DONAU CITY TOWER 1 – DEEP FOUNDATION, EXCAVATION AND DEWATERING SCHEME FOR THE 220 M TALL HIGH-RISE BUILDING IN VIENNA

Dietmar Adam [1], Roman Markiewicz [2], Josef-Dieter Deix [3]

ABSTRACT:
The Donau City Tower 1 is currently under construction and the first of two intended high-rise buildings in the so called „Donau City“ in the north of Vienna. The DC Tower 1 is the tallest building of the duo and comprises a height of 220 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 171 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.

1. Introduction

The DC Towers designed by Dominique Perrault will leave a distinctive mark on Vienna's skyline. Standing 220 metres tall, DC Tower 1 will not only be 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 is conveyed with concrete pumps, and about 22,000 tons of steel will be consumed. Up to date the building is almost finished, most of total 60 floors have been built. The high-rise building is 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 construction for a regular floor takes about 5 days. 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 is 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.

[1] Univ.Prof. Dipl.-Ing. Dr.techn. Dietmar Adam, Professor at Civil Engineering Department of Vienna University of Technology, Institute of Geotechnics – Centre of Ground Engineering, Soil and Rock Mechanics, Karlsplatz 13, A-1040 Vienna, Austria. Phone: 0043 1 58801 22100, Fax: 0043 1 58801 22199, E-mail: dietmar.adam@tuwien.ac.at
[2] Dipl.-Ing. Dr.techn. Roman Markiewicz, Geotechnik Adam ZT GmbH, Wiener Straße 66-72/15/4, A-2345 Brunn am Gebirge, Austria. Phone: 0043 2236 312244 22, Fax: 0043 2236 312244 99, E-mail: markiewicz@geotechnik-adam.at
[3] Dipl.-Ing. Josef-Dieter Deix, Porr Technobau und Umwelt AG, Abteilung Grundbau, Absberggasse 47, A-1100 Vienna, Austria. Phone: 0043 50 626 1271, Fax: 0043 50 626 1548, E-mail: josef-dieter.deix@porr.at

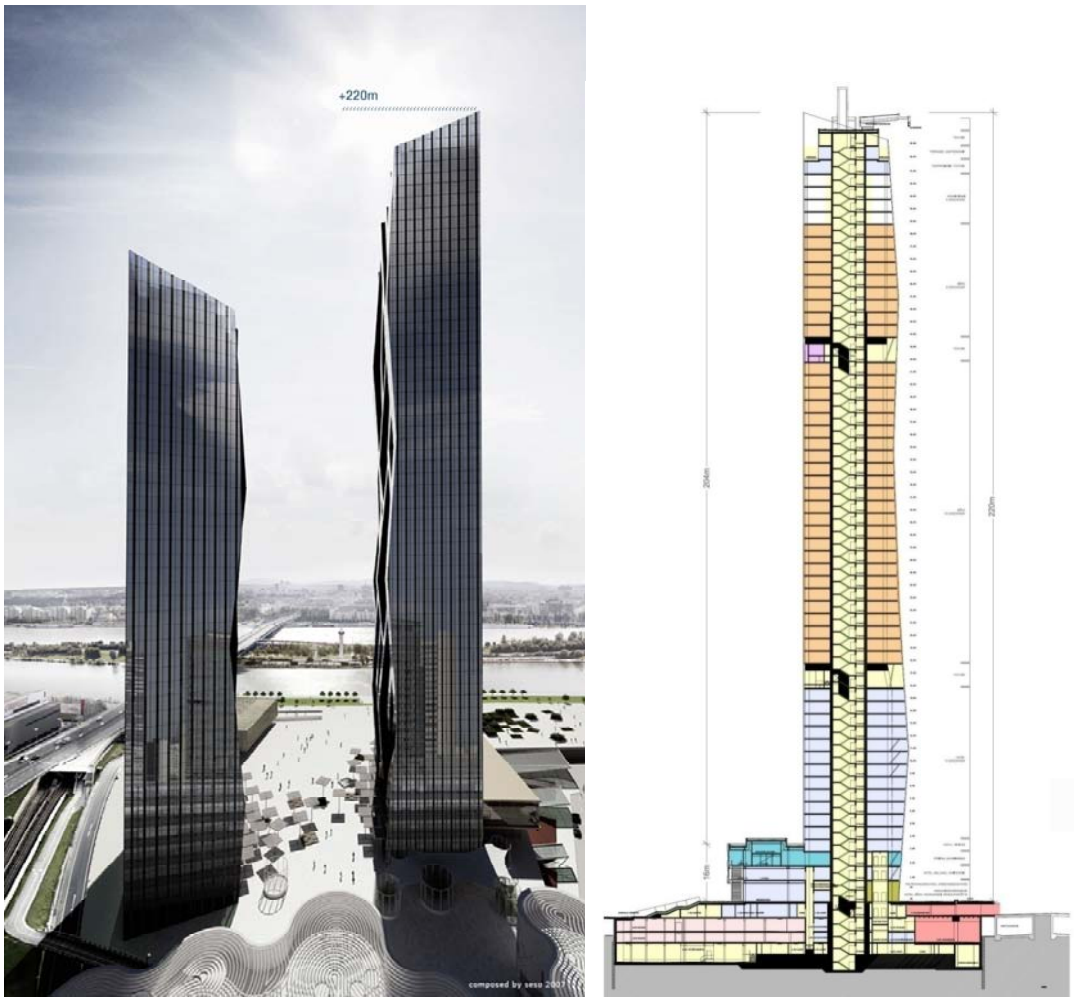


Figure 1. A new urban landmark in the north of Vienna: DC Tower 1 (220 m) to the right and DC Tower 2 (168 m) to the left. Typical cross section of DC Tower 1 including underground floors is shown on the right.

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 the complex geotechnical constraints for the execution of the deep foundation, the deep excavation, and the dewatering scheme.

The ground was explored by 6 core drillings in the design phase, more detailed important information could be generated by numerous drillings for the installation of a system of bored wells.

Beneath anthropogenic fill primarily consisting of loose to dense (sandy) gravel (very) loose to medium dense quaternary alluvial sandy gravel was found up to about 10.0 to 11.9 m beneath ground level. During construction the existence of fairly uniformly graded gravel with a very low content of sand and fine grains was revealed, in particular in the east of the construction site. 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 (see Fig. 2).

