14-04

Deep Foundations
Combined Pile-Raft Foundations of Frankfurt High-Rise Buildings

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1 Introduction

When considering high-rise structures in Germany, the place with the most impressive skyline is undoubtedly Frankfurt am Main (Fig. 1). After 1950 in Frankfurt and its metropolitan region a massive structural change took place. The service sector became more and more important and Frankfurt has grown not only in size, with new housing areas in the suburbs, but also in height.

Figure 1 Skyline of Frankfurt am Main

Meanwhile, for most new high rise structures, existing buildings will be replaced by new structures with higher loads to be transferred into the subsoil. For all areas of civil engineering, this is a demanding and complicated task, especially for geotechnical engineers when considering the danger of high settlements and tilting of the structure in Frankfurt clay.

The subsoil in Frankfurt mainly consists of a non-homogeneous, stiff and overconsolidated Frankfurt clay with embedded limestone bands of varying thickness. This layer is underlain by the Frankfurt limestone which consists of limestone and dolomite layers as well as algal reefs, marly calcareous sands and silts and marly clay. The rather thin top layer consists of quaternary sand and gravel. As the boundaries of soil layers are dipping according to Fig.2, the thickness of the settlement active Frankfurt clay varies below the foundation structures.

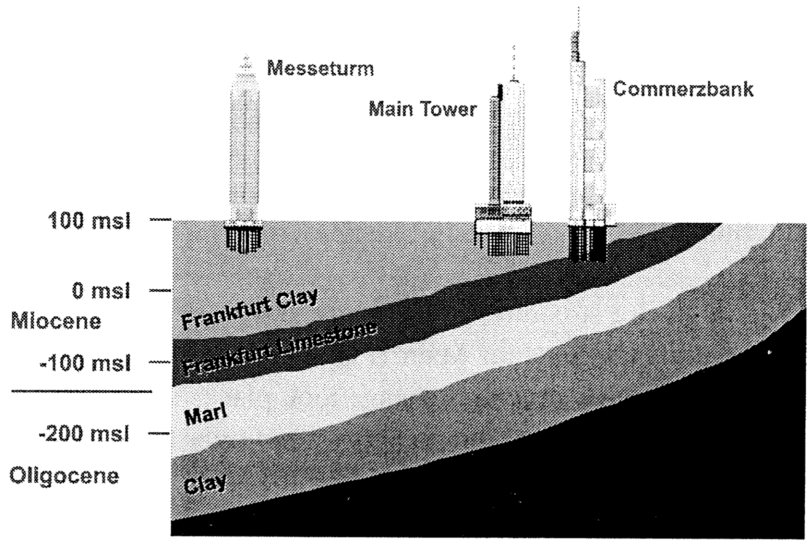


Figure 2 Subsoil of Frankfurt am Main

When planning foundations for high-rise buildings in urban areas, under these difficult conditions, a major task is the reduction of settlements and differential settlements of the structures as well as adjacent buildings. The aim is also to ensure their safety and serviceability under long live criteria and furthermore when considering the option of reuse of foundations. To realise these aims, the Combined Pile-Raft Foundation (CPRF) was developed. The CPRF is a meanwhile worldwide accepted approach that during the last two decades successfully has been used for foundations in the Frankfurt area, elsewhere in Germany and worldwide (Conte et al. 2003, Poulos 2001, Russo & Viggiani 1998, Poulos et al. 1997, Randolph & Clancy 1993, Randolph 1983, Cooke 1986). By using large 3D finite element simulations with a powerful pre- and post-processing the simulation and optimization of the often geometrical complicated foundation problem has become possible in an acceptable time frame.

To make clear the outstanding progress in using CPRFs, two examples of shallow founded high-rise buildings of the first generation are presented. These examples also illustrate that the settlement behaviour of Frankfurt clay can lead to a loss of serviceability of the entire structure if the risk is underestimated.

2 First generation of high-rise buildings in Frankfurt

For the first high-rise buildings which were founded on shallow foundations (2 - 4 m thick rafts) a settlement between 20 cm and 34 cm was observed (Katzenbach et al., 2001). Due to the problems of deflection and tilting considerable efforts had to be spent to correct the settlement behaviour of these buildings during the construction stage and later on.

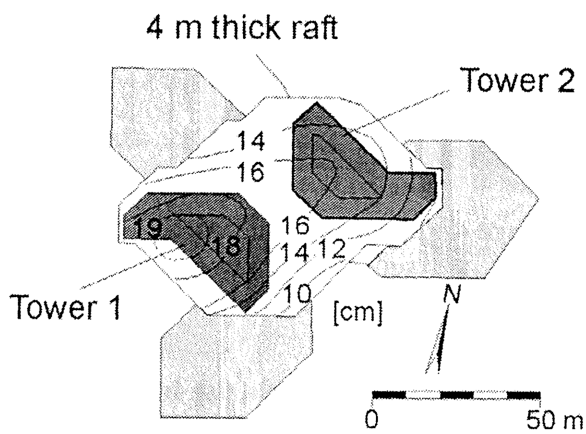


Figure 3 Deutsche Bank towers & settlement

One example, as given in Fig. 3, are the 158 m high towers of the Deutsche Bank with observed settlements of 10 cm to 22 cm until 1985 and resulting differential settlements of 12 cm. The towers are founded on a 80 m x 60 m shallow foundation with a thickness of 4 m. During construction it was attempted to keep the building from drifting out of plumb and it was finally accepted - although not recognizable - to have two not exactly vertical towers.

