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INDUSTRIAL APPLICATION

Assessing Settlement of High-Rise Structures by 3D Simulations

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Abstract: *A number of new high-rise buildings in the Frankfurt region are currently in their planning stage. Most of these high-rise structures will be erected on combined piled-raft foundations (CPRFs). Due to the complex interaction between piles, raft, and subsoil the difficult design of these foundations will be carried out by three-dimensional finite-element (FE) simulations. For the 121 m high* City-Tower*, which is currently under construction, the design procedure based on a three-dimensional FE simulation of the CPRF is described. The design process for the new 228 m high office tower* Max*, which will be located in the financial district of Frankfurt, in the direct vicinity of already existing high-rise buildings, has just started. To improve and verify the input parameters for the constitutive modeling and to allow for a cost optimized foundation design, a numerical back-analysis of the 110 m high* Eurotheum*, located close to the building site of* Max*, has been performed. For this building comprehensive measurements were carried out starting in the construction stage and lasting up to the present day.*

1 INTRODUCTION

After 1950 in Frankfurt and its metropolitan region a massive structural change took place. The service sector became more and more important. For the city development this process was comparable to the industrialization at the beginning of the 20th century. From the discussions and interviews being printed and published in daily newspapers one easily comes to the conclusion that the high-rise boom in Frankfurt for more office towers in Europe's new financial capital has just begun. Frankfurt grows not only in size, with new housing areas in the surrounding, but also in height (Figure 1). According to several statements of the planning board of the city of Frankfurt there are more than 22 spaces for potential office towers with at least 90 m (300 ft) of height. Together with the existing 73 high-rise buildings there will be nearly 100 towers within the city borders. In most cases, existing structures will be demolished and replaced by new constructions, a task which will be demanding for all areas of civil engineering, especially for geotechnical engineering as nearly all office towers will have to be founded on piled foundations or on combined piled-raft foundations (CPRFs).

When considering foundations for high-rise buildings in urban areas, a major task is the reduction of settlements and differential settlements of the new structures and adjacent buildings to ensure their safety and serviceability, especially under the long-life aspect and reuse of foundations. In many cases, the soil conditions can lead to deep foundations to transfer the high loads of the building into deep soil strata with higher bearing capacities. Compared to traditional piled foundations, where building loads are assumed to be transferred to the soil only by piles, the CPRF is a new approach. A CPRF consists of the three bearing elements piles, raft, and subsoil. Load sharing between piles and raft is taken into consideration, and the piles can be used up to a load level equal or greater than the bearing capacity of a comparable single pile. This design concept can lead to a considerable saving of construction time and resources compared to the traditional piled foundations.

2 BEARING BEHAVIOR OF A VERTICAL LOADED CPRF

According to its stiffness the CPRF transfers the total vertical load of the structure R_{tot} into the subsoil by

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Fig. 1. Skyline of Frankfurt am Main.

contact pressure of the raft R_{raft} as well as by the piles $\sum R_{\text{pile},i}$:

$$R_{\text{tot}} = \sum R_{\text{pile},i} + R_{\text{raft}} \tag{1}$$

In comparison with a conventional foundation design of a pile group, a new design philosophy with different and more complicated soil–structure interaction is applied for CPRFs. In this design philosophy, piles are used up to a load level that can be even higher than permissible design values for bearing capacities of comparable single piles. The distribution of the total building load between the different bearing structures of a CPRF is descry, bed by the CPRF coefficient α_{CPRF}, which defines the ratio between the amount of the pile loads $\sum R_{\text{pile},i}$, and the total load of the building R_{tot}:

$$\alpha_{\text{CPRF}} = \frac{\sum R_{\text{pile},i}}{R_{\text{tot}}} \tag{2}$$

To investigate the bearing behavior of a CPRF a number of different interactions as depicted in Figure 2 have to be considered. A suitable modeling technique has to include all these different types of interactions.

In Figure 3, qualitatively the obtainable settlement reduction $s_{\text{CPRF}}/s_{\text{RF}}$ is given as a function of the combined piled raft coefficient α_{CPRF} (Katzenbach et al., 1999a), where s_{CPRF} and s_{RF} are the settlements of the CPRF and a raft foundation (RF) of the same size. In general the value of α_{CPRF} varies between 0.4 and 0.7 (Katzenbach et al., 1998). For a value of $\alpha_{\text{CPRF}} = 0$ the load is transferred only through the raft whereas for $\alpha_{\text{CPRF}} = 1.0$ the load is transferred only through the piles.

