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Realistic Modelling of Soil-Structure Interaction for High-Rise Buildings

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Abstract

For a save design and construction and the technical and economic optimisation of deep foundation systems a realistic modelling of the soil-structure interaction is necessary. Especially for the hybrid deep foundation system Combined Pile-Raft Foundation (CPRF) this has to be considered. Based on an adequate soil investigation, in-situ pile load tests and a high-level design using the Finite-Element-Method (FEM) it is possible to design complex foundation systems for high-rise buildings even in soft soil conditions. For guarantee of the ultimate limit state (ULS) and the serviceability limit state (SLS) an independent peer review and the application of the observational method is necessary. The paper explains some special aspects of the optimisation process of the design and presents several projects from engineering practice, where the CPRF has been successfully applied.

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Keywords: Combined Pile-Raft Foundation, soil-structure interaction, pile load test, independent peer review, numerical simulations, back analysis, observational method.

1. Introduction

This contribution explains the aspects of soil-structure interaction by the hybrid foundation system Combined Pile-Raft Foundation (CPRF). The CPRF is a technically and economically optimised, complex foundation system. For a successful design the following aspects have to be considered:

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- qualified experts for planning, design and construction
- interaction between architects, structural engineers and geotechnical engineers
- adequate soil investigation
- consideration of the soil-structure interaction during design phase using the Finite-Element-Method (FEM) in combination with enhanced in-situ load tests for calibrating the soil parameters used in the numerical simulations
- quality assurance by an independent peer review process (4-eye-principle) combined with the observational method if necessary

Basically the design of deep foundations is regulated in several national standards and codes or for example in the Eurocode EC 7 [1]. Due to the fact that no international regulation existed until 2013 the international CPRF-Guideline [2] was developed by the Technical Committee TC 212 "Deep Foundations" of the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE).

Construction projects with a high complexity in design and construction and with a distinctive soil-structure interaction need an independent peer review process (4-eye-principle). Detailed explanations for this are given in [3, 4].

2. Combined Pile-Raft Foundation (CPRF)

The CPRF is a hybrid foundation system that combines the effects of a foundation raft and deep foundation elements like piles and barrettes [5, 6]. The bearing capacity and the deformation behaviour are affected by the interactions between the deep foundation elements, the foundation raft and the subsoil. For an optimised and safe design of a CPRF the calculation method has to consider these interactions [2, 7].

Due to the stiffness of the foundation raft the total load of the building Ftot,k is transferred into the subsoil via contact pressure under the raft σ(x,y) and via the deep foundation elements like piles. The total resistance Rtot,k(s) of the CPRF consists of the resistance of the raft Rraft,k(s) and of the resistance of the piles Σ Rpile,k,j(s) as explained in Eq. 1. The resistance Rpile,k,j(s) of a single pile "j" consists of the skin friction qs,k,j(s,z) and the base resistance qb,k,j(s), as show in Eq. 2 to Eq. 4. Fig. 1 shows the soil-structure interaction of a CPRF. The bearing capacity and the load-settlement behaviour are affected by the interactions between the different elements of the hybrid foundation system CPRF.

