Urban Development and Environment – Foundations, Retaining Walls and Associated Structures

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Foundation, deep excavation, and dewatering scheme for a 250 m tall high-rise building in Vienna

Fondation, fouille profonde et schéma de rabattement d'un terrain - immeuble de grande hauteur (250 m) - à Vienne

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ABSTRACT The construction of the Donau City Tower 1 has been recently accomplished; it is the first of two high-rise buildings in the so called „Donau City" in the north of Vienna. The DC Tower 1 is one of the tallest buildings in Europe and comprises a height of 250 m above ground, 20 m of underground floors, and a 35 m deep foundation. The execution of the deep excavation and the deep foundation of the Donau City Tower 1 made great demands on ground engineering. The geotechnical relevant works contained the construction of the pit supporting system for the excavation pit and deep foundation works for the tower consisting of numerous diaphragm wall elements with depths up to 30 m, and CFA piles for the foundation of shallow building parts. A special challenge was the design and execution of the dewatering scheme to remove the water in the quaternary soil and for lowering the water pressure in the tertiary soil layers.

RÉSUMÉ La construction de la "Donau-ville" tour 1 a été accompli récemment, comme le premier des deux grands immeubles à la dite «ville Donau», au Nord de la ville de Vienne. La tour «DC 1» en effet peut être considérée comme un des plus hauts immeubles en Europe, avec une hauteur de service de 250 m, 20 m de profondeur des sous-sols, et 35 m de profondeur des fondations profondes. L'excavation de la fouille et l'exécution des fondations profondes de la Tour DC 1 demandaient un haut niveau d'expertise de toutes sortes de procédés d'exécution géotechniques concernés. Parmi les procédés d'exécution les plus délicats il y avait certainement d'abord l'excavation et parois calés, mais en plus la construction de la fondation profonde en éléments multiples de parois moulés jusqu'à 30 m de profondeur, en combinaison avec des pieux CFA pour les parties basses de la tour. Le défi spécial était certainement la conception et installation de toute la partie du rabattement d'eau dans le sol des couches quaternaires, et de l'abaissement des pressions d'eau dans les couches tertiaires.

1 INTRODUCTION

The DC Towers designed by Dominique Perrault will leave a distinctive mark on Vienna's skyline. Standing 250 metres tall (including transmission mast), DC Tower 1 is not only Austria's and one of Europe's highest building but also a fascinating new urban landmark in the north of Vienna. The building volume comprises about 330,000 m³ and the total floor area is about 144,000 m³. About 110,000 m³ of concrete, thereof about 25,000 m³ of high-performance concrete C 50/60 and C 70/85, which was conveyed with concrete pumps, and about 22,000 tons of steel were consumed. The construction works were finished in autumn 2013, the building was opened shortly after. The high-rise building was constructed with a crane-independent self-climbing formwork for the building core and aluminium frame formworks for the ceilings sheltered by a 4-story high "wind shield". The extreme slenderness of the building at its narrow side with 1:11 respectively 20 m : 220 m is remarkable. The reinforcement content in the reinforced concrete elements up to 800 kg per m³ concrete is extraordinary high, in particular in the columns and walls of the building core. The huge masses of high-performance concrete was conveyed by pumps up to 150 m of height, and the thickness of the structural elements, e.g. the slab is about 4 m thick, the walls of the building core are 1 m thick, and the cross-sectional area of the columns is 1.20 m x 1.20 m.

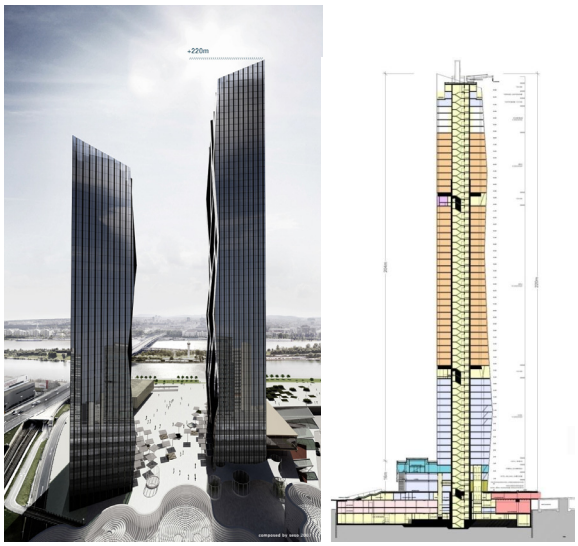

Figure 1. A new urban landmark in the north of Vienna: DC Tower 1 (250 m) to the right and DC Tower 2 (168 m).

