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Design and Construction of Deep Foundation Systems and Retaining Structures in Urban Areas in Difficult Soil and Groundwater Conditions

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Abstract

Economic and environment-friendly design focuses on a reduction of construction material used, construction time spent and energy consumed within the buildings construction and service time. Regarding deep foundation systems and retaining structures for excavations this paper highlights the important role of enhanced geotechnical design and independent quality assurance by means of the 4-eye-principle.

Selection and peer-review under responsibility of the Vilnius Gediminas Technical University

Keywords: deep foundation, deep excavation, soil-structure-interaction, independent peer review, 4-eye-principle, observational method, load test, Finite-Element-Method (FEM), back analysis.

1. Introduction

A successful design and construction of challenging project 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
- design of 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 processes and the observational method (4-eye-principle)

These aspects will be explained by two large construction projects in Germany and West Africa which are located in difficult soil and groundwater conditions. The last above mentioned aspect will be described in detail.

2. Quality assurance

The quality assurance is defined by the 4-eye-principle. This 4-eye-principle is a process of an independent peer review as shown in Fig. 1. It consists of 3 parts. The investor, the experts for planning and design and the construction company belong to the first division. Planning and design is done according to the requirements of the investor and all relevant documents to obtain the building permission are prepared. The building authorities are the second part and are responsible for the building permission which is given to the investor. The third division consists of the publicly certified experts. They

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are appointed by the building authorities but work as independent experts. They are responsible for the technical supervision of the planning, design and the construction.

In order to achieve the licence as a publicly certified expert for geotechnical engineering by the building authorities intensive studies of geotechnical engineering in university and large experiences in geotechnical engineering with special knowledge about the soil-structure-interaction have to be proven.

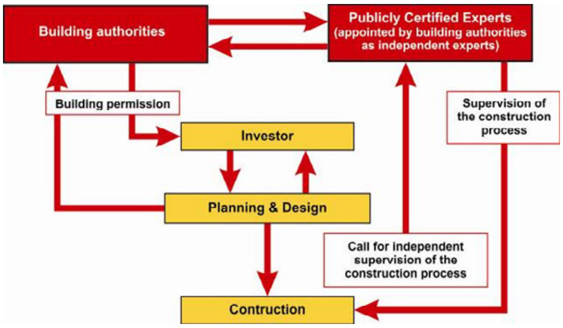

Fig. 1. Independent peer review process

The basis of the independent peer review by publicly certified experts for geotechnical engineering makes sure that all information including the results of the soil investigation consisting of laboratory and field tests and the boundary conditions defined for the geotechnical design are complete and correct.

In the case of a defect or collapse the publicly certified expert for geotechnical engineering can be involved as an independent expert to find out the reasons for the defect or damage and to develop a concept for stabilisation and reconstruction [1].

For all difficult projects an independent peer review is essential for the successful realisation of the project. These are for example deep foundation systems and retaining structures for deep excavations. The term "deep excavation" is established in the technical terminology of civil engineering [2–5]. For the quantitative demarcation the "Deep Excavation Index T_{EI}" is defined in Eq. (1).

$$T_{EI} = \frac{H_{res}}{E_S} \cdot 100 \quad [\mathrm{m}] \tag{1}$$

The deep excavation index T_{EI} is a relation of the horizontal forces H_{res} and the oedometric modulus E_S. The resulting horizontal forces H_{res} includes the excavation depth, the active earth pressure and the groundwater pressure. The oedometric modulus E_S describes the soil properties.

