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Excavation Pit for a Multifunctional Sports Arena in Brno, Czech Republic

Jan Štefaňák, Jan Minarčík, Martin Vítek, Petr Svoboda
Keller-speciální zakládání spol. s r. o., Czech Republic, jan.stefanak@keller.com

Václav Račanský
Keller Grundbau GmbH, Austria

ABSTRACT: This paper discusses the design and execution of a retaining structure for excavation pit for a multifunctional sports Arena built in Brno, Czech Republic. The retaining structure consists of an anchored contiguous pile wall with an axial distance up to 2.20 m. The free space between the piles is sealed by deep soil mixing (DSM) barrier constructed by triple mixing tool in permeable strata of quaternary gravels. In most of the pit perimeter, one row of anchors is used. In the less frequent sections with excavation depths up to 12.00 m, the structure is anchored at two levels. The upper row of anchors passes directly through the DSM body and the lower anchors passing through the core bores in piles. Prior to the commencement of design works, Cone Penetration Test probes were carried out at the site. Further, undisturbed samples of overconsolidated Tertiary clays were collected and subjected subsequently to triaxial (CIUP) tests. The engineering design approach used is presented, which takes the time effects on the strength of the Tertiary clays into account to deliver sustainable solution. Results of other possible analytical design approaches in terms of soil strength versus time of execution of structure are discussed. The paper also includes an evaluation of uniaxial compressive strength tests carried out on DSM core samples. Discussion of the results of monitoring of the structure (geodetic and inclinometric) is also encompassed.

KEYWORDS: Deep soil mixing, ground anchors, excavation pit, undrained shear strength of clays

1 INTRODUCTION

Construction of the new multifunctional arena in Brno, Czech Republic is designed to accommodate up to 13,000 spectators. It represents a strategically significant project commissioned by the Statutární město Brno, ARENA BRNO, a.s., Brněnské komunikace a.s. and Teplárny Brno, a.s. The external dimensions of the arena at ground level are 151 m × 108 m, with an above-ground height of approximately 29.5 m. The built-up area covers 16,470 m², and the total enclosed volume reaches approximately 475,000 m³ (Arena Brno, 2025).

The floor plan dimensions of the construction pit are 210 m x 110 m. The excavation depth of construction pit generally reaches 8.50 m, with local deepening up to 12.0 m. The Arena is being constructed near the Brno exhibition grounds.

2 GEOLOGICAL, ENGINEERING GEOLOGICAL AND HYDROGEOLOGICAL CONDITIONS

The region is characterized by a markedly differentiated terrain. The landscape comprises higher upland levels (approximately 350 m a.s.l.) and lower hilly zones (200–350 m a.s.l.), interconnected by depressions and basins—specifically, the Pisárky Basin, with an average elevation of 262.0 m a.s.l. The area of interest is situated above the valley of the Svratka River. The site´s pre-Quaternary bedrock comprises of Paleozoic sandstones. The uppermost layer of the sandstones is completely weathered into an eluvial deposit of sandy clay with a light reddish coloration. At greater depths, the sandstones exhibit varying degrees of weathering, ranging from highly weathered (strength class R5), moderately weathered (R4), to slightly weathered (R3).

The Quaternary layer mainly consists of fluvial gravelly and sandy soils, formed by the deepening of the Svratka River. The gravels typically consist of semi-rounded fragments of various sizes, alternating locally with coarse-grained sands. Beneath the fluvial gravels, marine Neogene sediments—clays—were encountered. These clays exhibit high plasticity and are generally of stiff to very stiff consistency. Plasticity index of clays I_p = 33 to 39; filtration coefficient $k = 4.6e^{-9}$ m/s to $8.6e^{-11}$ m/s. Neogene clayey sands were identified as thin interlayers between highly plastic clays. These sands are mostly dense.

Groundwater was detected in all boreholes, located in the layer of the fluvial gravels, at depths ranging from 2.6 to 4.2 m below ground level (Geodrill 2020, 2023)

2.1 *Supplementary Geotechnical investigation – In-situ testing*

Before finalizing the design work, additional investigations were undertaken to supplement the previous stages of site exploration. The scope of the investigation included the execution of additional boreholes, complemented by static cone penetration tests (CPT). Figure 1 compares core borehole log and the log of a close CPT probe within a typical geological cross-section.