2 GROUND CONDITIONS

The situation of the building site close to the Danube River, the complex ground conditions found during construction, the strongly varying stratification of the layers of the anthropogenic ground in the area of the former back water called "Kaiserwasser", and the irregularly layered tertiary soil strata generated complex geotechnical constraints.

Beneath anthropogenic fill (very) loose to medium dense quaternary alluvial sandy gravel was found up to about 10.0 to 11.9 m. The upper layer of the underlying tertiary sediments was formed by low-permeable clayey silt and silty clay comprising a thickness of about 0.3 to 4.7 m, which appeared to be an aquifuge. Underneath alternating sequences of silty fine sands, silty clays and clayey silts, and sporadic layers of sandy gravel were explored. Significant variations of the soil layer sequence between the western and the eastern area of the site had a distinctive impact on the construction works, in particular on lowering the water pressure in the tertiary sediments. While in the west directly beneath the aquifuge an advantageous thick permeable layer of sand was found, in the east a significant alteration of low-permeable clayey silts and permeable sands with small layer thickness made the dewatering in the tertiary aquifer more difficult (Figure 2). The groundwater level was explored in the quaternary alluvial sediments (aquifer) in a depth of about 2.6 m. Confined groundwater appears in the permeable sandy layers of the tertiary soil, the hydrostatic pressure corresponds approximately with the free groundwater water level of the quaternary aquifer.

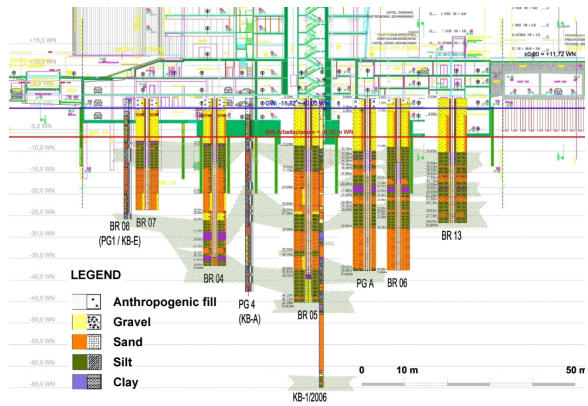

Figure 2. Geological longitudinal section (not to scale) and profiles of additional exploration borings. Shaded areas represent low-permeable layers in the tertiary ground (aquifuges).

3 EXCAVATION PIT

The pit wall served as a low-deformable temporary retaining structure against water and earth pressure from the quaternary soil layers and the adjacent motorway tunnels, and to resist uplift forces of the shallow building sections in the final state.

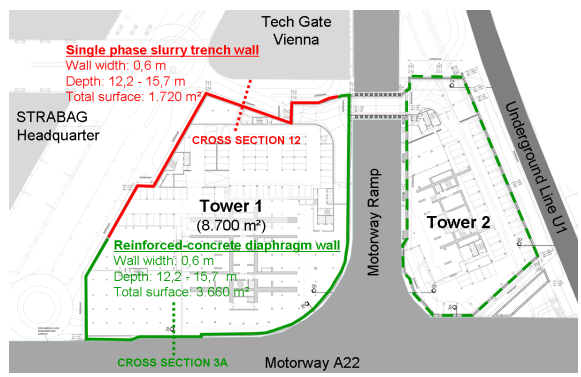

Figure 3. Excavation pit with location of the reinforced-concrete diaphragm wall and single phase slurry trench wall.