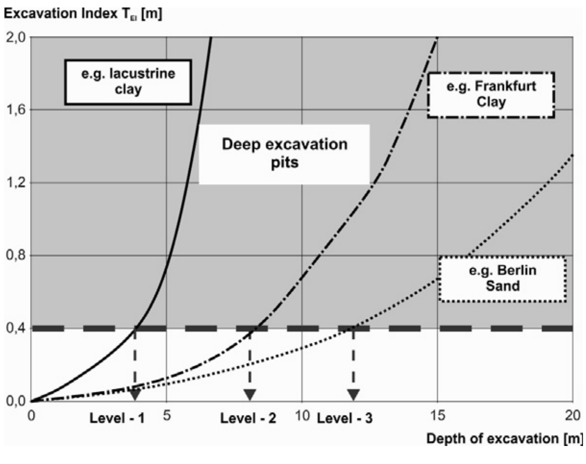

Fig. 2. Deep excavation indexes

With T_{EI} > 0.4 m an excavation can be classified as a deep excavation. Fig. 2 shows some examples for the deep excavation index T_{EI} > 0.4 m. Excavations are defined as deep when the excavation depth in lacustrine clay is less than 4 m, in Frankfurt Clay is less than 8 m and in Berlin Sand is less than 12 m.

3. Large construction project close to underground infrastructures

In the city centre of Frankfurt am Main, Germany, a large construction project with an area of about 30,000 m^2 is in progress. The existing buildings are deconstructed. One building in the north west of the project area will be modified. The new and the modified buildings are up to 23 storeys high and have up to 4 sublevels.

The project area is located north of the river Main. Fig. 3 shows an overview of the project. The project area borders on tunnels of the urban metro system in the west and the north. Additionally in the north a road tunnel is located directly above the metro tunnel with a distance to the planned northern buildings of less than 4 m. The challenges of this project are:

- the interaction between the existing buildings, the underground structures and the new respectively modified buildings
- the retaining systems close to the existing underground structures
- the deep foundation systems in difficult soil and groundwater conditions
- guarantee for stability and serviceability of the historic building in the south of the project area

The project is categorised into the Geotechnical Category GC 3 of the Eurocode 7 [6]. This is the category for very difficult projects or very difficult soil and groundwater conditions. Due to this fact an independent geotechnical expert for peer review and supervision is involved in the planning, design and construction phases and an extensive monitoring program according the observational method was installed.

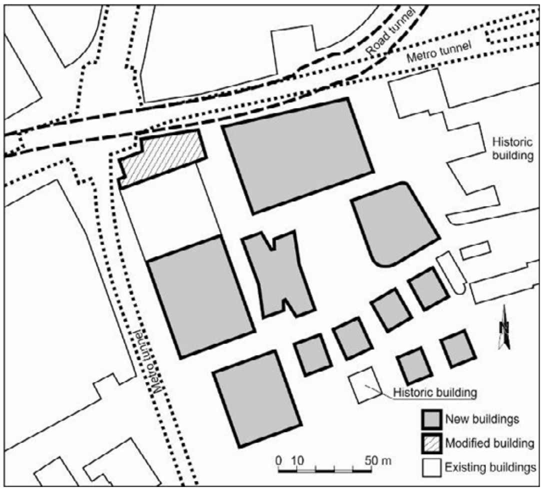

Fig. 3. Overview of the project area

3.1. Soil and groundwater conditions

The soil investigation was carried out up to a depth of 60 m below the surface.

The stratigraphy is as follows:

- filling up to a depth of 9 m below surface
- quaternary sand and gravel up to a depth of 11 m below surface
- tertiary Frankfurt Clay up to a depth of 35 m below surface followed by
- the Frankfurt Limestone

The tertiary Frankfurt Clay is a very settlement active, cohesive soil material that has a strongly time dependent deformation behaviour [7, 8]. In comparison to the quaternary and tertiary soil layers the Frankfurt Limestone is very stiff.

In the course of the geotechnical investigation program two aquifers have been encountered. The lower confined groundwater layer is located in the Frankfurt Clay and the Frankfurt Limestone. The upper groundwater level is located in the quaternary sand and gravel respectively in the filling. The groundwater levels are influenced by the river Main which is 50 m south of the project area.

