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Foundation of a high bay racking subject to extreme settlement requirements in Reykjavik

Fondation d'un entrepôt à hauts rayonnages soumis à d'extrêmes conditions de tassements à Reykjavik

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ABSTRACT: In Reykjavik, the capital city of Iceland, a high bay racking and a logistic center for a leading food distributer is under construciton. It is located on an artificial plot of land developed by the dredging technique. Due to extreme differential settlement requirements from 1:2000 to 1:3000, a 25m deep foundation into the sound bedrock is required. Thus a combination of ductile piles (system Keller KDP) and prestressed bar anchors have been installed at site. Additionally to the organisational matters on this jobsite, the Icelandic weather is a real challenge for the execution.

RÉSUMÉ: Dans le port de Reykjavík, capitale de l'Islande, un entrepôt à hauts rayonnages entièrement automatisé ainsi qu'un centre logistique pour un grossiste de produits alimentaires est en cours de construction. L'ouvrage se trouve sur un terrain remblayé à l'aide d'excavatrices-aspiratrices. En raison d'exigences de tassement différentiel très strictes, de 1:2000 à 1:3000, une fondation profonde de 25m jusqu'au rocher a été nécessaire. A cet effet, une combinaison de pieux ductiles du système Keller KDP, et de barres d'ancrages précontraintes, a été exécutée. Outre la logistique, les conditions climatiques islandaise ont été un réel défi pour l'exécution de ce chantier.

Keywords: Keller Ductile Piles; prestressed bar anchors; high bay racking; Load transfer method

1 INTRODUCTION

On the present work site a warehouse and a logistic center is planned to be in use end of 2019. The centerpiece of this project is a fully automatic high bay racking system integrated into a new warehouse – 136 m long x 62 m wide and up to 35m high. Beside the unusual strict settlement requirements for the foundation slab, the high wind loads of up to 250km/h have to be taken into consideration during all planning stages. Keller Grundbau Ges.mbH has already been contacted and involved during the planning phase of the foundation of the 8500 m² fully automatic warehouse and office area. The company had to find the best technical as well as economical solution to fulfill these

extreme settlement requirements. Many site tests have been performed to make sure that the offered solution is safe. In addition to pile driving tests, CPTs, core drillings and surveying of the rock profile ,the test field included the execution of 10 static load tests for micropiles (ductile piles for compression and bar anchors for tension loads) with test loads up to 2000 kN, representing the first static load tests on KDPs ever executed in Iceland.

2 SOIL CONDITIONS, PROJECT REQUIREMENTS AND PILE SOLUTION

The soil conditions are characterized by a 8 m thick gravel man-made backfill mixed with some stones and boulders followed by silt, clay, and sandy layers overlaying a basalt rock in a depth of 35 m. The fluctuation in length between two nearby test piles was significant and can be explained by the volcanic rocks that Iceland is made of. To ensure the bearing capacity of the developed area, a several meter thick layer of man-made fill was applied with preloading to reduce the expected settlements. Nevertheless this preloading turned out to be insufficient for the present project and a deep foundation was necessary.

Based on available data of the land reclamation, the rock profile has been estimated between 10 m and 35 m below ground level (Figure 1). Figure 1 shows in red 10m long piles and in blue 35m depth to the bedrock. This irregularity led to the decision to execute test piles.

The design of the foundation consisted in a pile-slab system made of a 60cm thick reinforced concrete slab and a deep foundation using ductile piles.

The settlement requirements for the foundation of the high bay racking were defined based on the slab regulations FEM 9.831 (1995) and FEM 9.832 (2001). Thus settlements of up to 1:3000 had to be complied with.

The Keller Ductile Pile (KDP) system has considerable advantages compared to other pile systems in regards to the total installed material. This induces a significant waste and thereby related costs reduction for the client. The flexibility in the execution process and the necessary equipment convinced the planners and the client to choose the KDP with 170 mm diameter and 9,0 mm wall thickness. All of them have been executed with a 270 mm grout cover over the full length.

As a result of the high wind speed in Reykjavik (250km/h) in combination with the chosen upper structure lead to local tensile forces of up to 4000 kN transfered into the foundation slab. Therefore prestressed double corrosion protected bar anchors with a 6 m bondlength are installed into the sound bedrock.