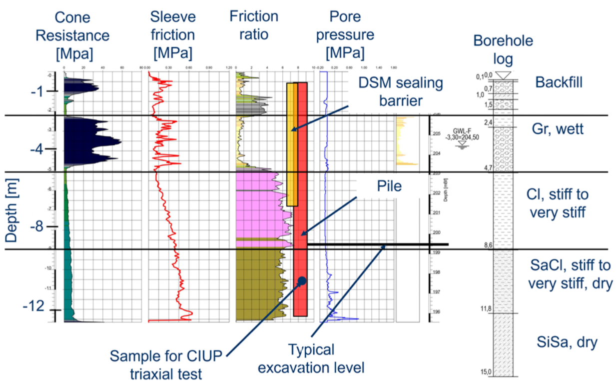

Figure 1. Comparison of borehole log and CPT probe log for typical cross-section

The comparison shows a strong correlation in terms of identifying lithological boundaries between different types of soils. The geological section in Figure 1 also contains a conceptual cross-section of the retaining structure, which is described in more detail later.

2.2 *Supplementary Geotechnical investigation – laboratory tests*

Undisturbed soil samples were collected from the depth of the anticipated pile wall embedding for triaxial testing (Chalmovský, 2023).

Consolidated undrained (CU) triaxial tests with isotropic consolidation and pore pressure measurement (CIUP) were performed on laboratory specimens with a diameter of 50 mm. Specimens were prepared from undisturbed core samples taken at a depth of 10.8 m. Prior to undrained shearing, the specimens were isotropically consolidated to effective confining pressures of σ'_3 = 100 kPa and 200 kPa. The shearing of fully saturated specimens was conducted under strain-controlled conditions at a deformation rate of 0.0050 mm/min,

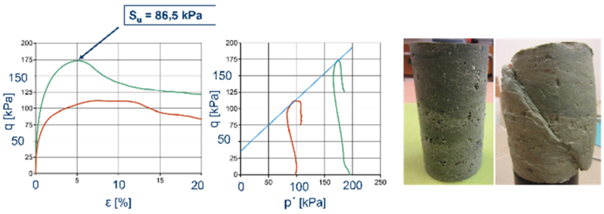

Figure 2. Results of CIUP Triaxial tests for the range of effective normal stresses from 100 kPa to 200 kPa

Based on the test results, the following effective peak shear strength parameters were evaluated: cohesion c'_p = 16.8 kPa and angle of internal friction φ'_p = 20.4°. Additionally, the peak short-term undrained shear strength was determined as c_u = 86.5 kPa.

3 TECHNICAL SOLUTION

The temporary retaining structure was engineered as a pile wall with technical watertightness ensured by a sealing barrier constructed using Deep Soil Mixing (DSM) technology. Retaining piles with a diameter of 880 mm were designed at center-to-center distances ranging from 1.9 m to 2.2 m. The toe of the sealing barrier was designed with at least 1.0 m embedment into the layer of Neogene clays. A schematic of the solution is shown in Figure 3.

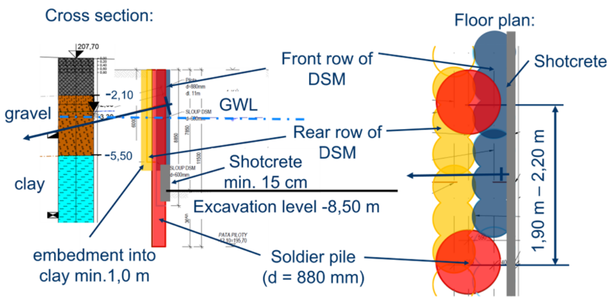

Figure 3. Typical cross-section through designed retaining structure

The barrier will consist of two rows of DSM lamellas. The face of the soil below the toe of the sealing barrier will be covered with a shotcrete layer of minimum thickness 15 cm. The final structure will have a single-sided formwork function. The exposed surface of the walls will be covered with a layer of shotcrete, with the final face offset by 5 cm from the new building structure.

Based on the excavation depth in specific section, temporary anchoring was designed in one or two anchorage levels. Temporary anchoring of the pile wall was designed directly through the sealing barrier in the first (upper) anchorage level and through individual piles in the second (lower) anchorage level. See Figure 4 for an anchoring scheme with details of the anchor heads.

