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Deep Foundation Systems for High-Rise Buildings in Difficult Soil Conditions

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ABSTRACT: The economic and environment-friendly design of deep foundation systems focuses on a reduction of construction material used, construction time spent and energy consumed within the buildings construction and service time. The special foundation system Combined Pile-Raft Foundation (CPRF), in-situ load tests on the construction yard and a sufficient soil investigation allow cost optimized constructions. In any case at challenging projects the observational method has to be applied for verification and quality control of the design. Regarding the 4-eye-principle as basis of the quality control management an independent peer review of the planning, design and construction phase is the guarantee for safety, serviceability and optimization of all projects. Optimization processes combined with a necessary quality management are explained in this paper on several challenging projects from engineering practice.

1. INTRODUCTION

The economic and successful design and construction of deep foundation systems for high-rise buildings is based on the following main aspects:

- qualified experts for planning, design and construction
- interaction between architects, structural engineers and geotechnical engineers
- adequate soil investigation
- analysis based on adequate calculation methods
- design of complicated deep foundation systems 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 and the observational method (4-eye-principle)

The aspects regarding analysis and design will be explained by large construction projects which are located in difficult conditions in terms of soil parameters, groundwater and existing structures in the area of influence. The aspects regarding the 4-eye-principle, quality assurance and the observational method are explained for example in [1 – 5]. For the design and construction of deep foundation systems for high rise buildings in urban areas high level engineering is necessary [6]. Especially if the structures are located very close together, as symbolized by the skyline of Frankfurt am Main, Germany (Figure 1), the soil-structure-interaction has to be considered.

Figure 1 Skyline of Frankfurt am Main, Germany

2. IN-SITU LOAD TESTS

The soil parameters are determined based on project- and site-related soil investigations with core drilling and laboratory tests. Those tests are important and essential for the initial definition of soil mechanical properties of the soil layers, but usually not sufficient for an entire and realistic capture of the complex conditions, caused by the interaction of subsoil and construction [1].

In order to reliably determine the ultimate bearing capacity of piles, load tests need to be carried out [2]. 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 induce the load in the pile using the pile itself partly as abutment. The results of the field tests allow a calibration of the numerical simulations. The principle scheme of pile load tests is shown in Figure 2.

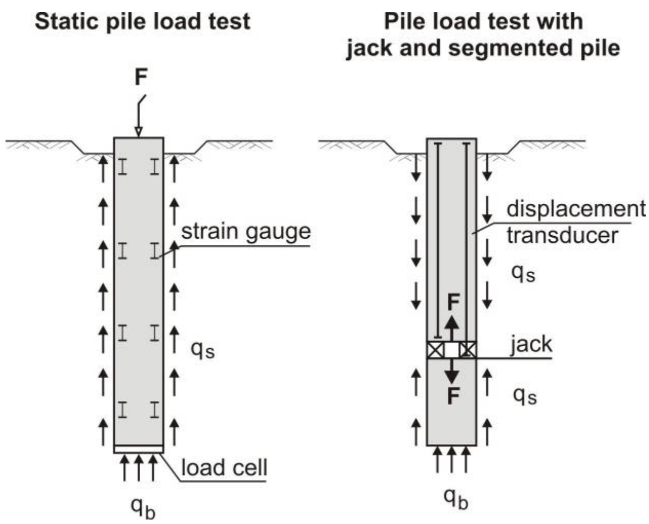


Figure 2 Principle scheme of pile load tests

3. 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. The bearing capacity and the deformation behavior are affected by the interactions between the deep foundation elements, the foundation raft and the subsoil. For an optimized and safe design of a CPRF the calculation method has to consider these interactions [7]. For design and construction of a CPRF the codes and regulations for classic pile foundations have to be considered. Additionally the CPRF-Guideline [8], developed in Germany, has to be observed. The international CPRF-Guideline is published by the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE).

3.1 Bearing capacity of a CPRF

Due to the stiffness of the foundation raft, the total load of the building $F_{tot,k}$ is transferred into the soil via contact pressure under the raft σ(x,y) and via the deep foundation elements like piles. The total resistance of the CPRF $R_{tot,k}(s)$ consists of the resistance of the raft $R_{raft,k}(s)$ and of the resistance of the piles $R_{piles,k,j}(s)$ as explained in Equation 1.

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

The resistance $R_{pile,k,j}(s)$ of a single pile j consists of the skin friction $q_{s,k,j}(s,z)$ and the base resistance $q_{b,k,j}(s)$ as explained in Equation 2 to 4.

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

$$R_{b,k,j}(s) = q_{b,k,j}(s) \cdot \frac{\pi \cdot D^2}{4} \tag{3}$$

$$R_{s,k,j}(s) = \int q_{s,k,j}(s,z) \cdot \pi \cdot D \cdot dz \tag{4}$$

Figure 3 shows the soil-structure-interaction of a CPRF. The bearing capacity and the load-settlement behavior are affected by the interaction between

- the foundation raft and the soil
- the piles and the soil
- the piles among each other
- the foundation raft and the piles