The method of dealing with differences in settlements between the towers and the adjacent lower building parts, included hydraulic pumping devices in all load carrying columns of the lower building parts, standing close to the towers. As a result the lower building parts could be regulated in their altitude by ± 8 cm in comparison to the two main towers. The differential settlements and their effects on the serviceability of the structure in the towers were overcome by assembly regulations for the facade and the elevators. The method of pre-installing hydraulic devices was also used when constructing the shallow founded Dresdner Bank. As depicted in Fig. 4 pressure cushions with a size of 5 m x 5 m were located under one corner. They initially were filled with water. After completing and adjusting the structure in the vertical position, the water was replaced by mortar. These complicated correction measures which often caused considerable problems, are insufficient and became unnecessary when using pile foundations and CPRFs some years later.

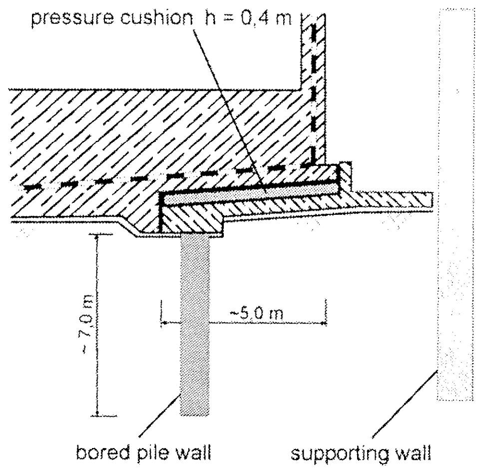


Figure 4 Dresdner Bank and hydraulic devices to adjust settlement behaviour

3 New generation of high-rise buildings in Frankfurt

The highest building in Frankfurt that is built on a CPRF, is the 256 m high Messeturm, constructed between 1988 and 1990 (Fig. 5). The initial settlements being calculated for a shallow foundation were about 40-50 cm with a differential settlement of about 15 cm. The settlement observed for the CPRF until 2000 was about 13 cm (Reul, 2000).

Designing CPRFs requires the qualified understanding of soil - structure interaction as presented in Fig. 6. According to its stiffness the CPRF transfers the total vertical load of the structure R_{tot} into the subsoil by contact pressure of the raft R_{raft} as well as by the piles $\Sigma R_{pile,i}$.

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

In comparison with a conventional foundation design of a pile group for CPRFs a new design philosophy with different and more complicated soil-structure interaction is applied. Piles are now used up to a load level which is much higher than permissible design values for bearing capacities of comparable single piles because the performance of the entire foundation structure has to be evaluated.

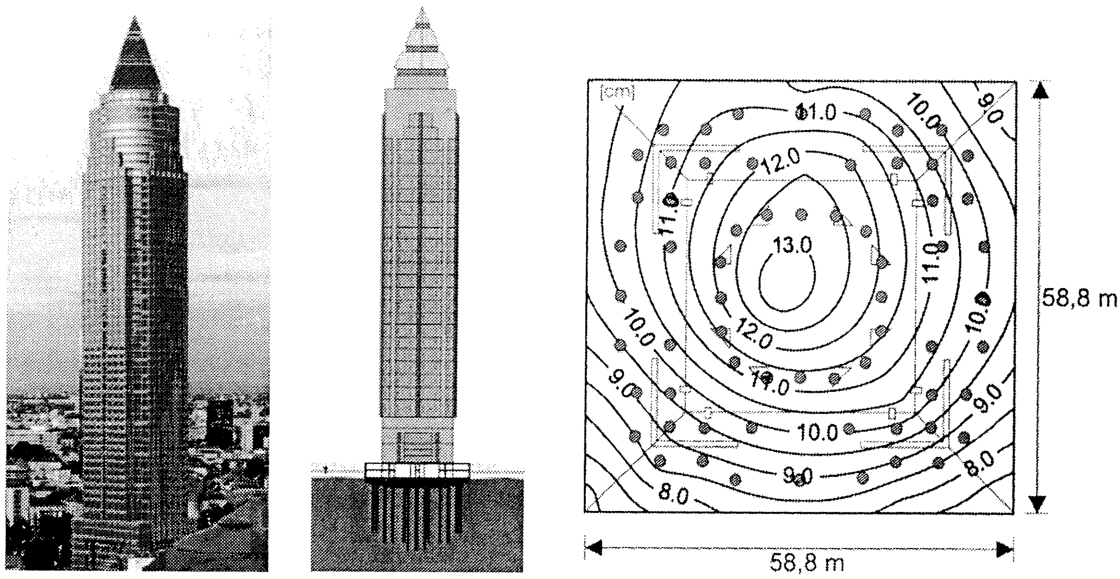


Figure 5 Messeturm in Frankfurt am Main & settlement

The distribution of the total building load between the different bearing structures of a CPRF is described by the CPRF coefficient α_{CPRF} (Eq. 2) which defines the ratio between the amount of load carried by the piles $\Sigma R_{pile,i}$ and the total load of the building R_{tot}.

