SETTLEMENT PREDICTIONS AND SURVEILLANCE DURING THE ENLARGEMENT OF THE METROSTATION MARIENPLATZ IN MUNICH

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Abstract. In response to increased passenger capacity requirements for public transport in Munich and with special regard to the 2006 Football World Cup, it was decided to enlarge the metro station Marienplatz, first taken into service in 1971. The finally approved concept provided for two extra tubes in parallel to the existing ones. By connecting the new and old tubes each with eleven cross cuts, the intended approximate doubling of the passenger platforms was achieved.

Because of the close vicinity of the new tunnel tubes to existing buildings – in particular the historic city hall situated right above – an innovative brine freezing technique was adopted in order to reduce the settlements to an absolute minimum. Starting from pilot tunnels, the soil body above the crown region of the new tunnels was frosted, providing both tunnel face support and a watertight shield towards the ground water above.

To predict the tunnel induced deformations and settlements, the tunnelling process was simulated by means of two-dimensional continuum finite element models of representative cross-sections under plain strain conditions. Three-dimensional redistribution effects were accounted for using the well established “α”- or “stiffness reduction” method. To consider the important influence of soil-structure interaction, a specific material law was adopted, that could be calibrated according to triaxial and oedometric test data and that accounted for different stiffness in primary and un-/reloading stress paths. The simulation of the mechanical behaviour of the frozen soil with respect to soil heaving and the generation of constraint stresses in the soil skeleton demanded additional considerations.

During the tunnelling process extensive accompanying measurements were undertaken. The soil freezing, the tunnelling process and the installation of the cross cuts caused extraordinary little surface settlement. Comparing the measured data with the predictions of the finite element analysis, a good correspondence could be identified.
1 INTRODUCTION

1.1 Background

The metro station Marienplatz in Munich was taken into service in 1971. The station consists of two separate platform tunnels situated right beneath the historic city hall. It is the most important station of the north-south metro lines 3 and 6 for passengers to and from the city center and at the same time provides connection to Munich's suburban trains. Furthermore the metro line 6 is the only public means of transport for the new Munich football arena that was taken into service in 2005. With special regard to the 2006 Football World Cup and in response to increased passenger capacity requirements for public transport in general, the Munich city council decided to enlarge the metro station Marienplatz.

The finally approved concept enlarges the existing passenger platforms by means of two new tunnels in parallel to the existing ones. New and existing tunnel tubes are connected each with eleven approximately 3.10 m wide cross cuts, by this assuring the intended approximate doubling of the platform passenger capacity.

1.2 Construction

In the beginning of the construction works for each of the two main tunnels a 30 meter deep pit in form of a shaft directly in front of the northern facade of the city hall has been developed. From these pits the pilot tunnels were driven in a depth of 17 meters under the surface for the designed soil freezing and the necessary drillings. At the bottom of the shaft in a depth of 30 m the tunneling works started. During the whole construction time all the necessary serving and disposal of bits and pieces of the tunnel construction site could be handled through out the pit shafts.

The new pedestrian tunnels – with an approximately 55 m² face in cross section were driven in traditional mining with two direction changes and direct contact to the existing tunnels. After finishing the tunneling works, eleven passes each between the old and new platform tunnel and a frontal connection to the existing transport building underneath the Marienplatz was carried out with required supporting measures for the existing structures. The construction work was finalized with the integration of the water pressure resisting inner lining of the new pedestrian tunnel as well as the lining of the shaft.

1.3 Involved parties

The successful cooperation of all parties involved in planning and construction was crucial for the realization of the very difficult task: The innovative proposal of the contractor Max Bögl was developed in cooperation with the engineering companies Bayreuther (geo technology and dewatering), Dr.-Ing. Orth GmbH (expert subsurface freezing), Univ.-Prof. Dr.-Ing. H. Schulz (general expert), planned and designed by the engineering company Schmitt Stumpf Frühauf and Partner (design and final planning) and commissioned by the main subway department “U-Bahn-Bau” of the City of Munich. Additionally involved were the proof engineer Dr.-Ing. Helmut Kupfer and the general expert of the client Mr. Dipl.-Ing. Paul von Soos. Parts of the monitoring program and accompanying tests concerning subsurface freezing were carried out by the chair of geo technology of Univ.-Prof. Dr.-Ing. Norbert Vogt at the Technical University of Munich.

