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Soil-Structure Interaction and ULS Design of Complex Deep Foundations

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ABSTRACT

In conventional design of deep foundations, some important positive effects evolving from the interaction of the bearing elements and the subsoil (Soil-Structure Interaction) are not utilised. These positive effects especially arise when using Combined Pile-Raft Foundations (CPRFs). The application of numerical methods during the design process of such foundations, which is explicitly allowed in Eurocode 7, is capable of regarding these effects. This paper deals with an approach using numerical methods within the ULS design for complex foundations and discusses case histories where CPRFs are used as a foundation for high-rise buildings in Frankfurt am Main. The paper will be finalised with an introduction to the Seasonal Thermal Storage where the piles of a deep foundation are used as energy piles to store or extract heat in the surrounding subsoil.

INTRODUCTION

Historically, the foundation technology started with timber piles which have been already used in prehistoric times, e.g. for the stone-age stilt houses at Lake Constance. The first high buildings have all been sacral buildings, e.g. the Pantheon in Rome or the Cathedral of Cologne which have been founded on rather solid ground. This enables the master builders to establish the buildings on shallow masonry foundations. Attempts to erect high buildings on softer ground led regularly to damages and tiltings, e.g. concerning the leaning tower of Pisa or the Holsten Gate in Lübeck. The first high profane buildings were erected in the last decades of the 19^{th} century in the United States of America after the invention of the elevator and the steel frame construction technique, like the Home Insurance Building in Chicago (1884) with 43 m or the Western Union Building (1872) in New York City with 71 m. This first generation of high-rise buildings could be founded shallowly tolerating a rather small quantum of settlements due to their restricted height. The next generation had turned from high-rise buildings to skyscrapers – especially in New York City – and therefore stronger foundations had to be constructed. The Woolworth Building (1913) was the first skyscraper for which a deep foundation was constructed in terms of a caisson-like foundation elements which directed the high loads of the structure onto the massive granite of the Manhattan peninsula.

In rather soft subsoils like in London or Frankfurt am Main, the high-rise buildings of the first generation have been founded on conventional rafts which all lead to settlements up to several decimeters and the endangering of the functionality of e.g. elevators; in some cases, special solutions had been developed, e.g. the compression cushions at the Dresdner Bank Tower in Frankfurt am Main which were designed for the correction of differential settlements (Katzenbach et al. 2006). A much better approach for safeguarding the stability and serviceability of high-rise buildings is the use of complex foundations. Complex foundations comprise foundations which utilise both shallow and deep foundation elements, mainly represented by the Combined Pile-Raft Foundation (CPRF). Contrary to conventionally designed raft or pile foundations, the design concept of a CPRF comprehends the interactions arising due to the complex mechanics in the interplay between raft, soil and deep foundation elementswhich will be highlighted in the next section.

SOIL-STRUCTURE INTERACTION OF COMPLEX DEEP FOUNDATIONS

Complex foundations are mostly represented by Combined Pile-Raft Foundations (CPRFs). CPRFs are generally designed for rather settlement sensitive soils, e.g. the clays in London (Hooper 1973, Cooke et al. 1981) or Frankfurt am Main (Sommer et al. 1985, Katzenbach & Reul 1997). The advantage of such a compound foundation is that the deep foundation elements can be set systematically at places with high loads from the superstructure in order to harmonise the settlements and to reduce the risk of punching and, thus, to effectively reduce the thickness of the raft (Love 2003). In this concept, the piles serve as settlement reducers rather than as elements for securing stability. So the piles are methodically loaded beyond the design value of an equivalent pile of a conventionally designed pile foundation. Such compound foundations are especially effective utilising the increase of stiffness with depth (Fig. 1) much better than a raft foundation which applies the largest stresses on the softer part in the upper region of the subsoil.

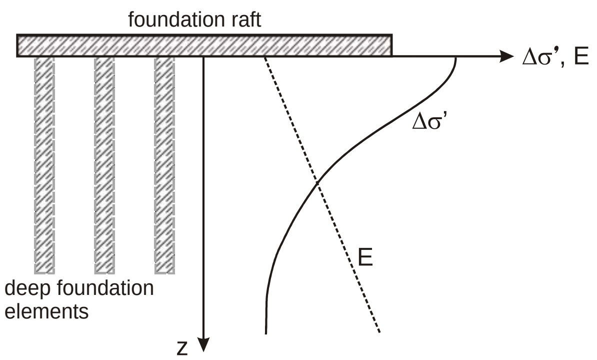

Fig. 1. Stress distribution beneath a foundation raft and increase of stiffness if soil with depth

One should bear in mind that the described concept can only be applied in a subsoil which does not contain strata at the pile bases much stronger and stiffer than the stratum right beneath the raft, otherwise the restricted displacement of the pile base is accompanied by attraction of stresses resulting finally in conventional end bearing piles as it has been observed e.g. for piles with the base in solid rock (Katzenbach et al. 1996).