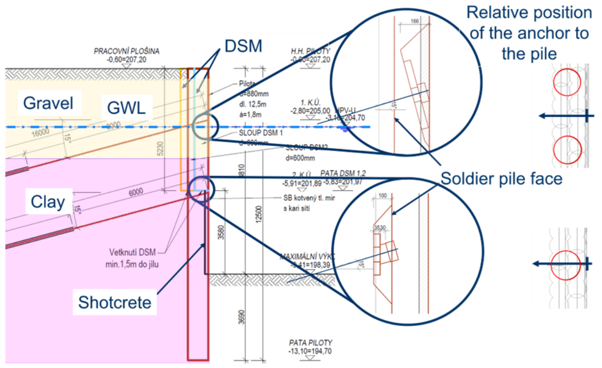

Figure 4. Details of the anchor heads in a section with two anchorage levels.

The construction process of a single DSM lamella consists of two basic phases. The first phase involves drilling a tool to the desired depth with little or no cement slurry supply. The drill-mixing tool consists of steel pipes with blades and paddles welded onto them – see Figure 7 below. A nozzle for feeding the slurry into the borehole is located at the tip of each pipe. There can be generally one to three paddles, which overlap each other. Each paddle forms a circle with a diameter of approximately 60 cm when rotating. Once the desired depth has been reached, the slurry is pumped through the drilling-mixing tool. While it is rotating, it moves upwards towards the head of the lamella. The disturbed soil is mixed with the cement suspension to form a mixture. The amount of suspension applied to the foundation soil, which is one of the main technological parameters, is controlled by the machine operator or the machine's computer independently. The full length of the lamella, or specific parts of it, is mixed 2-3 times, which is necessary for good mixing quality. The suspension-soil mixture creates a composite body with significantly higher physical and mechanical properties after hardening than the original soil. In terms of mechanical properties, the final product can be compared to jet grouting material.

4 GEOTECHNICAL DESIGN

To compute the internal forces in the designed structural elements, an iterative method was applied. The method evaluates the resistance on the passive side of the retaining structure, assuming elastic-plastic behavior of soil limited by the passive earth pressure. The approach to applying loads and determining the necessary embedment of piles below the bottom of the construction pit is described below in this chapter.

4.1 *Short-term and long-term stability of retaining structures*

The response of the soil to the loading is controlled by effective stress, regardless of the current magnitudes of excess pore water pressures. Figure 5 shows the total and effective stress paths, governed by the excavation of soil from the construction pit, at points on the critical shear planes. Excavation of soil reduces normal stress and increases shear stress. The pore pressure u_i after excavation of the construction pit (B-B') is lower than the pore pressure u_c in the steady state (C-C'). The total stress remains approximately the same over time after excavating soil from the pit (B'-C'). However, there is an increase in pore pressures due to reduction of suction which is caused by the dilatant behavior of overconsolidated clays. The result is a decrease in effective stress and a decrease in the factor of safety of the structure over time.

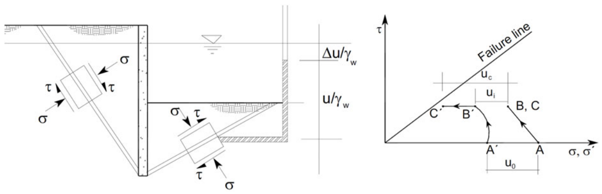

Figure 5. Change of stress and pore pressure under the bottom of the retained construction pit after excavation (taken from Atkinson (2007))

Currently, it is feasible to employ finite element method (FEM) simulations for modeling a coupled analysis of flow and consolidation processes. Thus, the factor of safety can be calculated at any given moment in time. However, the design of retaining structures under undrained short-term conditions still can be performed analytically using total strength parameters of soil. The main historical reason for using undrained analysis is that it eliminates the need to determine the values of excess pore pressures, which is necessary for determining short-term effective strength of the soil.

4.2 *Time-limit for undrained conditions*

The issue related to selecting analysis type in terms of short-term or long-term conditions is how long it is reasonable to consider the load conditions as undrained. Vermeer (1998) e.g., suggested an approach based on the classical theory of uniaxial consolidation described by the Equation (1):

$$T_v = \frac{c_v}{D^2}.t = \frac{k.E_s}{\gamma_w.D^2}.t \qquad (1)$$

where D is the depth of the retaining structure below the bottom of the construction pit, k is the filtration coefficient, E_s is the edometric modulus, γ_w is the unit weight of water, and c_v is the consolidation coefficient. He recommends considering conditions as undrained for the dimensionless time factor values Tv < 0.1.

