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Construction Lot U2/21- from geological ground model to successful excavation

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ABSTRACT: The construction lot U2/21 is part of the lines crossing project U2xU5, the most important climate protection and infrastructure project in Vienna. Based on a detailed 3D- geological ground model of the magistrate department MA 29 it could not be ruled out, that during excavation works, which are mainly situated in silty-clayey ground layers, sand pockets are encountered showing a decisive length and being filled with highly confined water. Advanced dewatering of the ground was not possible due to a dense development. Therefore, an innovative concept for excavation of the station tunnels was designed consisting of a prior excavated pilot tunnel supported with fiber reinforced shotcrete. Additional to the station tunnels a parallel running connection tunnel, 2 cross passages and 2 escalator tunnel are situated in the station area. A big shopping mall with a multilevel underground car park is in the immediate vicinity of the excavation works. Due to the sensible and dense development settlement compensation grouting is planned. Due to these complex boundary conditions, a 3D model for numerical calculations was established. The soil was modelled using the HSS- material model, the calculation values were determined based on extensive investigation works and laboratory testing as well as in situ borehole tests. The calculations were carried out considering consolidation (fully coupled) to take changes of the pore pressure and influences on the deformation field into account. The calculations were performed in real time to gain as realistic results as possible, taking development of stiffness and strength of shotcrete into account. For evaluation of compensation grouting volume expansion was applied on distinct elements. Various variants were investigated to evaluate the different influences on excavation areas and buildings. Based on detailed analysis the area of necessary settlement compensation measures could be optimized. The advantageous effects of the pilot tunnel could be verified by the results and were confirmed by the experiences during excavation.

KEYWORDS: 3D numerical modelling, soft soil, HSS material model, compensation grouting, pilot tunnel, dewatering

1 INTRODUCTION

Situated between the 6th and 7th district of Vienna, the construction lot U2/21 Neubaugasse is one of five newly built stations of the line crossing project U2xU5 for the Viennese Underground System. The existing U3 station runs longitudinally beneath Mariahilfer Straße, while the newly constructed U2 station crosses Mariahilfer Straße. Owing to its position within the urban rail network, the new station is expected to get the highest passenger volumes among all stations. Since the area is densely built up with narrow streets and high pedestrian traffic, specific challenges during design and the building process itself occurred.

The station (Figure 1) consists of three shafts, constructed using the cut-and-cover method, each up to 35 m deep, connected by two platform (station) tunnels and a connection passage for pedestrians, connecting the accesses from Mariahilfer Straße and Kirchengasse. The station tunnels, measuring 115 m in length and 4 m in width each, are linked by two cross passages and by the shaft in Lindengasse. Two escalator tunnels inclined at 24.5° were excavated from the cross passages providing a direct connection between the track level and the corresponding track level of the U3 station, which is designed as a stacked (double-level) structure. Both station tunnels are located entirely beneath densely built-up areas and were constructed using the shotcrete construction method following the principals of the New Austrian Tunneling Method (NATM). The distance from the tunnel crown to the ground surface ranges between 22 and 23 m, while the clearance to the basement floors of the buildings varies between 11 and 2 m.

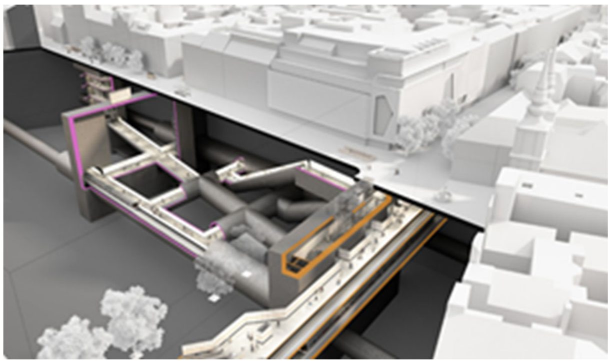

Figure 1. Overview of station Neubaugasse (Source: Wiener Linien)

2 GEOTECHNICAL GROUND CONDITIONS

From a geological perspective, the construction lot U2/21 Neubaugasse lies at the level of the Quaternary Arsenal Terrace. Starting at the surface of the ground the following ground types according to ÖGG (2023) are encountered within the excavation works. At the top anthropogenic embankments and, in some areas, loess or loess loam are encountered. These are underlain by sandstone gravels and, subsequently, quartz gravels. In some areas, alluvial fan sediments are intercalated between the two gravels.