The groundwater level was explored in the quaternary alluvial sediments (aquifer) in a depth of about 2.6 m beneath ground level. 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.

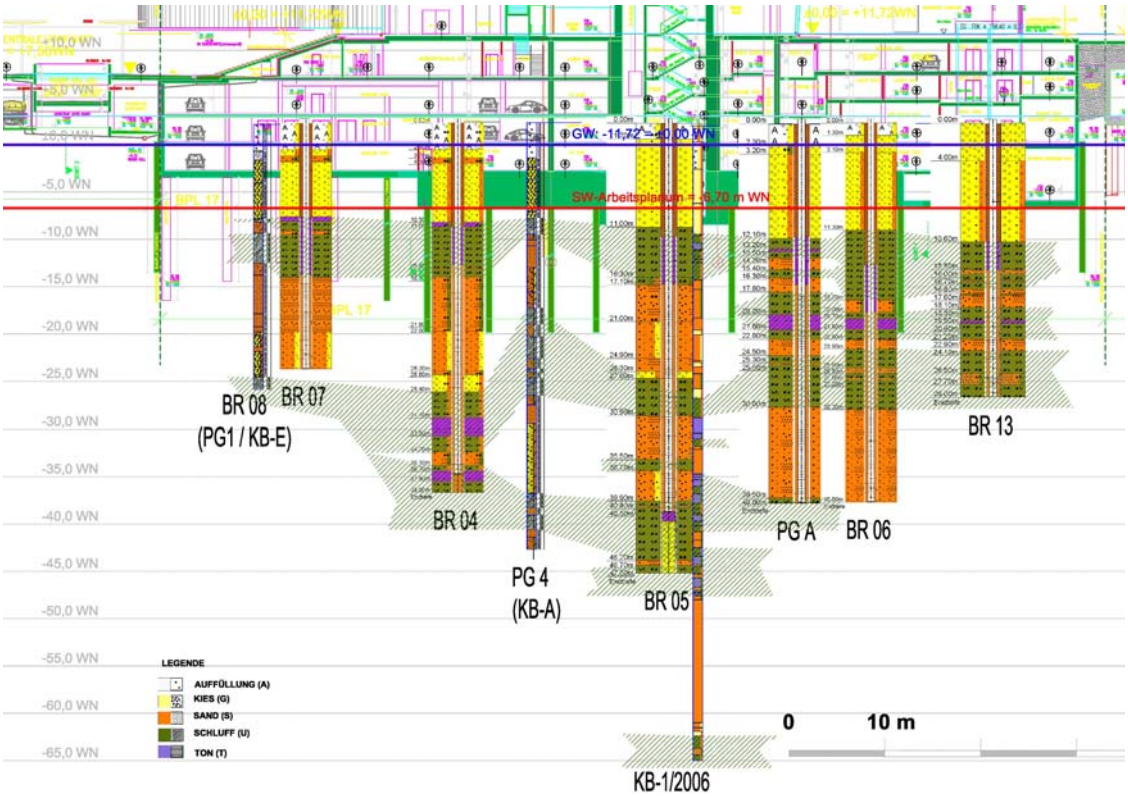


Figure 2: Underground floors and base slab; geological longitudinal section (not to scale) and profiles of additional exploration borings. Shaded areas represent low-permeable layers in the tertiary ground (aquifuges).

3. Excavation pit

The execution of an anchored reinforced-concrete diaphragm wall embedded in the upper aquifuge of the tertiary soil complex was defined in the tender documents for the pit wall. According to the building contract the design of the pit supporting system had to be accomplished by the construction company. On the one hand 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 on the other hand to resist uplift forces of the shallow building sections in the final state.

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. Since anchor heads had to be installed beneath the quaternary ground water level anchorage works were optimized in order to reduce time-consuming sealing measures to a minimum.

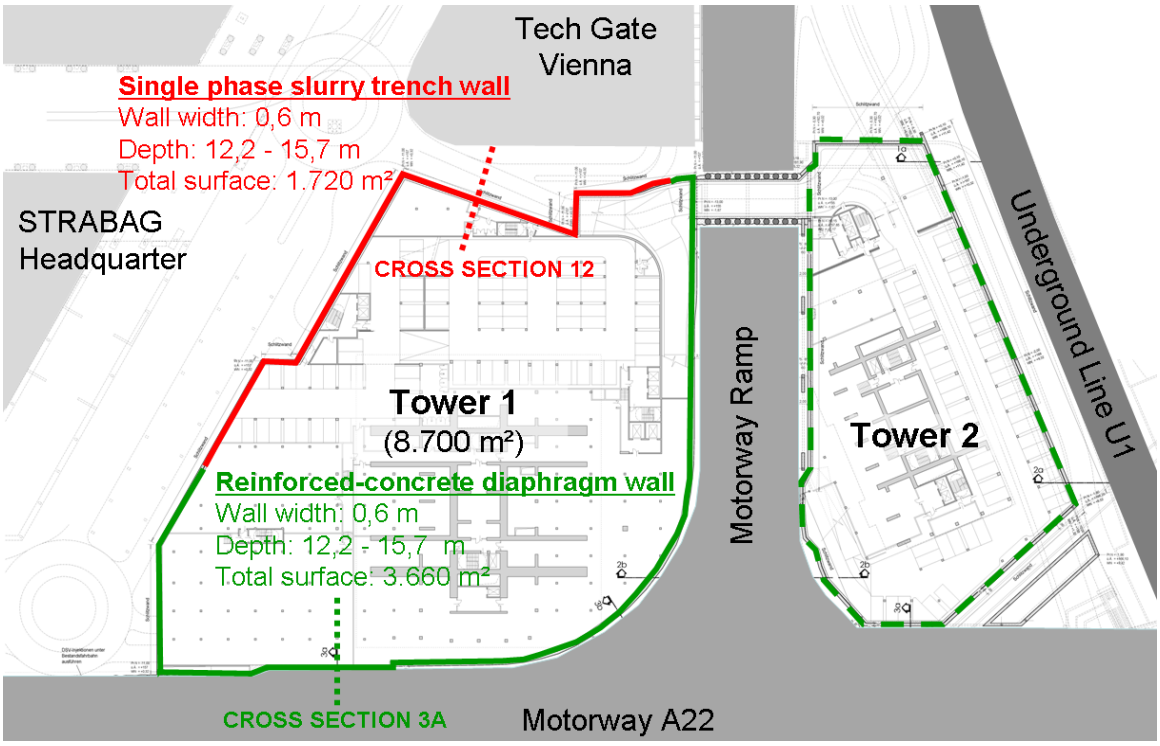


Figure 3: Loose porous gravels, practically no sand and fines, fairly uniformly graded material (left); formation of slopes in non-cohesive loose porous gravels (right).