The retaining structure for an excavation depth from 6 up to about 9.5 m beneath ground surface was realized with an anchored reinforced diaphragm wall and for an excavation depth up to 6 m with a single-phase slurry trench wall with steel girders comprising a thickness of 60 cm, which enclosed the pit with a total area of about 8,700 m².

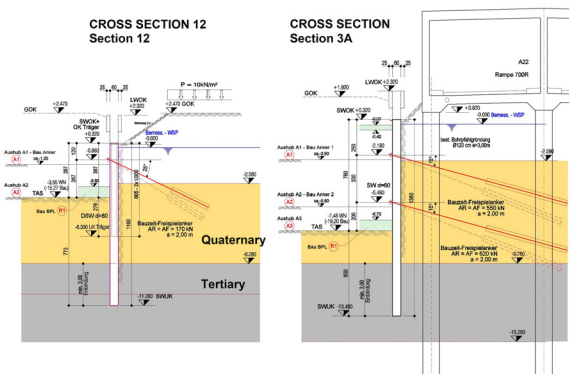

Figure 4. Excavation pit: typical cross sections of supporting system, single-phase slurry trench wall with inserted steel girder (left) and reinforced-concrete diaphragm wall (right).

4 DEEP FOUNDATION

Deep foundation for the high-rise building was separated from deep foundation of shallow building sections since the high loads from the tower have to be transferred deep into the ground in order to meet the requirements for maximum allowable settlements while the foundation of the shallow building sections together with the pit walls primarily serve to resist uplift forces in the final state. For the tower 171 reinforced-concrete diaphragm wall panels with depths up to 30 m and continuous flight auger piles (CFA piles) for the foundation of the shallow parts of the building were installed.

The installation of the deep foundation elements was accomplished from an excavation level close to the bottom line of the base slab about 7 m beneath the quaternary groundwater level, which corresponds to the hydraulic head of the confined tertiary groundwater. The confined groundwater conditions in combination with the alternating layering of fine grained and coarse grained soils was an extraordinary challenge for the installation of the diaphragm wall elements. A breakdown of the tertiary dewatering scheme would have caused fatal implications like washing in fines into the open trenches, hydraulic failure by heave and/or by uplift (buoyancy) of the low permeability ground layer beneath foundation base of the slab and endangering nearby structures.

4.1 Stability and settlement analyses

Taking into account earlier experiences with deep foundations of high-rise buildings in Viennese ground conditions the foundation concept was based on single reinforced-concrete diaphragm wall elements forming a “box foundation” consisting of consecutive cells, whereby the wall panels were arranged with a distance of not less than 4 m and not more than 7 m to each other. All elements had to be connected to each other at their heads via the flexural resistant base slab, consequently, the trapped soil within the foundation box transfers loads as well and not only the diaphragm wall panels, thus, following the principle of “pile raft foundations”. Cells of the box foundation were arranged in a way that the wall panels are loaded likewise. The single cells are closed at their edges. Those box foundations behave approximately monolithic and can be considered as “deeply embedded raft foundations” transferring loads not only via the raft (base pressure) but also along the vertical wall panels (skin friction).

Stability analysis

Following stability analyses were performed:

- Check of overall stability (ground failure) for the entire box foundation idealized as monolith considered as deeply embedded raft foundation (global safety factor $\eta = 3.5$).
- Check of load transfer via single panels for each element and both wall sides (increased allowable skin friction $\tau_{all,s} = 85$ kN/m², base pressure $\sigma_{all,s} = 700$ kN/m²).
- Comparable calculations were performed for the idealized monolithic box defining overall skin friction and base pressure (allowable skin friction $\tau_{all,b} = 60$ kN/m² along the circumferential vertical box surface and overall base pressure $\sigma_{all,b} = 700$ kN/m²).