Beneath the quartz gravels at a depth of 14m lies a thick layered package of Miocene deposits, which, in the U2/21 construction section, predominantly consist of clayey silts or silty clays (Vienna Tegel). Coarse silt and (silty) fine-medium sands layers with an extent of several tens of meters are also deposited in these layers. Due to their comparatively low thickness of up to approximately 1 m these sands do not represent an individual ground type. The tunnels are fully located within these Miocene deposits.

In terms of hydrogeological conditions, due to the topographical location, a shallow, sometimes – depending on precipitation conditions – only temporary, free groundwater level is present in some areas at the lower edge of the quartz gravel. Groundwater is also present in the coarse silt and sand layers within the Miocene deposits, but in contrast to the free Quaternary groundwater, this is highly confined and exhibits a correspondingly high-pressure level.

Therefore, these coarse silt and sand layers are particularly relevant from a geotechnical and tunnel engineering point of view. During design, main tasks were to ensure a reduction of water pressure on the shaft structures and to guarantee appropriate buoyancy safety during construction on the one hand. On the other hand, it was necessary to ensure appropriate hydrogeological conditions for the subsequent tunnel excavation works. Therefore, almost 100 gravitational wells with depths of up to 45 m were constructed to reduce the pressure level accordingly.

As a basis for designing the station structure and predicting the subsurface conditions as realistic as possible even during tunnel excavation, a 3D model containing geological and geotechnical boundary conditions was established based on the project-specific subsoil investigations and other boreholes already recorded in the City of Vienna's subsoil cadaster. Even the geological recordings of the drilled wells were integrated into the geotechnical 3D model to achieve a higher level of detail. Based on this detailed model it was possible to predict the anticipated subsoil conditions in the station area with high forecasting reliability by interpolating between single exploration outcrops (Figure 2).

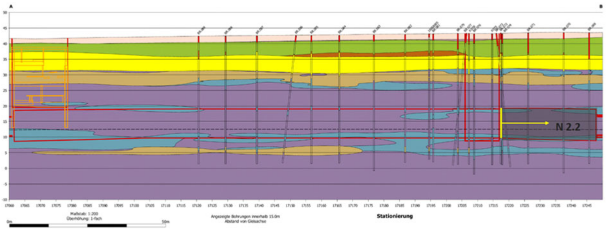

Figure 2. Longitudinal section of anticipated subsoil conditions in the area of construction lot U2/21

3 EXCAVATION CONCEPT

Based on the detailed 3D subsoil model, one of the main questions during design was whether the sand layers situated in the Miocene potentially containing water with high pressure will be encountered during excavation. Counter measures had to be evaluated and integrated into excavation design to guarantee a safe excavation process.

Since the U2/21 construction section is located in a very densely built-up area, and thus the planned release of Miocene groundwater from the surface by means of wells was not fully possible, an innovative concept for the Vienna subway construction was planned and implemented for the excavation of the two station tunnels.

The design contains the excavation of pilot tunnels with an excavation area of approximately 20 m² in the area of the subsequent station tunnel top heading. Therefore, the primary task of the pilot tunnels was to achieve secure groundwater relief from a smaller excavation profile compared to the station tunnels (approx. 95 m²). Furthermore, there are also other significant advantages for the subsequent tunnelling work in the inner-city area.

3.1 *Tunnel support*

During the excavation design of the pilot tunnels the following main facts had to be considered. On the one hand, a rapid ring closure had to be ensured to minimize surface settlements. On the other hand, careful use of resources within the pilot tunnel support measures and the requirement of their most gentle removing during the subsequent excavation of the station tunnels due to the urban neighborhood had to be guaranteed. Therefore, steel fiber-reinforced shotcrete with a thickness of 15 cm at minimum was foreseen. Fiber-reinforced shotcrete offers both high strength and the necessary flexibility for excavating the tunnel cross-section. The pilot tunnel is excavated in full face excavation with round lengths between 1.0 and 1.3 m. To enable the installation of advance stabilization measures, lattice girders were considered in the design additionally. These would only be installed if necessary. To further minimize the risk of encountering confined ground water during excavation of the station tunnels, appropriate drainage spears would be drilled in regular distances from the cross section of the pilot tunnels.