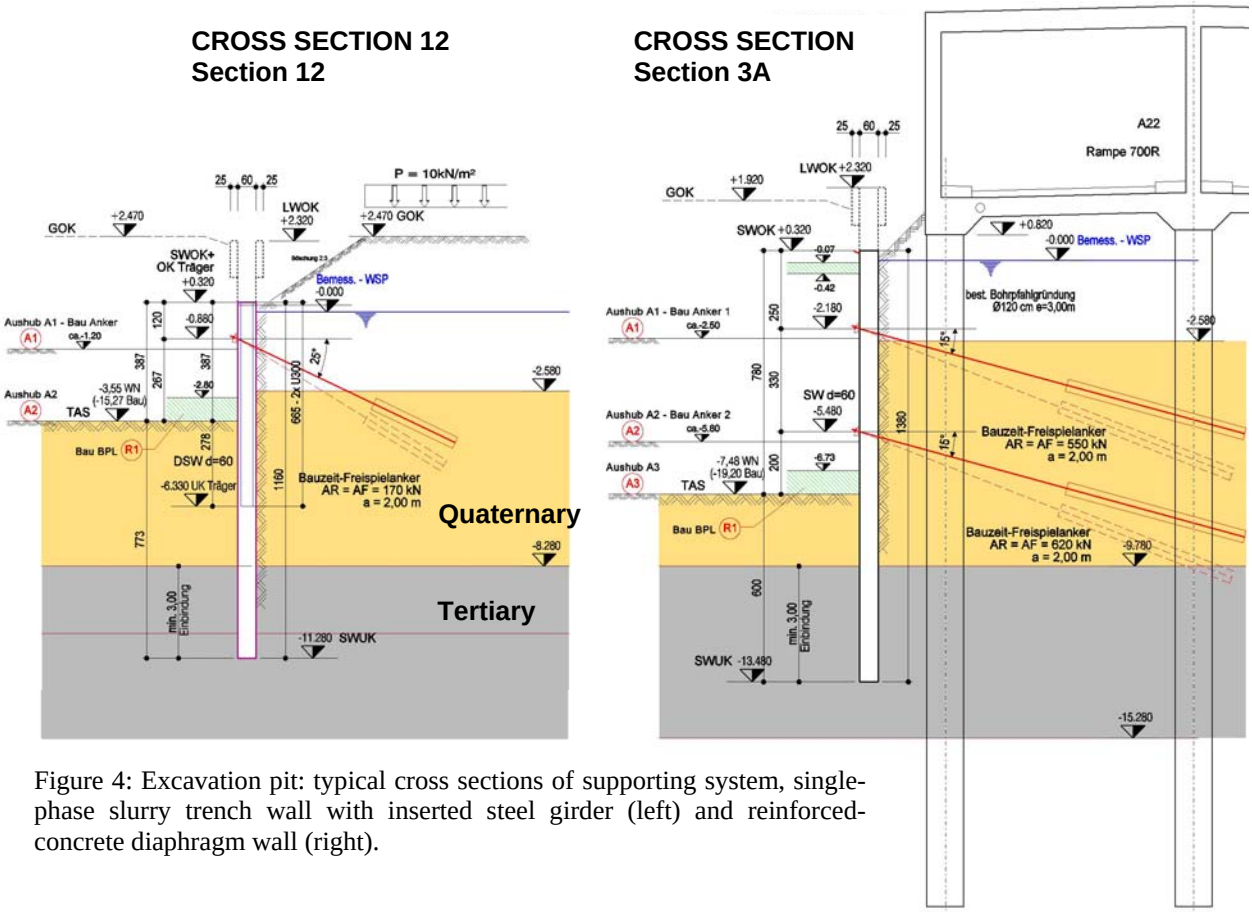


Figure 4: Excavation pit: typical cross sections of supporting system, single-phase slurry trench wall with inserted steel girder (left) and reinforced-concrete diaphragm wall (right).

During execution of the diaphragm walls loose porous gravels (practically no sand and fines, fairly uniformly graded) were encountered preventing the formation of a filter cake in the gravel along the open trench, thus, causing a significant loss of the slurry support effect and local failures of the gravelly trench walls. Since typical measures to stabilize the open trench, like increasing the rheological yield point of the slurry, adding of sand and bentonite pellets, etc. were not successful, low pressure grouting had to be deployed in order to prevent serious damages to sensitive structures (motorway tunnels) adjacent to the site. Thus, construction time was delayed for about three weeks. Subsequent determination of the grain size distribution on samples taken during excavation confirmed the existence of poorly graded gravel; in particular d_{10} was up to 10.5 mm, consequently, no sufficient support could be provided by the slurry in such soils.

The tight construction time required a sophisticated logistical concept for excavation and earthworks. Already during the installation of the pit walls a partial execution up to the quaternary ground water level was carried out. Immediately after completion of the supporting system the adjacent areas were excavated to the level of the anchor heads in order to enable the anchorage works without delays. Then the main focus of excavation was on the deep foundation area of the tower since the deep foundation elements had to be installed from the bottom level of the base slab. In total 45,000 m³ of soil was excavated, the average achievement per diem was 1,500 m³ including levelling the formation of the slab and removal of the excavation material by trucks.

The loose porous gravels (practically no sand and fines, fairly uniformly graded) required a re-grading of the slopes and re-contouring with blinding concrete or shotcrete.

Figure 5: Loose porous gravels, practically no sand and fines, fairly uniformly graded material (left); formation of slopes in non-cohesive loose porous gravels (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 (underground line, motorway, office buildings, etc.).

