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COMBINED FOUNDATION OF A HIGH-RISE BUILDING COMPLEX ON SAND: ANALYSIS AND OBSERVATION

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ABSTRACT

The building complex of the Beisheim Center at the Potsdamer Platz in Berlin has recently been completed. Different building parts had to be founded partially on the existing tunnel structures and partially on a system of piled-raft and raft foundations. The combined foundation was designed so as to minimize sectional forces due to the different foundation stiffnesses that were the unknowns in the analysis. Based on pile load tests the nonlinear stiffness of piles has been determined. For the design, both foundation components have been first decoupled and analysed separately. Subsequently, the interaction effects have been estimated by a simplified procedure. The pile configuration has been optimized by considering its effects on the superstructure. The performance of the combined foundations has been checked by comparing computed deformations with actual measurements at relevant points. Parallel to this, the system has been analysed by a 3D finite element model. The concept and the key design items of the combined foundation are presented, and a comparison is made between the estimated deformations and the measurements. The methodology for the analysis of building response to train induced vibrations is outlined.

Key words: monitoring, numerical modelling and analysis, piles, rafts, settlement, soil-structure interaction, vibration (**IGC**: E2/E12/H1)

INTRODUCTION

Piled rafts are a new foundation concept for high-rise buildings and have been successfully implemented in Germany since the beginning of the 1990's (Katzenbach et al., 2000). This foundation type is a viable alternative to conventional pile or raft foundations in competent ground. The implementation of this foundation type led to the abolishment of complicated settlement-correction techniques. During the past years, the computational methods available in combination with measurements on real projects have allowed the realistic modelling of the complicated bearing behaviour of that composite foundation system. A respective guideline (Hanisch et al., 2001) addresses the design methodology, the safety concept, the use of the observation method, the subsoil investigation program, the conditions for which pile load tests are necessary, and the requirements imposed on the calculation methods.

Almost exclusively, case studies in the literature refer to high-rise buildings or bridge foundations supported directly by piled rafts (e.g., Poulos, 2001). Here, a particular case of soil-structure interaction is considered: it involves a high-rise building supported partially by a piled-raft and partially by the adjacent caisson of a railway tunnel. The limitation of differential displacements between these two entirely different foundation parts governs the foundation design, i.e., within the ultimate state design philosophy the serviceability limit state is the critical one. In the sequel, the main features of the project are described, the analysis method adopted is outlined, and the results of measurements on the system behaviour are given.

PROJECT DESCRIPTION

The Beisheim Center building complex was the final project in the area of the Potsdamer Platz in Berlin's city centre. It consists of three major building units, as shown in Fig. 1. The particularity of the engineering design is associated with the fact that the mainline railway crosses the lot in a north-south direction in the form of a four-lane tunnel at a depth of 7 m below ground surface. This tunnel has been constructed as a monolithic structure partially as caisson and partially using the conventional cut-and-cover method. Thus, it does not have the expansion and the differential settlement joints between distinct blocks commonly installed in the other tunnel structures in the city. The thickness of the roof and base slabs are 2.0 m, and the side walls are 1.5 m thick. Along the eastern side of the building complex an additional tunnel of the light rail runs parallel to the property border. This tunnel lies at a depth of 12 m below ground surface and was built in the 1930's by the cut-and-cover method.

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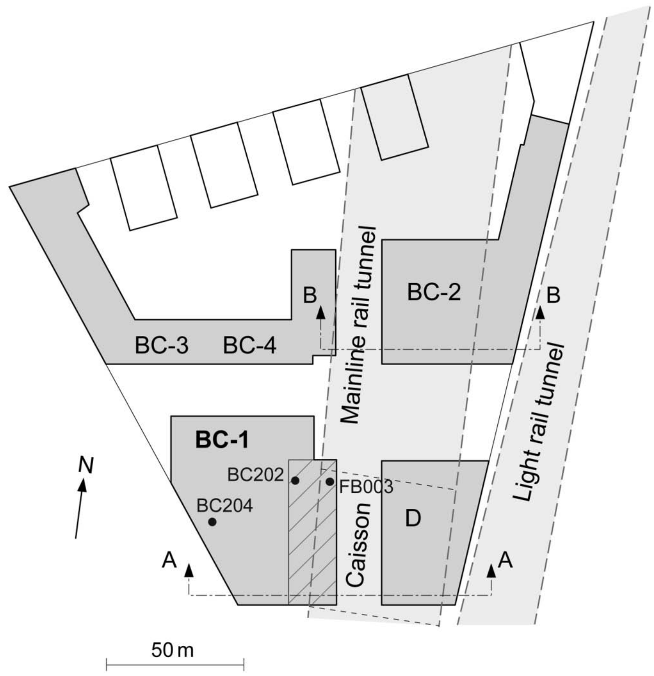