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

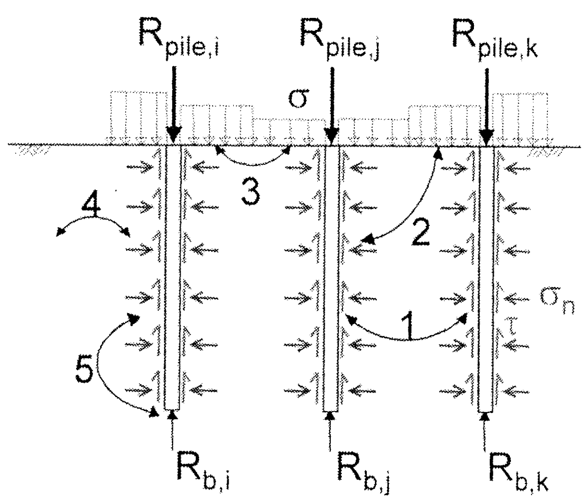


Figure 6 Soil-Structure interaction for a CPRF; 1 pile - pile interaction; 2 pile - raft interaction ; 3 raft - raft interaction; 4 pile - soil interaction; 5 pile base - pile shaft interaction

For a large number of high-rise buildings which have been instrumented by the Institute and Laboratory of Geotechnics of Technische Universität Darmstadt, the observed load share between raft and piles is depicted in Fig. 7 (Katzenbach et al., 2000). A CPRF coefficient of zero describes a raft foundation without piles, a coefficient of one represents a free standing pile group, neglecting the existence of a raft.

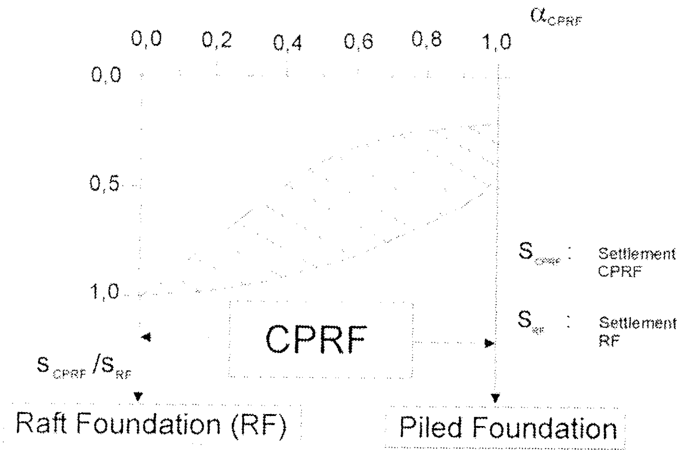


Figure 7 CPRF-coefficient

In order to investigate the bearing behaviour of a CPRF the aforementioned interactions as depicted in Fig. 6 have to be considered in a design process. Starting in the early 1980s first CPRFs were used for high-rise office buildings in Frankfurt am Main (Fig. 8), mainly to reduce settlements to practicable dimensions and, furthermore, to ensure serviceability by reducing differential settlements to a minimum in a more economical way than relying on raft foundations as illustrated for the first generation of high rise buildings. Compared to traditional pile foundations, CPRFs allow a saving of construction time and a considerable cost reduction.

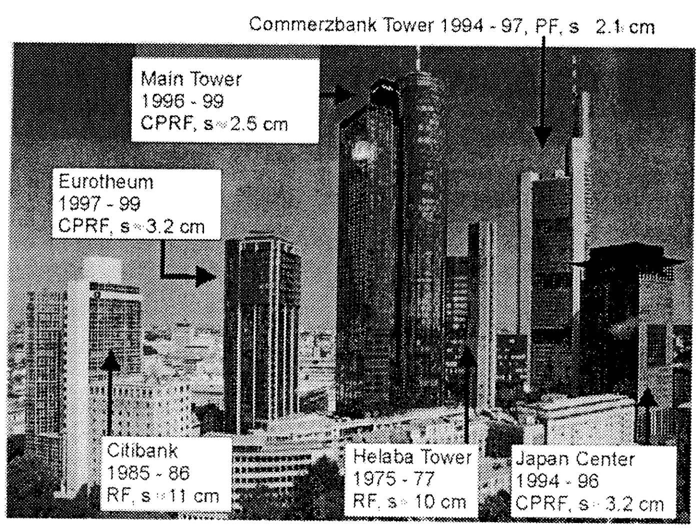


Figure 8 CPRF-coefficient

However, it became clear that the design and forecast of settlements requires, in general, the application of a powerful numerical tool. In the following example, the finite element method has been used to predict the settlement behaviour and performance of a high-rise building.

4 Finite Element simulation

In order to assess the load-settlement behaviour of the foundation and to predict the pile loads for the CPRFs 3D- finite element simulations are carried out. These simulations allow the consideration of complicated geometric shapes and provide a valuable tool to perform simulations with different pile configurations in order to optimize the foundation structure.

A constitutive model used for simulations should provide a reasonable good simulation of the stress-strain behaviour of soils, which depends on the stress path and the previous stress history. The material behaviour of the piles and the raft are simulated as linear-elastic in the finite element analysis. The soil is modeled with an elasto-plastic constitutive model consisting of two yield surfaces, the pressure dependent, perfectly plastic shear failure surface F_s (cone) and the compression cap yield surface F_c (cap) (Fig. 9). Stresses lying inside the yield surfaces do only cause linear elastic deformations. The Young´s modulus (E) is increasing with depth, the Poisson´s ratio (v) is assumed to be constant for the simulations. 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 behaviour of the cap yield surface is a function of the volumetric plastic strain which defines the actual cap position. The hardening/softening behaviour is based on back-analysis of pile tests and laboratory tests first presented in Katzenbach et al. (1994). Material parameters used in the following finite element analysis are summarised in Table 1.