In this context, one should also not forget that the piles of a CPRF exhibit a different load-displacement behaviour than piles of a conventionally designed pile foundation. This is due to the interactions between the foundation elements and the subsoil:

- Pile-Soil-Interaction
- Pile-Pile-Interaction
- Raft-Soil-Interaction
- Pile-Raft-Interaction

The stress states of the subsoil beneath a compound foundation are distinctly differing from the stress states in the subsoil surrounding a conventional pile. The part of the load which is transferred via the raft into the subsoil increases the hydrostatic pressure level at the pile shafts; this increase of the normal stresses due to the raft-soil-interaction from the stress level of a conventional pile foundation σ'_{pile} to the stress level of a CPRF σ'_{CPRF} be called $\Delta\sigma'_{compression}$ (Fig. 2). So the failure shear stress $q_{s,f}$ at the pile shaft according to the failure criterion of Mohr-Coulomb is computed by:

$$
\begin{aligned}
q_{s,f} &= \sigma'_{CPRF} \cdot \tan\varphi' + c' \\
&= \left(\sigma'_{pile} + \Delta\sigma'_{compression}\right) \cdot \tan\varphi' + c'
\end{aligned}
\qquad (1)
$$

So the maximum shear stress at the pile shaft is always larger for a CPRF than for a corresponding conventionally designed pile foundation. This corresponds well to experimental investigations (Vesic 1969, Ranganatham & Kaniraj 1978, Phung Duc Long 1993).

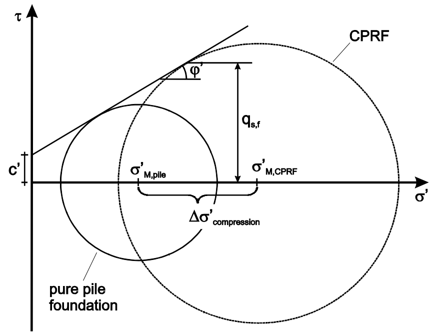

Fig. 2. Effect of increased normal stress level at a pile shaft of a CPRF in comparison to the pile shaft of a conventional pile foundation

The differences between the piles in a conventional pile foundation and the piles of a CPRF are displayed by means of numerical studies. The geometry of the modelled CPRF is depicted in figure 3. The thickness of the raft has been chosen to d = 1.0 m, the diameter of the piles to D = 1.5 m. The pile-pile-distance in the regular pile grid is e = 6D = 9 m. The length of the piles has been set to 30 m, which is a quite typical pile length in Frankfurt am Main (Katzenbach & Moormann 1999). The pile grid distance of e = 6D was chosen as a distance at which single piles do not interact (Cooke 1986); this has been targeted to demonstrate the influence of the raft-soil-interaction.

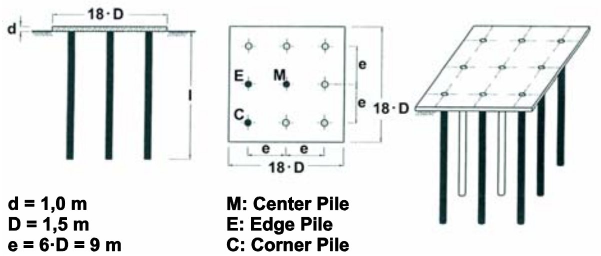

Fig. 3. CPRF model used for the estimation of the pile forces

The material behaviour of the piles and the raft is simulated as linearly elastic in the Finite Element analysis, whereas for the simulation of the material behaviour of the soil the modified Drucker-Prager cap model was used (Fig. 4). This constitutive 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 inside the yield surfaces do only cause linearly elastic deformations, while stresses on the yield surfaces 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 (Drucker & Prager 1952; Chen & Mizuno 1990). The basic material parameters – which have been determined within a continuous process of evaluating measurements and back-analyses on several high-rise projects in Frankfurt am Main (Katzenbach et al. 2005) – are shown in table 1.