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”. All wall panels are designed with approximately the same length and 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. The open arrangement of the panels simplified the installation of the diaphragm wall elements and made possible the run-off of the filtrate water during installation, thus, providing benefits for the stability of the open trench. 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. Therefore a global safety factor of $\eta = 3.5$ was defined by the geotechnical expert.
- Check of load transfer via single panels of the foundation box for each element considering both wall sides. Therefore an increased allowable skin friction of $\tau_{all,s} = 85$ kN/m² and a base pressure of $\sigma_{all,s} = 700$ kN/m² for the single element were defined by the geotechnical expert.
- Comparable calculations were performed for the load transfer of the idealized monolithic box taking into account overall skin friction and base pressure. Therefore an allowable skin friction of $\tau_{all,b} = 60$ kN/m² along the circumferential vertical surface of the box and an overall base pressure of $\sigma_{all,b} = 700$ kN/m² for the box were defined by the geotechnical expert.

Settlement analysis

Due to the location of the DC Tower 1 near settlement sensitive structures like the motorway A22, the motorway ramp, the underground line U1 and the striking distance of the DC Tower 2 influencing each other three-dimensional numerical settlement calculations had to be performed. The 3D Finite Element analysis was based on the Harding Soil Small constitutive model (HSS) to describe the soil behaviour with sufficient accuracy in order to estimate the settlement distribution taking into account the explored ground profile, a geological pre-overburden-pressure (POP) of 600 kN/m² and the adjacent DC Tower 2 to be built at a later stage. At the level of the base slab the average pressure of the settlement

relevant loads incl. the weight of the base slab 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 taking into account the influence of DC Tower 2. The lengths were staggered from 20 m, 25 m, 30 m to 25 m according to Figure 6.

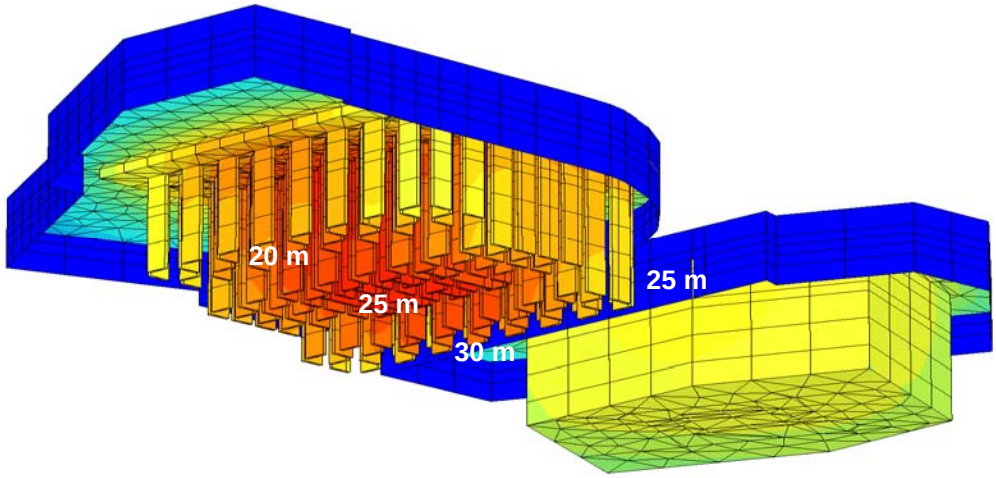


Figure 6: Settlement calculations; modelling of deep foundations for DC Tower 1 (detailed modelling of all foundation elements) and for DC Tower 2 (rough modelling as deep foundation block). Lengths of diaphragm wall panels for DC Tower 1 were staggered (20 m, 25 m, 30 m, and 25 m) in order to achieve uniform settlements taking into account the influence of 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. The underground line U2 will be influenced by construction of DC Tower 2 only in the range of about 10 to 20 mm.

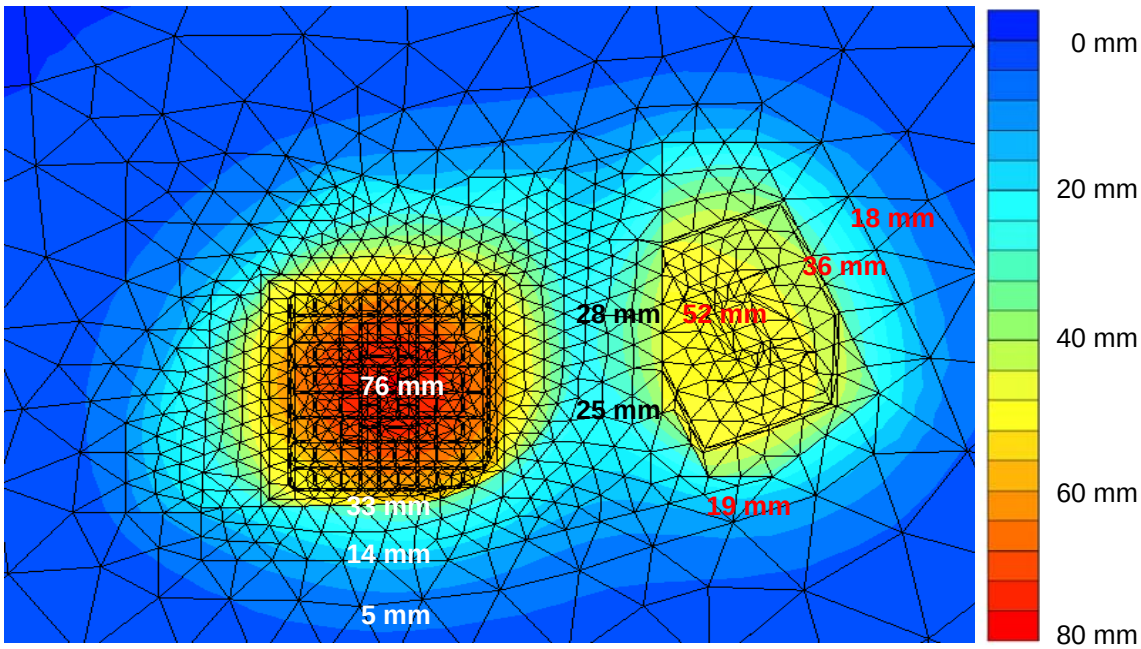


Figure 7: Result of settlement calculations for DC Tower 1 and DC Tower 2 after completion of both high-rise buildings influencing each other (Tschuchnigg & Schweiger, 2010, 2011, 2013).