3.2 *Dewatering*

The pilot tunnels will be driven in advance along their entire length from the shaft Lindengasse to the shaft Mariahilfer Straße respectively to the northern end of the construction lot. After completion of the pilot tunnel, the radial drainage and vacuum spears will be installed in the cross-section of the pilot tunnel to ensure advanced groundwater relief. To avoid interruption of station tunnel excavation and avoid softening of the temporary backfill of the tunnels, the water from these drainage bores will be collected and derived toward the Mariahilfer Straße shaft.

3.3 *Advantages*

In addition to the main task of the pilot tunnels, the safe groundwater drainage via a smaller excavation profile compared to the significantly larger station cross-section (approx. 95 m²), their use results in a number of other project-specific and technical advantages:

- Specific groundwater relief and dewatering in advance: The systematic drilling of drainage spears from the pilot tunnel allows safe groundwater relief in advance to the excavation of the station tunnels, even if vacuum application is necessary. Furthermore, the additional installation of drainage spears in the face of the pilot tunnels can lower the groundwater level in advance. The structural segregation of groundwater relief and excavation of the station tunnels also minimizes the risk of delays in the construction process.
- Reduced risk when encountering sand layers containing confined ground water:
  Compared to the cross section of the big station tunnels, the smaller cross section of the pilot tunnels minimize the danger when encountering these sand layers.
- Optimization of the excavation process of the station tunnels:
  Due to the fact that the cross section of the pilot tunnel lies within the top heading of the subsequent station tunnel, the excavation area of the station tunnel is reduced, and face stability is increased vice versa. Stresses in the subsoil can be partially redistributed und displacements during excavation of the station tunnel are reduced.
- Additional geological-geotechnical investigation: The pilot tunnels enable an additional investigation of the subsoil specifically regarding geological fault zones and water bearing formations. Therefore, the geotechnical model could be updated during construction and support measures could be adjusted to the new findings if necessary.

4 NUMERICAL MODEL

4.1 *Calculation Goals*

Due to the complex boundary conditions and foreseen excavation sequences, a 3D model had to be established to evaluate the effectiveness of the pilot tunnels with a focus on groundwater relaxation, possible interactions between single excavation steps and effects on the existing development consisting of high storage buildings with two underground parking decks in the immediate vicinity. Moreover, the necessity and effect of compensation grouting had to be evaluated in order to guarantee a most efficient and target-oriented approach.

On the one hand the aim of the model was to check the dimensions of the tunnel support measures consisting of fiber reinforced shotcrete and wire mesh reinforced shotcrete especially with focus on loading due to compensation grouting, and the face support measures consisting of rock bolts and wire mesh reinforced shotcrete. On the other hand, different sequences of compensation grouting had to be investigated to evaluate necessary heaves, to minimize tilting of the existing buildings and to examine tunnel loading.

To minimize model complexity as well as calculation time, symmetry of the station layout was taken into account when setting up the model geometry. Therefore, one boundary plane cuts through the connection tunnel referred to as construction component CC V. The cross passages, elevator- and station tunnels (CC N) are covered within the model. Instead of modelling the existing buildings discreetly a load distribution plate and equivalent surface load was applied.

The geotechnical model was transferred to the numerical model by assigning an appropriate material for each ground type to the corresponding FE- elements. Sand layers were not modelled discretely as they are not an individual ground type.

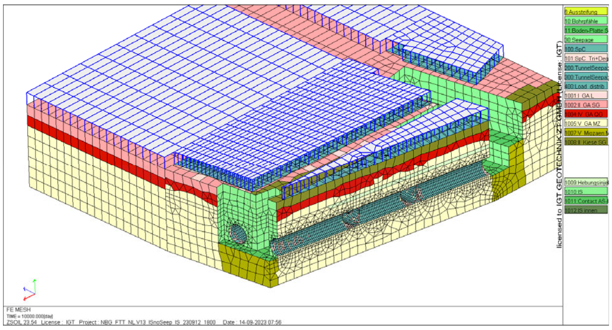

Figure 3. 3D model with station tunnel and ground types

4.2 *Modelling aspects*

4.2.1 *Material Model for Soils*

Investigating a deep-seated tunnel excavated in rock, a linear elastic ideal plastic material model with a failure criterion according to e.g. Mohr- Coulomb or Hoek-Brown is used and is normally sufficient. Regarding soils, mechanical behavior is characterized by a strong variation of soil stiffness which depends on the magnitude of strain levels occurring during construction stages. This behavior is modeled with non-linear elasticity. Moreover, pre-failure stiffness plays a crucial role in modeling typical geotechnical problems such as tunnel excavations in densely built-up urban areas.

