Stability and dewatering problems of deep excavations in Bratislava

Les problèmes de stabilité et d'assèchement des excavations profondes dans la ville de Bratislava

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ABSTRACT: Geotechnical design and execution of deep excavations of high-rise buildings in urban areas are presented in the paper. Design of retaining structure and subsoil behaviour based on the results of in situ measurement during execution with the aim to minimise the settlement of high-rise building are discussed. Limited space on the surface and ground conditions in Bratislava are important factors for design of foundations. Quaternary sediments, coarse grained soils, mainly sands and gravels reach down to the depths of 12 to 20 m, Neogene sediments, mainly clays and silty sands with confined ground water level occur deeper. Ground water level occurs from 2 to 6 m below the surface level. Several examples of excavations where the risks of local hydraulic failure were present are analysed. The paper summarizes stability and dewatering problems associated with design and execution of deep excavations in the city of Bratislava.

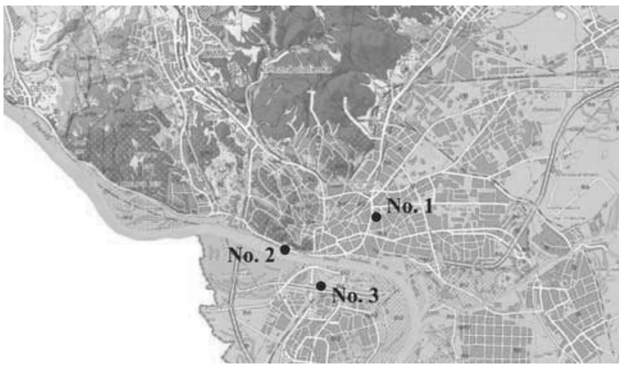
RÉSUMÉ : La conception géotechnique et l'exécution des fouilles profondes de bâtiments de grande hauteur dans les zones urbaines sont présentées dans le document. La conception de structure de soutènement et le comportement du sous-sol sur la base des résultats des mesures in situ lors de l'exécution dans le but de minimiser le tassement des bâtiments de grande hauteur sont discutés. L'espace limité sur la surface et les conditions du sol à Bratislava sont des facteurs importants pour la conception des fondations. Les sols à gros grains du Quaternaire, principalement les sables et graves descendent jusqu'aux profondeurs de 12 à 20 m, les argiles du Néogène et les sables silteux avec une nappe en charge se trouvent plus profonds. Le niveau de la nappe se trouve de 2 à 6 m en dessous du niveau de la surface. Plusieurs exemples de fouilles sont présentés où les risques d'une défaillance hydraulique locale ont été analysés. Le document résume les problèmes de stabilité et d'assèchement associés à la conception et à l'exécution des excavations profondes dans la ville de Bratislava.

KEYWORDS: retaining structure, deep excavations, geotechnical design, subsoil behavior

1 INTRODUCTION

Geological conditions in Bratislava are determined mostly by Quaternary sediments of the Danube river, Neogene sedimentsand Cenozoic crystaline rocks (biotite granites to granodiorites, diorite and gneises) of Carpathians. The most important construction activities in last 20 years have been concentrated in the Danube area, with the Quaternary sediments (coarse grained soils, mainly sands and gravels)with thickness of 12 to 15 m, in the city of and in near vicinity of Bratislava. South and southeastward the depth is increasing, up to 400 m under Gabcikovo (27 km from Bratislava). The Quartenary sediments are underlayed by fine-grained Neogene marine sediments with typical altering of fine grained and coarse grained sediments. Fine grained sediments are classified as clays from firm to very stiff consistency. Coarse grained sediments are classified as dense sands, silty fine sands and clayey sands. Under the Neogene deposits there are the same crystaline rocks as on the northern side of Bratislava (Kopecký, Černý. 2008). Underground water has two main horizonts. One is underground water in Quaternary deposits with free water table donated by water from precipitation and from Danube river. The second horinozont is groundwater confined in the Neogene deposits. The water is pressurized and recharged by underground water flowing from nearby Male Karpaty Mts.

Limited space on the surface and ground conditions in Bratislava are the important factors for design of foundations. Three examples of deep excavation where the risk of local hydraulic failure was present are analysed.