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 Fig. 8. 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 as well so that maximum settlements of about 55 to 60 mm in the tower area have to be expected in the end.

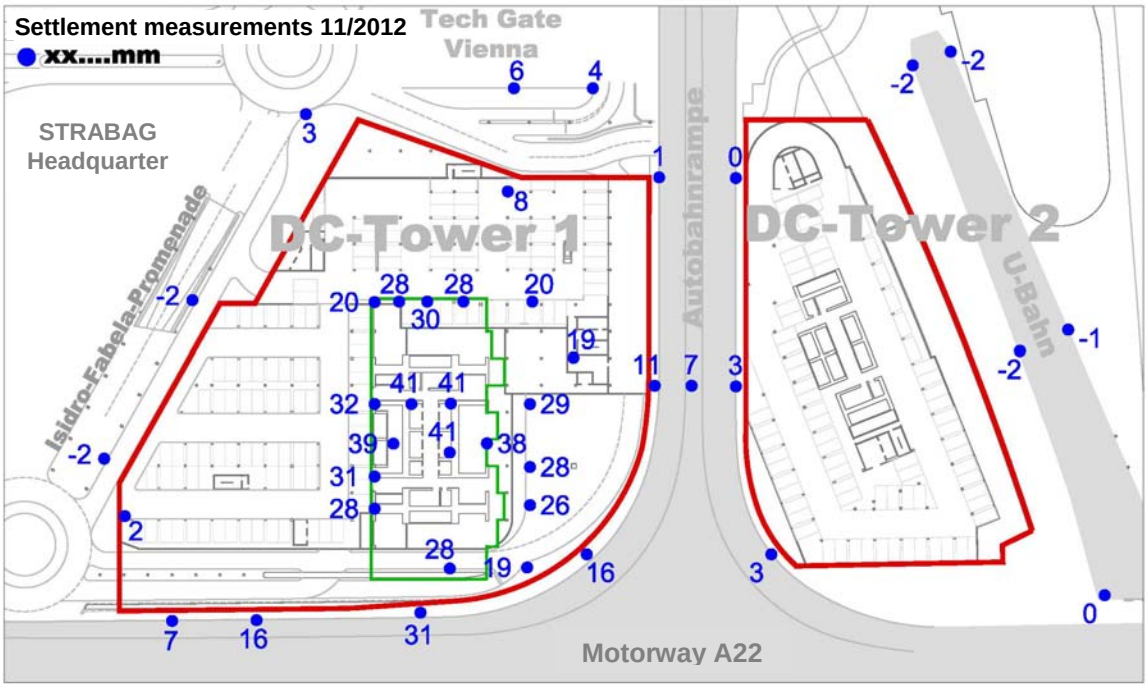


Figure 8: Settlement measurements after completion of building shell for DC Tower 1 in November 2012.

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. All in all 171 elements comprising a total surface of 16,500 m² were installed. In order to meet the requirements with respect to the tight construction time special measures had to be developed and executed.

Figure 9: Reinforcement cage for diaphragm wall element to be coupled by screwed joints (left); detail of screwed joint (right).

Coupling of reinforcement bars with screwed joints

Due to the fact that practically no tolerance for the installation of the 1.5 m long starter bars between the reinforcement cage of the diaphragm walls and the reinforcement of the base slab existed a novel connection system had to be developed. Instead of usual end-to-end main bars extending the head of the diaphragm wall panels the bars were equipped with screwed joints in recess units at the level of the element heads. Thus, the mobility of the rigs was not limited during the installation of the diaphragm wall elements, residual excavation could be performed without any hindrance as well as the cutting of the element heads, and the starter bars could be easily mounted manually after clearing the joints with high-pressure water jet.

Installation of guidance walls

The guidance wall consisted of two parts, firstly of a grillage of combined pre-cast and cast-in-place concrete walls comprising a height of about 0.6 m beneath the base slab level forming a rigid grid due to the spatial bracing, and secondly of a mobile pre-cast top frame segment in order to elevate the slurry level, thus, providing an increased excess slurry pressure to meet the required safety factor for internal stability against trench collapse. The shape of the top frame segment was adapted to the diaphragm wall grab and served to contain the overlay concrete. After completion of a diaphragm wall element the top frame segment was removed and located on the next guidance wall section. Due to the prompt removal the not stiffened concrete could be disposed easily. Time-consuming and costly cutting of concrete could be reduced to a minimum.

Figure 10: Grillage of guidance walls (left); pre-cast top frame segment (right).

Time saving measures

Pre-treatment of loose porous gravels with practically no sand and fines found during installation of the single-phase slurry trench and reinforced-concrete diaphragm walls caused delays in construction, which had to compensated by time saving measures focused on the deep foundation of the high-rise building since the construction of the base slab was on the critical path. For the construction of the diaphragm wall elements comprising a total area of 16,500 m² only 9 weeks were scheduled. In order to accomplish these works within this short period the maximum possible working time in two shift operation deploying two rig units was already envisaged in the construction schedule. However, the restricted space of about 70 m by 70 m in the area of the high-rise building did allow the deployment of one rig only. Thus, the intention was to extend construction works to a non-stop 24 hours 7 days a week operation. The request was granted by the authorities of the City of Vienna but strict obligations were imposed with respect to the operation time during weekends and the

transport from and to the site, which was prohibited in the night. Thus, the trench excavation was forced in the night and the excavated soil material was stored temporarily before it was removed during the daylight hours; likewise placing of concrete was carried out only during the day.

Figure 11: Installation of diaphragm wall elements.

Figure 12: Installed diaphragm wall elements and deep foundation works short before completion.

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. The decisive factors were the ground properties and the high groundwater level in connection with the assessment of the deep excavation level for installing the piles. By means of the confined tertiary groundwater with hydraulic heads almost up to the original ground level the required back pressure could not have been produced, thus, having caused hydraulic failure by heave within the encasing while boring. During the installation of continuous flight auger piles the

concrete pressure in the borehole, in the A-tube of the auger, and in addition the pump pressure resisted against the water pressure and the entry of soil particles. Subsequently, the 18 m long reinforcement cages were embedded into the fresh concrete by vibration.