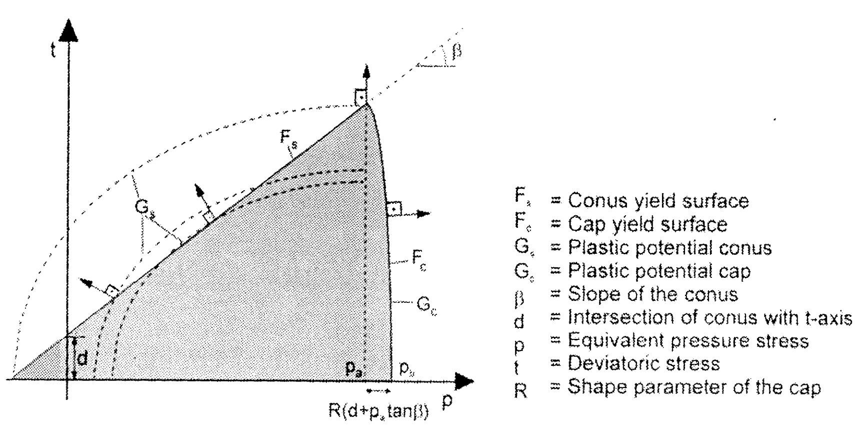


Figure 9 Drucker Prager / Cap Model, yield surface in the p-t-plane

The cap yield surface (F_c) 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. Plastic flow on the Drucker-Prager shear failure surface F_s produces plastic volume increase, which causes the cap to soften. The parameters β and d (Fig. 9) are derived from the angle of friction φ' and cohesion c′ of the soil.

Parameter	Unit	Soil	Raft & Piles
Young's modulus **E**	Mpa	50-270*	30000
Poisson's ratio υ	-	0.25	0.2
Coeff. Of earth pressure $\mathbf{K_0}$	-	0.5	-
Buoyant unit weight γ'	kN/m^3	9	13
Angle of friction φ'	°	20	-
Cohesion **c′**	kPa	20	-
Slope in p-t plane $\boldsymbol{\beta}$	°	37.67	-
Intersection **d**	kPa	42.42	-
Shape factor **R**	-	0.1	-

(*linear increase with depth)

Table 1 Material parameters used in finite element simulations

The transition from pile to soil is modelled as ideal contact, assuming that shear failure takes place in a narrow zone adjacent to the pile, which has the same material parameters as the surrounding soil. The constitutive model used at the Institute and Laboratory of Geotechnics of Technische Universität Darmstadt, was widely verified by numerical simulations of static pile load tests as well as by back analysing existing settlement data (Katzenbach et al., 1994, Reul, 2000). One example is the Eurotheum (Fig. 8) where the constitutive modelling and simulation parameters are given in Katzenbach et al. (2003).

5 City-Tower

The principal design procedure for a high-rise building foundation is described exemplarily for the office building CITY-TOWER (Fig. 10) which has recently been completed. The tower in the outskirts of Frankfurt is about 121 m high and founded in settlement active Rupel clay (Fig. 2) on a CPRF with large diameter bored piles. In 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 in full design life.

Based on the load distribution obtained from the structural engineer and the symmetry of the geometry the finite element mesh could be reduced to a half of the area to be considered with a total number of 10 365 elements (Fig. 11 & 12). Several simulations based on material parameters given in Tab. 1 have been performed to optimise the foundation design, to estimate the settlements and to assess the appropriate pile length, diameter and location of each pile under the raft.

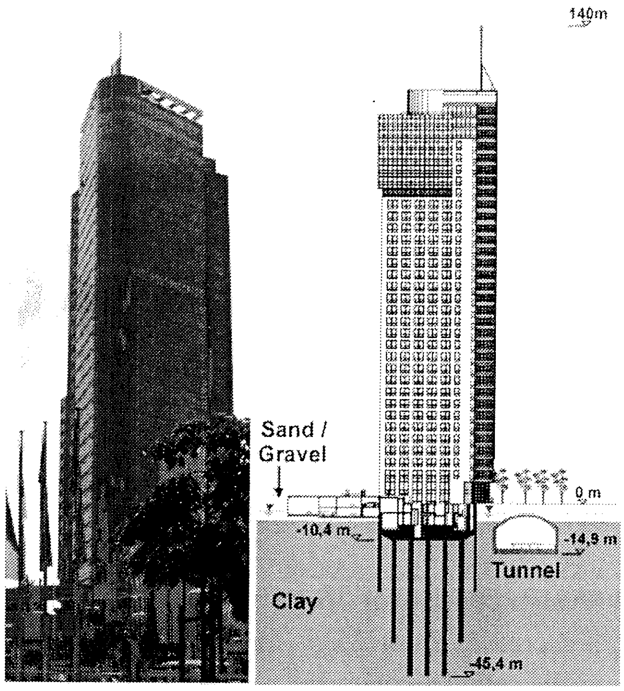


Figure 10 CITY-TOWER with foundation layout

These simulations also consider the overconsolidation of clay and the entire construction process including also the demolition of buildings that were existing on the building site, before the construction process of the CITY-TOWER was started. Due to this preloading of the subsoil, the results of the simulation are depending more on the stiffness of the soil rather than on the soil strength.