$$R_{tot,k}(s) = \sum_{j=1}^{m} R_{pile,k,j}(s) + R_{raft,k}(s) . \tag{1}$$

$$R_{pile,k,j}(s) = R_{b,k,j}(s) + R_{s,k,j}(s) . \tag{2}$$

$$R_{s,k,j}(s) = \int q_{s,k,j}(s,z) \cdot \pi \cdot D \cdot dz . \tag{3}$$

$$R_{b,k,j}(s) = q_{b,k,j}(s) \cdot \frac{\pi \cdot D^2}{4} . \tag{4}$$

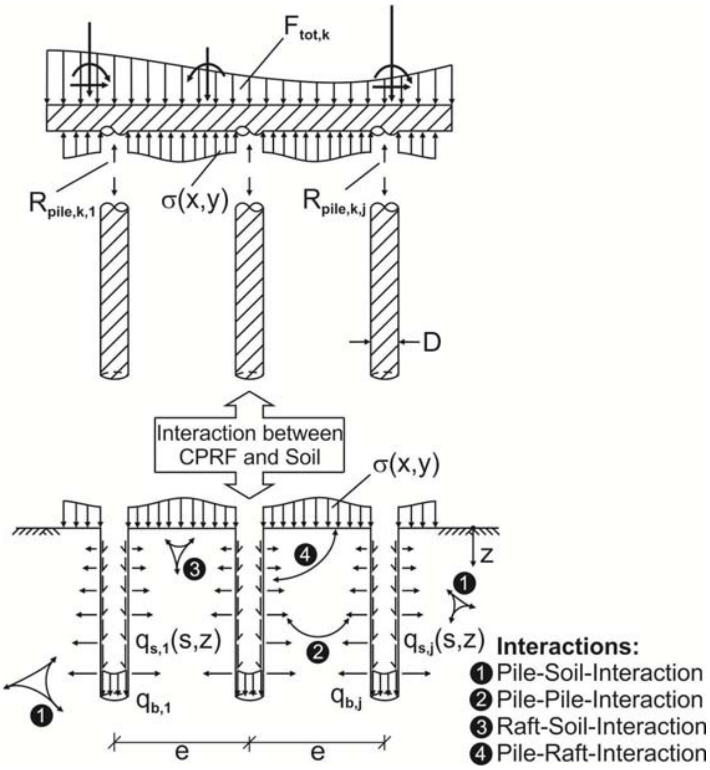


Fig. 1. Soil-structure interaction of a CPRF.

The distribution of the total building load between the different bearing structures of a CPRF is described by the CPRF coefficient αCPRF which defines the ratio between the amount of load carried by the piles Σ Rpile,k,j(s) and the activated total resistance Rtot,k(s) of the CPRF as shown in Eq. 5. The activated total resistance Rtot,k(s) is equal to the total load of the building Ftot,k.

$$\alpha_{CPRF} = \frac{\sum R_{pile,k,j}(s)}{R_{tot,k}}. \tag{5}$$

A CPRF coefficient of zero describes a raft foundation without any deep foundation element, a CPRF coefficient of one represents a classic pile group, neglecting the existence of a raft.

Compared to a classic spread or pile foundation the objectives and advantages of a CPRF are a reduction of settlements and differential settlements, an increase of the bearing capacity, a decrease of the bending load and finally a minimisation of the costs [8]. Further information about analysis and design of CPRF are given in [9-14].

3. In-situ pile load tests

Project- and site-related soil investigations with core drillings and laboratory tests are essential for the initial definition of soil mechanical properties of the single soil layers. But usually these investigations are not sufficient for an entire and realistic capture of the complex conditions, caused by the interaction of subsoil and construction [8].

In order to reliably determine the ultimate bearing capacity of piles, load tests need to be carried out [15]. For pile load tests often very high counter weights or strong anchor systems are necessary. By using the Osterberg method high loads can be reached without installing anchors or counter weights. Hydraulic jacks, so called Osterberg Cells (O-Cells), induce the load in the pile using the pile itself partly as abutment. For hybrid foundation systems like CPRFs load tests are a very good basis for the calibration of the numerical simulations by back-analysis.

4. Observational method

For projects with difficult boundary conditions and a distinctive soil-structure interaction it is necessary to apply the observational method to review the design during the construction time and, if necessary, during the service time of a structure. The observational method is always a combination of the common geotechnical investigations before and during the construction phase and the geodetic survey together with the theoretical modelling and a plan of contingency actions [1]. Fig. 2 shows the principle of the observational method. Only monitoring to ensure the stability and the serviceability of the structure is not sufficient and according to the standardisation not permitted for this purpose. Overall the observational method is an institutionalized controlling instrument to verify the soil and rock mechanical modelling.