3.2. Retaining system

The extensive monitoring program includes geotechnical and geodetic measurements. Geodetic measurement points are positioned on and around the project area:

- in the northern metro tunnel
- in the western metro tunnel
- in the northern road tunnel
- on the block foundations of the catenary support of the tram lines north of the project area
- inside and on the modified building in the north west
- inside and on the historic building in the south
- inside and on the buildings around the project area
- on the bridge over the river Main in the south west
- at the retaining systems

Additionally joint monitoring systems are installed in the metro and road tunnels. The following geotechnical measurement devices are installed on and around the project area:

- inclinometer behind the retaining systems
- load cells at struts
- load cells at anchors
- load cells under piles of the deep foundation systems
- contact pressure measurement devices under the rafts
- extensometer under the foundation raft

The groundwater levels are measured continuously in combination with the level of the river Main. In addition to the measurement program an emergency plan was developed.

Fig. 4. Retaining structure in the north of the excavation

The retaining system is an overlapping bored pile wall with an H-beam wall on the first 2 meters below the surface. The struts are angular steel tubes founded on the level of the raft foundation. Anchors were installed where it was possible regarding the underground structures. Fig. 4 shows on the left side the retaining structure in the north of the excavation. At that construction stage the excavation is nearly finished and the first parts of the superstructure are built. On the left side of Fig. 4 behind the retaining structure are the road and the metro tunnels.

Up to now no critical deformations and deformation rates occurred. For example the measurement results of one inclinometer behind the retaining structure in the north of the excavation are shown in Fig. 5 when the excavation was finished and the struts were installed. The maximum excavation depth is 10.5 m below the surface. The struts are installed 3 m and 6 m below the surface. The maximum horizontal deformation of the retaining structure is 1 cm at the surface and about 0.4 cm at the excavation level.

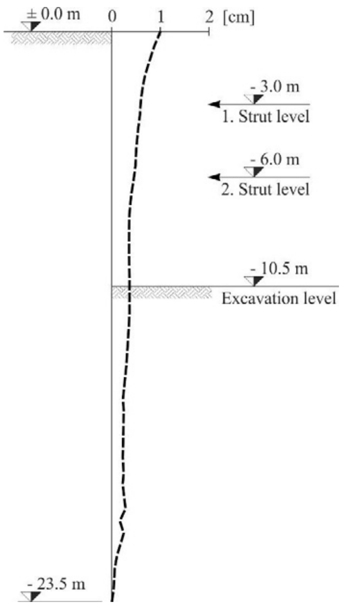

Fig. 5. Measured horizontal deformation of the retaining structure in the north of the excavation

3.3. Foundation system

Depending on the size of the loads and the nonuniform load distribution of the superstructure the foundation systems of the new buildings are rafts or Combined Pile-Raft Foundations (CPRF). The CPRF is an optimised foundation system for high-rise buildings [9–13]. The foundation of the modified building had to be intensified. Due to the limited space inside the existing building micropiles were used as additional deep foundation elements.

For an optimisation of the number and of the geometry and for the verification of the bearing behaviour of the micropiles in-situ load test were carried out. The tested micropiles had a length of 15 m and a diameter of 0.2 m. The test piles are used as foundation piles as well so they were not loaded until their ultimate bearing capacity but to the load they have to carry. Fig. 6 shows the result of one of the in-situ load tests. The pile was loaded up to 1,200 kN. The vertical displacement of the head of the micropile at that test load was 1.1 cm. The micropiles are able to transfer the intended load of the superstructure into the subsoil ensuring the stability and the serviceability of the whole modified structure.

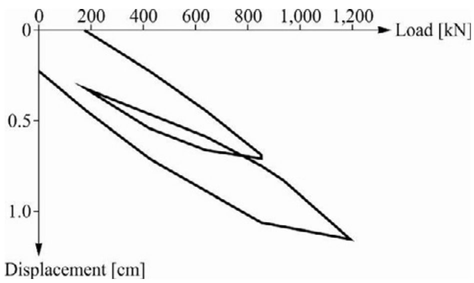

Fig. 6. In-situ load test of a micropile

4. Foundation of a high-rise building in settlement sensitive soil

At the coastline of West Africa a more than 75 m high-rise building in settlement sensitive soil is planned. The high-rise building will have up to 16 storeys. The foundation system is designed as a Combined Pile-Raft Foundation (CPRF). The annexe buildings are up to 60 m high and include apartments and parking levels. All structures will have one basement level. The whole structure will have a total load of more than 700 MN. Due to its complexity the project is categorised into the Geotechnical Category GK 3 of the Eurocode 7.