Therefore, it was decided to use the Hardening Soil Small Strain material model (HSS) to achieve a most realistic prediction of ground movements, load distributions within the soil and loading of the support measures.

Being aware of the availability of the brick model in the used software ZSOIL, which eliminates important drawbacks of the small strain overlay by Benz used in the original Hardening Soil-small (HS-small) model, it was decided to deactivate this option due to possible influences on the established parameter sets of the different ground types.

In case that for single ground types no HSS parameters were given in the geotechnical report, they were calculated based on common correlations to get calculation results as realistic as possible. Usually, the stiffness of soil increasing with depth is considered. This dependency can be expressed with the following correlation (Obrzud & Truty, 2020) e.g. for the triaxial modulus E50 defined as the secant modulus at 50 % of ultimate deviatoric stress q_f described by Mohr- Coulomb criterion (Equation (1)).

$$E_{50} = E_{50}^{ref} * \left(\frac{\sigma_3^{ref} + c * cot\varphi}{\sigma^{ref} + c * cot\varphi}\right) \qquad (1)$$

It must be noted that except for the oedometer modulus E_{oed} the reference stress is the horizontal stress.

4.2.2 *Material Model for Shotcrete*

Even today the main goal of design is to use construction material such as concrete and reinforcement steel as efficiently as possible. Therefore, regarding aging, creep, and stress dependence of shotcrete and especially pronounced capacity to take large strains at young age, the material model Aging Concrete as a Visco-elastic constitutive model was chosen. Thereby, load results in the shotcrete shells can be calculated as realistic as possible and shell thickness and reinforcement amounts can be minimized. The aging concrete model represents time dependent mechanical properties as well as rheological behavior of concrete in early age and consists of a set of parallel Maxwell units (Figure 5). Each unit is described by a maturity dependent Young's modulus

$$E_k(M) = W_k(M) * E \qquad (2)$$

and retardation time

$$\tau_k = E_k/\eta_k \qquad (3)$$

with Ek for spring elasticity, η_k for damper viscosity, W_k as a weighting factor (ΣW_k = 1) and M as a maturity measure expressed in time units. The parameters were calibrated based on a 3D-back calculation of an uniaxial compression test which was carried out with 40% of the uniaxial compressive strength.

All shotcrete linings were modelled with thin shells. These thin shell elements are divided into ten layers during calculation for appropriate stress integration. The corresponding thickness according to the design was assigned to the different tunnels during modelling (Figure 6).

4.2.3 *Modelling of Compensation Grouting*

To evaluate the influence of the compensation grouting on the tunnel lining and the existing development, the planned compensation grouting measures were modelled in a discrete way. Numerical modelling of Compensation Grouting is already described in numerous scientific articles (Schweiger et al. 2004; Ali et al. 2010). Thereby, different approaches of applying stresses or strains to the appropriate elements are taken.

In the present work volumetric strains were applied to selected elements representing the zone of compensation grouting. Two different variants of compensation sequences were investigated. Each variant consists of 2 phases. At variant 1 a pre-heave is applied in the first phase (1 % volumetric strain) and an additional heave of 0,5 % in the second phase before the excavation of the cross passages and the escalator tunnels. At variant 2 the contact injection is modelled in the first phase (0,5 % vol. strain) and all the necessary heave is applied in a second phase after excavation works are finished (2 % vol. strain). The evolution of imposed strains in time is driven by a load time function, so that applying of the strains is modelled linearly over a time span of 15 days. Afterwards, the imposed strains are kept constant. Generally, the area of compensation grouting was divided into a core zone with 100 % applied strains and a peripheral zone with only 50 %.