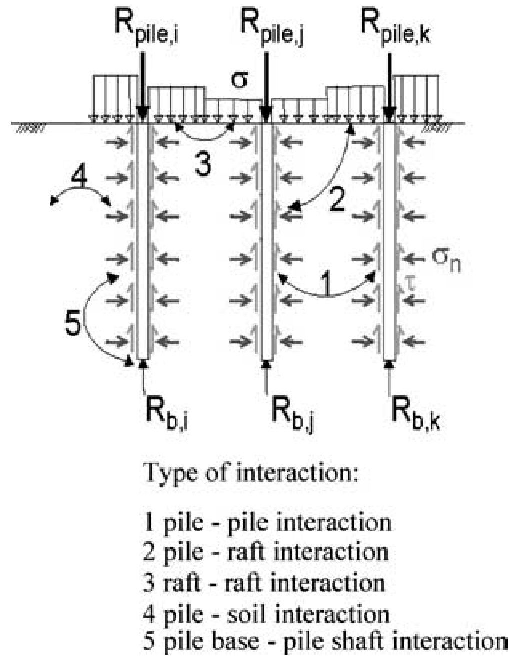

Fig. 2. Soil–structure interaction between raft, piles, and subsoil.

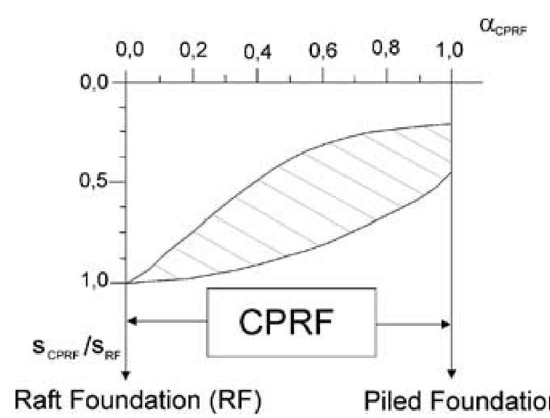

Fig. 3. Example for the settlement reduction of a CPRF as a function of α_{CPRF}.

3 EXPERIENCE GAINED ON CPRFS

The experience gained is based on settlement and load measurements on projects carried out so far, as well as on numerical computations and their validation with the help of the measurements. The use of numerical simulations has become an essential part of research performed to find a suitable design concept and a credible explanation of interactions.

Starting in the early 1980s, first CPRFs came under use mainly for high-rise office buildings in Frankfurt

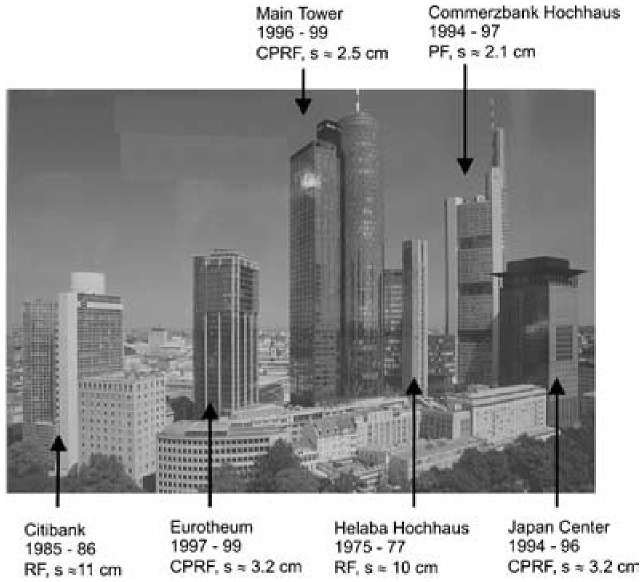

Fig. 4. Examples of deep foundations for high-rise buildings in Frankfurt am Main. RF: raft foundation, CPRF: combined piled-raft foundation, PF: piled foundation, s: settlement after finishing construction.

am Main (Figure 4) to reduce settlements to practicable dimensions and to ensure serviceability by reducing differential settlements to a minimum in an economical way. This undoubtedly would not have been possible to achieve with a simple raft. Compared to traditional piled foundations the cost reduction was immense.