Fig. 1. **Beisheim Center: plan view with location of observation points. The hatched area indicates the high-rise section of building BC-2**

The building complex Beisheim Center described herein consists of the following units: Building BC-1, which consists of apartments and a hotel has a 19-storey high rise section 70 m in height along the eastern side directly above the mainline rail tunnel, as indicated by the hatched area in Fig. 1, and a lower section, 36 m high, along the western side. The underground space has three basement levels that are reduced to one in the area above the tunnel. Building BC-2 is also partially founded on the mainline rail tunnel with ten storeys in its northern section and eleven storeys (height 37 m) in its southern section. Two and one basement levels are built in the area outside and directly over the tunnel, respectively. Building BC-3/BC-4 in the northern section consists of ten storeys and two basement levels (height 36 m). The constraints described above made a combined foundation for both buildings BC-1 and BC-2 necessary: The load transfer to the ground takes place partially through the tunnel structure and partially through a piled-raft foundation, as shown in the two typical cross sections given in Fig. 2. The present paper focuses on the design of building BC-1.

For the adjacent 18-storey high-rise building D, east of the property, only limited information is available. Its basement is monolithically connected to the mainline rail tunnel. The major part of the western high rise section, 65 m in height, is resting directly on the tunnel, while the eastern part with a height 36 m is founded on piles

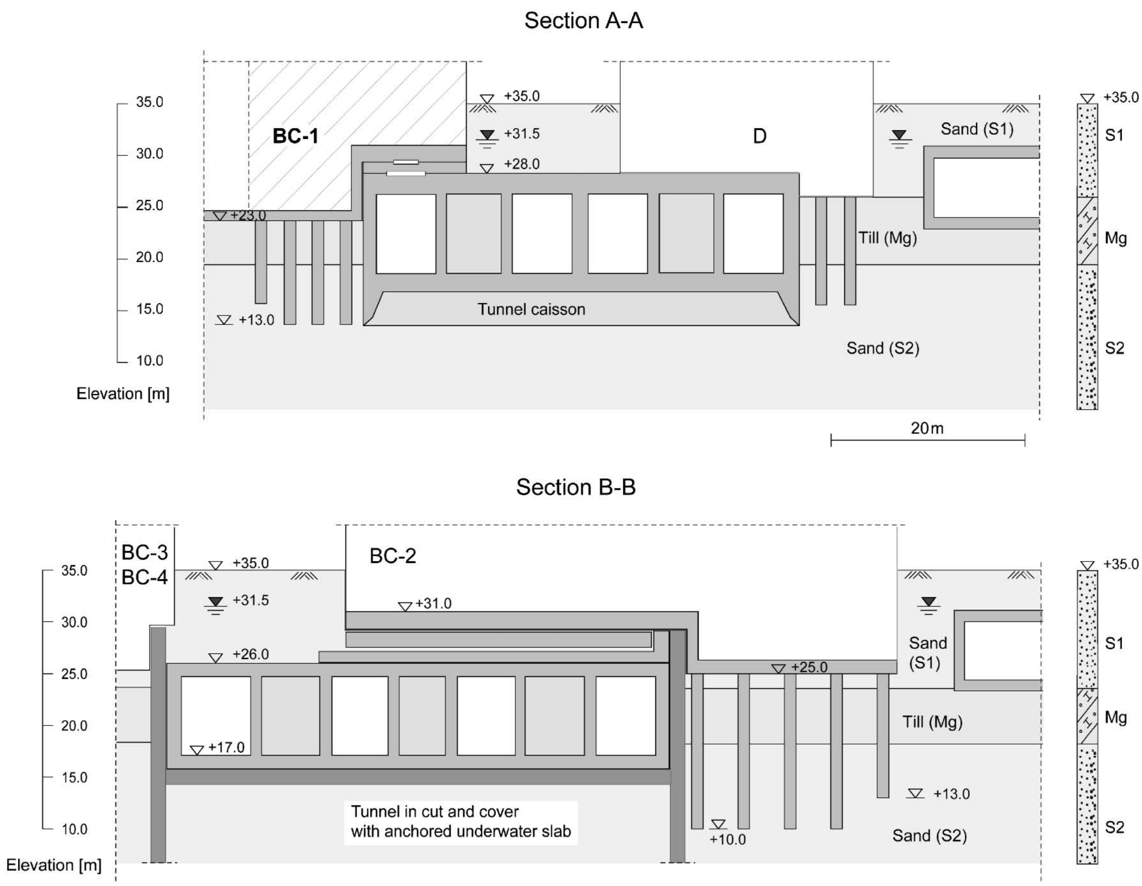


Fig. 2. **Cross sections A-A and B-B through the building complex**

(Fig. 2). Further constraints are defined by the underground station of Potsdamer Platz with access to the neighbouring buildings and railways in the south and by the neighbouring SONY Center building complex in the southwest.