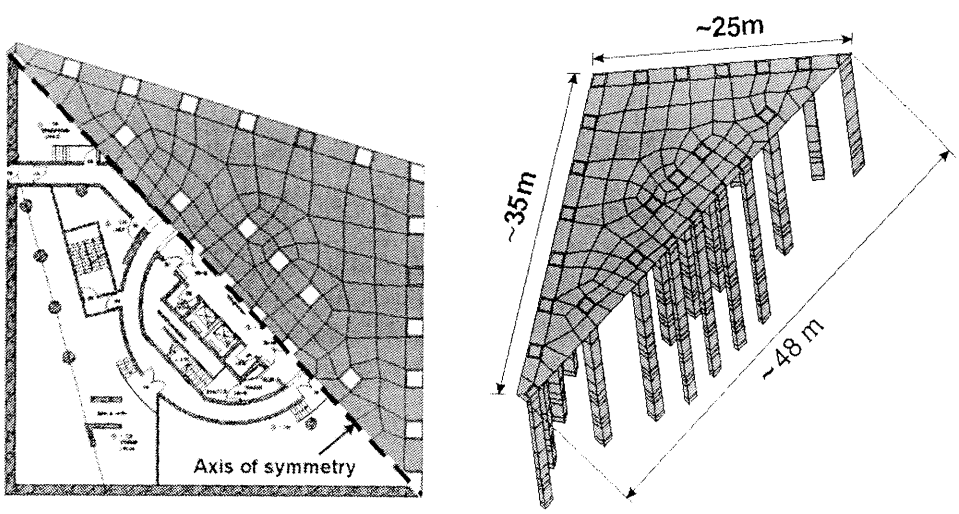


Figure 11 FE-Mesh of half of the raft area and 3D view of the CPRF with raft and piles

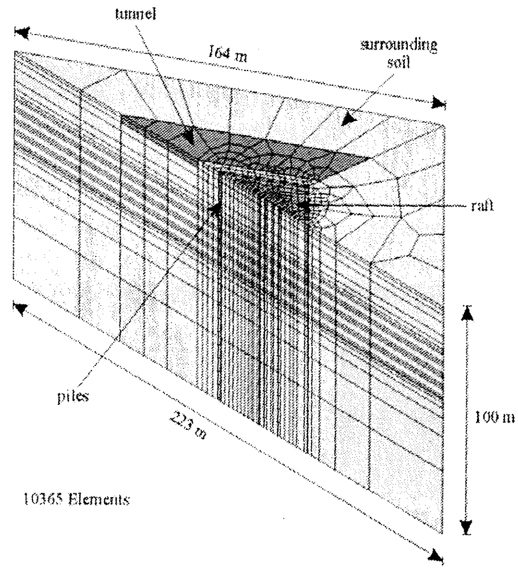


Figure 12 Entire mesh of the foundation system with surrounding soil

The final foundation design consists of 36 piles with a pile length between 25 m and 35 m. The pile length increases from 25 m for the outer piles to 35 m for the piles located in the centre 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 centre of the pile raft foundation after consolidation processes have ended. The differential settlement was calculated as about 1 cm between the center of the CPRF and its outer borders. Settlements after completion of the building are currently in the range of 2 cm and are still increasing due to consolidation effects.

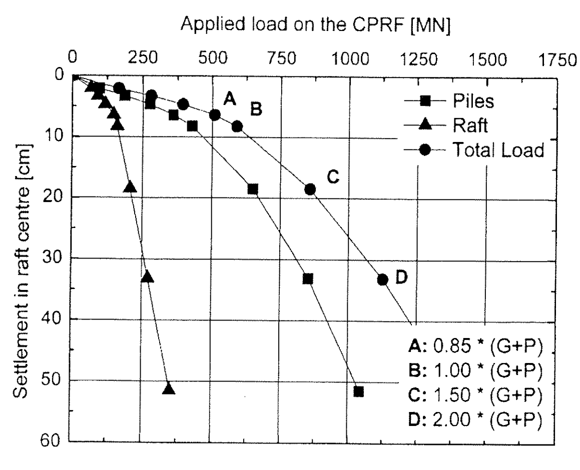


Figure 13 Load-settlement curves obtained from FE simulations

The measured pile loads are up to 8.5 MN and the contact pressures under the raft are between 60 and 70 kN/m^2. The water pressure below the raft is between 50 and 60 kN/m^2. The horizontal displacement of the adjacent tunnel was predicted as 0.5 cm - 1.4 cm. The largest horizontal displacement in vicinity of the tunnel was recorded by an inclinometer (Fig. 14) with about 1.5 cm. The displacement of the tunnel structure itself was below the predicted values. In Fig. 13 the load-settlement curves derived from one of the finite element simulations for the CPRF are given separately for the entire foundation structure, the piles and the raft. The letters A-D describe different loading levels of the foundation.

6 Observational method

As a matter of the rather extraordinary geometrical conditions and the special situation of the foundation adjacent to an existing tunnel, the CITY-TOWER required a comprehensive measuring program according to regulations of Eurocode 7.

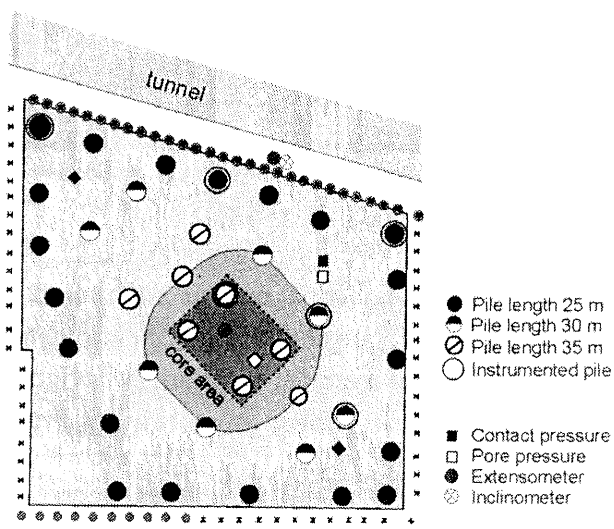


Figure 14 Measuring devices for the CITY-TOWER

The results of the geotechnical measuring program (Peck 1969, Katzenbach & Moormann 2003, Schmitt et al. 2002) - as an indispensable part of the safety concept - also allow a calibration of the numerical model that had been used to predict the load-settlement behaviour. The bearing behaviour of the piles is observed by 6 piles equipped with different measuring devices (Dunnicliff 1988). The general assembly consists of load cells at the pile bottom and on the pile top (Fig. 15 & 16) as well as 8 strain gages in four different depths along the pile length. The settlements adjacent to the new building are monitored with two multi point bore-hole extensometers up to a depth of about 70 m.