In the following, the example of the office tower City-Tower in Offenbach with its geometrical model of the continuum and the constitutive modeling is described.

4 AN EXAMPLE FOR THE DESIGN PROCEDURE FOR THE FOUNDATION OF HIGH-RISE BUILDINGS

The principal design procedure for a high-rise building foundation is described exemplarily for the office building City-Tower (Figure 5), which is currently under construction. The tower in the outskirts of Frankfurt is about 121 m high and founded in clay on a CPRF with large diameter bored piles. At a distance of about 4 m from the foundation of the tower a railway tunnel is situated 3 m below ground surface. An important task was to guarantee the serviceability of the tunnel during the whole construction process and further on. Numerical analyses were performed with a three-dimensional finite-element (FE) model at the Institute of Geotechnics in Darmstadt.

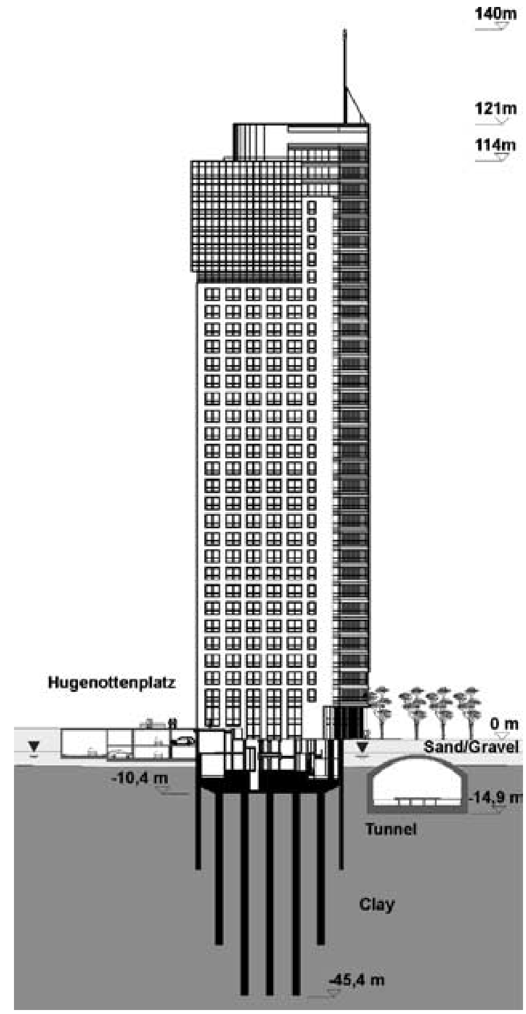

Fig. 5. Cross-section of the City-Tower.

5 FINITE-ELEMENT MESH

Based on the load distribution obtained from the structural engineer and the twofold symmetry of the geometry, the FE mesh could be reduced to a half of the area to be considered with a total number of 10,365 elements (Figure 6). The soil and piles are represented by first-order solid elements of brick and wedge shape, and the raft was modeled with first-order shell elements. For the contact zone between soil and foundation (raft and piles) thin solid continuum elements with the material behavior of the soil have been used. The three-dimensional mesh generation was performed by using the preprocessor PATRAN. FE simulations were carried out with the program ABAQUS.

Several simulations were performed to optimize the foundation design and to assess the appropriate pile length, diameter, and location of each pile under the

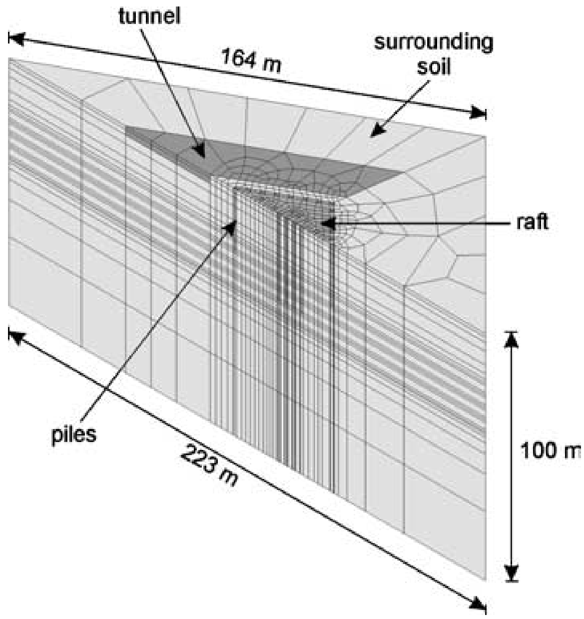

Fig. 6. FE mesh of the CITY-TOWER foundation.

raft. These simulations also consider the preloading of soil by old buildings that had been demolished before the construction process of the CITY-TOWER started. The final foundation design consists of 36 piles with a pile length between 25 and 35 m. The pile length increases from 25 m for the outer piles to 35 m for the piles located in the center of the raft. The diameter of all piles is 1.50 m, the thickness of the raft is about 3 m.