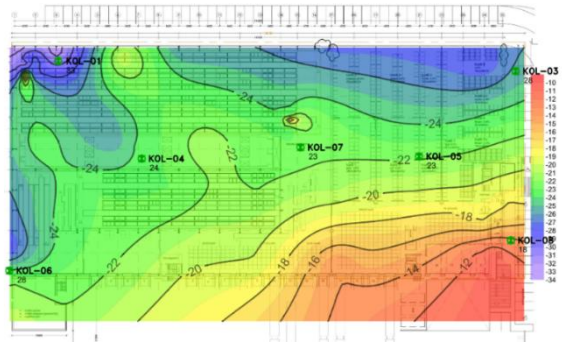


Figure 1: planview of the expected depth of the bedrock related to the current ground level

Figure 2 shows the whole construction site with the selected locations for the test fields. Testfield T1 and T2 are located in the critical high bay area and T3 is located where the shortest piles are expected to evaluate the differential settlements comming from the pile length itself. Figure 3 describes the detailed planview of one of the three testfields. Each testfield consists of four tension bar anchors and 3 ductile piles. Two are placed on the bed rock and one was limited to the length of the man made fill to evaluate the bearing capacity of this one. The bar anchors are drilled into bedrock to execute the static compression tests on the ductile piles placed in between.

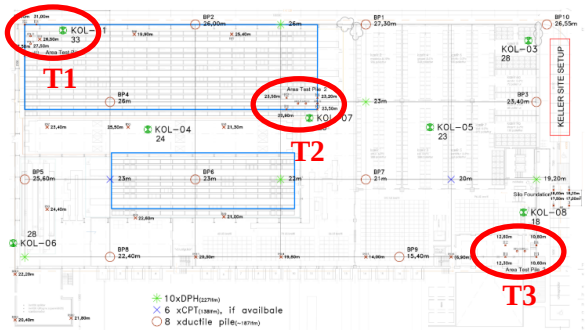


Figure 2: position of the performed test piles in addition to load test piles

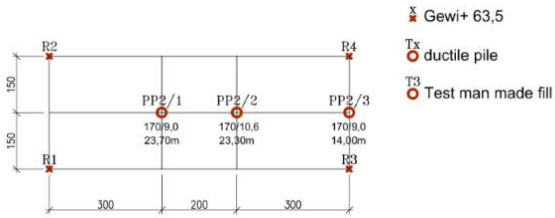


Figure 3: layout of compressive and tensile static load tests(test pile PP2/3 shorter in order to to evaluate skin friction and bearing capacity of the man-made backfill)

6 compression load tests and 4 tensile load tests as well as driving tests and test piles have been performed to check the depth of the rock and the driveability (Figure 2 and Figure 3).

3 PILE DESIGN USING LOAD TRANSFER METHOD

The Load Transfer Method (LTM) is a reliable method for the design of piled foundations showing simple graphs of the relevant results for combined pile-raft foundations and ground improvement systems with rigid columns. This method is based on load transfer curves (also „t-z“ und „q-z“ curves or mobilization functions) which have been developped by means of a database of instrumented pile load tests (Bohn, 2015; Bohn, 2016; Bohn et al., 2017). Cubic root curves have been proposed in these investigations and have been used for the present project (Tab.1).

The mobilization of skin friction depending on the displacement of the pile in the soil is described in each layer through non-linear load transfer curves (proposals in Bohn, 2015). The compression of the material of the pile or column is also taken into consideration. The limit values of skin friction and tip resistance $q_{s,ult}$ and $q_{b,ult}$ depending on the pile or column type can be determined with correlations according to the “Empfehlungen des Arbeitskreises Pfähle“ (DGGT, 2012) or in the best case scenario based on static load test results.

Figure 4 shows the required parameters as input data for a single pile calculation with the Load Transfer Method.

