each piezometer location. Based on these values, threshold values for each piezometers were determined including a safety margin and used as control parameter for allowing the successive increases in leg load on site. An example of comparison between these calculated excess pore pressure

control values and the measured values at two of the piezometers is shown in Figure 7. These show a good agreement between calculation and observations and gave good confidence in the rock-fill design during the jack up installation and testing.

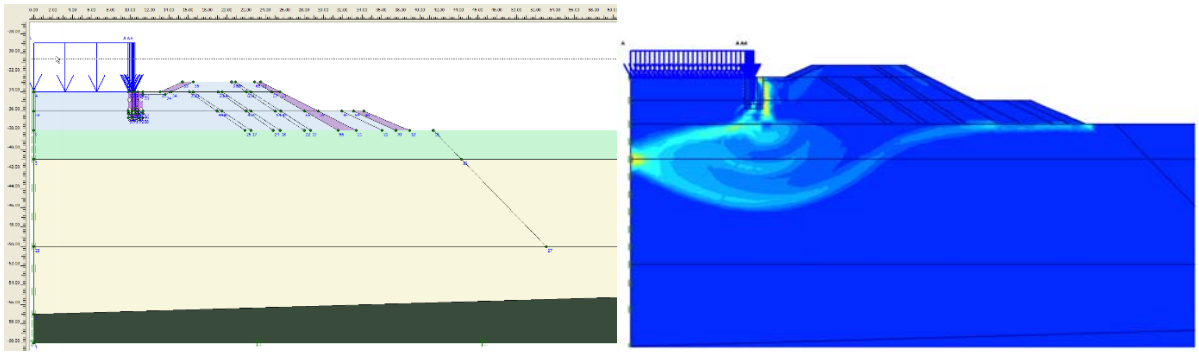


Figure 5 Model geometry (left) and typical failure surface in the 2D axi-symmetric FEM analyses

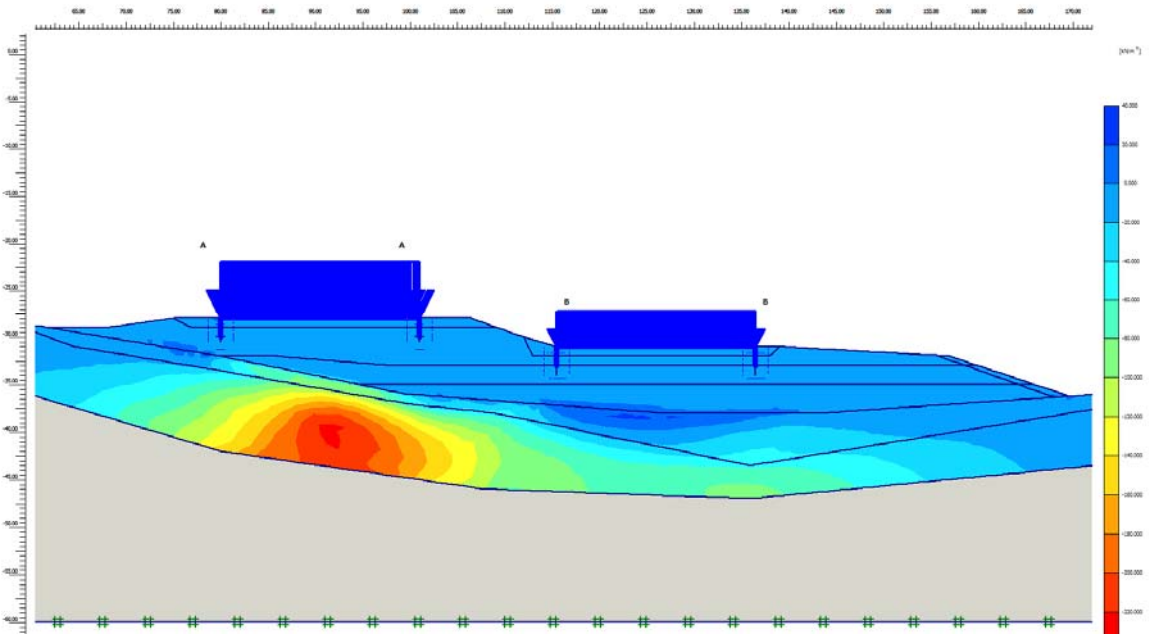
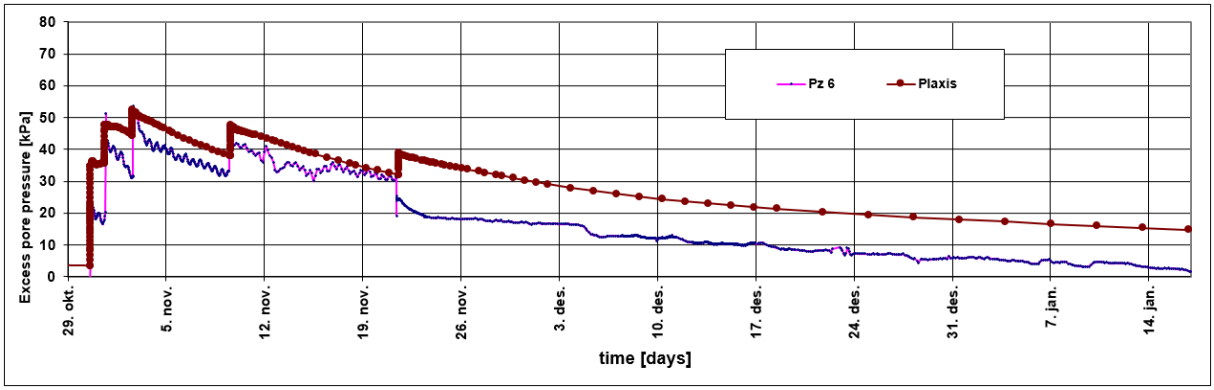