4.1. Soil and groundwater conditions

The soil investigation was carried out up to a depth of 80 m below the surface.

The following layers were detected:

- clayey sands at the surface followed by an
- alternating sequence of loose, medium dense and dense sand layers up to a depth of about 33 m below the surface followed by an
- alternating sequence of medium dense and dense sand layers and clay and silt layers with low to high plasticity down to the investigation depth

The groundwater level is detected close to the surface.

4.2. Combined Pile-Raft Foundation (CPRF)

The foundation system is designed as a 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 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 [14].

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 $R_{raft}(s)$ and of the resistance of the piles $R_{piles,k,j}(s)$ as explained in Eq. (2).

$$R_{tot,k}(s) = \sum_{j=1}^{m} R_{pile,k,j}(s) + R_{raft,k}(s) \tag{2}$$

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 Eq. (3) to (5).

$$R_{pile,k,j}(s) = R_{b,k,j}(s) + R_{s,k,j}(s) \tag{3}$$

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

$$R_{s,k,j}(s) = \int q_{s,k,j}(s,z) \cdot \pi \cdot D \, dz \tag{5}$$

Fig. 7 shows the soil-structure-interaction of a CPRF. The bearing capacity and the load-settlement behaviour 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

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 Eq. (6).

$$\alpha_{CPRF} = \frac{\sum R_{pile,k,j}(s)}{F_{tot,k}} \tag{6}$$

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.

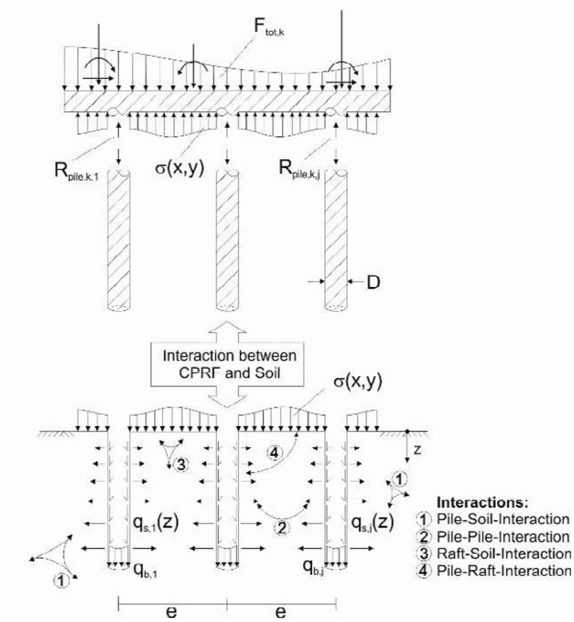

Fig. 7. Soil-structure-interaction of a CPRF

4.3. Optimisation of the CPRF

In order to assess the bearing capacity and the load-settlement behaviour and to calculate the internal forces of a CPRF three-dimensional simulations using the Finite-Element-Method (FEM) are suitable.

The simulations have to consider the non-linear behaviour of the soil and have to be calibrated on back-analysis of laboratory tests and in-situ load test.

At the current project the FEM-simulations for the CPRF are 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.