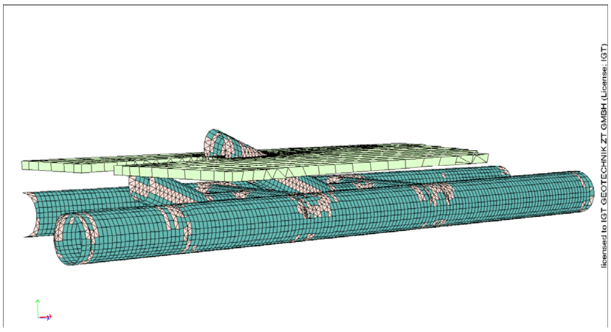

Figure 4. Elements for Compensation Grouting and SpC shell elements

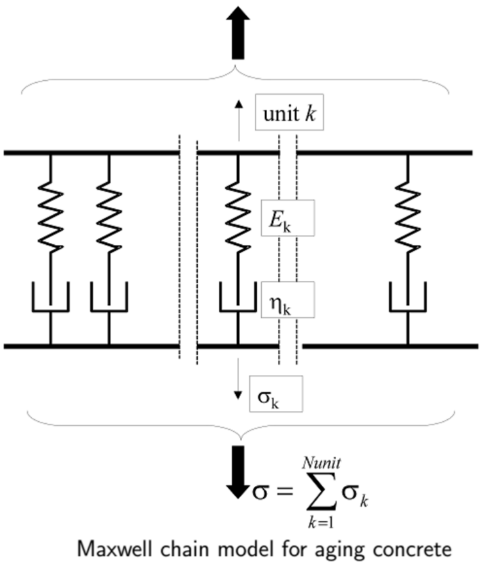

Figure 5. Modelling of aging concrete with parallel Maxwell units

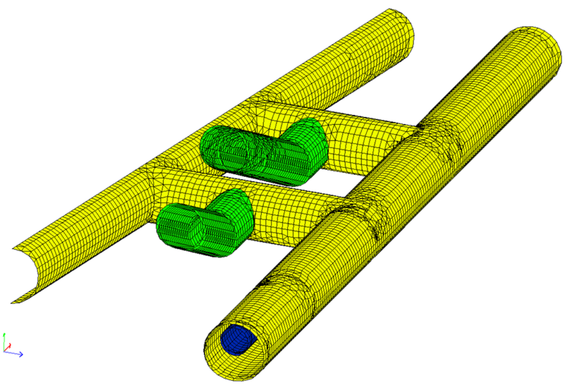

Figure 6. Shell elements representing shotcrete of all tunnel drives

The stiffness of the relevant elements is increased immediately prior to the first application of strain. Therefore, a material replacement must be performed to change the material model from HSS to a linear elastic – ideal plastic behavior with a failure criterion according to Mohr- Coulomb. An increase of strength of the elements is not considered as a conservative approach. To handle this material replacement a construction stage analysis is set enabled by a special incremental procedure to reproduce the correct and appropriate stress state in the relevant elements.

4.2.4 *Calculation steps*

In order to account for time-dependent behavior resulting from consolidation due to the presence of groundwater, the need of investigating the efficiency of the pilot tunnel on groundwater relaxation and to take the rheological behavior of shotcrete into account the calculation must be performed in real time steps. The duration of single construction steps and excavation velocities was specified on empirical values. Consolidation was considered by using a fully coupled system to capture the influence of deformations on the pressure field and vice versa. Therefore, time stepping during calculation has a big influence on the results and conclusions. To model the dewatering in the shaft vicinity, a permeable shotcrete shell and the free surface at the tunnel face, where groundwater can leak out, seepage elements were considered at appropriate time steps.

The in-situ stress state is calculated under consideration of the surface loads representing the existing buildings and traffic load. For each ground type the related lateral stress coefficient was set. Additionally, pre-overburden pressure was applied to the Miocene ground type. The further calculation steps contain the excavation of the vertical shafts Lindengasse und Kirchengasse consisting of bored piles, struts and base plate. Afterwards the first step of compensation grouting is performed. Then the two pilot tunnels are excavated. The remaining tunnel drives are considered in the appropriate time sequence according to the design.

5 RESULTS OF INVESTIGATIONS

Based on results of simplified numerical calculations it was possible to minimize the areal amount of the compensation grouting measures already in an early project stage. During detailed design the aim of the investigations was to determine settlements on the surface respectively the existing building, to assess the required shotcrete thicknesses for the individual tunnel drives and to determine displacements of the shotcrete shells. Furthermore, thresholds of the settlements and displacements had to be established within the scope of the Geotechnical Safety Management Plan according to ÖGG (2023).

When analyzing the results after the excavation of the pilot tunnels a water table drawdown in the immediate vicinity of the pilot tunnel can be observed (Figure 7). Due to the low permeability of the Miocene and the determined time stepping a steep drawdown cone occurs. Therefore, the aim of the pilot tunnel to obtain groundwater relaxation is achieved and the pilot tunnel fulfills its task.