Figure 6 Model geometry and generated excess pore pressure [kPa] distribution for the maximum leg load



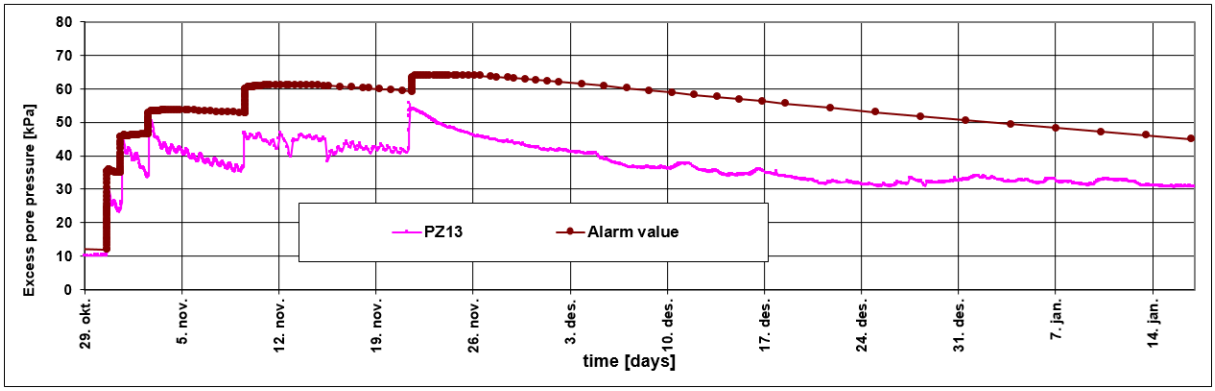


Figure 7 Examples of calculated Excess pore pressure vs. Measured excess pore pressure for piezometers 6 and 13.

An example of the model geometry used to estimate the 3D effects in PLAXIS 3D is shown in Figure 8. Results from such analysis where compared to 2D plane strain analysis with the same soil layering in order to establish the ratio between 3D and 2D capacities. It was determined through these analyses that the plane strain model would underestimate the rock fill capacity by about 20 to 30%.