Settlement analysis

Due to the location of the DC Tower 1 near settlement sensitive structures like the motorway A22, the

underground line U1 and the striking distance of the DC Tower 2 three-dimensional numerical settlement calculations were be performed. The 3D FE analysis was based on the Harding Soil Small constitutive model (HSS) to estimate the settlement distribution taking into account a geological pre-overburden-pressure (POP) of 600 kN/m² and the adjacent DC Tower 2 to be built later. At the level of the base slab the average pressure of the settlement relevant loads was calculated to 710 kN/m² and the maximum edge pressure to 967 kN/m² due to wind loads (Tschuchnigg & Schweiger, 2010, 2011, 2013).

Settlement calculations revealed that the two high-rise buildings influence each other. Consequently, the lengths of diaphragm wall panels for DC Tower 1 were adapted in order to achieve uniform settlements. The lengths were staggered from 20 m, 25 m, 30 m to 25 m according to Figure 5.

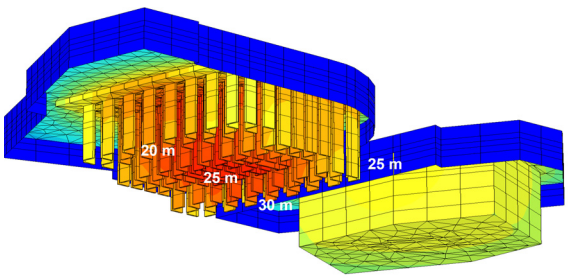

Figure 5. Settlement calculations; modelling of deep foundations for DC Tower 1 (detailed modelling of all foundation elements) and for DC Tower 2 (rough modelling as deep foundation block) (Tschuchnigg & Schweiger, 2010, 2011, 2013).

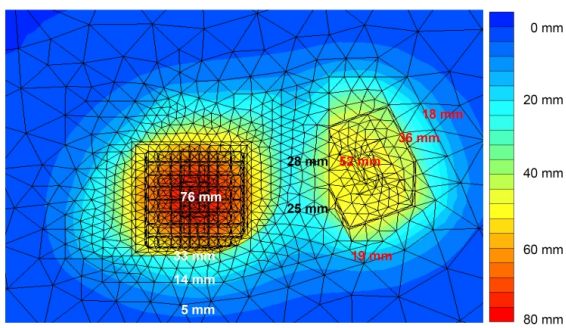

Figure 6. Result of settlement calculations for DC Tower 1 and DC Tower 2 (Tschuchnigg & Schweiger, 2010, 2011, 2013).

The maximum settlement after completion of DC Tower 1 was calculated to 76 mm in the centre of the tower area. Maximum settlements of less than 40 mm were expected for the motorway A22 and about 20 mm for the motorway ramp after completion of DC Tower 1 and about 25 to 50 mm after completion of DC Tower 2.

Settlement measurements after completion of the building shell for DC Tower 1 showed that the actual settlements were smaller than the calculated settlements. Maximum settlements in the tower area were determined to 41 mm, see Figure 7. However, it has to be taken into account that not all permanent loads (façade, interior, etc.) have been installed at this time. In addition, time dependent settlements (consolidation, creep) will contribute too so that maximum settlements of about 55 to 60 mm have to be expected.

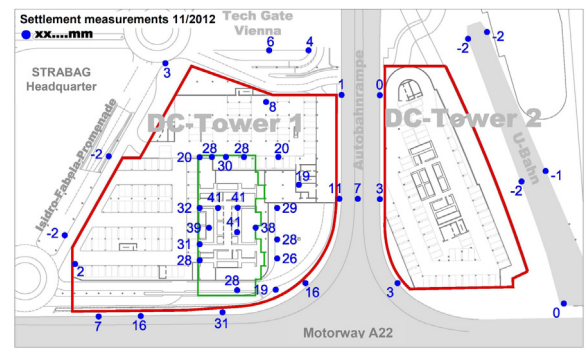

Figure 7. Settlement measurements after completion of building shell for DC Tower 1 in November 2012.

4.2 Diaphragm wall elements

The central focus of the foundation works was on the deep foundation of the high-rise building. The transfer of the loads of the 220 m tower is accomplished by a 4 m thick base slab resting on reinforced-concrete diaphragm wall elements with a cross section of 3.60 m x 0.60 m up to a maximum foundation depth of about 30 m beneath the base slab. 171 elements comprising a total surface of 16,500 m² were installed.