The main features of the geotechnical design and the excavation support are given by Richter et al. (2002). The foundation had to fulfil the criteria imposed with respect to bearing behaviour and serviceability of the structural elements, i.e., foundation slabs, basement walls, tunnel roof and walls. Due to the soil-structure interaction, the sectional forces are strongly influenced by the relative displacements between the various parts of this complex structural system. In this sense, the accurate prediction of the deformations governs the design. Maximum allowable values for differential settlements were not stated explicitly. Instead, a deformation analysis was performed first and the sectional forces were subsequently computed. By means of an iterative optimization procedure the stiffness of the piled-raft foundation was modified by changing the pile configuration until the sectional forces reached the allowable level.

In addition, the requirements imposed by the tolerable level of vibration and structure-borne noise also had to be considered in the design. The predicted values exceeded the tolerable ones, thus making necessary the installation of insulation mats between the mainline rail tunnel roof/walls and the foundation slab.

Next, the foundation concept is outlined and accompanied by the main outcomes of the analysis. It is followed by the results obtained during the monitoring of the building performance and a brief description of the vibration analysis.

SOIL CONDITIONS

The soil stratigraphy at the site is typical for Berlin: The top layer consists of a man-made fill followed by a medium density sand (S1) down to a depth of 12 m. It is followed by a layer of stiff to very stiff glacial till with 4 to 5 m in thickness underlain by the dense sand (S2). The groundwater table is located 3.5 m below ground surface along the entire building complex. The relevant soil parameter is its stiffness that is described in terms of a depth-dependent constraint modulus E_S as determined by the results of oedometer tests and a back calculation of observed settlements of high-rise buildings in Berlin (Richter, 1994). It is distinguished between first loading and reloading, the latter due to loading during the period of the glaciers. For first loading $E_S = E_0 \cdot \sqrt{z}$, while for reloading E_0 is replaced by E_W. Moduli are given in [MN/m^2], and z in [m] is the depth below ground surface. For the particular layers, the following values have been used: for sand (S1) $E_0 = 2$, $E_W = 6$; for glacial till (Mg) $E_0 = 5$, $E_W = 20$; for sand (S2) $E_0 = 30$, $E_W = 90$. These values have been verified by recent observations on high rise buildings in the city of Berlin. Values for the shear strength parameters in terms of angle of friction φ', cohesion c', and undrained shear strength s_u that were used in design are: sand (S1): $\varphi' = 34°$; till (Mg): $\varphi' = 28°$, $c' = 20$ kPa, $s_u = 200$ kPa; sand (S2): $\varphi' = 37.5°$.

FOUNDATION CONCEPT AND DESIGN

The foundation level varies within the footprint of the building complex. Common rafts have been successfully applied in the broader area for foundation levels deeper than the top of the glacial till layer. For the low-rise building sections with heights up to 35 m and 2 to 3 basements, the pressure under total load is approximately equal to 250 kN/m^2. For the high-rise section of BC-1 with a footprint area of 20 m by 50 m and height approximately 70 m the pressure under total load varies between 420 kN/m^2 and 570 kN/m^2. As stated above, in order to achieve an almost uniform settlement, the high-rise sections of BC-1 and BC-2 have been placed on piled-rafts. The lower buildings BC-3/BC-4 in the north and the western section of BC-1 are founded on common rafts.

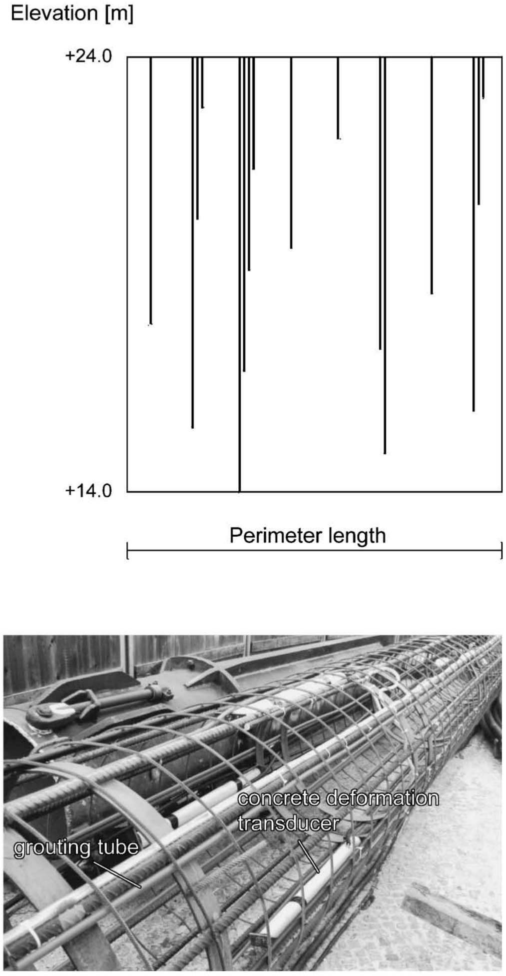
The polyurethane insulation mat for the vibration protection countermeasures that is placed between tunnel and building foundation is 25 mm thick with typical values of Young's modulus in the range of 2 MN/m^2. The exact type, and accordingly the stiffness, is selected dependent upon the applied contact pressure at the various locations of the foundation area.