The identification of all potential failure mechanisms is essential for defining the monitoring programme. The programme has to be designed in that way that all these mechanisms can be observed. The measurements need to be of an adequate accuracy to allow the identification of critical tendencies. The required accuracy as well as the boundary values need to be defined within the design phase and before the development of the monitoring programme. Contingency actions need to be planned before the construction works start considering the ductility of the bearing structures. The observational method must not be seen as a potential alternative for a comprehensive soil investigation campaign. A comprehensive soil investigation campaign is in any way of essential importance. Additionally the observational method is a tool of quality assurance and allows the verification of the parameters used in numerical simulations in the design phase. The observational method helps to achieve an economic and save construction [16].

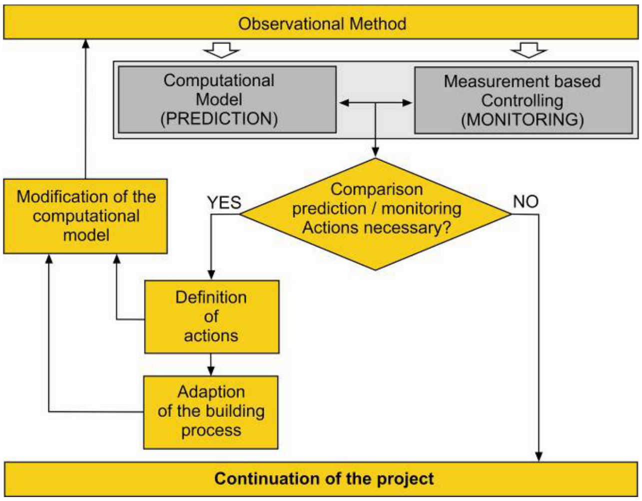


Fig. 2. Principle scheme of observational method.

5. Example from engineering practice: CPRF of a high-rise building in Frankfurt Clay

5.1. Project overview

The high-rise building “Messeturm” in Frankfurt am Main, Germany, is 256.5 m high. The foundation of the Messeturm is a CPRF which is based in the Frankfurt Clay (Fig. 3, left). The foundation raft is 58.8 m x 58.8 m wide with a maximum thickness of 6 m in the centre and a thickness of 3 m at the edges. The base of the foundation raft is about 11 m to 14 m below the ground surface. The raft is combined with 64 bored piles with a diameter of 1.3

m and a length of 30.9 m in the centre ring and 26.9 m at the edges (Fig. 3, right). The total building load, including 30 % of the live loads, is about 1,855 MN. The complex settlement behaviour is related to the load-deformation behaviour of the foundation system itself and the time-dependent load-deformation behaviour of the Frankfurt Clay. Therefore a monitoring programme was applied. The maximum measured settlements where about 13 cm in the centre of the CPRF and about 8 cm to 9 cm at the edges of the CPRF.

5.2. Optimisation of the CPRF

The CPRF was calculated with the FEM. Therefore a section of the foundation was modelled using the symmetry of the plan view (Fig. 5, left). The settlements of a pure raft foundation were calculated to 32.5 cm. The calculated settlements of the CPRF are nearly equal to the in-situ measured values mentioned above (Fig. 3, right). The CPRF coefficient is about αCPRF = 0.43 [17].

A pure pile foundation would have required 316 piles with 30 m in length. In comparison to the realised CPRF with 64 piles and an average length of about 30 m a pure pile foundation would have required more resources, e.g. concrete and energy, more time and would have been approximately 5.9 Million US$ more expensive.