Legend:
Cenozoic crystaline rocks: biotite granites, granodiorites, diorites,
Quaternary deposits: slope deposits, river deposits

Figure 1. Geological map of Bratislava region and location of case studies.

2 EXCAVATION FOR BUSINESS CENTER ON KARADZICOVA STREET

The first case study is excavation situated on the left side of Danube River in the city centre on Karadzicova street. Foundation pit was constructed for the high-rise building with 30 floors and 3 underground floors. The longer side of the foundation pit with the dimensions of 200x39 m and depth of 11.4 m was adjacent to existing buildings, two sides were adjacent to streets and only one short site was open to free space. This layout requires minimisation of horizontal deformation of the retaining structure. Furthermore, there was interest to use all space available on the construction lot, i.e. to exclude technologies that would interfere with the ground plan and decrease the usable volume of the underground floors.

In the first section of the foundation pit up to the depth 4.5 m, where dry conditions were expected, the soil nailing was used as support. The jet grouted wall in deeper section was anchored by one level of ground anchors 15 m long, with 25° inclination, the anchor force was 670 kN and spacing of the anchors 1.6 m. The retaining wall was prestressed in vertical direction due to the necessity to carry large bending moments (bending moment of 1332 kNm per meter of wall was expected by full excavation of the pit); this pre-stressing was supposed to limit the horizontal deformation (see fig. 2 and 3).

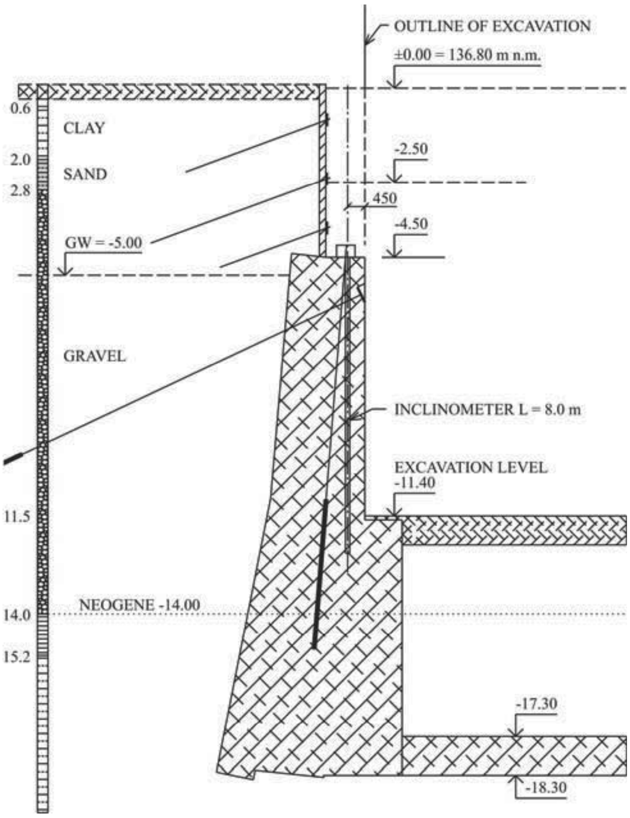


Figure 2. Cross section of retaining structure made by soil nailing and jet grouting.

According to geotechnical investigation the Neogene soils with high permeability are expected at the excavation base. Therefore a sealed bottom of the foundation pit was proposed by overlapping short columns constructed by jet grouting. This solution proved to positively influence the settlement of the building. Horizontal deformation in the level of the retaining wall 3.3 mm and 19.15 mm in the lower embedded part under the bottom of the excavation (in the direction of the excavation) were expected in geotechnical design. By full excavation the toe of retaining wall was expected to move into the foundation pit by 52.56 mm. Nine inclinometers were installed around the foundation pit in the wall constructed by jet grouting. The example of measurements from inclinometer no. 3, where the highest deformations (not higher than 2 mm in the direction into the pit) were indicated, is on figure 4.