The chosen concept of a combined foundation is exemplarily demonstrated here for building BC-1. The cross section is depicted in Fig. 2. The raft thickness varies between 1.0 m and 2.0 m with an average value of 1.5 m. The bored piles with a diameter of 1.2 m were enhanced in their bearing capacity by additional grouting of the shaft and base. Their length was restricted to 10 m in order to avoid perforation of the sealing layer that has been installed as part of the water-tightening system of the open pit excavation. Four pile rows have been placed in a parallel direction to the tunnel axis in a dense grid of 3.0 by 3.0 m. The smooth transition to the adjacent raft foundation of the lower buildings is achieved by reducing the pile length along the edge of the piled-raft to 8.0 m.

Pressure grouting of the pile base is applied by using a flexible bladder which is installed together with the reinforcement cage and permits distribution of the grout over the entire base area. Pressure grouting along the shaft is carried out by means of several small-diameter plastic tubes with a single outlet attached to the reinforcing cage. The arrangement of the valves around the pile shaft is made in such a way that each valve supplies roughly 4 m^2 of shaft area. After initial hardening of the pile concrete, the concrete cover is fractured by applying high water pressure. It is important that fracturing occurs at the right point of time. The pressure and volume of the grout as well as the water-cement ratio (< 0.7) and grouting rate are assessed from experience. Typically, a grouting pressure of 20 bar and a grout volume of 100 kg per valve are applied (Seitz and Schmidt, 2000).

Two pile-load tests were carried out during the design process in the area of the building construction. The piles were adequately instrumented in order to measure pile

(a)



(b)

Fig. 3. (a) Developed view of reinforcement cage (8ϕ25 mm) in the pressure-grouted portion of pile no. 1 with arrangement of grouting tubes; (b) instrumented pile with concrete deformation transducers and grouting tubes

head forces as well as the distribution of friction along the pile shaft (Geiss, 1999). Piles had a diameter of 1.20 m and lengths of 10 m (pile no. 1) and 15 m (pile no. 2), respectively. Piles were strengthened by grouting of the shaft and base. The arrangement of grouting tubes for pile no. 1 is shown in Fig. 3 together with a photograph of the instrumented pile. The success rate of grouting along the shaft (percentage of opened valves) was 88% for pile no. 1 and 67% for pile no. 2, respectively. Grouting of the base was performed only on test pile no. 2. Due to the lower grouting success pile no. 2 showed in the field trial a considerably lower stiffness than pile no. 1, although it was longer and thus reached deeper and stiffer soil layers. Results of the field trials for the two piles are shown in Fig. 4. For the pile construction a grouting success rate of 90% was prescribed and also maintained for all piles installed.

However, the observed behaviour of a single pile is not identical to that of a pile within a group with a dense grid: Due to the reinforcing effect of the neighbouring piles and the increase of confinement due to the load of the superstructure the effective pile stiffness is considerably higher. In the design, the constrained modulus corresponding to reloading was used to approximately capture these effects.

Since the aim of the design is the limitation of relative settlement, it is essential that the same assumptions regarding system modelling and effective soil stiffness are made for both components of the foundation, i.e., piled-raft and tunnel. The required low settlement level implies small strains in the soil thus justifying an equivalent-linear elastic analysis. Deformation moduli of the soil are adjusted to comply with the expected load level. Piled-raft and tunnel were first treated independently using as input the load intensity and distribution derived from the rigid base analysis of the superstructure.

The tunnel was modelled as a continuous box supported on springs by using a conventional finite element code. The soil spring constant was set equal to 30 MN/m^3. The value was selected from experience with past projects in the vicinity. Under maximum load, the analysis yielded an average vertical deformation of the tunnel beneath building BC-1 equaled 18 mm.

The current design methodology for piles and piled-raft followed in Germany is summarized by Vrettos (2007). The method for the analysis of the piled-raft foundation followed here is based on the simplified method suggested by Randolph (1994) and Fleming et al. (1992). The method allows for the calculation of the global settlement as well as the load distribution between pile group and raft. It is based on the solution for the stiffness of a pile-raft unit that is estimated through an approximation of the respective elastic continuum solutions. The overall stiffness of the pile group and raft system is determined in dependency of the stiffness of the pile group embedded in the soil, the stiffness of the raft resting on the soil, and an appropriate pile-raft interaction factor that is calculated by assuming that the settlement of the surrounding ground is decaying with distance according to a logarithmic law. The stiffness of the pile group is obtained as the weighted superposition of the stiffnesses of the individual piles according to the approximate formula suggested by Fleming et al. (1992), assuming that all piles are identical and uniformly arranged within the pile group.