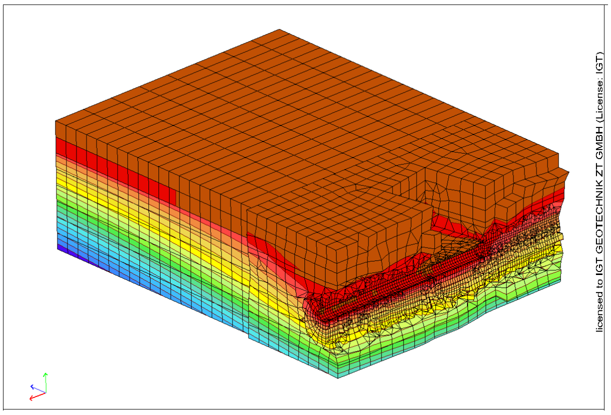

Figure 7. Pore pressure after excavation of pilot tunnels (qualitatively)

Figure 8 shows the calculated settlements on the surface based on variant 2 of compensation grouting when all tunnel drives are excavated before the second phase of compensation grouting is applied with reference time step after shaft excavation. As expected, the biggest settlements occur above the junction area of the tunnels and amount to approximately 35 mm. Imposing 2 % volumetric strains in phase 2 of the compensation grouting routine a heave of up to 38 mm occurs in the core zone (Figure 9). Therefore, the settlements due to excavation are compensated. For dimensioning of the tunnel lining this represents the most decisive load case. As is shown for a selected surface point above the station tunnel (blue line in Figure 10) imposing strains in phase 1 results in a heave of app. 10 mm.

The dimensioning of the shotcrete shell of the main tunnels could be performed successfully according to ÖGG (2018) even with comparatively low reinforcement content, moderate thicknesses and common concrete class. The fiber reinforced shotcrete shell of the pilot tunnels was dimensioned based on the guideline for fiber reinforced concrete of the German Committee for Structural Concrete (DAfStb, 2012) knowing shotcrete is excluded from this guideline. Experience has shown that this approach can be used successfully.

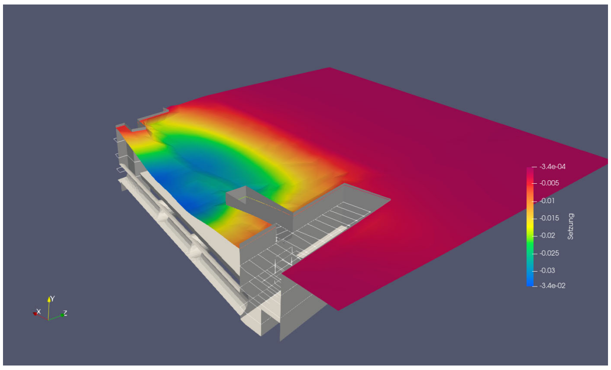

Figure 8. Calculated settlements on surface after excavation

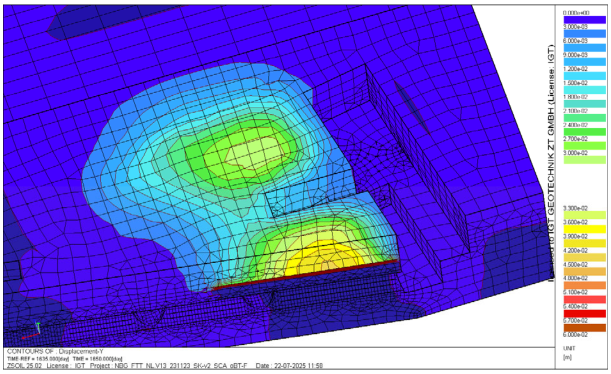

Figure 9. Resulting heave due to compensation grouting in phase 2

Based on dimensioning results, exceptions for compensation grouting works were defined to prevent local cracking of the shotcrete in the escalator tunnel, where it breaks through the compensation grouting layer.