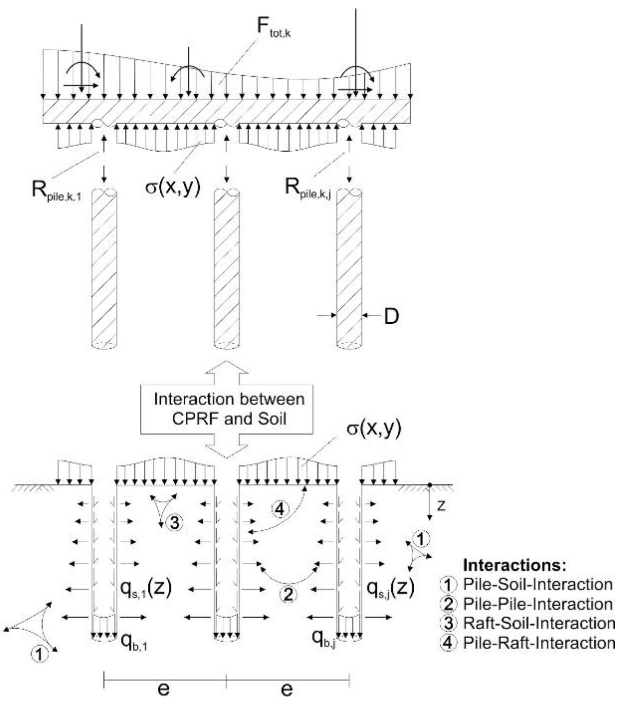


Figure 3 Soil-structure-interaction of a CPRF

The distribution of the total building load between the different bearing structures of a CPRF is described by the CPRF coefficient α_{CPRF} which defines the ratio between the amount of load carried by the piles $\Sigma R_{pile,k,j}(s)$ and the total load of the building $F_{tot,k}$ as explained in Equation 5.

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

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

3.2 Optimization of a CPRF

In order to assess the bearing capacity and the load-settlement behavior and to calculate the internal forces of a CPRF, three-dimensional simulations using the Finite-element-Method (FEM) is suitable. The simulations have to consider the non-linear behavior of the soil and have to be calibrated on back-analysis of laboratory tests and in-situ load tests.

The soil and groundwater conditions on the test site are as follows:

- 0 m – 10.5 m: sand, loose to medium dense
- 10.5 m – 33.5 m: sand, medium dense to dense
- 33.5 m – 38.5 m: clay
- 38.5 m – 44 m: sand, medium dense to dense
- 44 m – 52 m: clay and silt
- 52 m – 57 m: sand, medium dense to dense
- 57 m and deeper: clay and silt

The soil material parameters are summarized in Table 1.

Table 1 Overview of the soil parameters

	angle of friction φ‘ [°]	cohesion c‘ [kN/m²]
non-cohesive soil	30 - 32.5	-
cohesive soil	25	10

For the analysis of the load-deformation behavior the Finite-Element-Method (FEM) with non-linear constitutive equations (elastoplastic) was used. The groundwater level is at the ground surface.

For example the FEM-simulations for a CPRF can be calibrated on a pile load test carried out on the construction yard. For the pile load test, Osterberg Cells (O-Cells) were used. The test pile is divided into 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. The mesh of the FEM-simulation and the principle arrangement of the pile load test equipment with the 3 pile segments and the upper and lower O-Cell are shown in Figure 4.

For determination of the base resistance and the skin friction of pile segments in different soil layers, the O-Cells are activated differently. For determination of the skin friction and the base resistance of pile segment 3, only the lower O-Cell is activated using the segment 2 as abutment. For determination of the skin friction of pile segment 2, the upper O-Cell is activated and the lower O-Cell is unloaded. Pile segment 1 is 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 closed. The pile segments 2 and 3 are used as abutment.

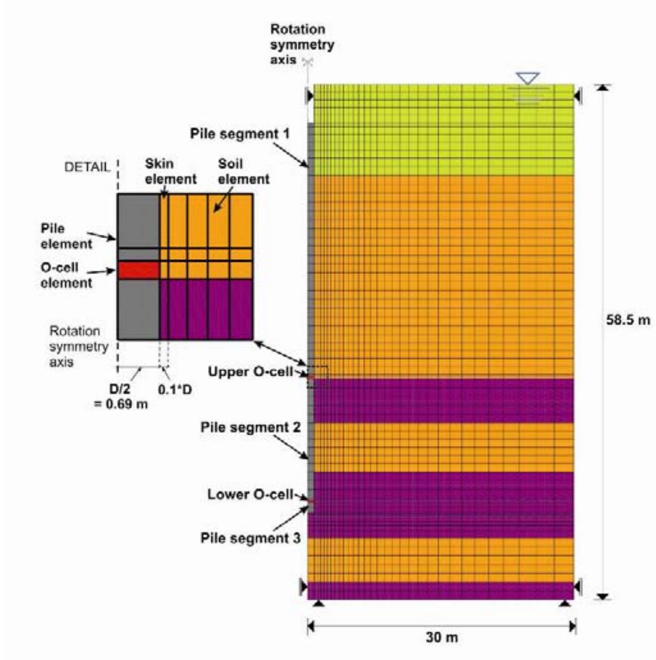


Figure 4 Simulation and principle arrangement of the pile load test

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 for 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 Figure 5. The comparison of the results shows a good accordance.

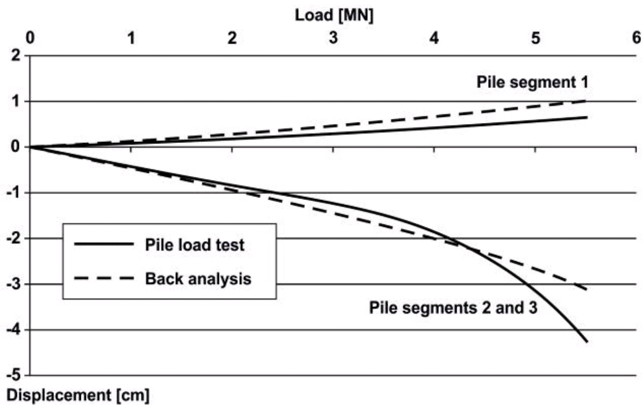


Figure 5 Results of the in-situ pile load test and the back analysis

The whole foundation system was designed by three-dimensional, non-linear FEM-simulations. For the three dimensional FEM-simulations the piles were modelled as square piles with the same shaft area as the constructed radial piles. The thickness of the raft is 1.8 m. The total building load is about 800 MN. The structural engineers defined a maximum differential deformation of 1:750. In this case the stability and the serviceability of the facade and not the inner concrete structure was essential. The length, the diameter and the number of the piles were optimized by the FEM-simulations and adapted to the bearing capacity and the load-settlement behavior of the CPRF. Figure 6 shows the final CPRF design illustrated by the FEM-simulation. The CPRF coefficient is $\alpha_{CPRF} = 0.8$.