The analysis for the system configuration described above yielded a global soil spring for the piled-raft equal to 33.3 MN/m^3. The load carried by the raft was 16% of the total load, the soil spring for the raft 5.43 MN/m^3, and the average spring constant for the individual piles 252 MN/m. The influence to the adjacent raft in the west is computed by replacing the piled-raft by an equivalent raft located at a depth equal to 2/3 of the pile length. The soil spring constant for this raft was computed equal to 10 MN/m^3.

Using this initial set of parameter values the detailed structural design of the combined foundation was carried out using a pseudo-coupled soil-spring method, whereby the soil moduli, and accordingly the spring constants, were adjusted to the actual load level. The approximate method proposed by Mayne and Schneider (2001) was

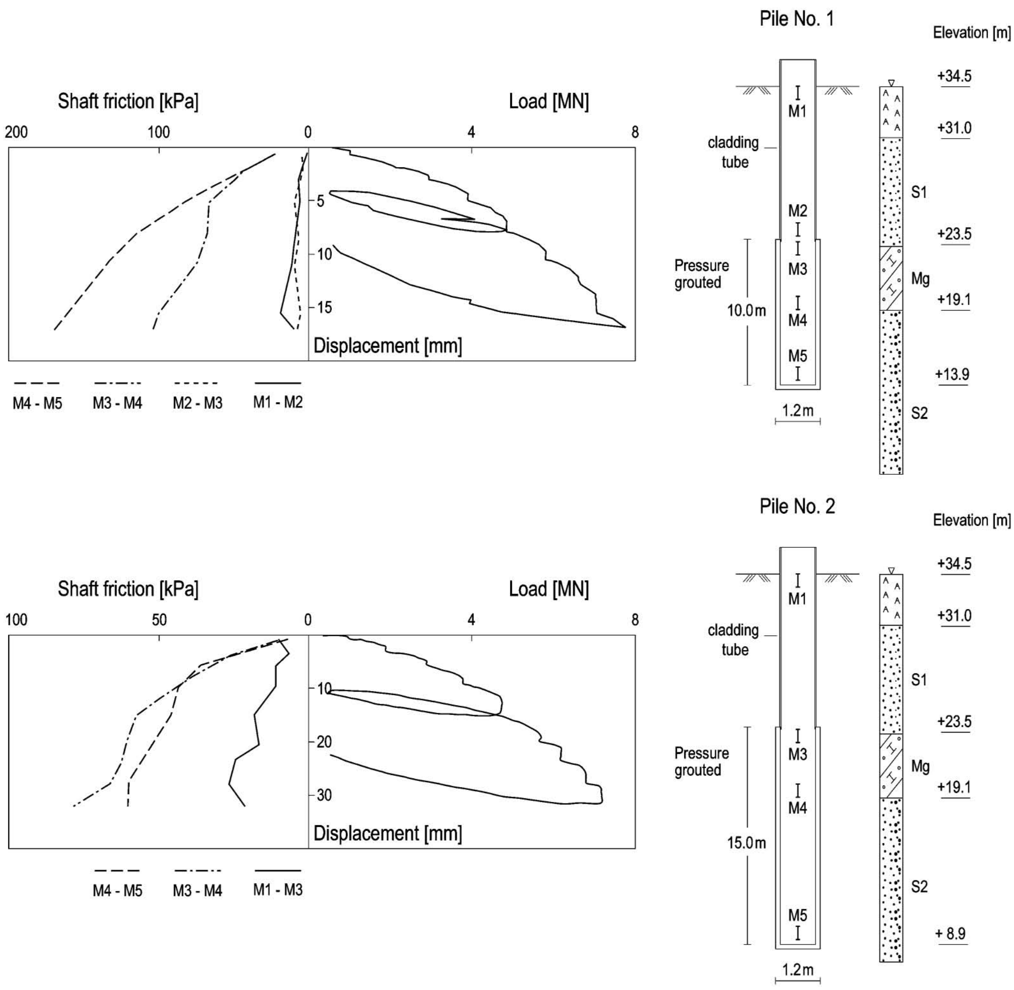


Fig. 4. Pile load test on pile no. 1 (top) and pile no. 2 (bottom). From left to right: shaft friction evolution with displacement in various sections along the pile; load-displacement curve; instrumented pile; soil profile

adopted, whereby the reduction of the soil modulus with increasing strain level is a function of the degree of shear strength mobilization.