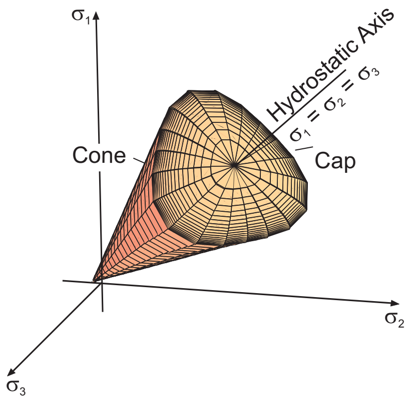

Fig. 4. Yield surfaces of the modified Drucker-Prager/cap model

The distribution of the Young's modulus E with depth z can be described by the simplified approach by Amann (1975):

$$E = 7MN/m \cdot (1m + 0.35 \cdot z) \tag{2}$$

This means that the stiffness of the Frankfurt Clay distinctly increases by depth *z*. Reul (2000) claims a similar, but non-linear relationship between *E* and *z*, which allows for rather surface near regions of the subsoil an approximation of E = 50 MN/m².

Table 1. Material parameters for Frankfurt Clay

Material Parameter	Symbol	Dimension	Value
Angle of friction	φ'	[°]	20.0
Cohesion	*c'*	[kN/m²]	20.0
Young's modulus	*E*	[MN/m²]	eq. 6 or 50
Poisson's ratio	ν	[-]	0.25
Unit weight	γ	[kN/m³]	19.0

Two simulations are picked out: one including the interaction between the raft and the subsoil (CPRF) and one at which the contact between raft and soil has been switched off, i.e. a pure pile foundation. The comparison of both computations is depicted in figure 5 for the centre pile. As expected, the piles of a conventional pile foundation behave within a certain load range pseudo-elastic until they reach a limit load. Then the settlements increase superproportionally with increasing load. This has to be ascribed to reaching the failure state at the shaft (upper right diagram in Fig. 5). The load-displacement curves of the CPRF piles do not exhibit this behaviour; in fact, the pile shaft resistance is still increasing after having exceeded the pseudoelastic range (lower right diagram in Fig. 5). The further increase in shaft resistance of the CPRF piles after exceeding this range has to be ascribed to the volumetric hardening due to the increase of the stress level (Fig. 6). This shows that the overall system strength and system stiffness of a CPRF is always larger than of a comparable pure pile foundation.

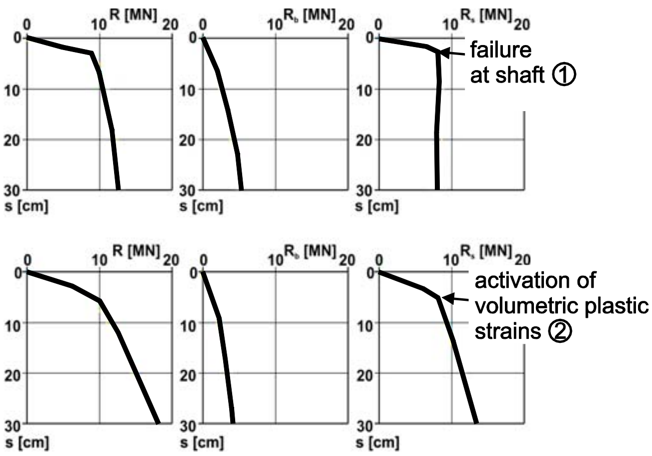

Fig. 5. Comparison between the load-resistance curves of the center pile of a conventional pile foundation (upper diagrams) and the center pile of a CPRF (lower diagrams) concerning the load-settlement behaviour with a distance in the pile grid of e = 6 D

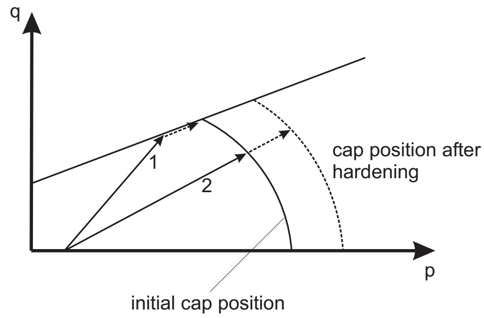

Fig. 6. Qualitative stress paths in the p-q-plane for points at the shafts of a pile of a conventional pile foundation (1) and a CPRF pile (2) causing activation of plastic volumetric strains due to volumetric hardening