4.3 Continuous flight auger piles (CFA piles)

Shallow building parts rest on deep foundation panels. Alternatively to encased bored piles defined in the tender continuous flight auger piles (CFA piles) were installed.

5 DEWATERING SCHEME

Before excavation and in addition to the pit wall construction the quaternary water within the enclosed excavation pit had to be removed from the ground by a system of bored wells. Moreover, the water pressure with hydraulic heads up to the quaternary groundwater level had to be lowered in the confined tertiary aquifer likewise by a system of wells in order to prevent hydraulic failure by heave and/or by uplift (buoyancy) of the low permeability ground layer.

According to Eurocode 7 (ÖNORM EN 1997-1 (2006)) the stability of a structure or of a low permeability ground layer against uplift shall be checked by comparing the permanent stabilising actions to the permanent and variable destabilising actions from water and, possibly, other sources. Verification for uplift (UPL) shall be carried out by checking that the design value of a combination of destabilising permanent and variable actions ($V_{dst;d}$) is less than or equal to the sum of the design value of the stabilising permanent vertical actions ($G_{stb;d}$) and of the design value of any additional resistance to uplift (R_d) fulfilling the inequality:

$$V_{dst;d} \leq G_{stb;d} + R_d \tag{1}$$

5.1 *Quaternary dewatering scheme*

The quaternary dewatering scheme served for the removal of water from the quaternary ground within the enclosed excavation pit. Due to the size of the pit, the ground water level, the extent of dewatering (drawdown elevation), and the porosity of the soil a total amount of 23,000 m³ was removed with 4 bored wells with a total pumping rate of 16 l/sec.

5.2 *Tertiary dewatering scheme*

The layout of the tertiary dewatering scheme was based on the results of the ground exploration. Taking into account the alternating layers of fine sands and silts it was assumed that the permeable layers (generally sands) communicate, thus, forming a common aquifer. In the following the main data of the tertiary dewatering scheme to lower the confined water pressure consisting of 15 bored filter pipe gravel/sand wells (filter pipe diameter 6 inches) and 8 piezometers is presented:

- Water permeability of aquiferous layers: $k_f = 1 \cdot 10^{-4}$ bis $5 \cdot 10^{-5}$ m/s
- Well depth: depending on local ground between T = -20 m WN and T = -35 m WN

For verification of the assumed data and the hydraulic model and for checking the layout of the tertiary dewatering scheme a pumping test was carried out resulting in an average water permeability of $k_f = 5.2 \cdot 10^{-5}$ m/s, thus, confirming the assumptions with sufficient accuracy.

5.3 *Installation of wells and piezometers*

The quaternary and tertiary wells were bored from the original ground level by percussion core drillings forced down by hydraulic hammer blows.

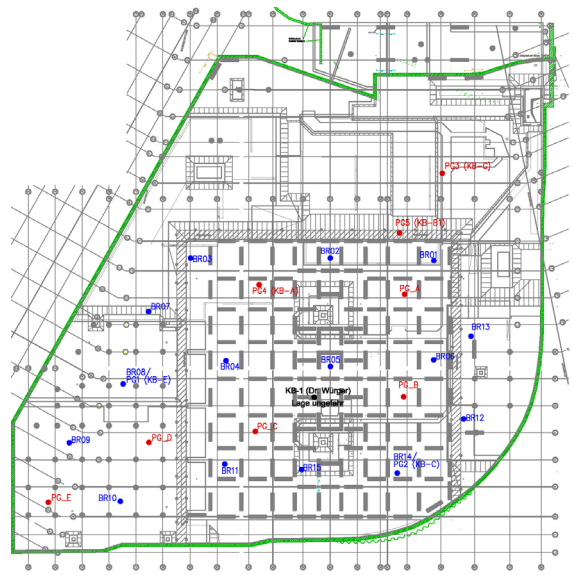

Figure 8. Situation of the CFA piles (circular, grey) and diaphragm wall panels in the tower area (rectangular, grey); wells (blue) and piezometers (red) for the tertiary dewatering scheme.