Another recommendation employed for practical design of retaining structures e.g. is the so-called Frankfurt rule (Frankfurt Stadbahnrichtlinie, 1991). It stipulates that the undrained shear strength of Frankfurt clay may be used to design the retaining structure, provided that after an excavation step with a maximum depth of 8.0 m under bracing—at latest four weeks after starting excavation of the last 2.5 m—the retaining wall is supported against the building foundation or equivalently.

4.3 *Effect of soil structure on the short-term strength of fissured clays*

As noted by (Janbu, 1997) and (Laufleur, 1988), simple total stress analysis can lead to erroneous designs with an overestimated factor of safety. One of the reasons might be the anisotropy of the overconsolidated clay because of secondary developed texture. It can be formed by secondary effects acting on already sedimented soil, e.g. 1/ the cracking of more brittle layers deposited between plastic layers susceptible to remolding, 2/ the weathering of non-clay particles in clayey material by deep Pleistocene frost, etc. Clay layers are also interspersed with sand layers of varying thickness and of different strength parameters than the surrounding clay (Mencl, 1966). The designers of retaining structures designed in the 1970s (before the wide spread of numerical methods into practice) were aware of these factors. An analytical approach was therefore used at that time for the design. It is not consistent in terms of soil mechanics but is conservative compared to the total stress analysis. This approach was in slightly modified form applied to the design of the above presented structures. The concept of this approach is described below.

4.4 *Geotechnical model of retaining structure*

The geotechnical model of area of interest can be simplified into two main layers 1/ quaternary gravel layer and 2/ underlying layer of overcosolidated neogene clays, see Figure 6. The redistributed active earth pressure was considered on the active side of the wall. Magnitude of active pressure referred to effective unit weight of the gravel and total unit weight of the clay.

Previously published Czechoslovak recommendations (Gregor, Klein, 1977) state that in clays with a plasticity index $I_p > 20$, it is not necessary to consider full hydrostatic pressure for short-term conditions. This rule can be applied if it is structurally ensured that water cannot infiltrate behind the wall or if the preventive drainage system is constructed. Hence, linearly increasing hydrostatic pressure was considered in model starting from the design groundwater level to the depth of bottom of the DSM lamellas. The load from hydrostatic pressure was modeled as horizontal pressure acting on the wall.

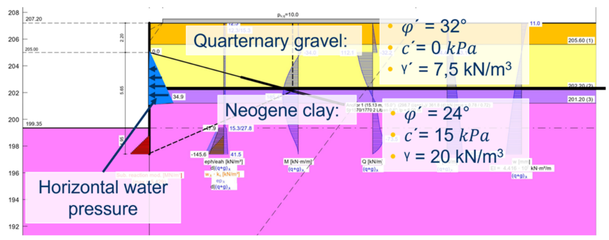

Figure 6. Geotechnical model of typical analyzed cross-section

The analysis yielded the required embedment length of the pile below the bottom of the construction pit d = 1.95 m. This model was further modified for the design - the effective weight of the soil was considered in front of the wall on the passive side. This achieved a reduction in mobilizable passive resistance, leading to additional design safety and a required embedment depth of d = 2.95 m. For comparison, simple total stress analysis was performed with undrained shear strength c_u = 80 kPa, which is in line with the results of laboratory tests. The resulting required length d = 1.45 m in such case.

5 IMPLEMENTATION OF DESIGNED STRUCTURES

5.1 *Preproduction tests of DSM sealing barrier lamellas*

Prior to commencing production of the sealing barrier, a mixing test was carried out in advance at the site. The objective of the test was to set the technological parameters of the production process (number of mixing cycles) and material parameters (amount of cement and amount of mixed suspension) to achieve the mechanical properties of the composite as specified in the design. Figure 7 shows a diagram of a lamella created by a three-paddle mixing tool in the geological conditions encountered.