DESIGN PROCESS FOR COMPOUND FOUNDATIONS IN ACCORDANCE TO EC7

In this section we present a design process for complex foundations like Combined Pile-Raft Foundations or rafts on barrettes (Katzenbach et al. 2003). As a a standardisation background is needed for the design process, we refer to Eurocode EC 7. In EC 7 complex foundations are not explicitly given a section, but they are rather handled like deep foundation elements. The general ultimate limit state design for piles according to EC 7 is ruled by the following inequation:

$$F_{c;d} \le R_{c;d} \qquad \text{resp.} \qquad F_{c;k} \cdot \gamma_F \le \frac{R_{c;k}}{\gamma_R} \tag{3}$$

This means that the design values of the acting forces $F_{c;d}$ compressing the pile must always be less than or equal to the maximum design value of the associated resistance force $R_{c;d}$ in ULS. (Design values are factorised values, e.g. for reduced strength parameters or increased loads to consider a safety level; they are indicated by "d". The not factorised values are called characteristic values and indicated by "k".)

This methodology can be transferred to the design process of CPRFs. For a CPRF, the inequation for the proof of the ultimate limit state is formed by the sum of acting forces on the CPRF $\Sigma F_{c;d}$ and the overall resistance of the CPRF $R_{tot;d}$ in ULS:

$$\sum F_{c;d} \leq R_{tot;d} \qquad \text{resp.} \qquad \sum F_{c;k} \cdot \gamma_F \leq \frac{R_{tot;k}}{\gamma_R} \tag{4}$$

It is important to keep in mind that the bearing capacity of the single piles is not regarded in this context.

The overall resistance force is – analogously to the pile resistance – dependant on the settlement. The overall resistance force of a CPRF in ULS is defined as that point at which the increasing of the settlement becomes more and more superproportional (Fig. 7).

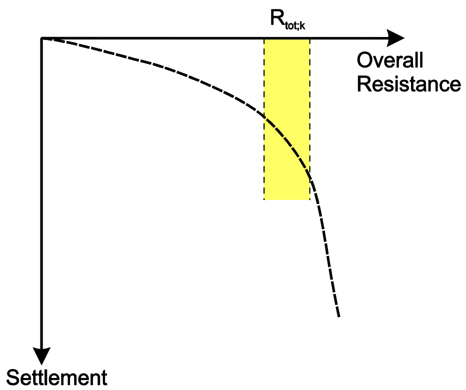

Fig. 7. Non-linear system behaviour of a CPRF and determination of the overall resistance in the ULS by means of a distinctly recognisable failure state

As according to EC 7 no numeric determination of the safety level is required, it is sufficient to safeguard that there will occur no failure before the subsequent resistance force level – derived from the ULS condition (4) – is reached (Fig. 8):

$$R_{tot;k} \geq \sum F_{c;k} \cdot \gamma_F \cdot \gamma_R \tag{5}$$

Due to the favourable interactions within a CPRF, a very distinct failure appears quite rarely, in most cases there is a smooth increase in the slope of the resistance-settlement-curve, so eq. (5) will apply in many cases.

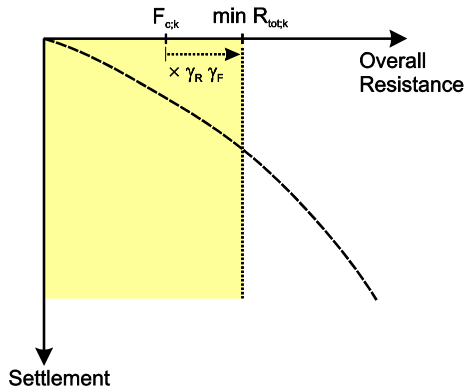

Fig. 8. Non-linear system behaviour of the CPRF and minimum distance between applied characteristic loads and overall resistance of the CPRF

The here displayed methodology is performed on the subsequent example.

DESIGN EXAMPLE

The presented design example contains a foundation system which has already been presented in section 2; the numerical model utilises the double symmetry of the foundation system so only a quarter of it is modelled (Fig. 9).