Figure 13: Installation of continuous flight auger pile (CFA piles) from deep excavation level.

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.

The conditions for hydraulic failure of the ground can be expressed in terms of total stress and pore-water pressure or in terms of effective stresses and hydraulic gradient. For failure by heave, both total and effective stresses are applied. Total stress analysis is applied to failure by uplift. 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:

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

$V_{dst;d}$ … destabilising permanent and variable actions

$G_{stb;d}$ … stabilising permanent vertical actions

R_d … additional resistance to uplift

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 by pumping from 4 bored wells with a total pumping rate of 16 l/sec. After the water was pumped out the operation of the dewatering system was continued in order to discharge seepage water from leaking pit walls and the base of the excavation, and moreover from precipitation.

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 (filter pipe diameter 6 inches) is presented:

- Calculative thickness of aquifer: M = 18.0 m
- Water permeability of aquiferous layers: $k_f = 1 \cdot 10^{-4}$ bis $5 \cdot 10^{-5}$ m/s
- Fictitious radius of excavation pit: R = 49.5 m
- Drawdown elevation in the centre of excavation pit: s = 7.0 m
- Maximum groundwater flow to excavation pit: Q_{max} = about 50 l/s
- Well depth: depending on local ground between T = -20 m WN and T = -35 m WN

The discharge of the collected water was carried out through the public sewer system and with a recharge system at some distance from the excavation.

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. The evaluation of the test resulted 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 (Note: rotary core drillings were not deployed due to smearing the fines over the edge of the boreholes and, thus, reducing the permeability significantly along boreholes). Undisturbed samples taken by a core-catcher were available. The well and piezometer design could be customised exactly to the explored soil layers.

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 in exactly the same way 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.

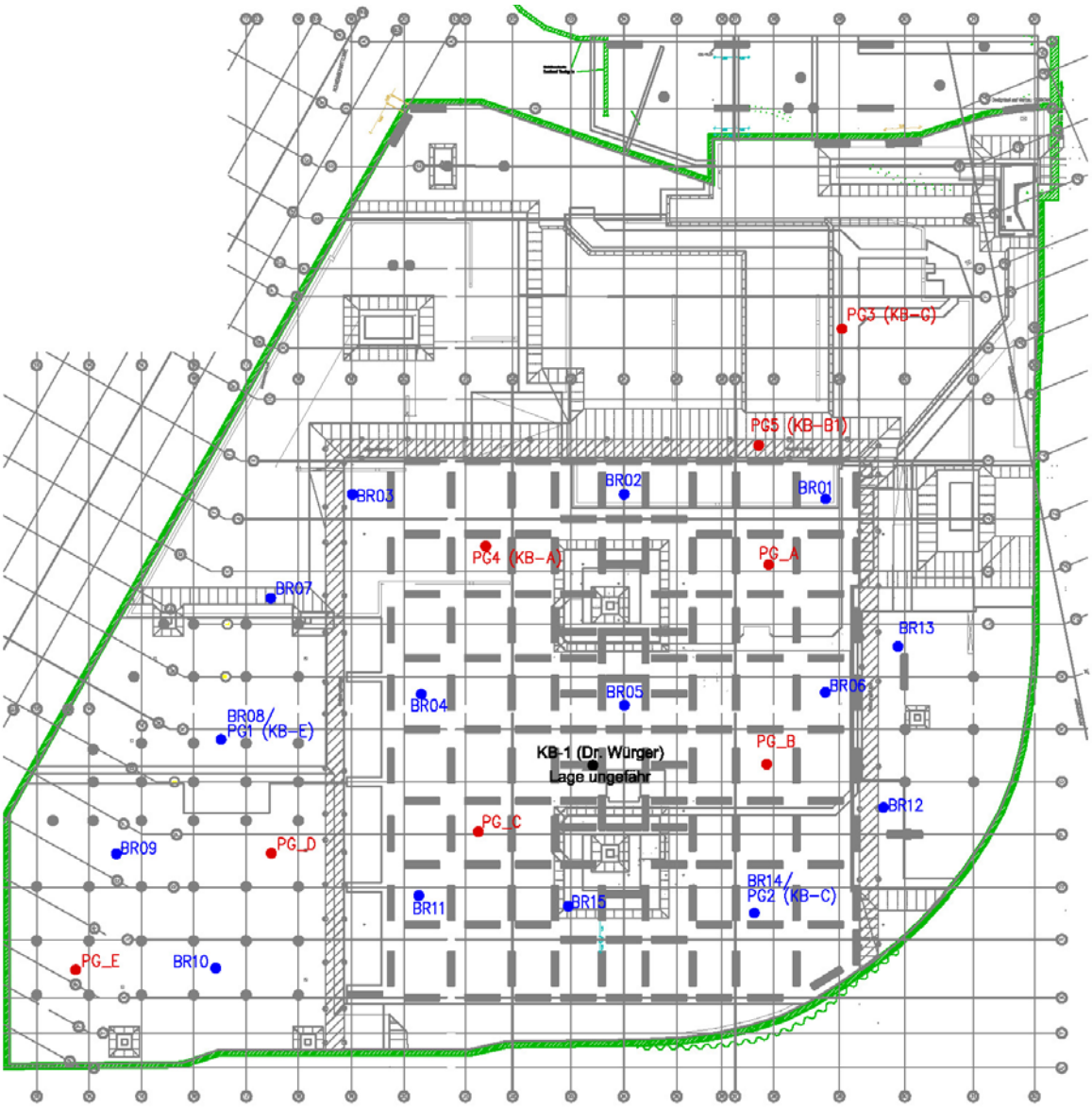


Figure 14: Situation of the CFA piles (circular, grey) and diaphragm wall panels in the tower area (rectangular, grey); wells (blue) and piezometers (red) for the tertiary dewatering scheme.

The design was separately done for each well and piezometer depending on the explored layering of the ground. 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. After installation of wells and piezometers the well development was accomplished bottom-up step-by-step by deploying a packer pump in order to remove the sand fraction as far as possible. Since wells and piezometers were installed from the original ground level they had to be lowered step-by-step depending on the respective excavation level.