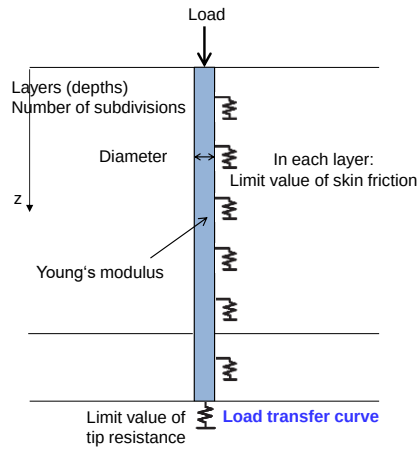
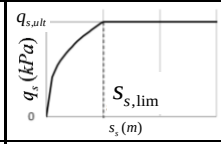
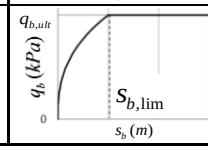


Figure 4: Single pile case and LTM input parameters

For combined systems with large foundation slabs where the soil below the slab takes a part of the loads, the system is considered as a unit cell (Figure 5). In the case of a combined pile-raft foundation as considered in this project, no load distribution layer is used and there is a direct contact between slab and the piles. The settlement of the soil layer between the piles is described by the soil constrained modulus. For piles resting on a stiff layer, the calculation shows that the load transfered into the soil is small.

		Mathematical expression	Curve shape	Deformation parameter
Cubic root curves	**Shaft**	$q_s = \min\left(\left(\frac{s_s}{s_{s,\lim}}\right)^{1/3} \cdot q_{s,ult}; q_{s,ult}\right)$		Limit settlement $s_{s,\lim}$: = 0.018 m
	Tip	$q_b = \min\left(\left(\frac{s_b}{s_{b,\lim}}\right)^{1/3} \cdot q_{b.ult}; q_{b.ult}\right)$		Limit settlement $s_{b,\lim}$: = 0.1·B

Tab 1: Load transfer cubic root curves (Bohn, 2015; Bohn et al., 2017)

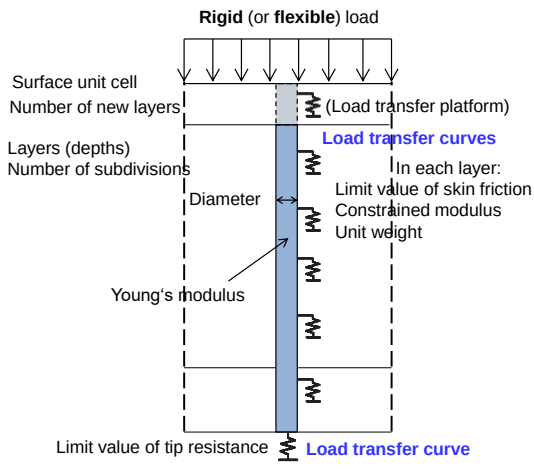


Figure 5: Unitcell (infinite grid) and LTM input parameter

For this project in Iceland the performed static load tests represent the input data for the design of the final foundation system. The bar anchors have been tested on tension loads (no contribution of the soil between the piles), which corresponds to the real behaviour in the final system. The ductile piles and the soil layers under the slab are loaded as a combined system in the planned foundation. For these piles, the following design steps are considered:

- Back-calculation of the static load tests on single piles for the estimation of the skin friction and tip resistance values. A precise evaluation of the values would be only possible with instrumented load tests. However a reliable estimation can be reached thanks to the different pile lengths that have been tested;

- Use of these skin friction and tip resistance values as input data for the design of the combined system including the pile-raft interaction.

Figure 6 and Figure 7 shows for example the results from the back-calculation of the static load tests for test pile PP1.1 with 31.8 m length for a total load of 900 kN (corresponds to a unit cell or a reference surface of 3 m x 3 m with 100 kN/m² load). An equivalent elastic modulus has been determined considering of the steel and concrete section area. In Figure 7, the graphical results show the settlement, skin friction mobilization and load over depth for the 500 kN load step. Figure 8 shows the correspondence between the calculation results and the measured load-settlement curve after back-calculation.