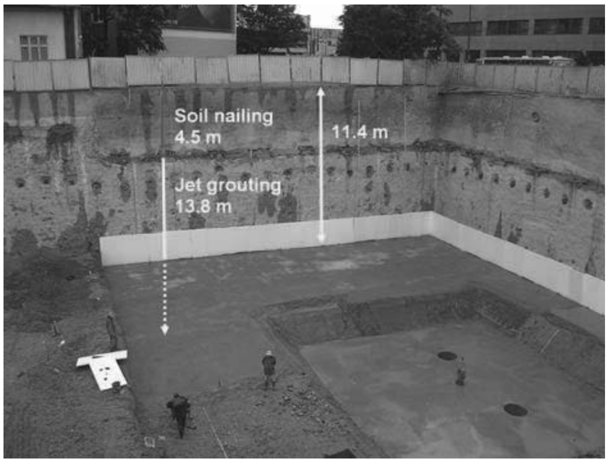


Figure 3. View into the excavation. Retaining structure made by soil nailing and jet grouting.

Similar results with even lower deformations were measured by other inclinometers. This favourable outcome was achieved thanks to high stiffness of the retaining wall prestressed in vertical direction. The stiffness of weak sandy clays under the bottom of excavation has been increased by constructing a horizontal barrier by jet grouting.

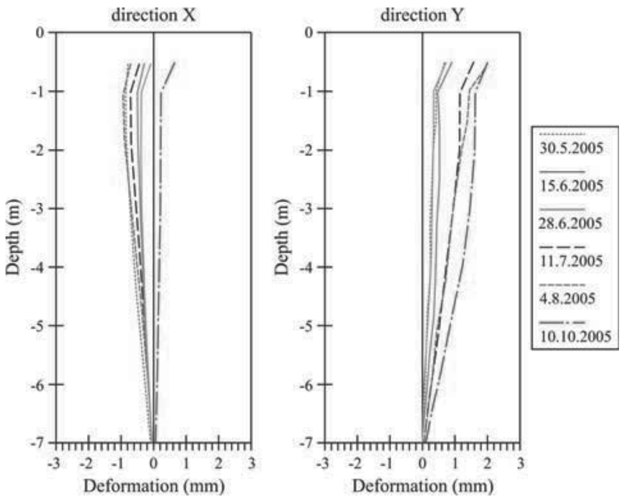


Figure 4. Horizontal deformations of retaining structure during construction.

3 EXCAVATION FOR RIVER PARK

The second case study is the foundation pit with dimensions of 265x53.5 m and depth of 9.0 m (fig. 5) situated directly at Danube riverside (in the distance of 12 m to Danube river). Retaining walls for foundation pit were designed as a secant pile wall embedded 1m into the weathered rock base. Danube coarse grained soils are well-known by their susceptibility to piping. The analysis of the risk of filtration failures is important in geotechnical design. Risk of the internal erosion (structural erosion) is affected by granularity of gravel or sand soils, situated in this region. Therefore the analysis of seepages effect on safety of construction and its subsoil it is necessary to carefully observe mainly development of filtration velocities or hydraulic gradients, which are decisive from the viewpoint of reviewing filtration stability of soils (Bednárová et al., 2010). According to geotechnical investigation the value of permeability coefficient (hydraulic conductivity) of gravel k is in the range from 1.5×10^{-2} to 6.5×10^{-3} m/s. The characteristic value of k equal to 1×10^{-5} m/s was evaluated from the results of water pressure tests. The permeability coefficient $k = 1 \times 10^{-7}$

m/s was used in pile walls design (representing mainly leakages between the piles). Seepage in the foundation pit consists of wall and bottom rate of seepage. The seepage through the pile wall on section 1 was 0.008 l/s and can be obtained from equation

$$q_{w1} = \frac{k_s}{2\,t_s}\left(H^2 - d^2\right) = 7.7 \text{ x } 10^{-6}\, m^3 s^{-1} \quad (1)$$

where H is the height of the water column,

d embedding of the wall under the bottom of the foundation pit.

The predicted flow rate at the pit through walls was 2.12 l/s in the section farther from Danube and 2.23 l/s in the section closer to Danube.

Flow rate through the bottom was determined by the relation

$$Q_b = k\, i\, A_d = 0.0567 \text{ m}^3/\text{s} = 56.7 \text{ l/s} \quad (2)$$

The total rate of seepage can be given as

$$Q = Q_{w1} + Q_{w2} + Q_b = 61.55 \text{ l/s} \cong 62 \text{ l/s} \cong 223 \text{ m}^3/\text{h} \quad (3)$$

In case the groundwater level reaches 138.0 m above sea level (i.e. level of the pile heads), the rate of seepage in the pit increases to 266 m^3/h.