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 15 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 (e.g. BR12, BR13) did not yield the flow rate as intended. It was found out that the alternating layered ground with only very thin water permeable soil layers (see Fig. 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 Fig. 2).

By means of the findings from the start-up phase 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 piezoemters, which were used as discharging wells and to enable a more specific monitoring in sensitive geological areas (i.e. with significant alternation of soil layering).
- Installation of two additional wells in sensitive geological areas (i.e. with significant alternation of soil layering).
- In the centre of the excavation pit 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

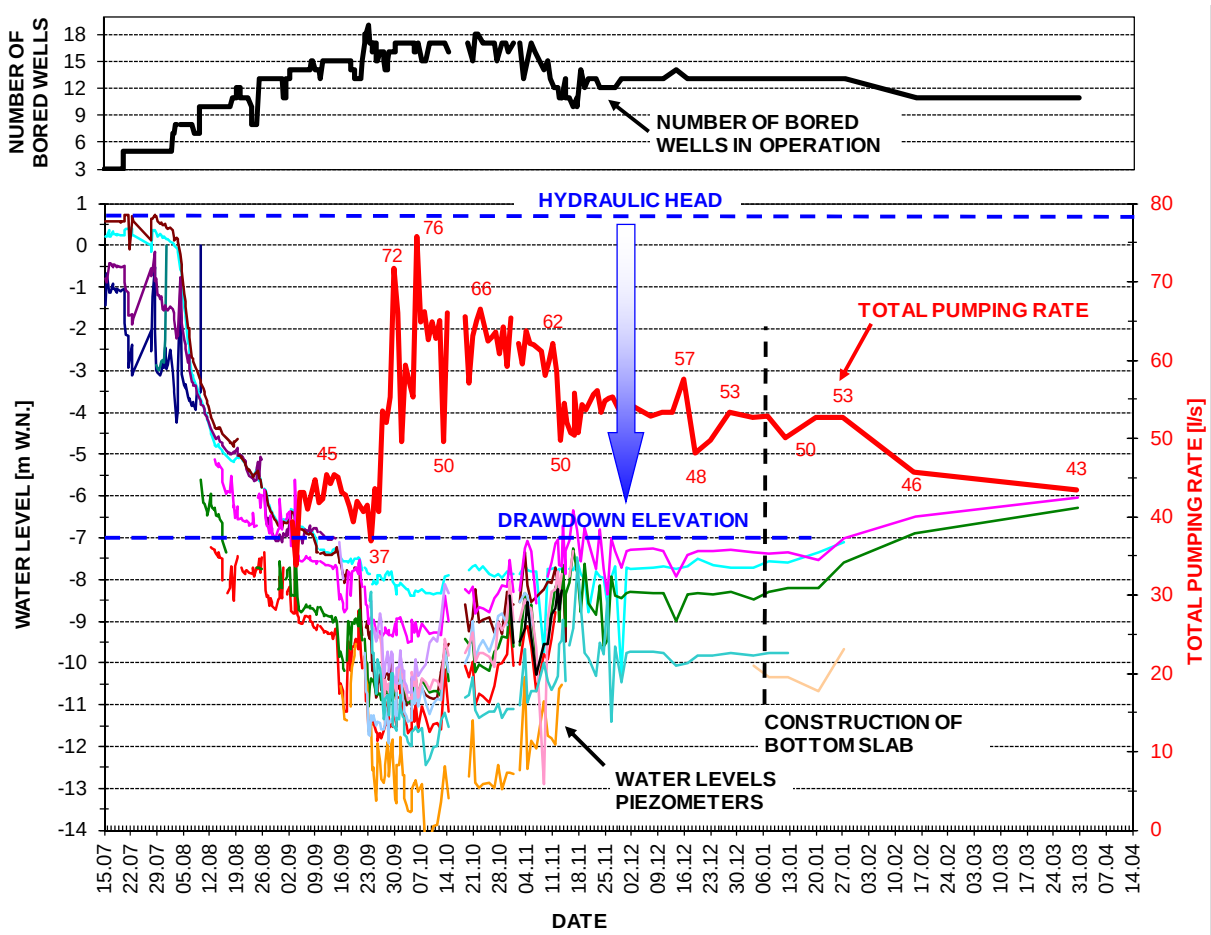


Figure 15: Monitoring of dewatering; water levels in piezometers, total pumping rate and number of bored wells in operation for lowering the water pressure in the confined tertiary aquifers with a hydraulic head up to the quaternary groundwater level.

From 21 September 2010 some 4 additional wells were put into operation (see Fig. 15), thus, dropping the drawdown elevation. However, Figure 15 shows that the water level in the piezeometers dropped already some days before; this was caused by the well development (removal of sand fraction) of the new wells and, thereby, by operating the wells ahead.

During installation of deep foundation elements (CFA piles and diaphragm wall 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 elements all wells in the periphery of about 3 m were temporarily deactivated and used as piezometers.

References

Adam, D. (Geotechnik Adam ZT GmbH) (2010). *Donau City Tower 1, A-1220 Wien, Konzept tertiäre Grundwasserentspannung.* Geotechnische Stellungnahme (in German).

Blaasch, G. (2012). *Bau des Donau City Towers 1 in Wien (Österreich).* www.baumaschine.de / Betontechnik- Schalung - BauPortal 10/2012 (in German).

Deix, J.-D., Adam, D., Markiewicz, R. (2011). *Die tiefe Baugrube des Donau City Towers – komplexer Spezialtiefbau unter schwierigen Randbedingungen.* Mitteilungshefte der Gruppe Geotechnik Graz, Heft 40, Beiträge zum 26. Christian Veder Kolloquium, Tiefe Baugruben unter schwierigen Randbedingungen, S. 247-262, 28. und 29. April 2011, Graz (in German).

Pelzl, M. (2007). Auszug aus der Fundierungsstatik für die Einreichplanung (in German).