The total load (dead load G + service load P) of the building considered within the simulation is about 600 MN. The settlement calculated for $G + 1/3P$ reaches a maximum of about 6 cm at the center of the CPRF. The differential settlement is about 1 cm between the center of the CPRF and its edges. The horizontal displacement of the adjacent tunnel with a predicted value of 0.5–1.4 cm was within the acceptable range.

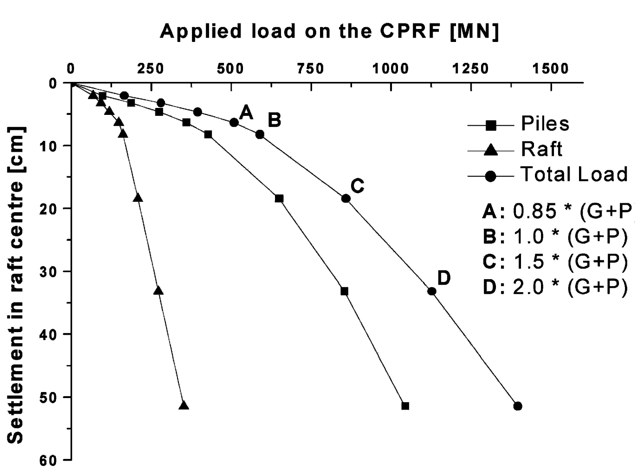

Fig. 7. Load–settlement curves derived from FE simulation.

In Figure 7, the load–settlement curves derived from one of the FE simulations for the CPRF are given for the entire foundation structure, the piles and the raft. Some characteristic load levels are marked (A–D).

6 MATERIAL MODELS

The soil, in reality a multiphase medium consisting of solid, liquid, and gas, was simplified as a single-phase medium. Thus consolidation effects have been neglected. As shown in Reul (2002), the consolidation has only minor effects on the final settlements and load distribution between piles and raft. The material behavior of the piles and the raft was simulated as linear elastic in the FE analysis, whereas for the simulation of the material behavior of the soil the elasto-plastic cap model as implemented in ABAQUS was used (Figure 8). The model consists of two yield surfaces, the pressure dependent, perfectly plastic shear failure surface F_s (cone) and the compression cap yield surface F_c (cap). Stresses lying inside the yield surfaces cause only linear elastic

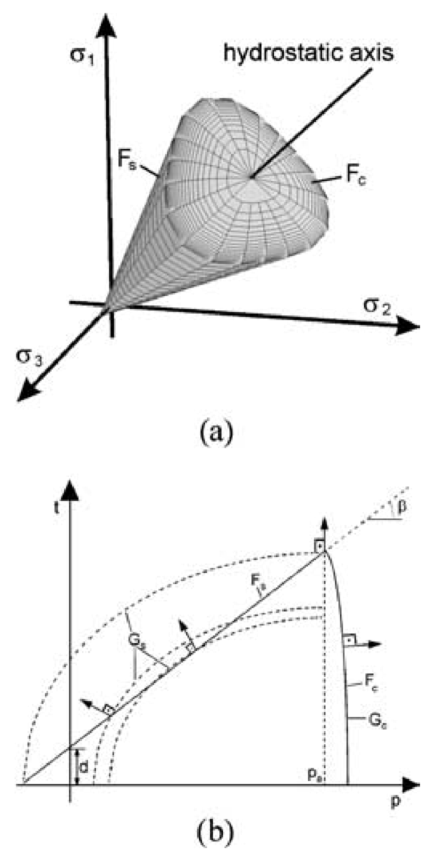

Fig. 8. Drucker Prager/Cap model: (a) yield surface in the principal stress space and (b) yield surface in the p-t plane.