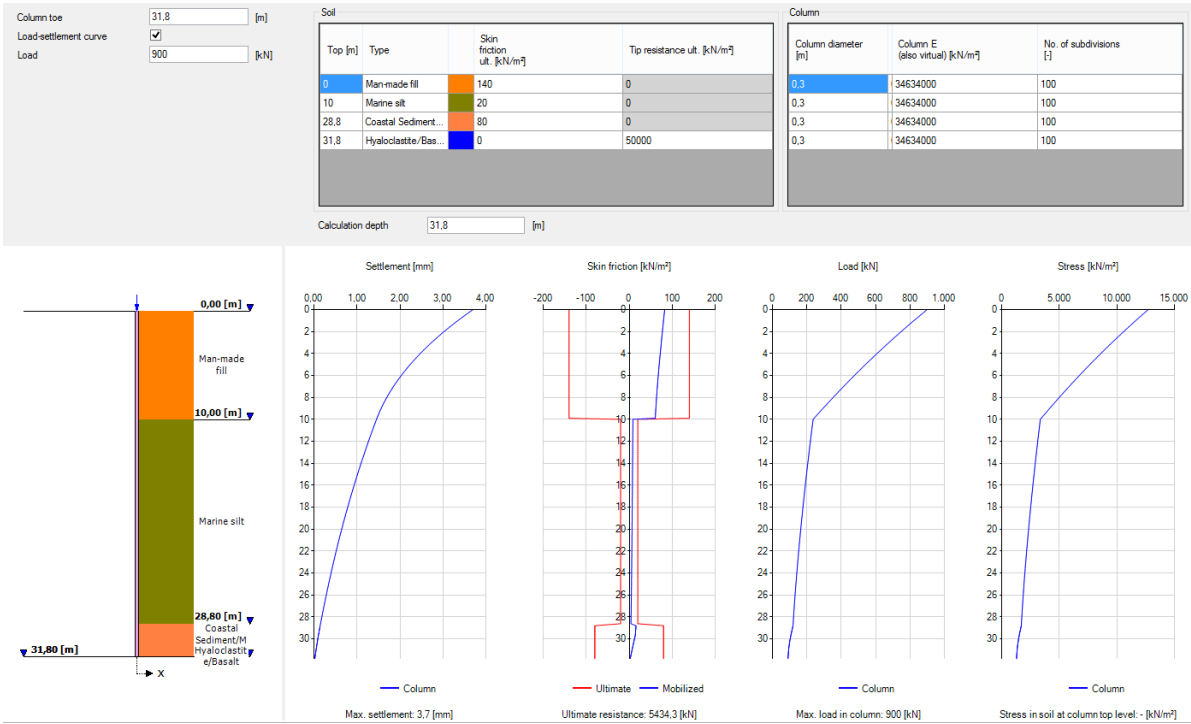


Figure 6: Detailed result from the back-calculation of the static load test for ductile pile PP1.1 for a total load of 900 kN

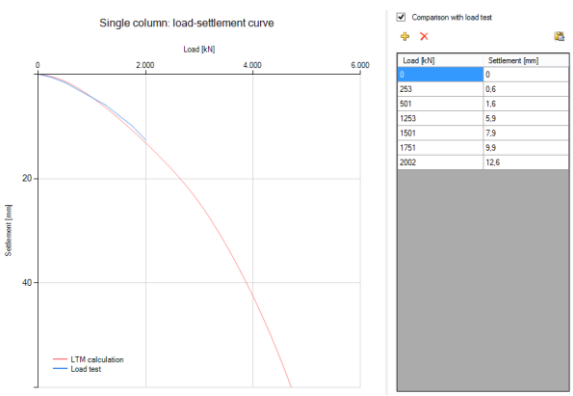


Figure 7: Result from the back-calculation of the static load test for ductile pile PP1.1

The back-calculation results for the values of skin friction and tip resistance showed for all static load tests a good correspondence, except for the man-made fill layer which shows highly heterogeneous behaviour over the whole area. However this layer has no influence on the overall system with pile-raft interaction. Due to the stiff slab, the settlement in the soil and the settlement in the pile at the top of the unit cell are equal. Therefore no skin friction (dependent on the displacement between pile and soil) can be mobilized directly beneath the slab. This has been checked using different values of skin friction in this layer in the overall pile-raft system.

Figure 8 shows the result for the pile-raft system for a 3 m x 3 m pile gridin terms of graphical results of settlements, mobilized skin friction ald load in soil and pile over depth. The small negative skin friction values in the lower part of the man-made fill are simply due to error tolerances in convergence algorithm. Basically the complete system shows to be very similar to a pile foundation since only a small part of the load is taken by the soil beneath the slab. The rock layer is that stiff that almost no settlement arise at the pile tip. The pile-raft interaction leads to the effect that the skin friction in the first layer can not be activated.