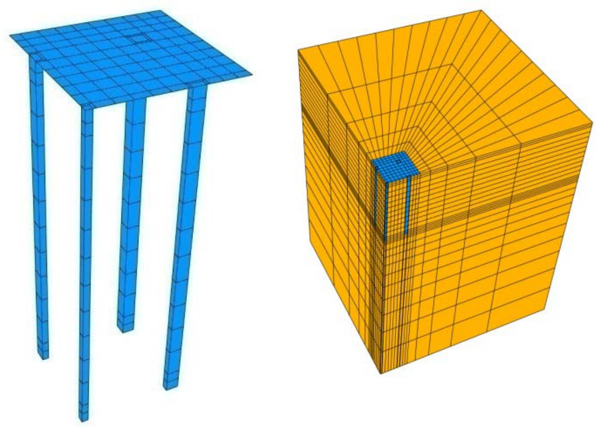

Fig. 9. Discretisation of the example CPRF and the whole model

The forces conducted by the superstructure sum up to steady actions of $\Sigma G_{c;k} = 90$ MN and variable actions of $\Sigma Q_{c;k} = 30$ MN.

With this numerical model, a numerical load test by steadily increasing the loads has been performed to generate the characteristic relationship between the settlement of the raft and the total load which is equal to the overall resistance of the CPRF. In figure 10, the evolution of the overall resistance is drawn versus the settlement of the centre of the raft.

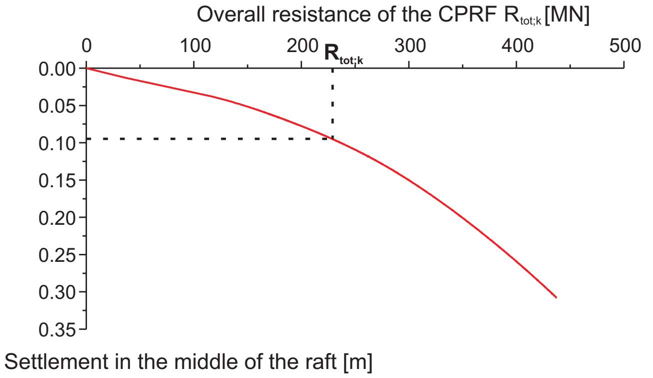

Fig. 10. Evolution of the overall resistance of the example CPRF due to the settlement in the middle of the raft

The evolution of the overall resistance force shows no distinct failure state but rather a continuous decrease of the system stiffness after having left a pseudo-elastic range. This has to be ascribed to the increasing inelastic volumetric deformations which occur due to the hardening of the cap in the constitutive model. Due to this fact, no unique resistance force in the Ultimate Limit State (ULS) can be deducted and equation (5) will be used.

The design value of the total load on the CPRF is computed by summing up the steady actions $G_{c;k}$ and the variable actions $Q_{c;k}$ times the related partial safety factors:

$$F_{c;d} = \sum G_{c;k} \cdot \gamma_G + \sum Q_{c;k} \cdot \gamma_Q \qquad (6)$$

As the partial safety factors are not uniform in the CEN states, the German factors are chosen according to DIN 1054 (2005):

$$\gamma_G = 1{,}35 \quad ; \quad \gamma_Q = 1{,}5 \quad ; \quad \gamma_R = 1{,}4$$

Applying these safety factors on equation (6) we receive:

$$\begin{aligned} F_{c;d} &= \sum G_{c;k} \cdot \gamma_G + \sum Q_{c;k} \cdot \gamma_Q \\ &= 90\,\text{MN} \cdot 1{,}35 + 30\,\text{MN} \cdot 1{,}5 = 167\,\text{MN} \end{aligned} \qquad (7)$$

According to equation (5) the result of equation (7) is multiplied by the safety factor for the overall resistance and applied:

$$F_{c;d} \cdot \gamma_R = 167\,\text{MN} \cdot 1{,}4 = 234\,\text{MN} \qquad (8)$$

Regarding figure 10, it can be seen that up to this loading no failure occurs. Thus, the stability of the foundation has been proved.

With the presented results of the numerical load test, it is also possible to investigate the Serviceability Limit State (SLS) because it bears the advantage of a physically orientated computation model.

DESIGN EXAMPLES OF CPRFs

For receiving an adequate, reliable and transferable computational model it is necessary to validate the model features and assumptions by performing back-analyses on antecedent cases. In the following section two examples of numerical back-analyses are introduced, the high-rise buildings Messeturm and Eurotheum in Frankfurt am Main.

The numerical back-analysis of the Messeturm aimed on two objectives. At first the analysis was undertaken to gain knowledge about the interactions and the load distribution within the CPRF. The second reason was to validate and calibrate the computational model.

The FE model used for the analysis mapped an eighth of the whole raft (Fig. 11) utilising the threefold symmetry of the construction. The analysis comprised several steps including the excavation process and the groundwater lowering and re-rising (Reul 2000).