Each single construction method used for the platform extension at the subway station Marienplatz in Munich is state of the art – but its combined application to cope with the complexity of the construction tasks was innovative and remarkable.
2 GEOLOGICAL AND HYDROLOGICAL SITUATION

The geological and hydrological conditions beneath the city hall are very well explored both from the first tunnel driving process (1967-1969) and recent drillings and pumping tests. The top layer of fine to coarse quaternary gravels some 2 to 5 m thick is way above the driving level. The two new tubes with approximately 7 m diameter are located about 25 m under the surface. The geological situation in-situ is characterised by alternating layers of tertiary sands and clays. The borders between these layers fluctuate greatly. As a result the roofs of the new tunnel tubes are mainly located in the lower tertiary clay layer, but are also locally touching the upper sand bed carrying unconfined groundwater. The floor of the drives is largely cutting into the lower sand bed which is subject to confined groundwater conditions.

3 COMBINATION OF SUITABLE CONSTRUCTION METHODS

3.1 Excavation shafts for launching of the tunnel works

The excavation shafts located directly in front of the northern façade of the city hall were used during the whole construction time for every kind of supporting, supplying and disposing work. The water resistant excavation pit of the shaft was constructed by contiguous bored piles. The diameter of the piles was 1.20 m, the distance in axis was 1.00 m. In the floor plan the piles were positioned in an oval form. This form allows to omit any braces in the excavation as the forces can be hold by the oval ring alone. The excavation works had to be stopped for the starting face of the pilot tunnels in the depths of approximately 17 meters under the surface. After the driving of the pilot tunnels, the shafts could be sunk down to the floor level of the new tubes. Due to the design of the special form of the shaft, the requirements for op-
timal stiffness combined with maximum working space in the shaft and a quick construction process could be fulfilled.

3.2 Pilot tunnels

The pilot tunnels above the crown of the main tubes were carried out with the pipe driving method using a compressed air-hooded shield. The diameter of the tunnels was 2.00 meters. The tunnelling started from an intermediate excavation level of the shaft. Out of the pilot tunnels the required drillings for the soil freezing lances were performed. The direction or length of the lances could be easily adapted to the existing building structures.

3.3 Soil freezing

The frozen soil body above the crown region of the new tunnels provided both tunnel face support and a watertight shield towards the ground water. Due to the separation of advance support and driving, a fast and continuous drive was possible. A spacious lowering of the ground water level was avoided and only temporary ground water relaxation in the lower sand layer was required.

No other method of face support or drainage system would have been adapted as exactly to the existing structures as the frozen soil. To realize the special geometric “umbrella” form of the frozen soil body using many single drillings fitted with icing lances, three-dimensional design drawings had to be done before. Sometimes under construction while drilling some obstacles occurred. In this case the drilling for the icing lances had to be stopped and a new position for the drilling had to be found as substitute. This re-design had to be carried out immediately and parallel to the construction work. Despite of the obstacles which occurred
while drilling, the target was reached. Due to the quick and exact re-design methods it was secured that a continuous frozen soil in exact geometric from was developed.

3.4 Driving

Under the shelter of the frozen soil body the tunnel driving process could be performed with a minimum of supporting measures. The shotcrete drive was executed as a full-face excavation with advance crown. The excavated cross-section was about 55m². The length of the advance in the crown amounted to 1m, in the bench and floor to 2m. For driving a tunnel excavator and an excavation attachment were used. The static shotcrete thickness was 30cm, with additional 3cm “sacrificial layer “in direct contact to the frozen roof. Four-belt trapezoid lattice arches were installed in the crown. The shotcrete lining was reinforced in the wall and floor with Q 188.

4 FINITE ELEMENT SIMULATION

4.1 Requirements

To predict the tunnel induced deformations and settlements with sufficient accuracy, the tunnelling process needs to be captured by adequate finite element computations. During the tunnelling procedure significant stress redistribution in the surrounding soil skeleton takes place, interacting with the stresses and deformations of the shotcrete outer lining. Therefore, the realistic simulation of the actual soil-structure interaction is of decisive importance and demands for the adoption of a suitable material law. Due to the inherent nonlinear character of the analysis it is obligatory to simulate every subsequent step in the loading history of the construction process. At the same time it is important to be able to perform parameter studies within a reasonable timeframe.