At the beginning of water pumping, static reserve of groundwater from closed space was pumped from the foundation pit besides the inflow. The volume of saturated gravels is 92170 m^3, of which volume of water by the assumption of 25 % active porosity is 23043 m^3. This amount of water is presumed to be pumped in 9 days (during excavation works) when pumping more water than 30 l/s over pumped water flow into the foundation pit.

In order to secure dry foundation pit it was recommended to modify the length of the piles according to the actual conditions even though it might result in longer piles in certain sections compared to the design. Seven pumping wells with diameter 600 mm were proposed with recommendation to verify the actual pumped amounts. Contractor did not trust the assumption of the calculation and constructed 10 wells.

During the excavation of the foundation pit, a measuring of the groundwater movement was performed at all 10 pumping wells (water was permanently pumped only from 2 wells).

Parameters of groundwater movement were determined by the single borehole indicator method, based on the principle of diluting the indicator. NaCl was used as the indicator. Evaluating NaCl concentration was performed by the Radelkis OK 104 tool/instrument on battery source, which recorded the conductivity of the saline solution. Sensors of the Radelkis set with prolonged cable were used as detectors. The value of filter speed (v_f) in each depth level, as well as the average value for the whole measured wall, was calculated according to the relation

$$v_f = \frac{\pi\, d\, \beta}{4\, \alpha\, \delta\, t} \ln \frac{c_o - c_p}{c - c_p} \quad (4)$$

where d is the inner diameter of the borehole ($d = 0.8$ m);

α - coefficient of drainage impact of the borehole; base on comparable experience the value $\alpha = 2$ was used;

β - ratio used for volume decreasing ($\beta = 1$);

δ - coefficient considering the sealing impact on the ratio of flow concentration ($\delta = 1$);

t - time equal to the difference between c_o and c;

c_o - initial concentration of the indicator;

c - concentration in time t;

c_p - background of the indication substance before indicating the environment.

Average value of filter speed for the test section can be determined by the relation

$$\bar{v}_f = \frac{\sum v_{fi}\, \Delta h_i}{\sum \Delta h_i} \quad (5)$$

Evaluated filter speed in all measured objects depend on the location and ground conditions (gravel). No well showed an anomaly of permeability of depth that would indicate increased inflows from certain depth level or from the bottom of the foundation pit. In 2 wells where the water was pumped, filter speed in the range of 1.4 to 4.7 x 10^{-3} m/s was measured, while in other cases it was $v_f = 2.2 \text{ x } 10^{-4}$ to 6.7 x 10^{-5} m/s. Very low seepage ratio was measured in line where jet grouting by the pile wall was performed, which proves reliability of the sealing system. Depth division of the filter speeds confirmed non-homogenity of the gravel location (in some depth the sand fraction was missing).

Figure 5. Foundation pit with dimensions of 265 x 53,5 m and depth of 12 m next to Danube river.

Stable pumping at 20 l/s decreased the groundwater head under the level of excavation base. Decreasing the groundwater head in the area of the foundation pit and evaluation of the movement tests by diluting method indicated that lesser amount of water is required to be pumped from the foundation pit than expected amount of water in the project design (62 l/s). In conclusion it was stated that the retaining structure created a foundation pit with required sealing function.

When preparing the base foundation for placing the underlying concrete, the excavation in the north-western corner of the foundation pit deepened for 0.3 m more than was necessary and minor water seepage occurred along the pile wall that held at the same level for several days. It was confirmed by leveling that the groundwater head in the wet area by the pile wall was 0.48 m bellow the upper edge of the base concrete and held at the same level. Performed test of natural conductivity of the water seepage helped in identifying its origin. Two locations of leakage were confirmed in the wet area along the pile wall. It was evaluated as the water seepage into the foundation pit area through the contact of the piles connection in the wall where the discontinuities in the rock massive were collectors of the water. Drainage to the nearest active pumping well had to be installed in order to solve the problem. At the same time it was necessary to replace existing soil by gravel without the sand fraction in the foundation pit area and thoroughly compact the layers of this soil replacement, in order to eliminate the risk of higher deformations of the future base structure.