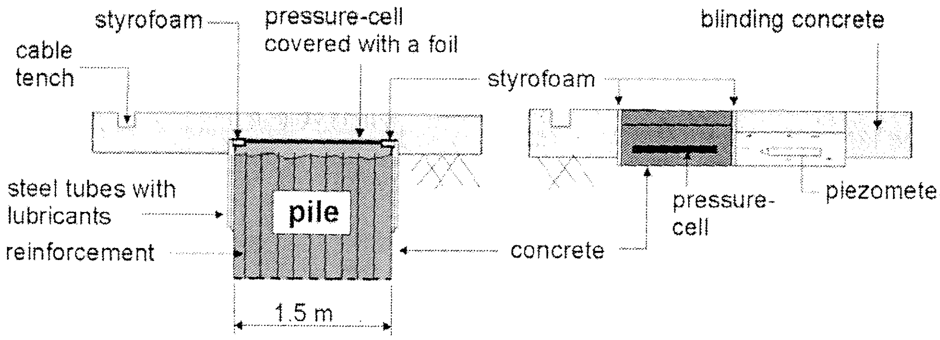


Figure 15 Measuring devices – load cells & piezometer

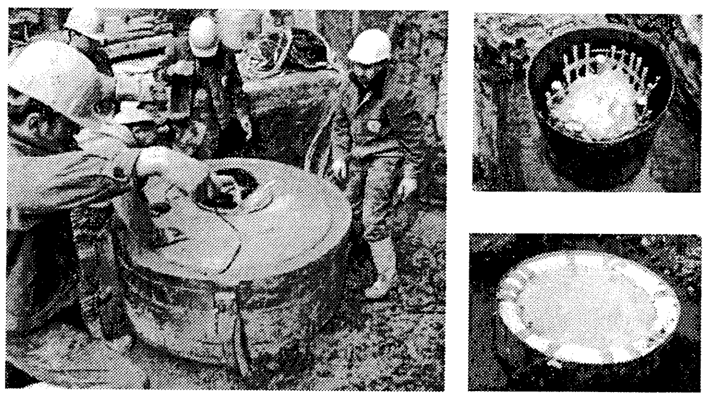


Figure 16 Installation of a load cell on a pile head

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. The number of installed measuring devices compares quite well with the measuring devices installed for similar projects in the Frankfurt area (Fig. 17).

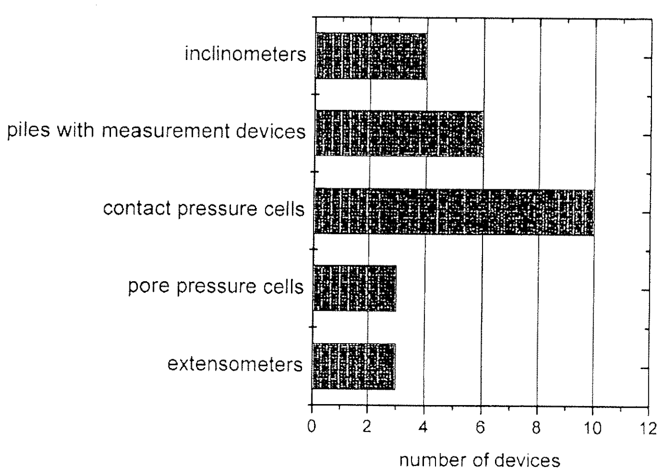


Figure 17 Measurement devices installed for CPRFs, average values from 14 high-rise projects in Frankfurt

7 The German guideline for Combined Pile-Raft Foundations

Based on a large variety of parametric studies with numerical simulations and the extensive experience on CPRFs gained by long term monitoring of the foundation behaviour, the German guideline for Combined Pile-Raft Foundations was developed by Prof. Katzenbach (TU Darmstadt, Geotechnics) and Prof. König (Uni Leipzig, Structural Engineering) under the leadership and the finacial support of "Deutsches Institut für Bautechnik (DIBt)", Berlin (Hanisch et al. 2000). The new CPRF-guideline (German name: KPP-Richtlinie) gives guidance to several aspects regarding the design, the safety concept, the limits of application, the use of the observational method and the construction of CPRFs. It also gives a guidance for the practicing engineer on an adequate soil investigation program, including also the matter of drilling and the question regarding in which cases static axial pile tests are required (Hanisch et al. 2000, Katzenbach & Moormann 2001). Furthermore, the guideline clarifies aspects on what is required and expected from an appropriate calculation method and which requirements a calculation method should fulfil when it is used to design a CPRF.