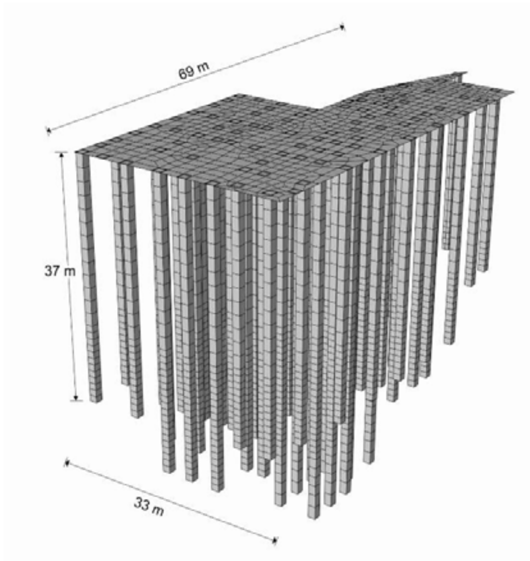


Figure 6 Final CPRF

During the construction phase and for the first years of service time of the building loads of the piles, stresses under the raft and the deformation behavior of the CPRF will be measured by a monitoring program according to the requirements of the observational method.

4. EXAMPLES FROM ENGINEERING PRACTICE

In this chapter several examples from engineering practice are presented. The examples are large challenging constructions projects with classic pile foundation systems or CPRFs.

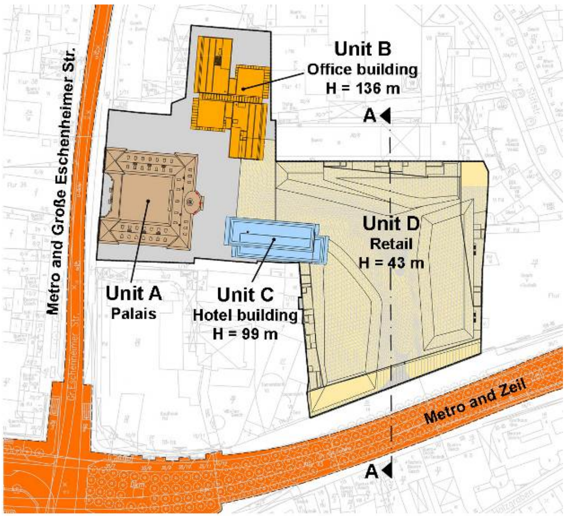
4.1 Classic pile foundation for a high-rise building in clay and limestone

4.1.1 Project description

In the center of Frankfurt am Main, Germany, on a construction site of 17,400 m^2, the high-rise building project "PalaisQuartier" has been realized (Figure 7). The construction was finished in 2010. It is located next to one of the most frequented shopping streets in Germany, the "Zeil" [9].

Figure 7 Project "PalaisQuartier", Frankfurt am Main, Germany

The complex consists of several structures with a total of 180,000 m^2 floor space, thereof 60,000 m^2 underground (Figure 8). The project includes the historic building "Thurn- und Taxis-Palais" whose facade has been preserved (Unit A). The office building (Unit B), which is the highest building of the project with a height of 136 m, has 34 floors each with a floor space of 1,340 m^2. The hotel building (Unit C) has a height of 99 m with 24 upper floors. The retail area (Unit D) runs along the total length of the eastern part of the site and consists of eight upper floors with a total height of 43 m. The underground parking garage with five floors spans across the complete project area. With an 8 m high first sublevel, partially with mezzanine floor, and four more sublevels the foundation depth results to 22 m below ground level. Thereby excavation bottom is at 80 m above sea level (msl). A total of 302 foundation piles (diameter up to 1.86 m, length up to 27 m) reach down to depths of 53.2 m to 70.1 m above sea level depending on the structural requirements. The pile head of the 543 retaining wall piles (diameter 1.5 m, length up to 38 m) were located between 94.1 m and 99.6 m above sea level, the pile base was between 59.8 m and 73.4 m above sea level depending on the structural requirements. As shown in the sectional view (Figure 8), the upper part of the piles is in the Frankfurt Clay and the base of the piles is set in the rocky Frankfurt Limestone.



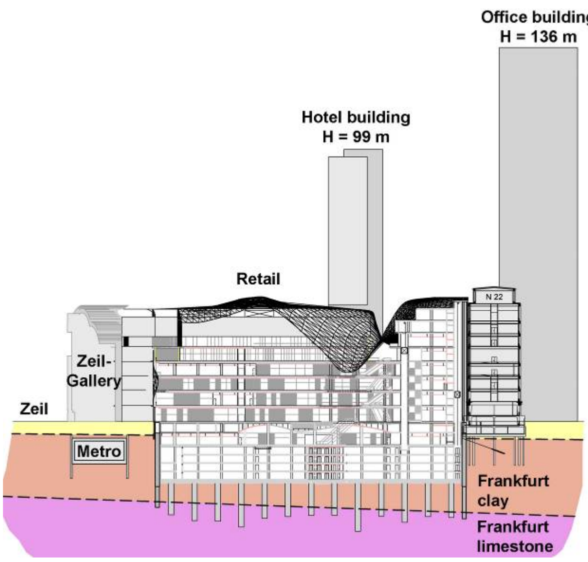
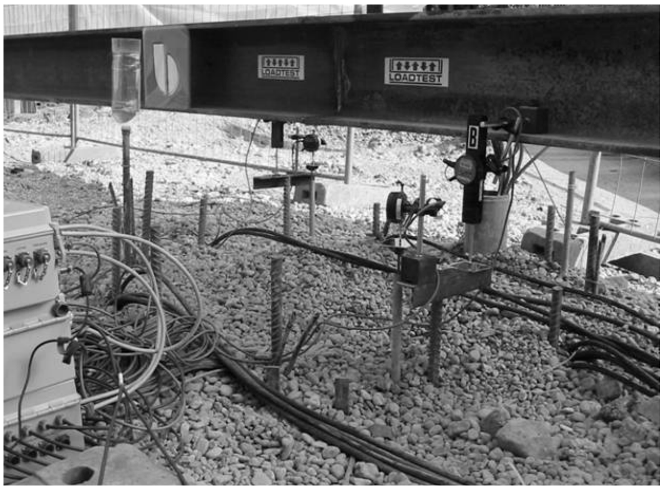