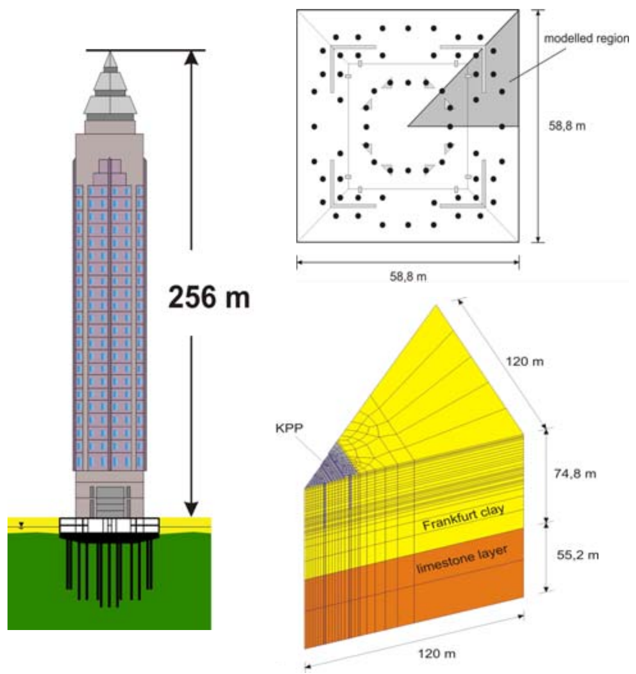

Fig. 11. Messeturm, Frankfurt am Main: View, ground plan and FE mesh

The material behaviour of the piles and the raft were simulated as linear-elastic in the Finite Element analysis, whereas for the simulation of the material behaviour of the soil an elasto-plastic model was used (Fig. 4).

The calculated settlements of 17 cm differed slightly from the measured values of 13 cm. The basic shape of the settlement distribution of the raft is nearly equal in both cases (Fig. 12). However, the results of the numerical analysis are matching the measurement data very good.

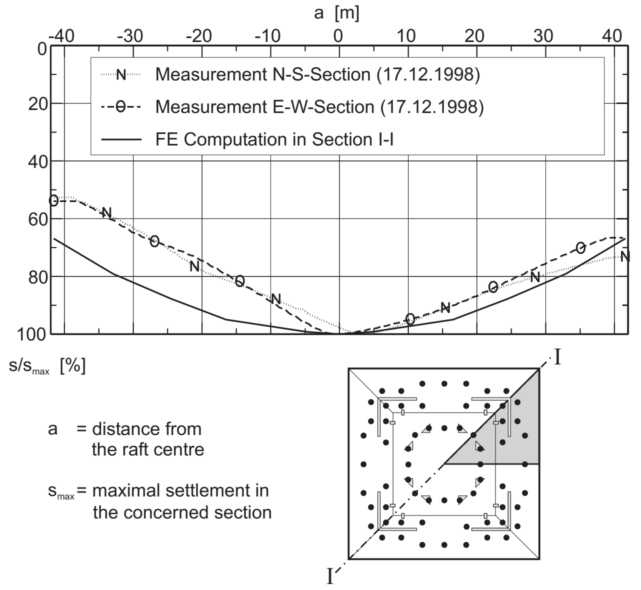

Fig. 12. Distribution of the relative settlements of the Messeturm CPRF (Reul 2000)

The herewith validated computational model was adopted to subsequent analyses, e.g. the simulations for the Eurotheum building or the Maintower high-rise building (Moormann & Katzenbach 2002). The construction of the Eurotheum building (Fig. 13) started in 1997 and lasted until 1999. The foundation is a CPRF with 25 piles, diameter of 1,5 m and pile length between 25 m and 30 m depending on the position of the pile. The total vertical load of the Eurotheum is about 550 MN. The Eurotheum consists of a tower area (Fig. 13) with a height of 110 m and a ground area 28 m × 28 m and an adjacent annex with six floors (Schmitt et al. 2002).