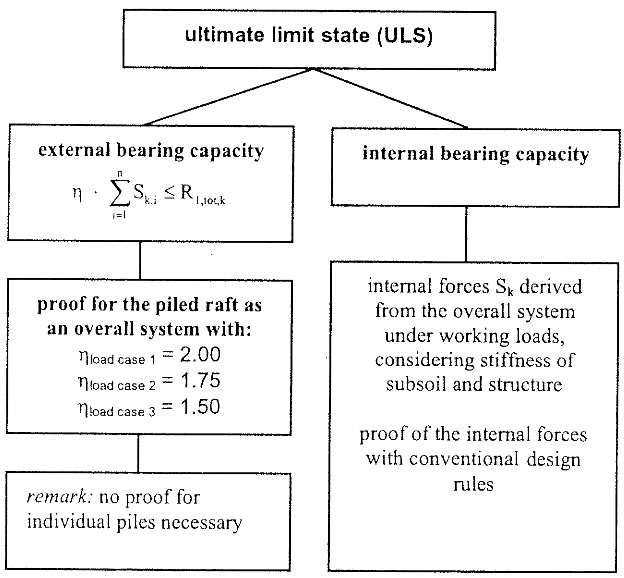


Figure 18 CPRF-guideline: Ultimate limit state (ULS) approach

The guideline distinguishes between the external and internal bearing capacity and follows the limit state design philosophy. Within the limit state design method the performance of the whole structure as well as a part of it is described with reference to a set of limit states beyond which the structure fails to satisfy fundamental requirements. In the Eurocode a distinction is made between ultimate limit state (ULS) and serviceability limit state (SLS). Ultimate limit states are situations involving different kinds of collapse, failure and excessive deformations prior to failure, and situations where there is a risk of danger to people and/or severe economic loss.

The ULS (Fig. 18) is separated into two parts. Proofing the external bearing capacity ensures that the overall system consisting of soil and foundation elements like raft and piles are supporting the working load of the building. The applied global safety factor is $\eta=2$ (case D in Fig. 12) for load case 1 (applicable for dead loads and regular working loads).

In the formula depicted in Figure 18, $S_{k,i}$ is the characteristic value of action i and $R_{1,tot,k}$ gives the characteristic value of the total resistance of a CPRF which can be derived from the calculated load-settlement curve of the whole system. The internal bearing capacity is defined by the bearing capacity of the different parts of the reinforced concrete structure itself. Attention is drawn to the fact that, compared to classical pile foundations, no proof for the external bearing capacity of each individual pile is necessary which in turn, leads to the enormous economic advantages of CPRFs.

The serviceability limit state (SLS) corresponds to conditions beyond which specified requirements for the structure and its use are no longer met. This applies to deformations, settlements and vibrations in normal use under working loads, such that the serviceability of the structure is not guaranteed.

The SLS condition (Figure 19), to be satisfied, is that the design value of the action effect E is less than the limiting value of the deformation of the structure at the serviceability limit state, where C is the resistance property for SLS. Corresponding to ULS the internal serviceability is related to the construction materials used for different foundation parts.

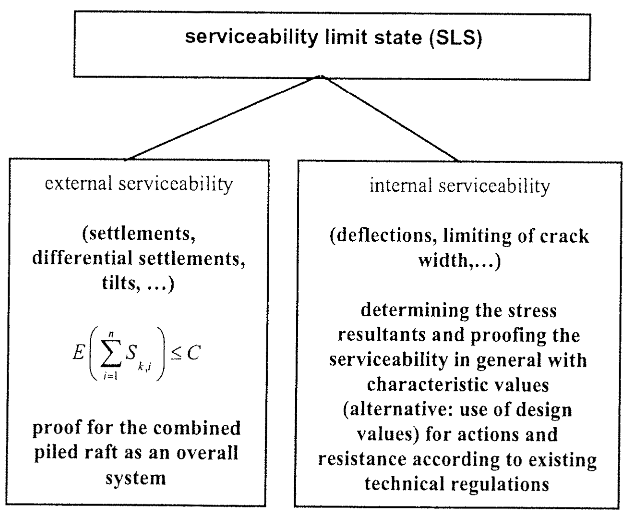
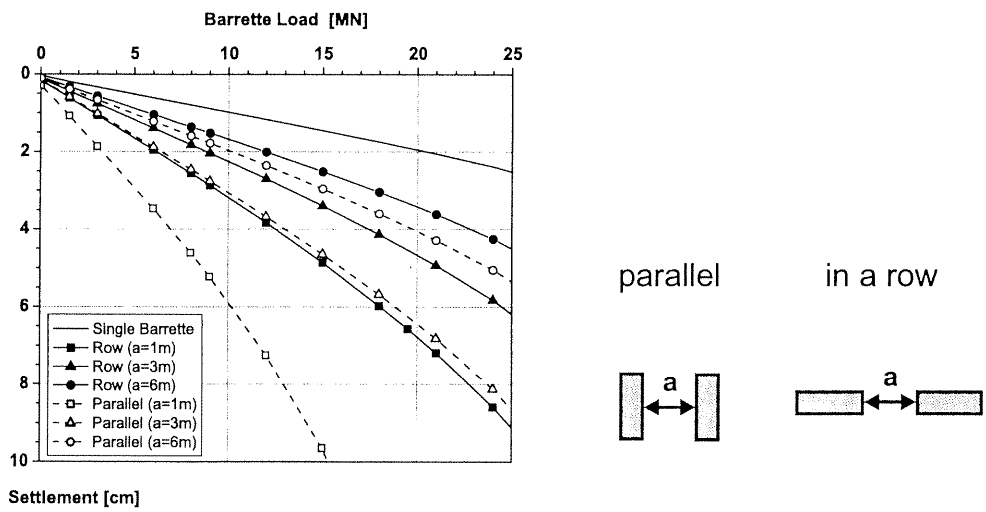


Figure 19 CPRF-guideline: Serviceability limit state (SLS) approach

8 Use of barrettes for extraordinary loading

In the case of very high building loads the design concept of CPRF results in a large number of high loaded piles with a length of more than 50 m. In this context the idea of replacing the piles by barrettes and creating a Combined Barrete-Raft Foundation (CBRF) can be a more economic design approach. In order to understand the barrette to barrette interaction a basic investigation using finite element modelling was carried out. The aim was to give recommendations on the impact of different barrette arrangements and to assess the load-settlement behaviour.