Figure 8 PalaisQuartier complex in Frankfurt am Main, Germany: plan view (top) and cross section A-A (bottom)

4.1.2 Optimization of the foundation system

Regarding the large number of piles and the high pile loads, a pile load test has been carried out for optimization of the classic pile foundation. Osterberg-Cells (O-Cells) have been installed in two levels in order to assess the influence of pile shaft grouting on the limit skin friction of the piles in the Frankfurt Limestone (Figure 9). The test pile with a total length of 12.9 m and a diameter of 1.68 m consists of three segments and has been installed in the Frankfurt Limestone layer 31.7 m below ground level. The upper pile segment above the upper cell level and the middle pile segment between the two cell levels can be tested independently. Pile shaft grouting has been carried out for the middle pile segment. In the first phase of the test, the upper part was loaded by using the middle and the lower part as abutment. A limit of 24 MN could be reached (Figure 10). The upper segment was lifted about 15 mm, the settlement of the middle and lower part was 10 mm. The mobilized shaft friction was about 830 kN/m^2.



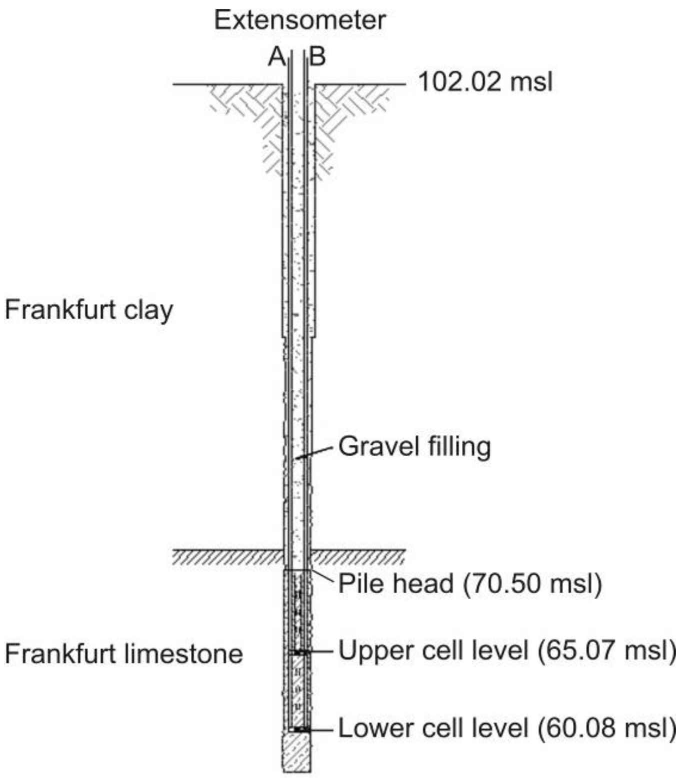


Figure 9 Pile load test setup

Subsequently the upper pile segment was uncoupled by discharging the upper cell level. In the second test phase, the middle pile segment was loaded by using the lower segment as abutment. The limit load of the middle segment with shaft grouting was 27.5 MN (Figure 10). The skin friction was 1,040 kN/m^2, this means 24 % higher than without shaft grouting. Based on the results of the pile load test using O-Cells, the majority of the 290 foundation piles were made by applying shaft grouting. Due to the pile load test the total length of was reduced significantly. The reduction is about 15 % of the total pile length.

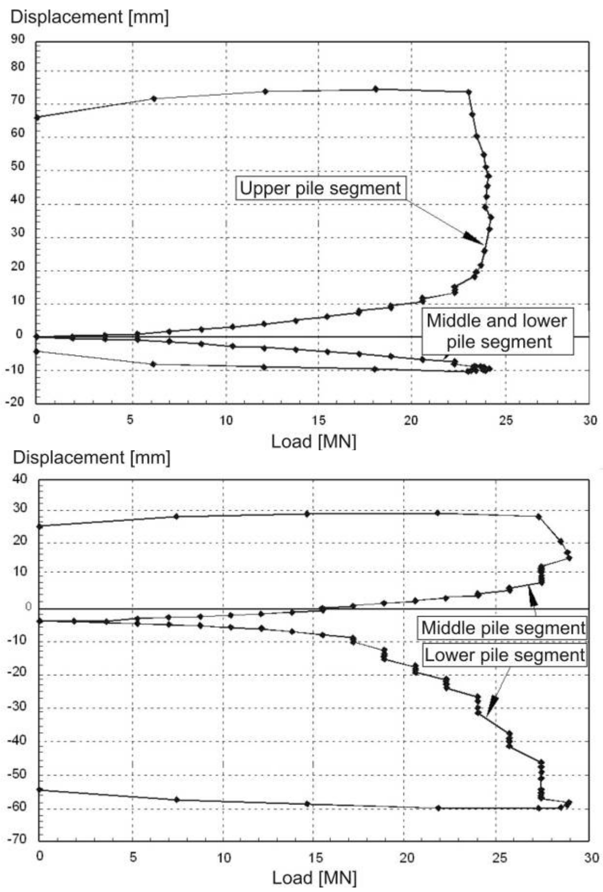


Figure 10 Load displacement curve of test phase 1 (top) and test phase 2 (bottom)

4.2 Classic pile foundation for a high-rise building in clay marl

4.2.1 Project description

Near the central station of Kiev, Ukraine, the International Business Center is under construction. The new building contains a 32 storey high building, a shopping mall, office buildings and an underground parking under the whole construction.