By means of an iterative optimization process the stiffness of the piled-raft was modified by changing the pile geometry (length, distance) until the sectional forces in the building structure reached the desired level. The non-linearity of soil resulted at the end in a 20% reduction of the initial soil stiffness that in turn implies an increase of the load share carried by the raft to 21% and a reduction of the pile spring constant to 192 MN/m. The settlement of the piled-raft under total load is computed equal to 17 mm, i.e., is of the same order as that of the tunnel. The additional settlement due to the compressibility of the insulation mats was estimated approximately equal to 5 mm. The selected configuration yielded an almost uniform loading of the piles with an average pile load of 4.7 MN. To consider the inherent uncertainty in the soil behaviour the computed differential deformations were additionally varied within a prescribed range of $\pm 15\%$ and the unfavourable combination was subsequently used for the structural analysis. For the settlement prediction, it was assumed that the effective, settlement-inducing load is equal to 70% of the maximum load, which also includes the variable loads.

The design of the piled-raft foundation for building BC-2 followed exactly the same lines. In this section the pile distance was set equal to 5 m and the length of the shaft-grouted piles was increased to 15 m.

SETTLEMENT MEASUREMENTS

The relevant measurements accompanying the construction process refer to vertical displacements at points in the tunnel as well as on the building shell. The position of the monitoring points is shown in Fig. 1. We hereby present results for point FB003 on the tunnel wall, point BC202 on the high-rise founded on a piled-raft, and BC204 on the low-rise building founded on common raft. The evolution of settlement is plotted against the various construction phases in Fig. 5.

Measurement data show the expected heave due to the excavation of the open pit and the subsequent settlements due to the loading by the superstructure. The maximum vertical heave of the tunnel caisson amounts to 13 mm and is attributed to unloading due to the excavation above and on the side of the tunnel as well as due to excess water pressure exerted on the tunnel base slab. This

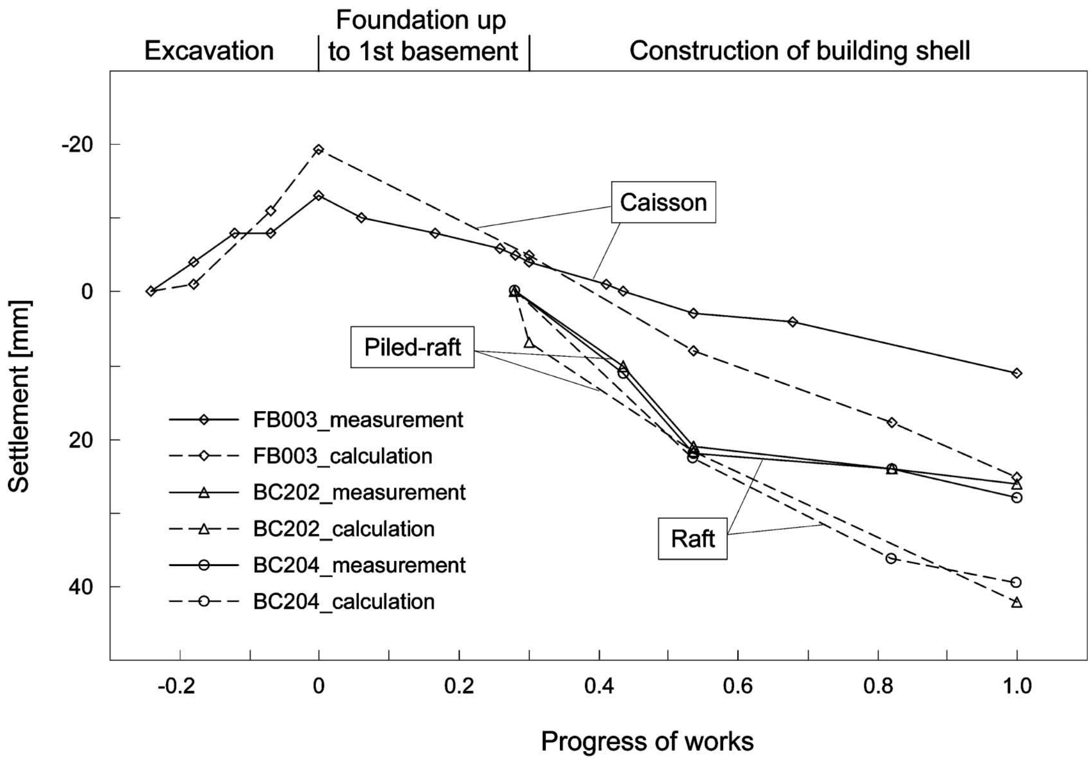


Fig. 5. Evolution of vertical displacements with construction process: measurements and results of the 2D finite elements analysis