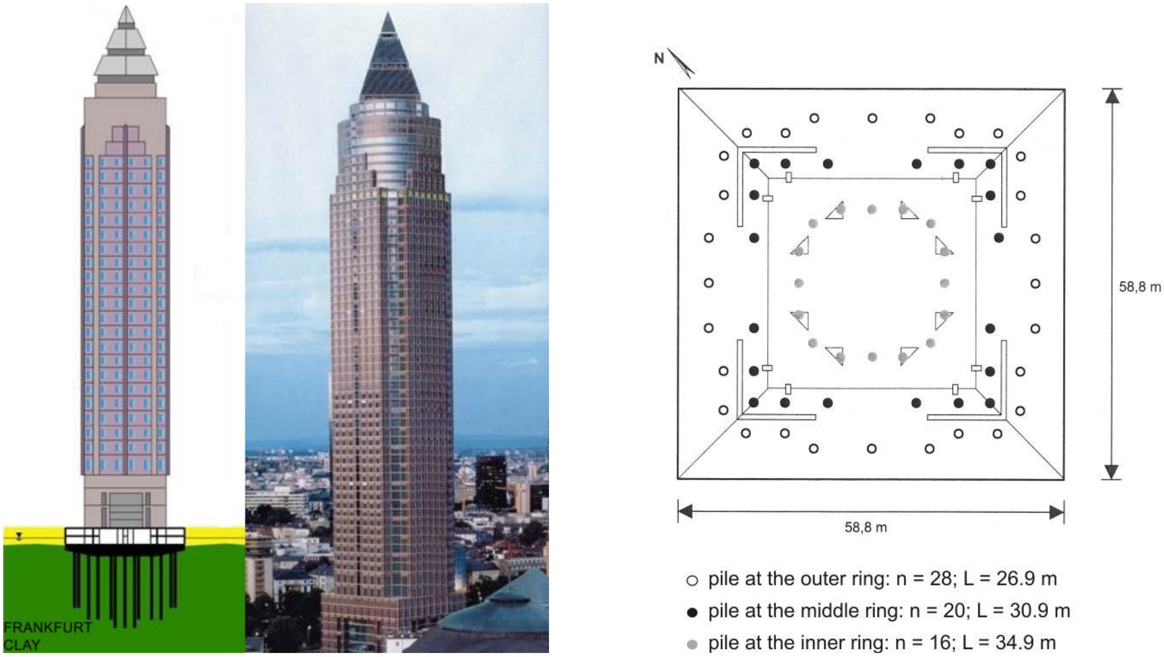


Fig. 3. Messeturm in Frankfurt am Main (left) and alignment of the CPRF (right)

Example from engineering practice: CPRF of a high-rise building in clay marl

5.3. *Project overview*

In the scope of the project Mirax Plaza in Kiev, Ukraine, 2 high-rise buildings, each of them 192 m (46 storeys) high, a shopping and entertainment mall and an underground parking are under construction (Fig. 6, left). The gross area off the project is about 294,000 m2 and cuts a 30 m high natural slope.

The geotechnical investigations have been carried out to a depth of 70 m. At the surface a fill of 2 m to 3 m was detected. Beneath the fill is a layer of quaternary silty sand and sandy silt with a thickness of 5 m to 10 m. Underneath this layer tertiary silt and sand with a thickness of 0 m to 24 m were detected. Beneath the tertiary silt and sand a layer of tertiary clayey silt and clay marl of the Kiev and Butschak formation with a thickness of about 20 m was detected. From this layer down to the investigation depth tertiary fine sand of Butschak formation was found.

The groundwater level is in a depth of about 2 m below the ground surface.

5.4. Optimisation of the CPRF

For verification of the shaft and base resistance of the deep foundation elements and for calibration of the numerical models, pile load tests have been carried out on the construction site. The piles had a diameter of 0.82 m and a length of about 10 m to 44 m. Using the results of the load tests, the back analysis for verification of the FEM-simulations was done. The soil properties determined with the results of the back-analysis were partly 3 times higher than indicated in the geotechnical report. Fig. 4 shows the results of the load test No. 2 and the numerical back analysis. Measurement and calculation show a good accordance.