The soil conditions at the construction site are as follows:

- fill to a depth of 5 m to 8 m
- quaternary silty sand and sandy silt with a thickness of about 20 m
- tertiary dense silty sand (Charkow formation) of about 8 m
- greenish silt layer with a thickness of about 2m (Kiev formation) which represents the transition to clay and clay marl
- clay marl of the Kiev formation in a depth of 30 m

The ground water level is in a depth of about 3 m below the ground surface. The soil conditions and a cross section of the project are shown in Figure 11.

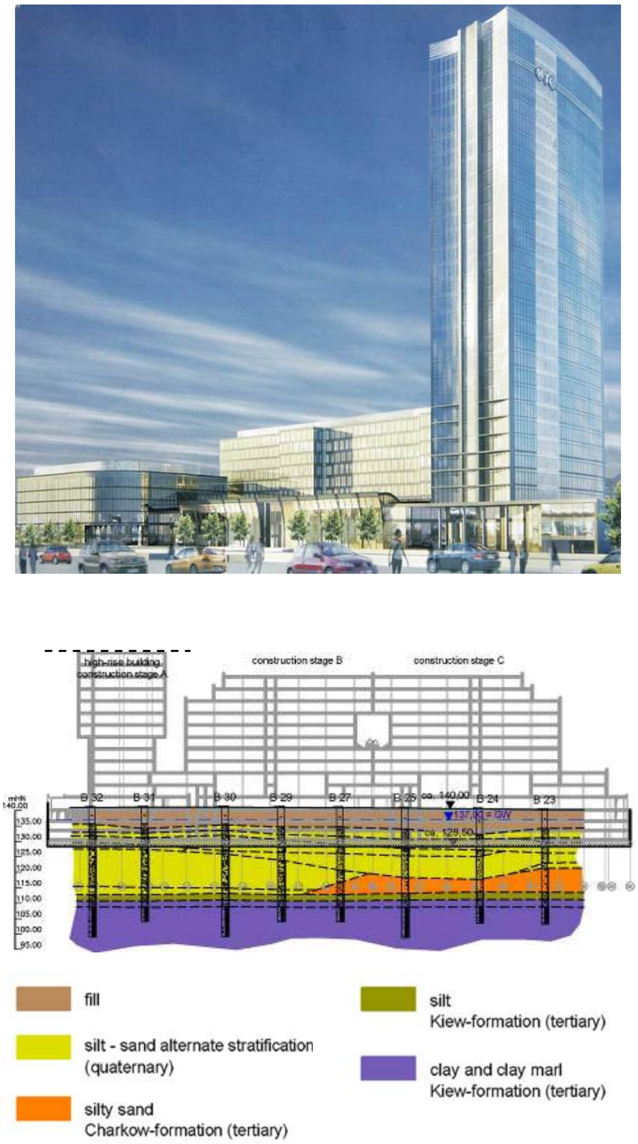


Figure 11 Animation of the finished project (top) and soil conditions (bottom)

4.2.2 Optimization of the foundation system

For optimization of the foundation system, pile load tests using O-Cells were carried out installing a pile with a diameter of 0.88 m and a length of 12 m. Figure 12 shows the test setup.

The base of the pile is located in a depth of 37 m below the ground surface. The empty drilling hole was filled with gravel. Two O-Cells were installed at two levels. The lower O-Cells were installed 0.5 m above the base of the pile in the Kiev clay marl. The upper O-Cells were installed approx. 5.5 m above the pile foot.

The upper pile segment is 6.5 m long and is located in the quaternary sandy silt and in the clay of the tertiary Kiev formation. The test had two different load phases. The load phases are shown in Figure 13.

In the test phase 1 the upper O-Cells were activated in order to determine the skin friction of pile segment 1.

In test phase 2A the lower O-Cells were activated. The upper O-Cells were unlocked so no load transmission was possible to the upper pile. In test phase 2B the upper O-Cells were locked and the load in the lower O-Cells was increased to determine the base resistance. The maximum base resistance of the pile was much higher than the limited load of the O-Cells so no collapse was achieved. The load displacement curves of the phases 1 and 2 are displayed in Figure 14.