Pelzl, M., Deix, J.-D. (2013). *Donau-City-Tower 1, Fundierung des höchsten Gebäudes in Wien.* Mitteilungshefte der Gruppe Geotechnik Graz, Heft 49, Beiträge zum 28. Christian Veder Kolloquium, Tiefgründungskonzepte, Vom Mikropfahl zum Großbohrpfahl, S. 163-180, 4. und 5. April 2013, Graz (in German).

Tschuchnigg, F., Schweiger, H.F. (2010). *Study of a complex deep foundation system using 3D Finite Element analysis.* Numerical Methods in Geotechnical Engineering (NUMGE 2010), Numerical Methods in Geotechnical Engineering – Benz & Nordal (eds), Taylor & Francis Group, London, ISBN 978-0-415-59239-0.

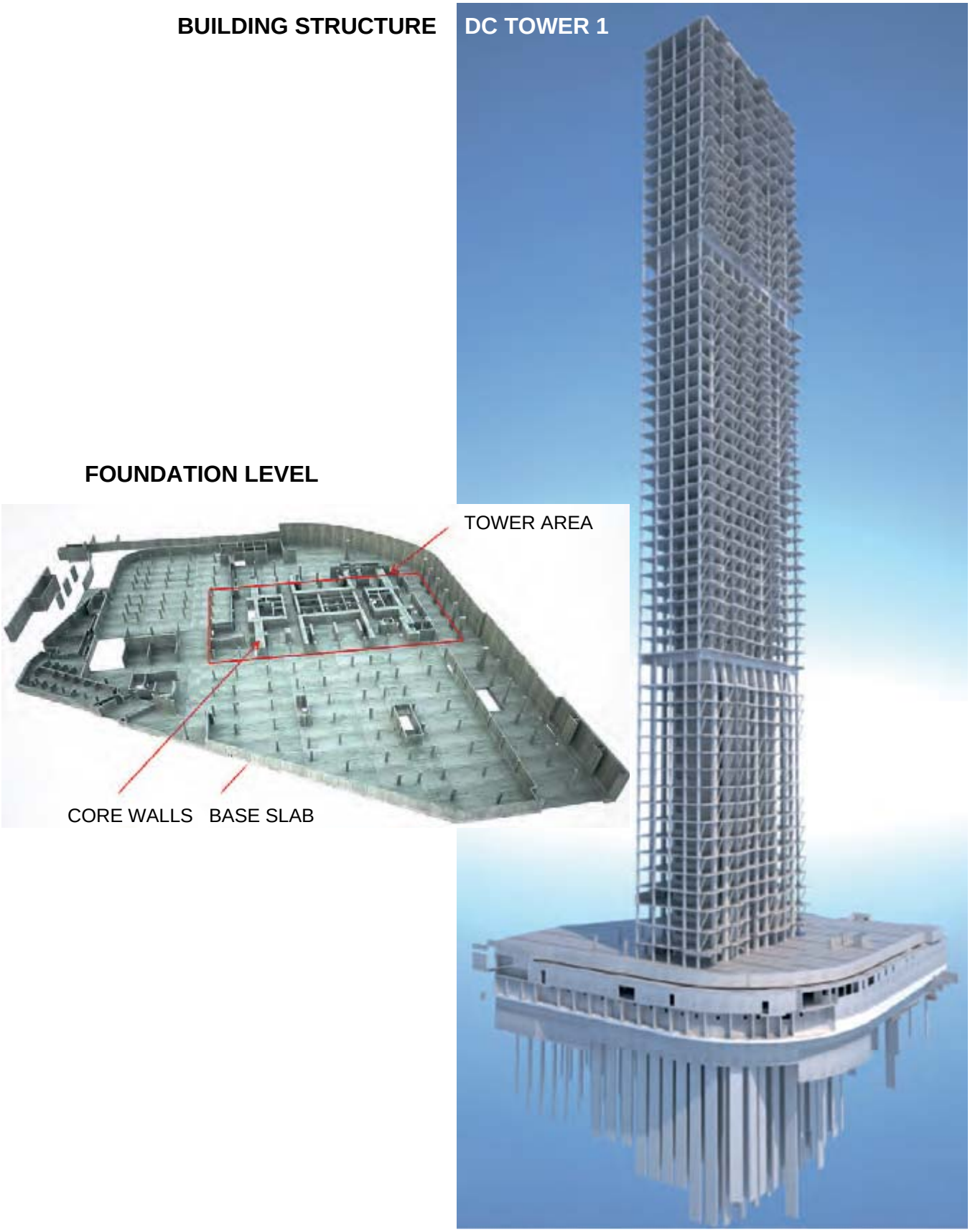
Tschuchnigg, F., Schweiger, H.F. (2011). *Comparison of deep foundation systems using 3D Finite Element analysis.* Proc. of IACMAG 2011 – Melbourne, Australia, 9–11 May 2011.

Tschuchnigg, F., Schweiger, H.F. (2013). *Setzungsprognose für den Donau-City-Tower mittels 3D FE-Analyse.* Mitteilungshefte der Gruppe Geotechnik Graz, Heft 49, Beiträge zum 28. Christian Veder Kolloquium, Tiefgründungskonzepte, Vom Mikropfahl zum Großbohrpfahl, S. 181-193, 4. und 5. April 2013, Graz (in German).

Weigl, G., Deix, J.-D., Adam, D. (2012). *Donau-City Tower – Ein neues Wahrzeichen für Wien.* Tagungsband Betontag 2012, 19. – 20. April 2012, S. 77-81, Österreichische Bautechnik Vereinigung, Austria Center Vienna, Wien (in German).

Würger, E. (2007). *Geotechnisches Gutachten über die Untergrundverhältnisse und die Gründungsmöglichkeiten für den Neubau der Tower 1 und 2 der Donau City Wien, in 1220 Wien, Katastral Gemeinde Kaisermühlen, Bauplatz 10* (in German).

ÖNORM EN 1997-1 (2006). *Eurocode 7: Geotechnical design, Part 1: General rules,* Österreichisches Normungsinstitut, 1020 Wien (in English).



DC Tower 1; structure of the high-rise building: deep foundation up to 30 m made of diaphragm wall panels and CFA piles, 4 m thick base slab, building core, top-rated-supports, beams, ceilings, and 2 m thick outrigger reinforcement levels at heights of 60 m and 150 m (Blaasch, 2012).