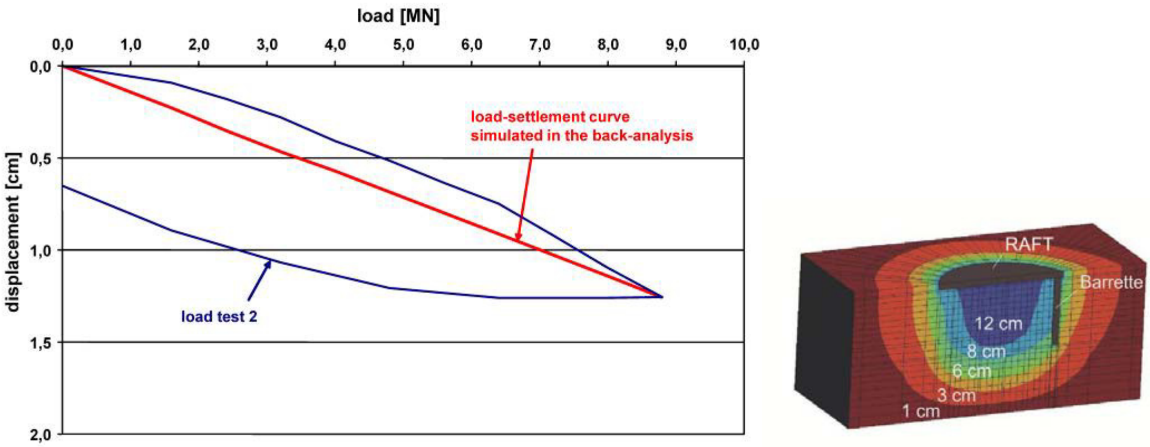


Fig. 4 Results of the in-situ load test and the numerical simulations (left) and FEM-model and calculated settlements in [cm] (right).

The obtained results of the pile load tests and of the back analysis were applied in 3-dimensional FEM-simulations, taking advantage of the symmetry of the footprint of the building (Fig. 4). The overall load of Tower A is about 2,200 MN and the area of the foundation is about 2,000 m2.

The foundation design considers a CPRF with 64 barrettes with 33 m in length and a cross section of 2.8 m x 0.8 m. The raft of 3 m thickness is located in Kiev Clay Marl at about 10 m depth below the ground surface. The barrettes reach the Butschak Sands.

The calculated loads on the barrettes were in the range of 22.1 MN to 44.5 MN. The loads on the outer barrettes were about 41.2 MN to 44.5 MN which significantly exceeds the loads on the inner barrettes with the maximum value of 30.7 MN. This behaviour is typical for a CPRF. The deep foundation elements at the edge of a CPRF activate a bigger soil volume that is not influenced by neighbouring elements. The CPRF coefficient is αCPRF = 0.88. Maximum settlements of about 12 cm were calculated due to the settlement-relevant load of 85 % of the total design load. The pressure under the foundation raft calculated does not exceed 200 kN/m2 in most areas, at the edge of the raft the pressure reaches 400 kN/m2. The calculated base pressure of the outer barrettes has an average of 5,100 kN/m2 and for inner barrettes an average of 4,130 kN/m2. The mobilised shaft resistance increases with the depth reaching 180 kN/m2 for outer barrettes and 150 kN/m2 for inner barrettes.

Regarding the complex foundation system the observational method was applied. Especially the distribution of the loads between the barrettes and the raft is monitored. For this reason 3 earth pressure devices were installed under the raft and 2 barrettes were instrumented over the length.

In the scope of the project the new allowable shaft resistance and base resistance were defined for typical soil layers in Kiev. This unique experience will be used for the high-rise buildings of new generation in Ukraine.

The CPRF of this project is the first CPRF in Ukraine. Using the advanced optimisation approaches and taking advantage of the positive effects of a CPRF, the number of barrettes could be reduced from 120 barrettes with 40 m length (classic pile foundation) to 64 barrettes with 33 m length (CPRF). The foundation optimisation leads to a considerable decrease of the utilised resources (cement, aggregates, water, energy etc.) and leads to cost savings of about 3.3 Million US$.

6. Example from engineering practice: CPRF of a high-rise building in very soft soil

6.1. *Project overview*

A more than 75 m high-rise building in settlement sensitive soil has been constructed at the coastline of West Africa. The high-rise building has up to 16 storeys. The foundation system is designed as a CPRF. The annex buildings are up to 60 m high and include apartments and parking levels. All structures have one basement level. The whole structure has a total load of more than 700 MN. Due to its complexity the project is categorized into the Geotechnical Category GC 3 of the Eurocode EC 7 [1]. The Geotechnical Category GC 3 is the category for the most difficult projects that require a high level design and an independent peer review.