The barrettes analysed in the numerical investigation are 1 m in thickness, 3 m in length and 30 m in depths. The distance between two free standing barrettes which were investigated, varies from 1 m to 6 m. Two geometrical configurations were analysed, an arrangement in rows and a parallel arrangement of the barrettes (Figure 20). In both configurations the barrettes are installed in clay. The clay was modelled with a linear increasing Young's modulus with an average value of E = 35 MN/m².



a) b)

Figure 20 Load-settlement curves (a) and barrette arrangement (b)

The load-settlement curves reveal the influence of the barrette to barrette interaction on the deformation behaviour. The settlement increases with decreasing distance between the barrettes. For the same loading the settlements are higher if the barrettes are arranged in a row compared to a parallel arrangement.

To analyse the influence of the barrette to barrette distance, the changes in settlement based on the results of the calculation with 1 m distance are shown in Figure 21. The figure depicts the settlement reduction for two load steps of 8 MN and 21 MN. Especially for the parallel arrangement the increase of the horizontal distance between the barrettes leads to a significant settlement reduction. For the parallel arrangement also the value of the load has an important influence on the decrease of settlement (Fig. 21).

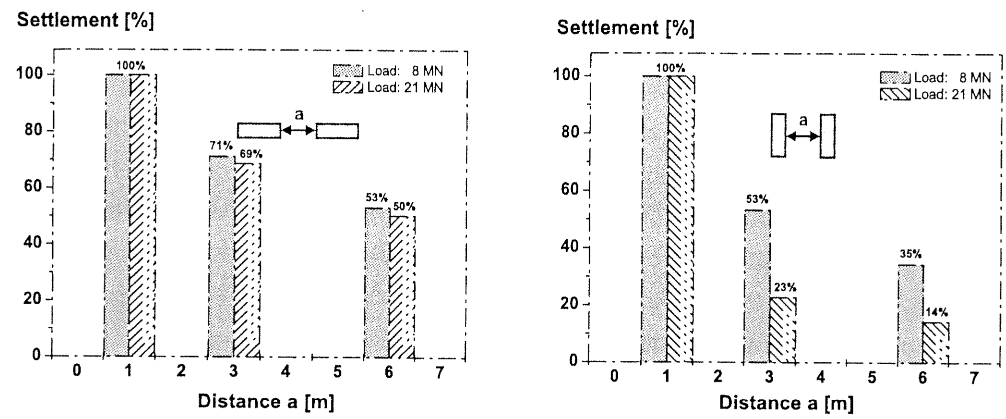


Figure 21 Dependency between barrette arrangement, loading and settlement

To reduce the mutual interaction between two barrettes it is recommended to arrange the barrettes with a distance of a > 3 m especially for a parallel arrangement.

To investigate the comparability of the CBRF and the CPRF two FE-Simulations with identical boundary conditions have been carried out. The models used for the foundations are shown in Fig. 22. The CBRF was modelled with 32 barrettes each with a cross section of 1.0 m x 3.4 m and a length of 45 m. The CPRF consists of 65 piles with a diameter of 1.5 m and the same length. The barrettes have been arranged according to the aforementioned recommendations. The surface of all piles provided to mobilize skin friction is almost equal to the surface of the barrettes.

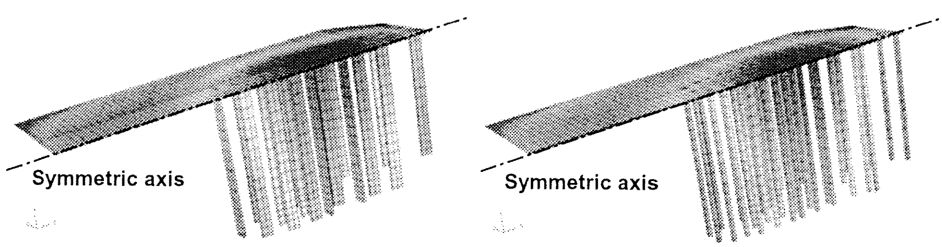


Figure 22 Finite element model of the CBRF (left) and the CPRF (right)

For a total load of 2,500 MN the settlements of both foundations are about 18 cm but for the CBRF only the half number of foundation elements is necessary.

9 Conclusions and outlook

The CPRFs of high-rise buildings completed during the last years in the area of Frankfurt, have shown that by choosing the foundation concept of a CPRF, a considerable settlement reduction of more than 50 % compared to a simple raft foundation can be achieved.

During the design process of a CPRF based on finite element analysis, as described before, a strong co-operation between the geotechnical and structural engineer is necessary to guarantee a safe and economic construction (Katzenbach et al., 1999). In this context, an important part of the design work of the geotechnical engineer is also reviewing and assessing the effects of results from the geotechnical analysis on the structural design.

The choice between a CPRF and a system that is using barrettes instead of piles, will mainly depend on the local market prices for the construction of a conventionally bored pile and for the construction of a barrette.

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