The quaternary wells comprised a drilling diameter of 324 mm and were bored to the depth of the tertiary aquifuge. The filter pipe diameter was 150 mm (6 inches) and the slot size of the filter pipes was 1 mm. The tertiary wells were telescoped with a final diameter of 273 mm due to the well depths between -20 m and -36 m WN. The tertiary wells were designed as the quaternary wells. The filter sand was

customised to the respective soil of the permeable layers, the grain size varied from 1 to 2 mm.

The piezometers were equipped in the same way as the wells in order to use them as wells as needed. This measure was part of the safety concept.

5.4 Start-up of tertiary dewatering scheme

Lowering the water pressure was started already after completion of the first series of wells; more wells were put into operation successively. Finally, in the week from 7 to 14 September 2010 all 15 wells were operated at once. Figure 9 shows that the water level in the piezometers dropped in a range from -7.0 m WN (PG4) to -9.3 m WN (PG_C). At that time the total pumping rate was about 45 l/sec.

Although this start-up phase confirmed the functioning of the tertiary dewatering scheme it was revealed that some individual wells did not yield the intended flow rate. The alternating layered ground with only very thin water permeable soil layers (see Figure 2) made a higher flow rate impossible. Moreover, a test with 11 deactivated wells showed a phreatic rise of the tertiary groundwater of about 4 m within 20 minutes only (!). The cause of this unexpected rise was a highly permeable tertiary gravel layer observed over a wide area in a depth of about 25 m WN (see Figure 2). By means of the findings following additional measures were carried out:

- Retrofitting of some individual piezometers to discharging wells. Thus, the total pumping rate could be increased significantly.
- Installation of alternative piezometers in order to replace those piezometers, which were used as discharging wells and to enable a more specific monitoring in sensitive geological areas.
- Installation of two additional wells in sensitive geological areas.
- In the centre 2 additional wells were bored to a depth of each 32 m in order to accomplish a deeper lowering of the tertiary groundwater and, thus, to extend the contingency reserve (to comply with the safety concept).

5.5 Operation of tertiary dewatering scheme

From 21 September 2010 some 4 additional wells were put into operation (see Figure 9), thus, dropping the drawdown elevation. However, Figure 9 shows that the water level in the piezometers dropped already some days before; this was caused by the well development of new wells and, thereby, by operating wells ahead. During installation of deep foundation elements the total pumping rate was about 65 l/sec in average. The hydraulic pressure adjacent to the concurrently installed diaphragm wall element dropped beneath the required drawdown elevation (-7.0 m WN) to level -8.5 m WN. In order to prevent bentonite slurry seepage output from the open trench due to the artificially produced groundwater flow during the installation of the diaphragm wall all wells in the periphery of about 3 m were temporarily deactivated and used as piezometers.

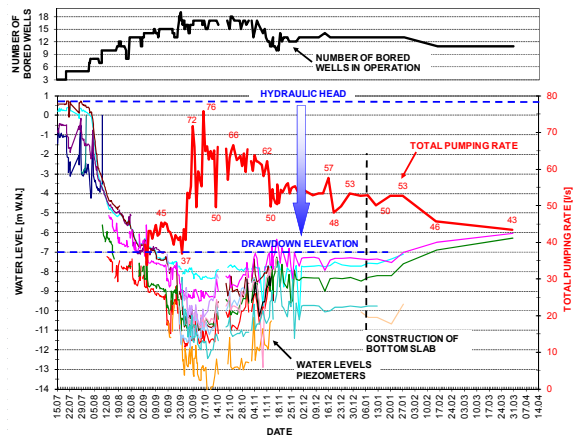

Figure 9. Monitoring of dewatering; water levels in piezometers, total pumping rate and number of bored wells in operation for lowering the water pressure in the confined tertiary aquifers with a hydraulic head up to the quaternary groundwater level.

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