confirms that the tunnel has been loaded during the construction process since it is part of the excavation support system. At the reference measurement of the piled-raft (at 28% of progress of works) the heave of the tunnel caisson was 5 mm. The direct interaction between tunnel and building is activated after the foundation slab overhangs the tunnel. At the end of works the final maximum absolute settlements were 11 mm on the tunnel (observation point FB003) and 26 mm on the building piled-raft foundation (point BC202). Thus, the observed settlement difference during the simultaneous loading of tunnel caisson and piled-raft equals 10 mm. This settlement value is further reduced if the 5 mm compression of the insulation mat is considered. The remaining $10-5=5$ mm must be interpreted as the sum of the compression of the tunnel walls and extension of the building basement walls. It should be noticed that the observed differential settlement between the low-rise (common raft) and the high-rise building section (piled-raft) is almost negligible.

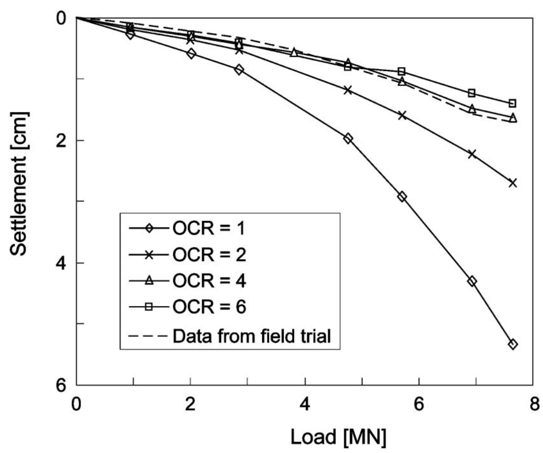


Fig. 6. Load settlement curve for pile no. 1: field trial vs. finite-element analysis

FINITE-ELEMENT COMPUTATIONS

The final design was performed by the method described above. In the frame of a post-construction detailed study, a non-linear analysis using finite elements has been carried out. Because of the large system dimensions a 3D numerical analysis is practicable only for parts of the entire system. The codes PLAXIS 3D Tunnel and PLAXIS 2D (Brinkgreve and Broere, 2004) are used for the analyses.

In the first step, the behaviour of a single pile with variable length and grouting success rate is analyzed. For the top sandy layer (S1), we use a Mohr-Coulomb elastoplastic constitutive law while for the glacial till (Mg) and the underlain sand (S2) the built-in Hardening-Soil model with stress-dependent deformation moduli is adopted. The closest agreement with the trial test data is obtained by assuming an overconsolidation ratio OCR equal to 4 (cf. Fig. 6). By increasing the OCR, the share of shaft friction in the total resistance also increases, reaching at the maximum pile load of 7.7 MN a value of 78%. For an OCR equal to 4, the ratio between base resistance and shaft friction agrees with the measured values as well.

In the next step, we investigate the effects of the approximation of the 3D model by a plane strain 2D model

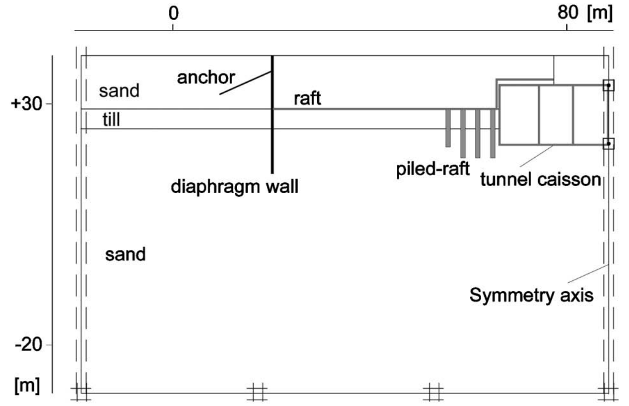


Fig. 7. Finite element system with boundary conditions for the 2D deformation analysis of the foundation of building BC-1 in the cross section through the observation point BC204

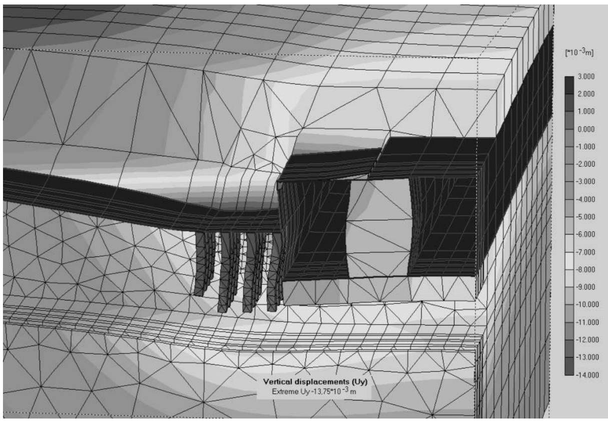


Fig. 8. Deformed shape of the 3D finite element model of the subsystem foundation BC-1 with symmetry axis at the tunnel center

that is commonly used in design. In the simplified 2D model, the piled-raft is replaced by a raft embedded at the level of the tunnel base and loaded with an average pressure of 200 kN/m^2 while the tunnel is loaded with 500 kN/m^2 (cf. Fig. 7). In the 3D model, the pressure of 500 kN/m^2 on the tunnel is applied only over an area 20 m in length in the direction of the tunnel longitudinal axis. As expected, the 2D model yields considerably larger settlements, in the present case by a factor of 2.