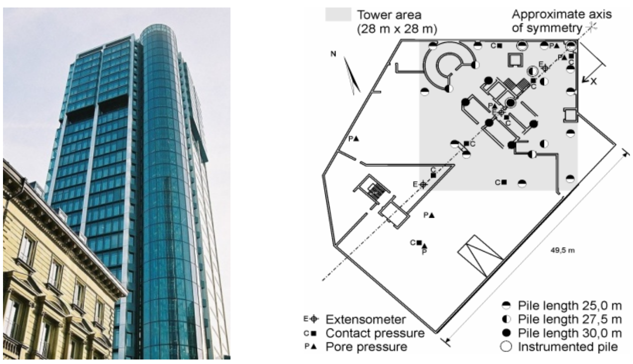

Fig. 13. Eurotheum, Frankfurt am Main: View and ground plan with piles and measurement devices

The location of the geotechnical measurement devices and the piles is shown in the ground plan of the building (Fig. 13). Four piles were equipped with load cells at the pile head in order to observe the bearing behaviour of the piles. The contact pressure of the raft is measured at seven locations, the pore pressure at six locations. The settlement of the building is observed by geodetic measurements.

The numerical back-analysis of the Eurotheum has been carried out with the three-dimensional Finite Element model shown in figure 14. Due to the approximate symmetry of the geometry and loading of the tower it was possible to reduce the geometry of the Finite Element mesh to one half of the real geometry. With a step-by-step-analysis the construction process including the excavation for the basement, the pile and raft installation and the gradual loading has been simulated.

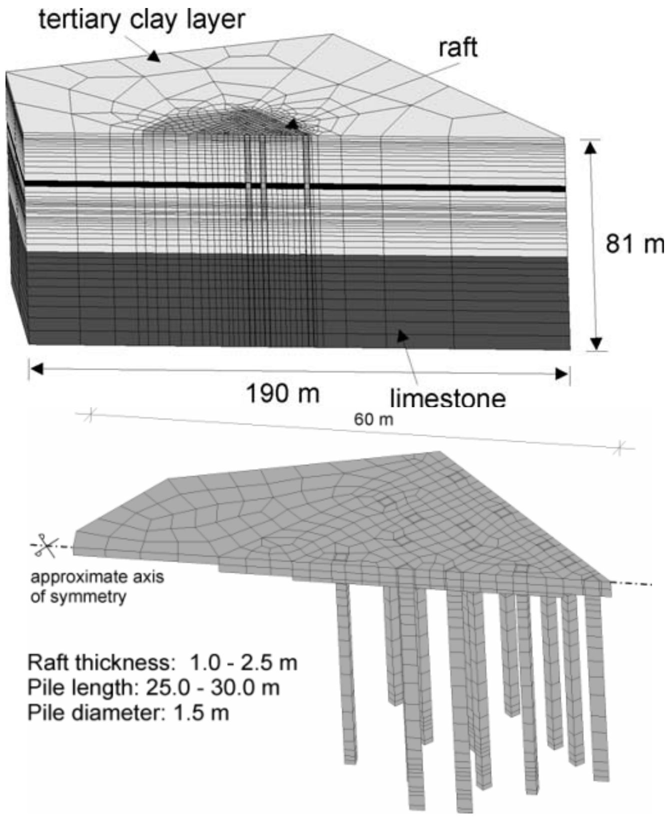

Fig. 14. FE mesh of the Eurotheum (Schmitt et al. 2002)

The settlements along the axis of symmetry obtained by the FE analysis and the associated measurements are displayed in figure 15. The maximum settlement for the Eurotheum observed at the end of the construction of the high-rise building was 3 cm. The calculated settlements of up to 4 cm are final settlements and do not consider the consolidation process which was still running when the measurements were performed.

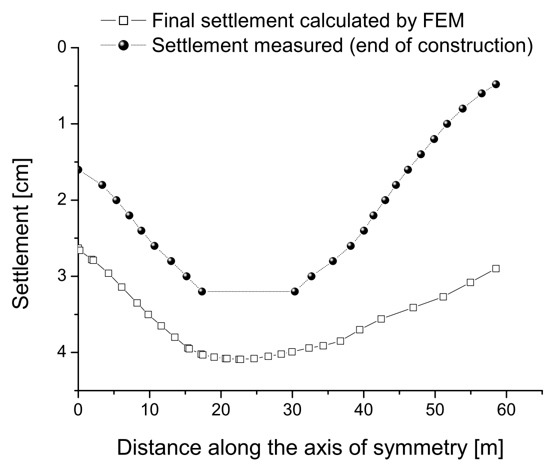

Fig. 15. Measured and computed settlements of the Eurotheum (Katzenbach et al. 2002)

SEASONAL THERMAL STORAGE

Heating and cooling of the building interior to establish comfortable climatic conditions cause a relatively large energy demand in any building. Especially high-rise buildings show a particularly high demand, since such buildings usually have a high ratio of facade surface to office space. Rising prices for gas, oil and electrical power as well as environmental considerations give rise to alternatives to the conventional concepts of energy supply.