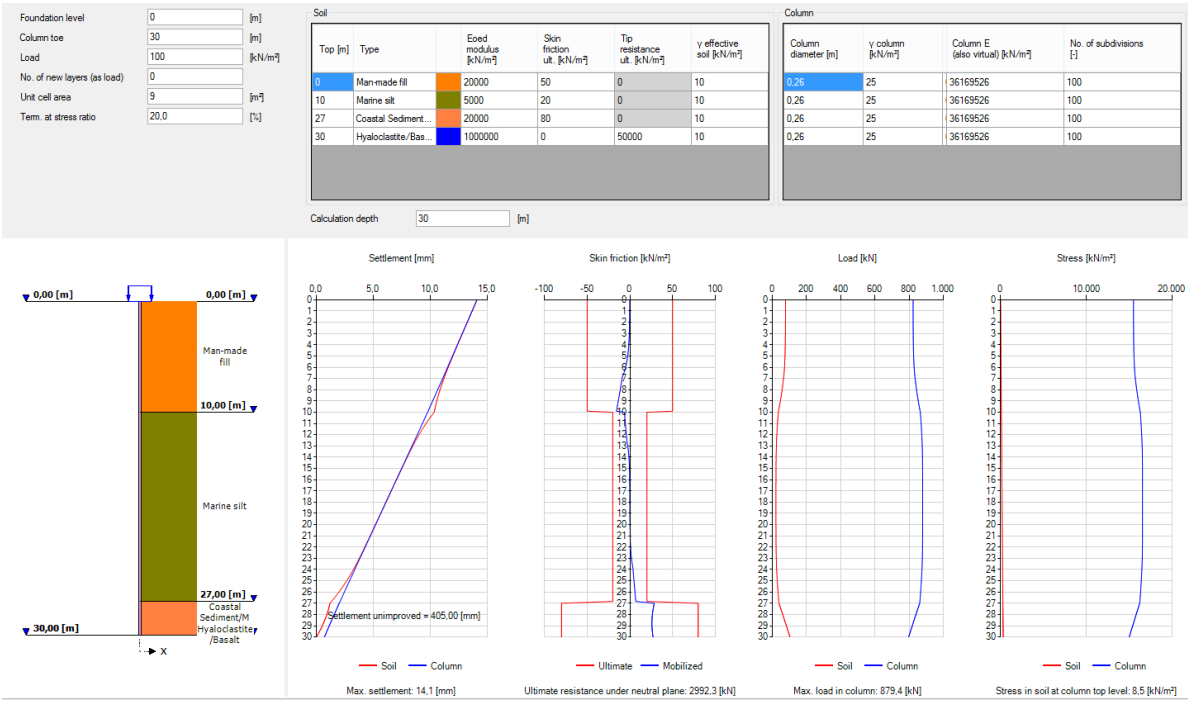


Figure 8: Detailed result of the calculation for the complete system of a unit cell of 3 m x 3 m with 100 kN/m² load for 30 m bedrock level

This explains why the settlement of around 14 mm in the unit cell system of 3 m x 3 m with 100 kN/m² is larger than the settlement of a single pile with approximately 4 mm.

Since the load is mainly concentrated on the pile itself, the calculation as a point-supported slab with the stiffness taken from the overall system is representing well the actual foundation system. Those stiffnesses were calculated with the LTM-model for pile lengthes of 10 m to 34 m depending on the site area. For the different pile grids planned on the construction area, the grid area (reference surface per pile) has a negligible impact on the settlements.

The stiffness has been increased by 15 % to 30 % in the corners and edges of the foundation slab based on the analysis using the finite element method (see next section).

The settlement and tilting of the concrete slab can be calculated with a software taking into consideration the slab stiffness and the irregular load distribution on the slab. Figure 9 is an example of the software GGU Slab used for this application showing the settlements over the area. The strict settlement requirements for the whole slab were fulfilled thanks to the appropriate pile grids depending on the different location under the building.

4 COMPARISON OF THE CALCULATION WITH THE FINITE ELEMENT METHOD

Independently from the beforehand described calculation done by Keller, an independent third party has reviewed the calculation since the very beginning of the project. Therefore the pile foundation based on static load tests has been modelled with the software Plaxis 3D (version 2016.2.0.0) and then compared to the results from the load transfer method, as described before. Both calculation methods reached a very good correspondence for the pile stiffness as well as for the tilting of the foundation slab. A back-calculation of the load tests and modelling of the

pile-raft effect with both calculation methods as described in section 3 have been performed and compared with each other. Figure 9 and Figure 10 are showing an example of the 3D model and the settlements calculated for one load case.