Furthermore, we investigate the accuracy of modelling the pile rows (3D) by means of equivalent walls (2D). Within the frame of an engineering approximation, friction forces at both sides of the wall are reduced in dependency on the pile diameter and distance, and the base resistance of the equivalent walls is neglected. Although this approximation can not capture the through-the-soil interaction between the individual piles, the simplified model yields overall system stiffness values comparable to those of the 3D model.

Finally, a detailed 3D analysis of the relevant partial system consisting of tunnel, piled-raft of the high-rise, and raft of the adjacent low-rise buildings is carried out. The deformed shape is depicted in Fig. 8. Compared to a 2D analysis, the maximum deformation values are smaller by a factor of approximately 3. In addition to the aforementioned shape factor 2D/3D of 2, the much coarser finite-element grid used in the 3D model due to the limits imposed by the computer code results in a stiffening of the system in the order of approximately 50%. The observed settlements lie between the 2D and the 3D computed values.

VIBRATION PROTECTION

The second part of the design addresses the building protection against traffic induced vibration in the tunnels underneath and adjacent to the buildings. The analysis of vibration propagation from the source (rail track) to the receiver (floor) requires a complicated procedure involving a number of unknowns. The maximum tolerable vibration level with respect to human sensitivity is defined in terms of an intensity perception factor in the DIN 4150. The tolerable level of the structure-borne noise was specified by the client's consultant.

The vibration propagation including soil-structure interaction is computed by means of a simplified 2D global model that is set-up as follows: The soil is represented by a layered half-space approximation and the piles by linear springs. Pile stiffnesses are computed from the dynamic response analysis of the respective pile group. The eigenfrequencies of the relevant building floors are first determined in a 3D model. The floor cross-section properties are then modified so as to yield in the equivalent 2D model approximately the same eigenfrequency and central deflection of the 3D model.

Since, at that time, train operation along the mainline railway tunnel underneath the high-rise hadn't commenced, the input could not be directly measured. Instead, a one-third octave band spectrum of the train induced vibrations, as given in Table 1, was specified on the tunnel walls by the consultant of the railway company and was also part of the planning approval procedure. This vibration level had to be guaranteed by the railway company by appropriate countermeasures, in this case through the installation of a heavy mass-spring system with a tuning frequency of 8 Hz. The prediction of the vibration level in the building consisted of four distinct steps: i) computation of one-third octave floor response spectrum for an excitation by a unit load on the tunnel slab, ii) computation of the one-third octave spectrum on the tunnel wall for a unit load excitation, iii) computation of the one-third octave difference spectrum on the tunnel wall from the specified one and the spectrum due to the unit load excitation, iv) superposition of the difference spectrum and the computed floor spectrum. The predicted level of structure-borne noise was higher than the tolerable one. The installation of insulation mats between the tunnel roof and walls and the foundation slab was therefore necessary. The implications for the structural design are given above.

For the light rail that continued operation during construction, it was possible to directly measure the emitted

Table 1. One-third octave spectrum of the effective vibration velocity at the tunnel walls of the mainline railway

f [Hz]	8	10	12	16	20	25	32	40	50	63	80	100
v [(μm/s]	11	11	7	5	4	4	6	9	8	4	2	1

vibrations both in the tunnel and also in an adjacent borehole. The analysis was then carried out using transfer functions for the individual floors. In order to reduce the vibration level that affected a number of buildings along the train line, it was decided to reduce the vibration level in the source though the installation of sub-ballast mats.

CONCLUDING REMARKS

With increasing use of urban space for high-rise buildings next to new/existing infrastructures, design for serviceability will continue to gain importance, and will often determine the requirements imposed on the foundation system. The settlement behaviour of the unconventional combined foundation presented herein showed that the available methodologies to capture soil-structure interaction effects are capable in predicting the behaviour of real-life situations. Both in simplified hand-calculations and in elaborate numerical methods using advanced constitutive models, the value of the stress-level dependent soil stiffness is the governing parameter in realistically assessing the deformation behaviour of this type of foundation. For complex situations or when information on the soil behaviour is limited, the observation of settlements is an indispensable part of the safety concept.

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