6 COMPARISSON WITH OBSERVED SYSTEM BEHAVIOR

In the construction phase the excavation speed of the pilot tunnel was increased in the numerical model due to a request of the construction company. The aim was to check if an overutilization of the shotcrete takes place due to higher loading and lower concrete strength. The verifications were successful, and the excavation speed was implemented. During excavation of the pilot tunnel no sand layers with confined water were encountered. Therefore, no systematic dewatering measures for ground water relaxation ahead of the tunnel face were necessary. Nevertheless, decisive findings for the future excavation of the main tunnels could be made due to additional geological and geotechnical investigations. Based on the improved decision bases the excavations in construction Lot U2/21 could be finished successfully in August 2025.

Comparing the calculated with the measured settlements (orange line in Figure 10) on the surface matched to the beginning of excavation in the pilot tunnel shown in Figure 10 it gets obvious, that calculated and observed settlements are very similar for the excavation phase. Nevertheless, a difference occurs during shaft excavation and accompanying dewatering. As is shown, observed settlements amounts up to 20 mm whereas the calculated ones only show a few millimeters. It must be noted that shaft excavation in the numerical model lasts approximately twice as long as in reality. Therefore, the difference can be attributed to differing soil properties such as permeability and stiffness. When analyzing the evolution of the observed settlements it gets obvious that compensation grouting was only performed to a low extent in the subject area. Subtracting the primal heave due to phase 1 of compensation grouting in the calculation, resulting settlements are very similar in the calculation and reality. This may be caused by the lower permeability of the modelled soil (sand layers were not modelled discretely) and ongoing dewatering during excavation within the calculation whereas dewatering in reality is mainly determined by the thickness and lateral extend of these sand layers. Investigations of the water content in the Miocene layer of in situ soil samples and of soil samples freshly extracted from the tunnel face showed similar values.

Figure 11 shows a comparison of observed and calculated displacements of the pilot tunnel and the station tunnel. It becomes obvious that the chosen material model aging concrete with the selected parameters represents the reality very close. It has to be mentioned that the amount of observed displacement depends very much on an early zero reading of the installed measuring points. Measurement precision must be limited by ensuring adequate boundary conditions such as air and target quality.

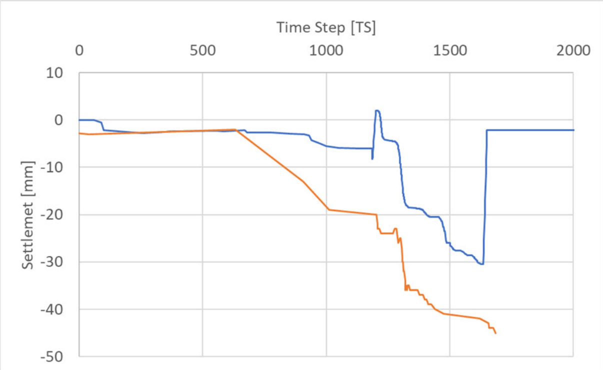

Figure 10. Settlement on selected surface point

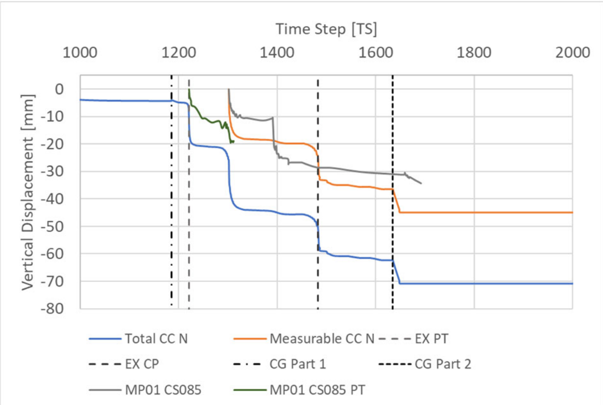

Figure 11. Vertical Displacement in Station Tunnel (CC N, EX…Excavation, CP…Cross Passage, PT…Pilot Tunnel, CG…Compensation Grouting, MP…Monitoring Point, CS…Monitoring Cross Section)

7 CONCLUSIONS

The current numerical investigations were a very valuable tool for verification of different construction methods under complex boundary conditions. To get the most realistic results and ensure a safe and economical construction material models for soil and construction materials play a key role. Furthermore, sophisticated material models guarantee saving resources, which nowadays is a main task in the design phase. For future construction projects in similar ground conditions back calculations of the observed system behavior must be performed to complete results of ground investigations and ensure most accurate results. Regarding to the boundary conditions in the project area the pilot tunnels represent a very useful tool for additional ground investigations and reducing water pressure with decreased risk when encountering sand layers containing confined ground water.

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