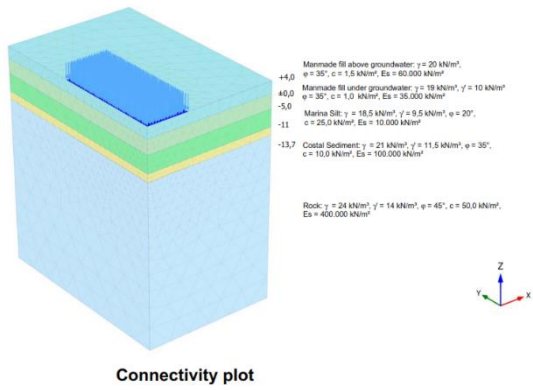


Figure 9. Example of the used pile model showing the different soil layers and parameters used for the calculation with Plaxis 3D

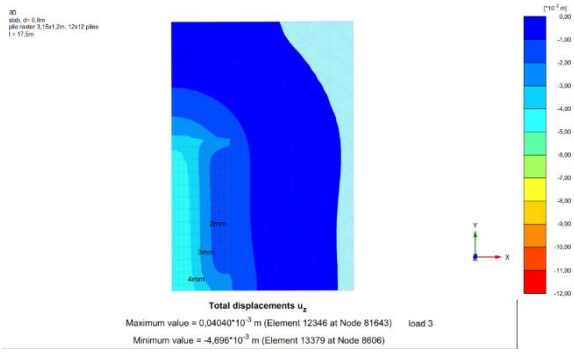


Figure 10. Example for settlement calculation of the pile supported foundation slab using Plaxis 3D

5 EXECUTION

1 900 Keller Ductile Piles (KDP) 170/9,0 mm with a total length of 44000 m in combination with 2500 m of permanent prestressed bar anchors were installed on the site. 2 piling rigs were required for the execution of the KDP and 1 double head drill rig for the tension piles. In order to keep the schedule a detailed logistical concept was required from the beginning since the rigs, mixing plants as well as the material had to be imported. More than 80 maritime containers have been shipped during the construction time. Figure 11 shows the components of the ductile pile that have been driven into the soil using a hydraulic hammer.

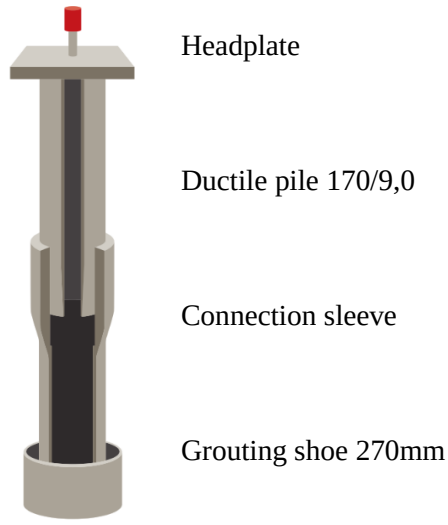


Figure 11: Components of a ductile pile for compression loads

A standard rig is the hydraulic excavator givingflexibility in the execution. Depending on the chosen ductile pile and the necessity of the ring space injection, hydraulic hammers are used to drive the piles. In Figure 11 the site setup as well as the installation process of a fully grouted ductile pile is shown. In this project all the piles were installed until reaching the sound bedrock. Due to a required positioning accuracy and the observed material in the top backfill layer, all piles were pre-drilled in the first 7 meters. The recording of the driving time per meter allows to conclude about the bearing capacity of the respective soil layer based on the beforehand executed load tests.

The construction schedule was met although the weather condition were complicated during the winter months. Finally the whole foundation

works were handed over in time to the concrete contractor.

6 CONCLUSION

The applied foundation system represented by a combination of Keller Ductile Piles and permanent prestressed bar anchors was executed in time thanks to the high flexibility and productivity despite a delayed construction start. Due to the missing experience with piling works in Iceland, a test field was executed prior to the start of the detailed design of the foundation. The whole design for the pile foundation was done based on the filed testing with two different concepts (load transfer method and finite element method) in order to fulfill the extreme settlement requirements. The results of the load tests allowed the adaptation of the models to the local conditions for the different bedrock depth encountered over the site. Herewith an economical and efficient planning was ensured. The KDP also convinced all involved parties since wasting of the resources could be avoided due to prevention of process-related overlengths. This project illustrates once more the added value of load tests on the respective construction area in order to establish the ideal design and execution technique related to the local conditions.

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