Table 1
Material parameters

	Tertiary clay	*Limestone*
φ' (deg)	20	15
c' (kN/m^2)	20	1,000
E (MN/m^2)	E^*	2,000
ν	0.25	0.25
K_0	0.50	0.5
γ (kN/m^3)	19	20–24
γ' (kN/m^3)	9	10–14
α	0.0	0.001
β (deg)	37.67	29.53
d (KN/m^2)	42.42	2,114
K	0.795	0.841
R	0.1	0.001

$^*E = 58 + [\tanh(\frac{z-30}{15}) + 1] \cdot 0,7 \cdot z$, where φ' is the effective friction angle; c', the cohesion, E, Young's modulus; ν, the poisson ratio; K_0, the coefficient of earth pressure at rest; γ, the total unit weight; γ', the buoyant unit weight; α, the shape factor for the transition surface; β, the slope of yield surface F_s in the p-t plane; d, the intersection of yield surface F_s with the t-axis; K, the shape parameter of yield surface F_s; R, the shape parameter of yield surface F_c; z is the depth within tertiary clay layer.

deformations. The Young's modulus (E) increases with depth, the Poisson's ratio (ν) was assumed to be constant for the simulations (compare Table 1). Stresses on the yield surfaces do lead to plastic deformations. The shear failure surface is perfectly plastic, whereas volumetric plastic strains can lead to a hardening or softening by changing the cap position.

The hardening/softening behavior of the cap yield surface is a function of the volumetric plastic strain, the hardening function is derived from hydrostatic triaxial tests. This yield surface may change in size, position, or shape as the soil is loaded successively to higher stress levels. On the Drucker–Prager shear failure surface F_s the material dilates while on the cap surface it compacts. The plastic flow on the Drucker–Prager shear failure surface F_s produces plastic volume increase, which causes the cap to soften. The constitutive model gives the possibility for a reasonably good simulation of the stress–strain behavior of soils and depends on the stress path and the previous stress history. The Drucker–Prager failure surface can be written as

$$F_s = t - d - p \tan\beta = 0 \tag{3}$$

The cap surface with its elliptical shape is written as

$$F_c = \sqrt{(p - p_a)^2 + \left(\frac{Rt}{1 + \alpha - \frac{\alpha}{\cos\beta}}\right)^2} - R(d + p_a \tan\beta) = 0 \tag{4}$$

The plastic flow is defined by a flow potential, which is associated on the cap area and nonassociated on the failure yield surface. It consists of an elliptical part in the cap region defined by

$$G_c = \sqrt{(p - p_a)^2 + \left(\frac{Rt}{1 + \alpha - \frac{\alpha}{\cos\beta}}\right)^2} \tag{5}$$

and a second elliptical part in the failure region given by

$$G_s = \sqrt{[(p_a - p)\tan\beta]^2 + \left[\frac{t}{1 + \alpha - \alpha/\cos\beta}\right]^2} \tag{6}$$

with

$$t = \frac{1}{2}q\left(1 + \frac{1}{K} - \left(1 - \frac{1}{K}\right)\cos(3\theta)\right) \tag{7}$$

where d is the intersection of the yield surface F_s with the t-axis (derived from cohesion c'); p, the hydrostatic stress; q, the Mises equivalent stress; K, the shape parameter of yield surface F_s; R, the shape parameter of yield surface F_c; p_a, the initial cap position; p_b, the compression yield stress; α, the shape factor for a transition surface (not applied here); β, the slope of yield surface F_s in the $p-t$ plane (derived from internal angle of friction φ'); θ is the lode angle.

The constitutive model used at Technische Universität Darmstadt was validated by numerical simulations of static pile load tests as well as by back analyzing existing settlement data of foundations. A complete set of material parameters is given in Section 9.

7 INTERACTION BETWEEN GEOTECHNICAL AND STRUCTURAL ENGINEER

An important part of the design work of the geotechnical engineer is reviewing and assessing the effects of results on the structural design (Katzenbach et al., 1999b). The amount of results of FE analysis is huge and only few data is actually in the interest of the structural engineer. Considering the fact that large three-dimensional elasto-plastic FE simulations are very time consuming and that most structural engineers do not use continuum elements but shell elements for structural design purposes, a system must be found to meet the available analyzing tools of the structural engineer. As shown in Figure 9, a two-dimensional model was used to design the raft (thickness and reinforcement). The three-dimensional system was reduced to a two-dimensional FE model consisting of shell and spring elements representing raft and piles.