Following the creation of the experimental lamella, it was excavated to verify the expected geometry of the body. Samples of mixture were taken during mixing to determine the uniaxial compressive strength of the final composite. The mean characteristic value of the uniaxial compressive strength was *UCS* = 2.90 MPa. The results of the laboratory tests confirmed the assumptions regarding the strength of the composite considered in the static calculation. It was utilized in the proof of the capacity of the anchor area against punching. The second proof was elaborated for capacity of arch virtually formed in the sealing barrier body to resist loads from the active side of the structure.

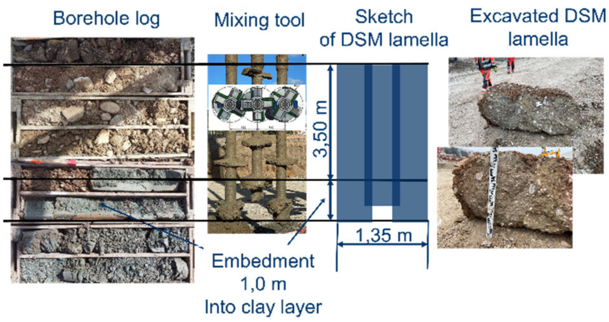

Figure 7. Illustration to preproduction test with photo of relevant bore core (left) scheme of a three-paddle mixing tool (middle); and the excavated body of test lamella (right)

5.2 *Construction process*

The primary wall construction process consisted of two steps. The first step involved installing two rows of DSM sealing lamellas. The second step involved drilling piles through the DSM body at intervals of 2-3 days. Once the piles and DSM walls had hardened, excavation of the soil in the pit began.

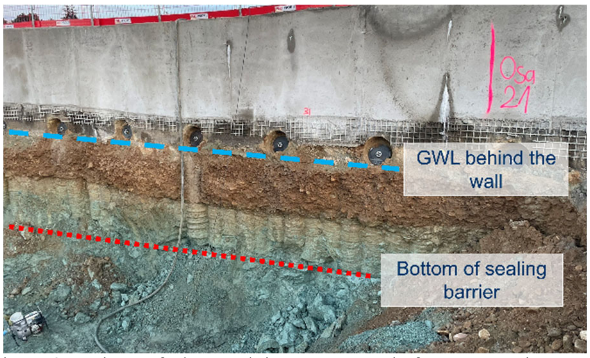

Figure 8. View of the retaining structure before excavation and application of shotcrete on the lower level. The clear interface between the sealing barrier created in layer of quaternary gravel and the unimproved layer of tertiary clay is visible.

Throughout the excavation, the DSM surface was levelled by milling to prepare the surface for application of shotcrete. In the first phase, the pit was excavated only to the first anchor level. Temporary anchors were then installed. After the anchors had been cured and tensioned, further excavation of the pit continued either to the final level of the excavation or to the second anchor level (depending on the depth of the pit in specific cross-section). Milling the exposed surface of clay followed by shotcrete application was carried out in the deeper parts of the wall below the DSM barrier. A view of the wall before milling the face is shown in Figure 8. Photo in Figure 9 shows a panorama view of the completed retaining structures at the start of work on concreting the foundation slab.

Figure 9. Panoramic view of the completed retaining structure at the start of concreting the foundation slab

Approximately 3.2 kilometers of retaining piles were installed, along with more than 4.8 kilometers of temporary strand anchors. To seal the excavation pit, approx. 900 DSM lamellas were mixed. Roughly 4,100 square meters of shotcrete were applied to the exposed surface of the retaining structure. Foundation piles were installed by cased drilling from the level of the working platform for installation of the first row of anchors. The deepest borehole for the pile with diameter 1500m and length of 26.0 m was 36.0 m.

6 CONCLUSIONS

The design and execution of the temporary excavation support system were provided by KELLER – Speciální zakládání (CZ). The executed retaining structure design proved to be economically and environmentally sustainable. Replacing the originally designed jet grouting by DSM technology simplified the work process and eliminated the need for jet grouting spoil suspension disposal. Deformations of wall were monitored during excavation using inclinometer measurements in the typical and in the deepest section of the wall. The maximum magnitude ranged up to approximately 12 mm in the deepest section. In the section of typical depth, the deformations in the order of millimeters were observed, in range of measurement accuracy. Megnetoelastic sensors Dynamag were installed on selected anchors to monitor the anchor force. The forces observed after the excavation ranged between 89% to 93% of the calculated characteristic anchor forces. The safety of the design was thus confirmed as well.

7 ACKNOWLEDGEMENTS

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