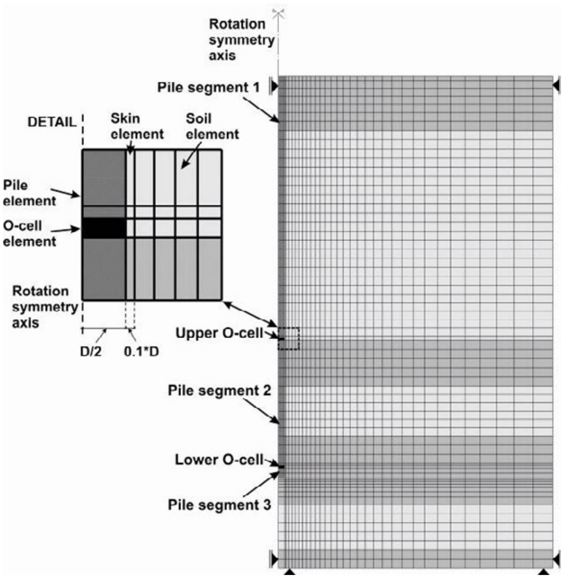

Fig. 8. Simulation and principle arrangement of the pile load test

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 load O-Cell was loaded and the lower O-Cell was closed. The pile segments 2 and 3 are used as abutment.

Fig. 8 shows 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-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 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 Fig. 9. The comparison of the results shows a good accordance.

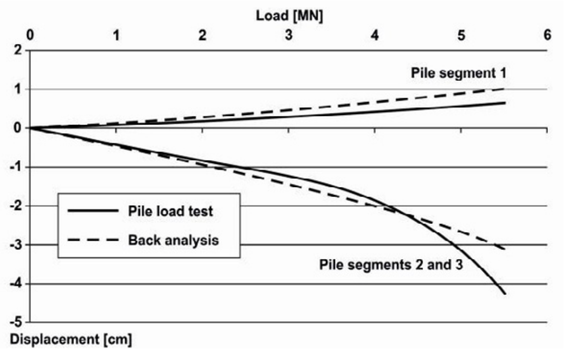

Fig. 9. Results of the in-situ pile load test and the back analysis

The whole foundation system was design by three-dimensional, non-linear FEM-simulations. The length, the diameter and the number of the piles were optimised by the FEM-simulations and adapted to the bearing capacity and the load-settlement behaviour of the CPRF. Fig. 10 shows the final CPRF design illustrated by the FEM-Simulation. The CPRF coefficient is $\alpha_{CPRF} = 0.8$.

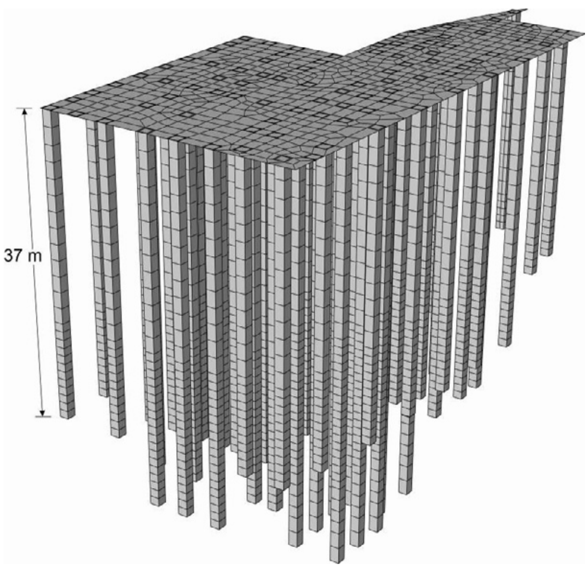

Fig. 10. Final CPRF

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 behaviour of the CPRF are measured by a monitoring program according to the requirements of the observational method.

5. Conclusions

The paper shows on two examples the important role of enhanced geotechnical design and the independent quality assurance by means of the 4-eye-principle.

The paper also explains how to optimise geotechnical structures without loosing safety and serviceability.

Various types of retaining structures are available and state of practice. The “Deep Excavation Index” T_{EI} allows accounting for the governing parameters bringing along the particular challenges in design of deep excavations including geometry and soil properties.

For the optimisation of deep foundation systems in-situ load tests on the construction yard are very utile. The results are used for determination of possible bearing capacities including the load-deformation behaviour and they can be used for calibration of numerical analysis, e.g. FEM-based simulations.

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