4 DEEP EXCAVATION IN PETRZALKA

Quaternary gravel sediments on the right side of Danube were characterized as loose to medium dense coarse grained soils with the values of density index I_D varying from 0.25 to 0.63

and deformation modulus E_{def} = 30 up to 112 MPa according to geological investigation by dynamic penetration testing up to the depth 40m. Locally in layers with fine-grained gravel with minimal sand filling the value if I_D was 0.15 and deformation modulus E_{def} = 16 MPa. Average thickness of the quaternary gravels was 13 m. Neogene sediments were represented mostly by sandy clays (*saCl*) with firm filling of clay, with sand locations with addition of fine-grained soil (less than 15 %), to a small extent also with silts of middle plasticity to clays of high plasticity in depths of 19.8 to 20.6 m and 25.3 to 26.4 m. The value E_{def} determined by dynamic penetration tests was 20 to 28 MPa in Neogene soils.

Contact stress in base foundation of the high building was 500 to 800 kPa. Limit settlement determined the type of foundation (rafts or piles). Based on the information about disproportionately high compressibility of the subsoil, ground treatment by deep vibratory compaction was performed. Base foundation was located in the depth of 4.0 m under the surface. The raft is 1.4 m thick, in deepened parts 2.0 m. Density of compaction points was in raster of 2.2 x 2.4 m for stress of 800 kPa, under pad footings and strip footings in raster 2.5x 2.5 m to 1.75 x 3.5 m for stress of 500 kPa and under pads and strips in raster 1.8 x 1.8 m for stress of 800 kPa. The length of the piles for stress of 800 kPa was in depth of 3.0 m from the working base under the surface 10,0 m (i.e. till the Neogene in the depth of 13.0 m), shortened to 7 m in area with load of 500 kPa.

This arrangement was based on previous comparable experience in Quaternary gravel sediments of Bratislava where the verified deformation modulus of the vibrated gravel columns reached the average value of E_{def} equal to 500 MPa and increasing of the deformation modulus of average environment was proved by the values E_{def} from 250 to 300 MPa. The calculation of the average deformation modulus of improved subsoil in the area A in question is according to the relation

$$E_{def} = \frac{A_g\ E_g + A_s\ E_s}{A} \tag{6}$$

where A_g is the area of vibrated stone columns
E_g – deformation modulus of the vibrated stone columns
A_s – area of the unimproved soil
E_s – deformation modulus of the unimproved soil

After constructing the vibrated stone columns, dynamic penetration tests were performed, located in the middle of the vibrated stone columns. Increase in the deformation modulus with depth was also proved, which was taken into account in the calculation of predicted settlement based on the monitored data. Average value of E_{def} = 284 MPa was measured under the base foundation up to depth of 2 m. The predicted final settlements are summarized in the table 1.

Table 1. The values of settlement calculated for different foundations.

Foundation	*Settlement* *s* (mm)
Spread foundation without soil improvement	104.46
Spread foundation with soil treatment by stone columns (E_{def} based on comparable experience)	67.01
Spread foundation with soil treatment by stone columns (E_{def} based on site testing)	64.55
Pile foundation	73.97

During the construction of the high building the vertical and horizontal deformations were monitored. The measured value of settlement after consolidation process reached 52.2 mm.

5 CONCLUSIONS

The key value in the design of high building foundation in deep excavation pits is the limit usability state. Most questions are raised by the simulation of soil deformation characteristics and expected groundwater inflows. Good prognosis of deformations is based on combination of determining correct soil deformation characteristics verified by laboratory and field tests in the whole deformation zone and selecting the right calculation method. The project must respond to the architect's requirements by suitable design, its monitoring and potential modification of stiffness of the retaining structure, subsoil and additional sealing elements.

Geotechnical design and execution of deep excavations of high-rise buildings in urban areas are presented in the paper. Design of the retaining structure and subsoil behaviour based on the results of in situ measurement during execution with the aim of minimising the settlement of high-rise building are discussed. The paper summarizes stability and dewatering problems associated with design and execution of deep excavations in the city of Bratislava. The results of geotechnical calculations have been compared to the results of in-situ measurements.

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