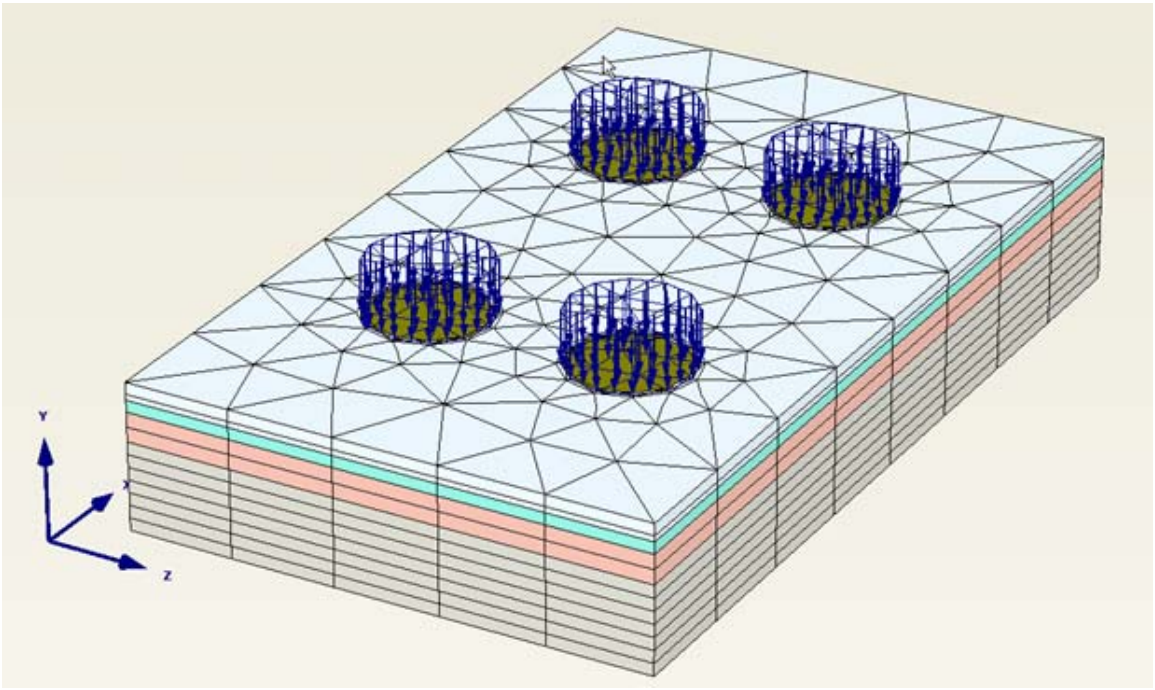


Figure 8 Typical model geometry for the 3D effect study

ANALYSIS OF ADD ON INCLINATION – SOIL STRUCTURE INTERACTION

The first PLAXIS calculations carried out with solely vertical loads on the add-ons showed that at the end of the legs load test, the inclination of the spud-cans would be approximately 0.3 degree. However, these calculations do not account for the moment induced in the leg when the add-on is inclined and which has the effect to level the add-on back to a horizontal position.

A structural calculation was therefore carried out based on an in-house model of the jack-up. The add-ons were forced to rotate by 0.23 degrees when submitted to a vertical force of 8000 kN, leading to a utilization of 0.87 in the connections between the old spud-can and the add-ons. This allowed calculating the reaction moment created by the leg on the add-on due to the stiffness of the legs. This reaction moment was then added to the vertical load in the PLAXIS calculation in order to simulate how this would affect the actual add-on inclination. Doing this, the model more accurately allows for soil structure interaction and therefore

calculates the actual inclination of the add-ons at the end of the test, which in this case was found to be around 0.15 degrees.

CONCLUSIONS

The rock fill geometry, construction sequence and jack-up test loading sequence have been established based on FEM analyses of the rock fill. In order to validate those analyses and to control the rock fill stability during the jack-up stay in Eydehavn, the excess pore water pressure, rock fill displacement, leg loads and legs RPDs were monitored continuously during the critical phases of the project and regularly outside of these phases.

The monitoring of the leg loads and RPDs showed that during the period where the load distribution on the platform remained constant, only minor changes in the measured load values where registered while the RPD values remained almost constant confirming that the jack-up and rock-fill were stable.

The settlement measurements carried out along the project showed a consolidation in line with the calculations carried out, with total settlements up to 12 cm from the weight of the platform acting on the rock-fill. The horizontal displacements over this period were in the order of 2 cm to 7 cm in a direction consistent with the seabed slope.

Continuous monitoring of the excess pore water pressure was done during jack-up installation and leg load testing. The recorded values where compared to the values calculated with PLAXIS and found generally to be less than the alarm values that were set-up. This monitoring also supported the load step durations during the jack-up leg load testing and gave flexibility for the activities of the jack-up. The excess pore pressure records showed that the seabed and rock-fill fullfilled the safety requirements at all times during the jack-up stay in Eydehavn.

ACKNOWLEDGEMENT

The authors would like to thank A.S. Nymo Yard in Eydehavn for the permission to publish this work and the co-operation on the project presented.

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