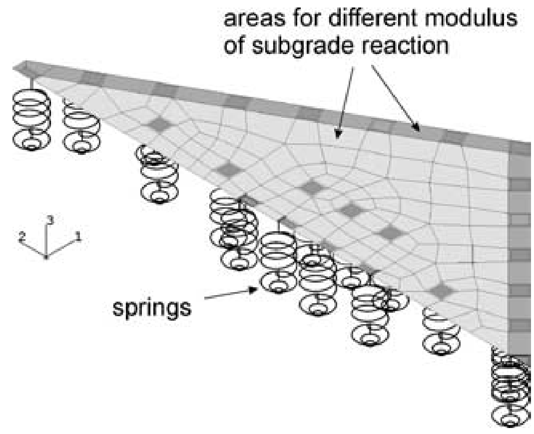

Fig. 9. Simplified model for structural design purposes.

The spring stiffness c_{vi} for the two-dimensional model was derived from the three-dimensional model (Figure 6) by

$$c_{vi} = \frac{Q_i}{s_i} \tag{8}$$

where Q_i represents the pile load and s_i the settlement of the pile (i). The spring stiffness increases from 135 up to 210 MN/m depending on the position of the pile. The modulus of subgrade reaction for the raft area in the two-dimensional model was obtained by back-analyzing the settlements calculated by the three-dimensional model.

8 THE OBSERVATIONAL METHOD—MONITORING THE FOUNDATION

As a matter of the rather extraordinary geometrical conditions and the special location of the foundation adjacent to an existing tunnel, the City-Tower required a comprehensive measuring program according to regulations of Eurocode (EC) 7.

With the results of the geotechnical measuring program, as an indispensable part of the safety concept, it is possible to perform a validation of the numerical model that had been used to predict the settlement behavior of the foundation. The bearing behavior of the piles is observed by six piles equipped with different measuring devices (Figure 10).

The general assembly consists of load cells at the pile bottom and on the pile top as well as eight strain gages in four different depths along the pile length. The settlements adjacent to the new building are monitored with two multipoint bore-hole extensometers up to a depth of about 70 m. The vertical displacement of the adjacent tunnel is monitored by geodetic leveling, whereas the horizontal displacement is observed by an inclinometer installed behind the new bored pile wall (Figure 10).

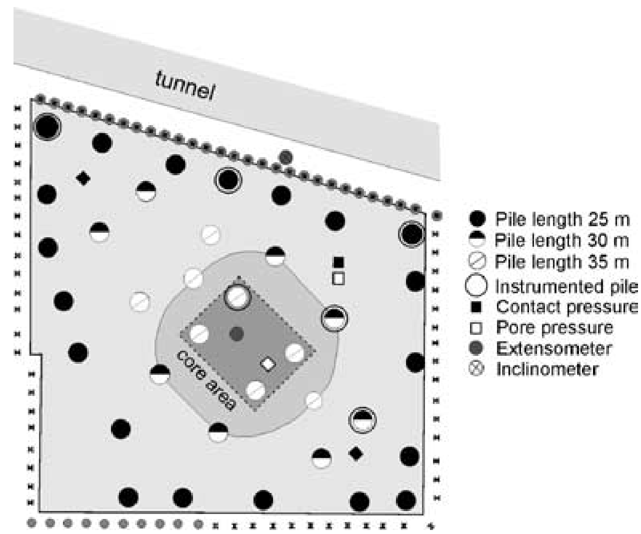

Fig. 10. Ground plan of the City-Tower including geotechnical measurement devices.

9 IMPROVEMENT OF CONSTITUTIVE MODEL BY BACK-ANALYZING THE EUROTHEUM

The planning and design process for a new 228 m high office tower called Max has just started. Some details on this project will be given in Section 10. To improve and verify the input parameters for the constitutive modeling of the new office tower Max, a numerical back-analysis of the 110 m high Eurotheum, which is located close to the building site of Max, has been carried out. The Eurotheum consists of a tower area (Figure 14), height 110 m, ground area 28 × 28 m, and an adjacent area with six floors. For the Eurotheum, comprehensive measurements have been carried out starting in the construction stage and lasting up to the present day. The Eurotheum with its 30 floors, shown in Figure 11, was constructed between 1997 and 1999.