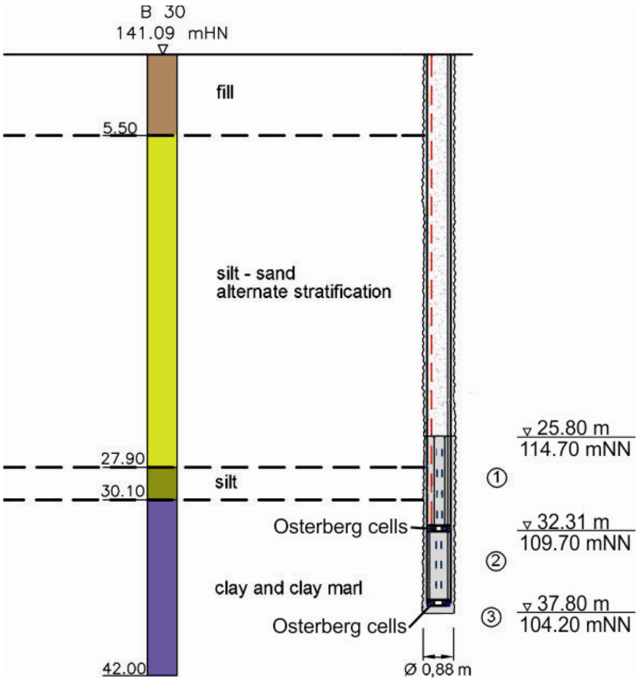


Figure 12 Setup of the pile load test

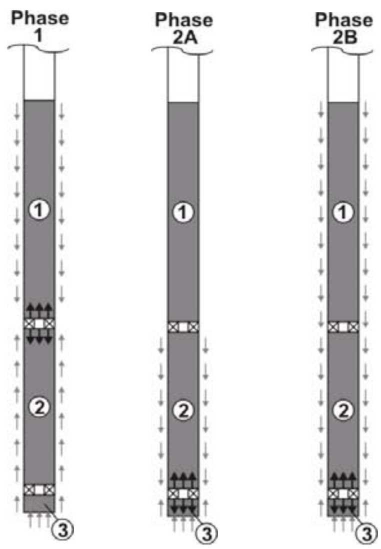


Figure 13 Load phases of the pile load test

The foundation of the high-rise building of the International Business Center Solomenka was planned with 99 barrettes with the dimensions of 2.8 m x 0.8 m up to 6.8 m x 0.8 m and a length up to 46 m. Due to the results of the pile load test the dimensions of the barrettes were optimized. The total length of the barrettes was reduced by approximately 25 %.

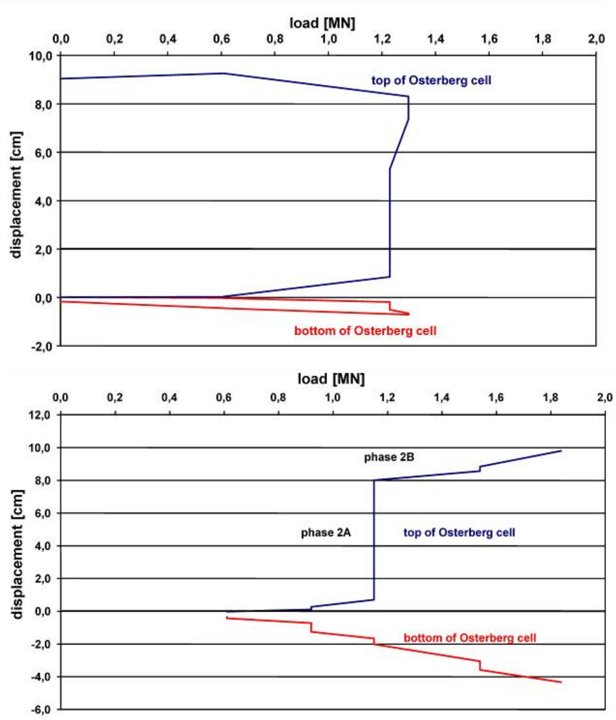


Figure 14 Results of the test phase 1 (top) and 2 (bottom)

4.3 CPRF for a hilgh-rise building in clay marl

4.3.1 Project description

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 (Figure 15). The gross area of the project is about 294,000 m^2 and cuts a 30 m high natural slope.

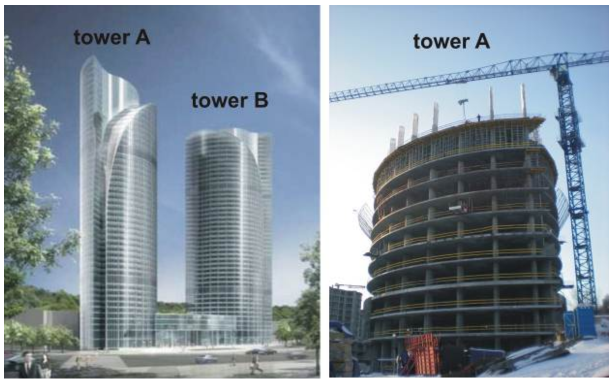


Figure 15 Mirax Plaza Kiev: animation tower A and B (left), tower A under construction (right)

The geotechnical investigations have been executed 70 m deep. The soil conditions at the construction site are as follows:

- fill to a depth of 2 m to 3 m
- quaternary silty sand and sandy silt with a thickness of 5 m to 10 m
- tertiary silt and sand (Charkow and Poltaw formation) with a thickness of 0 m to 24 m
- tertiary clayey silt and clay marl of the Kiev and Butschak formation with a thickness of about 20 m
- tertiary fine sand of the Butschak formation up to the investigation depth

The ground water level is in a depth of about 2 m below the ground surface. The soil conditions and a cross section of the project are shown in Figure 16.

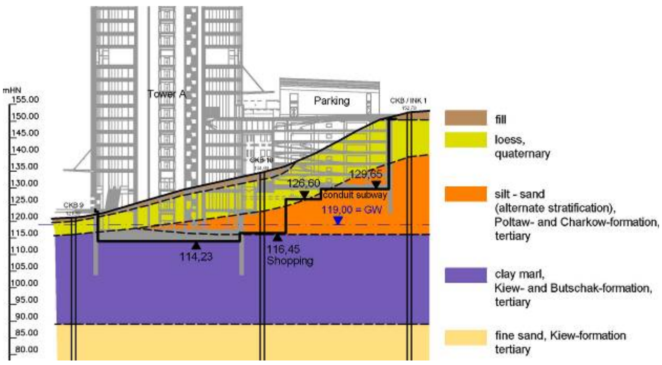


Figure 16 Soil conditions and cross section of the project area

4.3.2 Optimization of the foundation system

For verification of the shaft and base resistance of the deep foundation elements and for calibration of the numerical simulations, pile load tests have been carried out on the construction yard. 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 in accordance with the results of the back analysis were partly 3 times higher than indicated in the geotechnical report. Figure 17 shows the results of the load test No. 2 and the numerical back analysis. Measurement and calculation show a good accordance.