The soil investigation was carried out down to a depth of 80 m below the surface. At the surface clayey sands have been detected. Until a depth of 33 m below the surface is an alternating sequence of medium dense and dense sand layers. Down to the investigation depth follows an alternating sequence of medium dense and dense sand layers and clay and silt layers with low to high plasticity. The groundwater level is close to the surface.

6.2. Optimisation of the CPRF

For the determination of the bearing capacity, the load-settlement behaviour and the internal forces of the CPRF 3-dimensional simulations using the FEM are necessary. The simulations considered the non-linear behaviour of the soil and had been calibrated by back analysis of laboratory tests and in-situ load tests.

For the in-situ load test Osterberg Cells (O-Cells) have been used. The test pile had 3 parts: the upper pile segment 1, the middle pile segment 2 between the upper and the lower O-Cell and the lower pile segment 3 (Fig. 5, left).

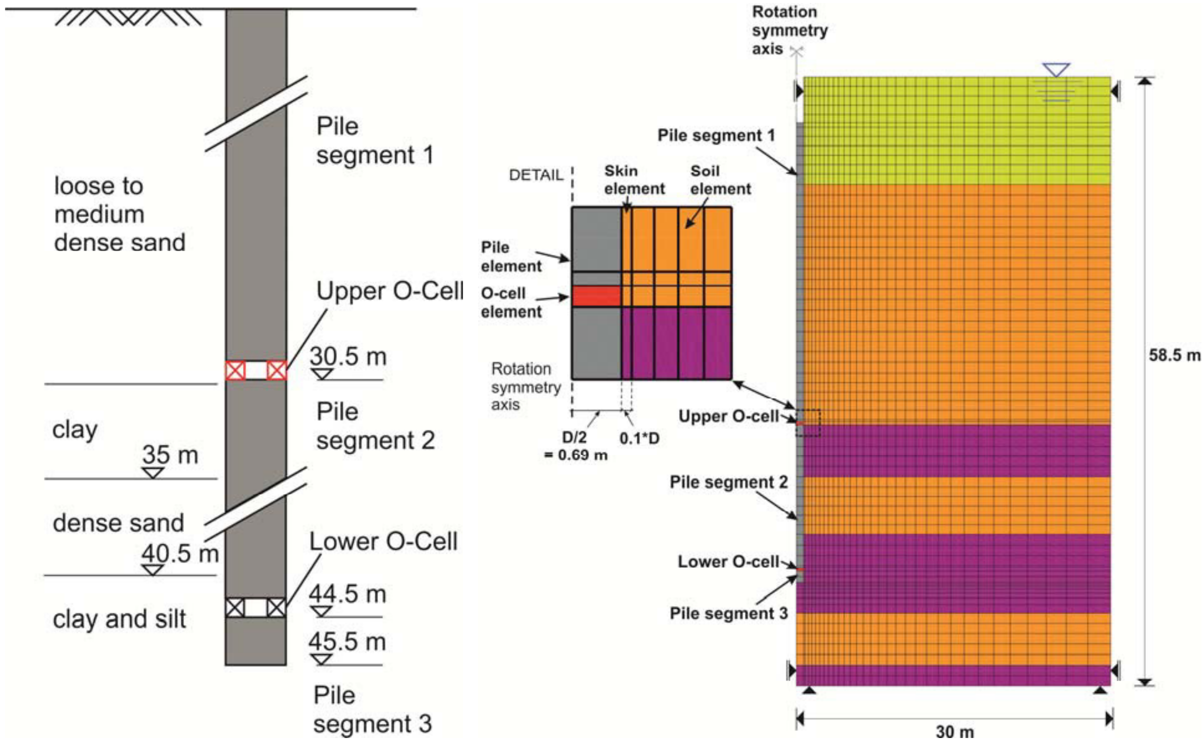
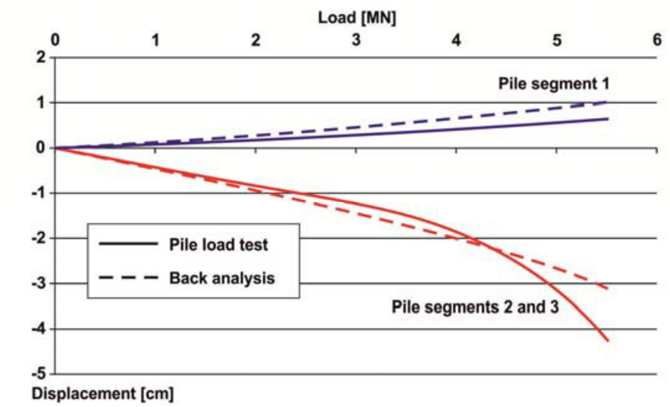