The foundation is a CPRF with 25 piles, diameter of 1.5 m and pile length between 25 and 30 m depending on the position of the pile. Down to 9 m under the surface the subsoil consists of quaternary sand and gravel. This stratum is followed by a tertiary clay layer with a thickness of about 49 m and a stratum called Frankfurt limestone down to great depth. The total vertical load of the Eurotheum is about 550 MN.

9.1 Measurements

The observational method for large civil engineering projects like high-rise buildings is in the actual state of

Fig. 11. EUROTHEUM.

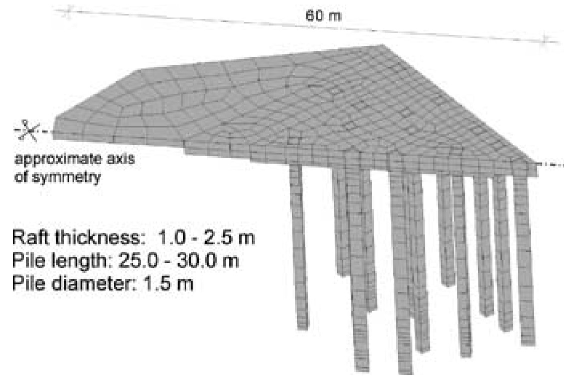

Fig. 12. FE mesh with raft and piles of the EUROTHEUM.

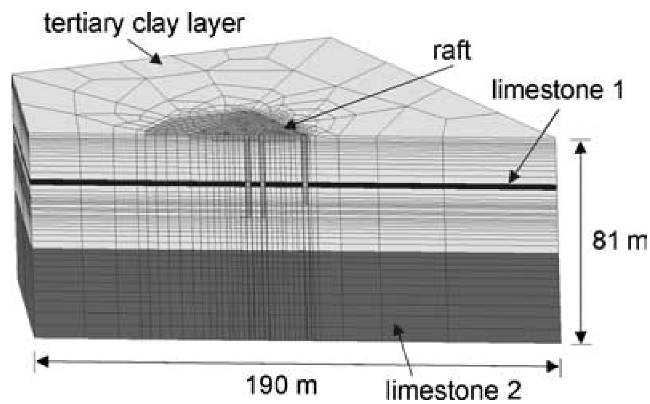

Fig. 13. FE mesh of the EUROTHEUM foundation.

knowledge an important part of the safety concept. In terms of the EC 7, CPRFs are classified as structures of the highest geotechnical category (category 3) and the foundation behavior has to be monitored. Therefore, geotechnical measurement devices were installed and geodetic surveying has been carried out for all high-rise structures constructed in Frankfurt in the last decade. In Figure 14, the ground plan of the EUROTHEUM is shown with the location of all geotechnical measurement devices. All sensors are linked to an automatically operating monitoring network, which allows an online supervision of the measurement data. The objective of the measurements is to monitor the load-bearing behavior of the CPRF, for example, the load share between raft and pile and the pile–pile interaction. For the EUROTHEUM, four piles were equipped with load cells at the pile head to observe the bearing behavior of the piles. The contact pressure of the raft is measured in seven locations, the pore pressure is measured in six locations. The settlement of the building is observed by geodetic measurements.

9.2 Back-analysis

The back-analysis of the EUROTHEUM has been carried out with a three-dimensional FE model, which is partly shown for the raft and piles in Figure 12. The entire mesh is given in Figure 13. Due to the approximate symmetry of the geometry and loading of the tower it was possible to reduce the geometry of the FE mesh to one-half of the real geometry. The mesh consists of about 22,000 elements, mainly eight-node brick elements. For modeling of the subsoil the cap model, which was described before, has been used. The material parameters used for the tertiary clay layer and the limestone layers are given in Table 1. The shear parameters (φ' and c') of the clay were obtained as mean values from triaxial tests. Results of several pressiometer tests carried out in Frankfurt clay show that the soil stiffness increases with depth. The distribution of the Young's modulus with depth was derived from several back-analyses.

With a step-by-step analysis, the construction process including the excavation for the basement, the pile and raft installation, and the gradual loading have been simulated. Some results obtained from the FE analysis and the corresponding measurements are displayed in Figure 15. The diagram shows the settlements of the raft along the axis of symmetry obtained from measurements and from the FE simulation. Origin and direction of x are shown in Figure 14.