Additional to their structural function all constructional elements with direct soil contact - primarily foundation piles but also rafts and retaining walls - can be equipped as energy exchangers for using the adjacent subsoil either as energy source or energy storage.

To enhance a standard foundation pile into an energy pile it is equipped with loops of plastic tubes carrying an energy exchanger fluid. These tubes are attached to the reinforcement cage before lowering it into the borehole, see figure 16.

Fig. 16. Reinforcement cage with attached tubes and measuring equipment

The temperature difference between the fore flow of the energy exchanger fluid and the soil surrounding the pile causes an energy flow and thereby the thermal activation of the surrounding soil for either energy withdrawal or deposit. This allows for two fundamentally different strategies to utilise the subsoil: energy extraction and the so-called seasonal thermal (energy) storage. Energy extraction in combination with a heat pump is usually solely used for heating purposes.

Seasonal Thermal Storage makes use of the energy capacity of the subsoil. Energy is cyclically with the seasons withdrawn and deposited under and over the natural temperature level of the soil. Therefore in times of energy overage in the building during the summer months energy is transferred to the soil from the building or external sources, e.g. process-induced heat such as from cooling appliances. By fall the soil has experienced a rise of temperature by several degrees, and in winter the stored energy can be withdrawn for heating by inverting the process in the ground. In spring the soil has been cooled down and is ready for summer operation again.

Figure 17 displays both states of operation with the three involved systems: the energy piles in the subsoil and the building heating and cooling systems respectively.

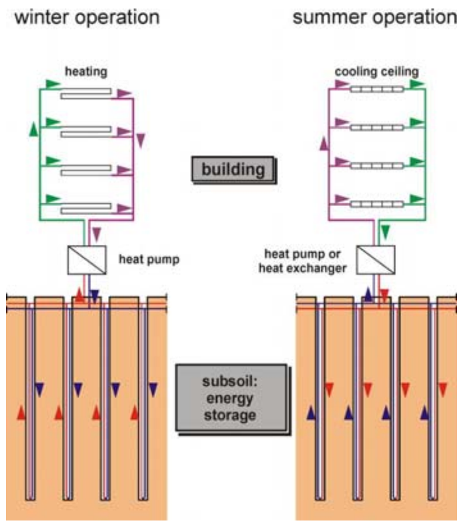

Fig. 17. Seasonal Thermal Storage System

In Frankfurt am Main, a number of large buildings have been equipped with systems for geothermal use of the subsoil. Examples among others are the high-rise buildings Gallileo (Katzenbach et al. 2001) and the Maintower (Fig. 18).

The Thermal Storage Systems are designed using the basic heat transfer equation (Eq. 9):

$$\mathrm{div}(\lambda\,\mathrm{grad}\,T) - (\rho \cdot c)_W\,\mathrm{div}(v \cdot T) + \mathrm{div}(D_\lambda\,\mathrm{grad}\,T) + \dot{Q}_i = \rho \cdot c\frac{\partial T}{\partial t} \quad (9)$$

Here, the first summand denotes conduction, the second summand convection and the third summand dispersion. Heat sources are considered by $\dot{Q}_i$ and the right side of the equation represents the temporal change of temperature.

Fig. 18. Maintower and Gallileo in Frankfurt am Main

Since the overall energy storage capacity and output power are strongly dependent on the soil and ground water conditions, a specific geothermal exploration is essential designing the building service system properly. A well designed system can be cost efficient, environmentally friendly and therefore saving energy resources.

CONCLUSIONS

It has been shown that numerical methods, especially the Finite Element Method, are capable of acting as a reliable tool for the proof of safety in the Ultimate Limit State (ULS) for complex foundations because they can map all relevant effects of the Soil-Structure Interaction. Moreover a concept for a design process for complex foundations based on numerical methods is introduced and proved to be conform to EC 7. In this context, it has to be stressed that it is indispensable to develop a calibrated and validated computational model for a reliable design process.

The Soil-Structure Interaction occurring at a complex foundation like a CPRF causes a lot of favourable effects which can be utilised by engineers, finally leading to an effective cost and resource reduction compared to conventional design processes, e.g. for conventional pile foundations.

Recent developments extended the functionality of deep foundations towards a geothermal usage of the subsoil for running the building.

All these research outcomes clearly lead to an increase of efficiency concerning costs and resources both during construction and service of high-rise and large buildings.

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