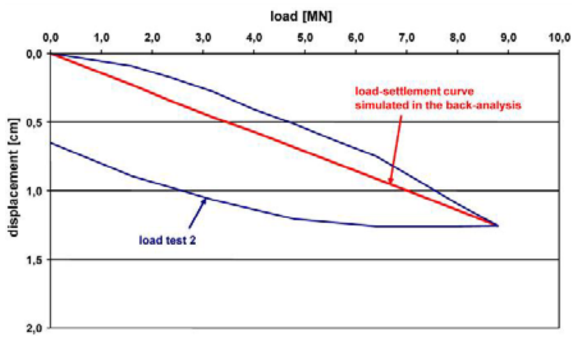


Figure 17 Result of the in-situ load test and the numerical simulation

The obtained results of the pile load tests and of the executed back analysis were applied in 3-dimensional FEM-simulations of the foundation for Tower A, taking advantage of the symmetry of the footprint of the building. The overall load of the Tower A is about 2,200 MN and the area of the foundation about 2,000 m^2 (Figure 18).

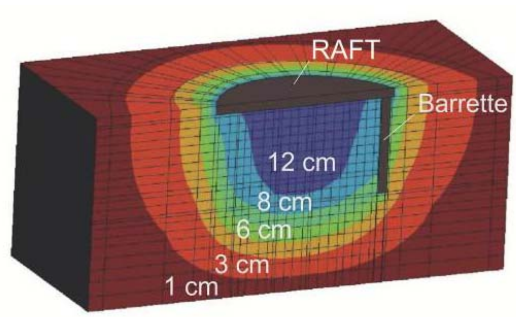


Figure 18 FEM-model of the CPRF of tower A and calculated settlements in [cm]

The foundation design considers a CPRF with 64 barrettes with 33 m 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 are penetrating the layer of Kiev Clay Marl reaching the Butschak Sands.

The calculated loads on the barrettes were in the range of 22.1 MN to 44.5 MN. The load 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 behavior 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 neighboring elements. The CPRF coefficient is α_{CPRF} = 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 is calculated in the most areas not exceeding 200 kN/m^2, at the raft edge the pressure reaches 400 kN/m^2. The calculated base pressure of the outer barrettes has an average of 5,100 kN/m^2 and for inner barrettes an average of 4,130 kN/m^2. The mobilized shaft resistance increases with the depth reaching 180 kN/m^2 for outer barrettes and 150 kN/m^2 for inner barrettes.

During the construction of Mirax Plaza, the observational method according to EC 7 [1] is 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 (most loaded outer barrette and average loaded inner barrette) were instrumented over the length.

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

The CPRF of the high-rise building project Mirax Plaza represents the first authorized CPRF in the Ukraine. Using the advanced optimization approaches and taking advantage of the positive effect of CPRF, the number of barrettes could be reduced from 120 barrettes with 40 m length to 64 barrettes with 33 m length. The foundation optimization leads to considerable decrease of the utilized resources (cement, aggregates, water, energy etc.) and to cost savings of about 3.3 Million US$.

4.4 Combined foundation system on a fault zone

4.4.1 Project description

In Darmstadt south of Frankfurt am Main, Germany, the conference center Darmstadtium was realized directly over the Rhine Valley fault. The challenging construction was finished in 2007. Figure 19 gives an impression of the whole structure.

Figure 19 Conference center Darmstadtium, Darmstadt, Germany

The results of the soil investigation in the planning stages showed that under the building the Rhine Valley Fault divides the subsoil in two parts (Figures 20 and 21). In the northern and western parts of the project area, the soil consists of the sediments of the

Rhine valley. In the eastern and southern parts of the project area, the soil consists of rock, the so called Granodiorit.

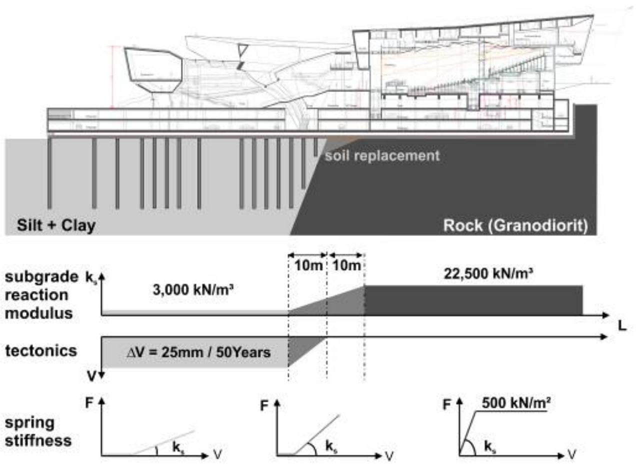


Figure 20 Excavation of the Darmstadtium

4.4.2 Optimization of the foundation system

Up to now the tectonic processes along the Rhine Valley Fault did not fade out. In the west of the Rhine Valley Fault the surface of Darmstadt sinks up to 0.5 mm per year. The foundation system and the superstructure had to be dimensioned for this tectonic deformation. In the rock a raft foundation and in the area of the Rhine Valley a CPRF was constructed. At the transition of the stiff rock to the sediments a soil replacement was carried out (Figure 21). Additionally to the combined foundation system and the soil replacement the superstructure is constructed with joints to allow differential deformations.