The maximum settlement for the EUROTHEUM observed so far is about 3 cm. Due to consolidation the settlement is still increasing. The final settlements calculated in the FE analysis reach a maximum value of

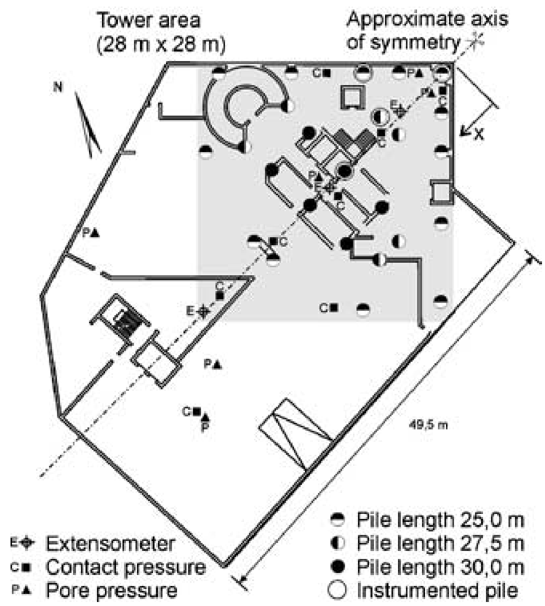

Fig. 14. Ground plan of the EUROTHEUM including geotechnical measurement device.

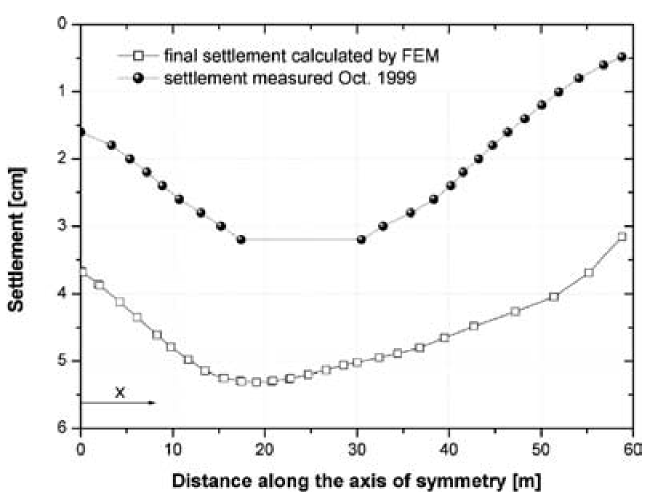

Fig. 15. Settlement of the EUROTHEUM.

about 5.5 cm. Further modifications of soil modeling parameters will be considered to optimize the settlement behavior of the FE model.

10 ASPECTS FOR THE DESIGN OF THE NEW HIGH-RISE BUILDING MAX

The new high-rise building MAX (Figure 16), in the heart of Frankfurt, is now in its planning and design stage. The tower will have a gross storey area of 95,000 m^2 on 64 floors and a height of 228 m. It will cost about

Fig. 16. MAX site.

€600 million and is supposed to be completed in 2006. The office tower will be located in the financial district of Frankfurt in the direct vicinity of already existing high-rise buildings (shaded area in Figure 16) and will be constructed on a CPRF. The design will be optimized by FE analysis comparable to those carried out for the CITY-TOWER and the EUROTHEUM. The EUROTHEUM is located on the opposite side of the street (Figure 16) with similar ground conditions. This makes the experience gained by back-analyzing the existing structure of the EUROTHEUM a valuable tool to optimize the prediction analysis of settlement behavior for MAX. At the same time it allows for a very economic foundation design by saving resources, time, and manpower.

11 CONCLUDING REMARKS

The use of numerical simulations for assessing the settlement behavior of high-rise structures became a powerful tool for the design process. Considering the fact that in most urban areas settlement sensitive traffic and supply networks are also located below the subsurface in the vicinity of new foundation structures, a more detailed mesh generation in these areas will be necessary. This consequently increases the number of elements for a simulation. The settlement predictions presented within this contribution neglect the effect of consolidation. Further research is necessary to improve knowledge about these effects and to consider them in future simulations. The simulations presented before, with an average number of elements between 10,000 and 25,000, require generally about 18 hr of computational time on a Sun-Ultra 2 workstation. Both increasing the number of elements and especially the consideration of consolidation will lead to an enormous increase of computational time. Thus efforts considering these aspects are strongly related to future developments in computer technology.

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