Fig. 5 Setup of the pile load test (left) and FE-Model for back analysis (right).

The pile segments were actived differently to determine the base resistance and the skin friction. For determination of the skin friction and the base resistance of pile segment 3 only the lower O-Cell was activated using the segment 2 as abutment. For determination of the skin friction of pile segment 2 the upper O-Cell was activated and the lower O-Cell was released. Pile segment 1 was the abutment for this test phase. For determination of the skin friction of pile segment 1 the upper O-Cell was loaded and the lower O-Cell was fixed. The pile segments 2 and 3 were used as abutments. Fig. 8 shows on the right the mesh of the FEM simulations and the principle arrangement of the pile load test equipment with the 3 pile segments and the upper and lower O-Cells.

The results of the back analysis by FEM simulations are the basis for the adjustment of the estimated soil parameters and were used to verify the developed, simplified stratigraphy for the analysis of the whole foundation system. The results of the pile load test in-situ and of the back analysis are drawn in Fig. 6 on the left. The comparison of the results shows a good accordance.

The length, the diameter and the number of piles of the whole CPRF were optimised by the FEM simulations. Fig. 6 shows on the right the final CPRF design. The CPRF coefficient is αCPRF = 0.8. During the construction phase and for the first years of service time the loads of the piles, the stresses under the raft and the deformation behaviour of the CPRF are measured by a monitoring programme according to the requirements of the observational method [16].



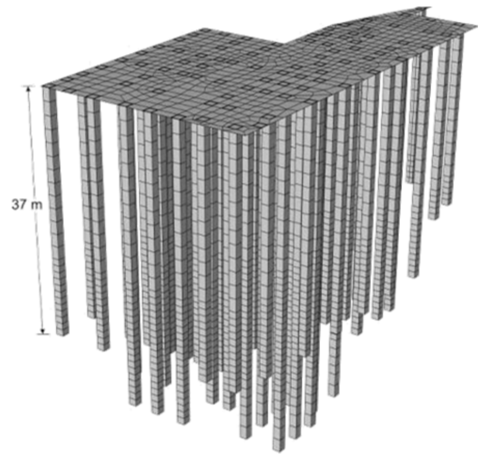


Fig. 6 Results of the in-situ pile load test and the back analysis (left) and the final CPRF (right).

7. Conclusions

In many geotechnical challenges deep foundations are the only possible technical solution for the implementation of large construction projects. The economic and environment-friendly design of the deep foundations focuses on a reduction of construction material used, construction time spent and energy consumed within the buildings construction and service time [4]. CPRFs are a hybrid deep foundation system using the bearing capacity of the piles and the raft.

Generally CPRFs belong to the Geotechnical Category GC 3. For a safe and optimised design and construction numerical simulations using FEM, an independent peer review and the application of a monitoring programme are necessary [18, 19].

With in-situ load tests on the construction site the base resistance and the skin friction of deep foundation elements are determined [20]. In addition the in-situ load tests are very utile for the calibration of the numerical models. The numerical simulations have to consider the non-linear behaviour of the soil material.

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