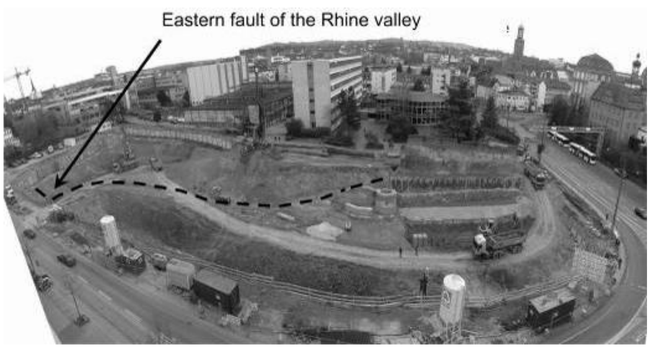


Figure 21 Cross section of the Darmstadtium

4.5 CPRF in Frankfurt Clay

4.5.1 Project description

During the independent peer review process for a new high-rise building in Frankfurt am Main, Germany, the publicly certified expert for structural engineering informed the building authorities to involve a publicly certified expert for geotechnics.

Due to the complexity the construction project is categorized into the Geotechnical Category 3 according EC 7. This is the category with the highest requirements. The foundation system was planned as a Combined Pile-Raft Foundation (CPRF). The CPRF guideline documented in [7] and [8] requires to involve a publicly certified expert for geotechnics for the review process as well. It was estimated in the early planning stage that the soil and ground water conditions are common for the Frankfurt subsoil:

- fill to a depth of 10 m followed by quaternary gravel to variable depth
- beneath the quaternary gravel is the Frankfurt Clay consisting of stiff to semi hard clay with thin limestone layers and sand layers
- the basis is the Frankfurt Limestone

Due to this estimation the foundation system was planned as a CPRF reaching the Frankfurt Clay. Figure 22 shows the finished high-rise building.

Figure 22 View on the finalised high-rise building

4.5.2 Optimization of the foundation system

For the foundation a CPRF was planned like shown on the top of Figure 23. During the review process additional soil investigations were carried out. The results indicated that there is a discontinuity in the intersection between the settlement active Frankfurt Clay and the rocky Frankfurt Limestone of several meters. The top of the Frankfurt Limestone is not as deep as estimated so the piles of the CPRF would have been partly in the limestone (Figure 23 in the middle).

In this case the piles would have had a much higher resistance due to the very stiff Frankfurt Limestone. Damages would occur in the foundation raft and the superstructure because the piles founded in the rock would get a higher load than the other piles founded in the Frankfurt Clay. Differential settlements would also occur.

On basis of the report of the publicly certified expert for geotechnics, the design of the foundation system was adapted and the piles of the CPRF were shortened (Figure 23 on the bottom). The whole construction was realized without any damage. The different stages of the foundation design are shown in Figure 23:

- on top: first planning stage
- middle: after the additional soil investigations
- bottom: realized foundation system after the report of the publicly certified expert for geotechnics

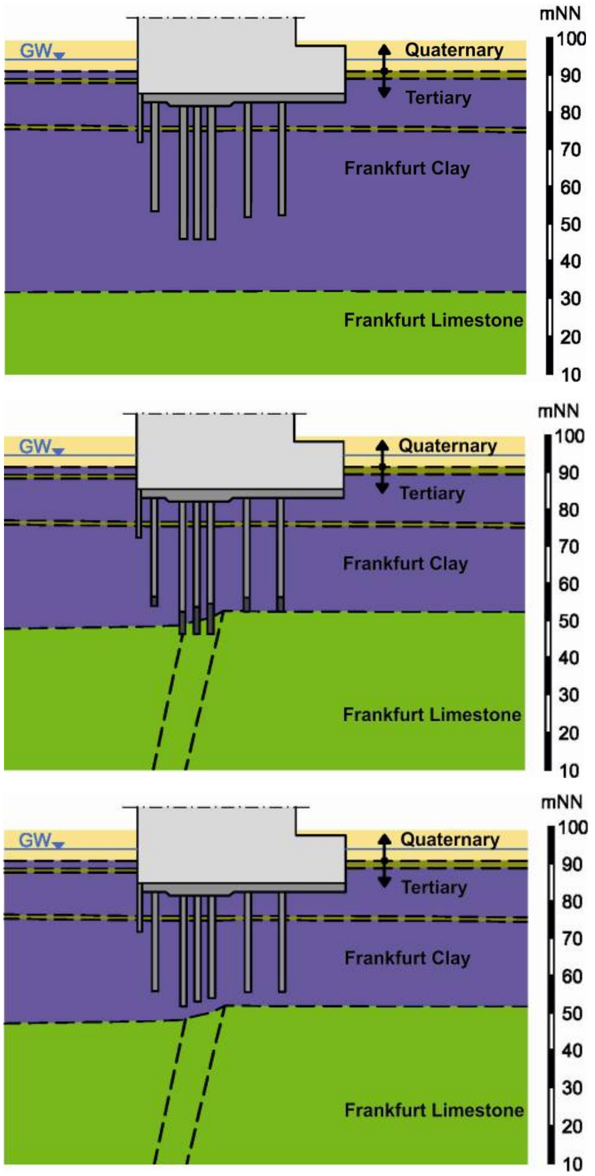


Figure 23 Development of the deep foundation system in different planning stages

5. CONCLUSION

An adequate soil and ground water investigation program is the basis for a safe and economic design and construction of geotechnical structures. For complex construction projects the observational method can be applied for verification of the results of the soil investigation and of the analyses carried out during the design phase. In any case the 4-eye-principle is the only possibility for maximum risk mitigation. Essential parts of the 4-eye-principle are the publicly certified experts for geotechnics and the independent peer review process.

For the optimization of deep foundation systems, in-situ load tests on the construction site are very utile. As shown by the presented examples the results are used for determination of possible bearing capacities including the load-deformation behavior and they can be used for calibration of numerical analysis, e.g. FEM-based simulations. The results of the in-situ load test have to further positive effects. On one hand the detected shaft resistance and base resistance may help to reduce the total pile length significantly. On the other hand the results give the possibility to check the estimated bearing capacity of the deep foundation elements for risk mitigation.

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