4.2 Solution strategy

To consider the important influence of soil-structure interaction, the soil body was modelled as a nonlinear continuum adopting a specific material law (Granular Hardening). This material law could be calibrated according to triaxial and oedometric test data and it accounted in particular for different stiffness in primary and un-/reloading stress paths.

![Triaxial response "Granular Hardening"](image)

Figure 3: Characteristic stress path for triaxial loading with un-/reloading sequence

The tunnelling process was simulated by means of classical two-dimensional continuum finite element models of representative cross-sections under plane strain conditions. In reality, not only stress redistribution in the cross-section plane can be observed. In the working face
region stress redistribution effects in the third direction along the tunnel axis take place, as well – resulting in a load release of the working face section. The two-dimensional simulation accounted for this effect by means of the well-established so-called “α”- or “stiffness reduction” method. This method involves an artificial relaxation of the sections of the model that are going to be excavated, i.e. removed from the model, during the next step – thereby the relaxation reflecting the load release effect due to the three-dimensional stress redistribution at the working face. The amount of artificial relaxation is subject to calibration and controlled by the relaxation factor $\alpha$.

The stiffness-reduction method represents an approved, fast and efficient simulation approach, that yields sufficient accuracy and allows parameter studies within a reasonable timeframe. For Munich soil conditions it can be resorted to comprehensive experience made in the past with this method.

As outlined above, numerous nonlinear construction stages had to be considered in the analysis. In the order of the loading history of the construction process, the following steps were covered:

- In-situ stress state prior to construction activity
- Ground water relaxation (drainage)
- Installation of the pilot tunnel
- Soil freezing process
- Tunnelling process (excavation and subsequent securing by shotcrete outer lining)

![Figure 4: Concept of the $\alpha$-method](image-url)
Defrosting of frozen soil body
- Cross cut installation process
- Installation of inner reinforced concrete lining
- Drainage turning-off

Due to the inherent nonlinear character of the analysis it was obligatory to first generate a loading state of the model that represented the actual state prior to beginning of the construction activities. This was basically achieved by simulating the historic construction process of the existing tunnel tube – starting from a homogenous stress state. The so obtained loading state of the model then served as the starting point for the subsequent analysis of the tunneling process for the new tunnel tubes. From the complete analysis chain stated above, the aspects concerning the simulation of the soil freezing / defrosting with the impact of the frozen soil on the tunneling process and the simulation of the cross cutting process are highlighted in the following chapters.

4.3 Analysis principles for the freezing process and the behaviour of frozen soil

Starting from the pilot tunnel the cooling lances were driven towards the new tunnel location in a fan-like manner. They were passed through by –40°C cold brine, serving as coolant. By this, a pore-water freezing process in the adjacent soil was initiated, finally forming a wedge shaped frozen soil body above the location of the new tunnel tubes. The freezing process implicates a remarkable temporary strength and stiffness conditioning for the affected soil regions. On the other hand significant volumetric expansion can take place, which is, however, constrained by a limiting maximum compressive stress state.

<table>
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Table 1: Mechanical properties of frozen soil in-situ, Dr.-Ing. Orth GmbH

This mechanical behaviour had to be accounted for in the analysis and investigated with respect to permissible soil heaving and the generation of constraint stresses in the soil skeleton. For the actual configuration – due to the limiting compressive stress state – the freezing
affected sand layer did not experience volumetric expansion. Volumetric expansion was restricted to the clay domain and amounted to an effective value of 0.14%.

The volumetric expansion of the frozen clay region is of isotropic nature and therefore could be simulated in the analysis by an equivalent thermal supply for the affected finite elements up to reaching the limiting compressive stress state.

In analogy to the freezing process, the loss of strength and stiffness caused by the defrosting process of the frozen soil body – including a significant interference to the affected soil skeleton - also had to be simulated in the finite element calculations. Unlike the volumetric expansion, the contraction of the defrosted soil body is of anisotropic nature depending on the prevailing stress state within the affected region. This effect could be simulated with an artificial relaxation of the affected finite elements causing them to contract in a realistic way.

Figure 5: Expansion and contraction of the soil body due to freezing / defrosting (scale factor 400)

As the actual tunnel driving process was performed under the shelter of the frozen soil body, additional considerations concerning the calibration of the analysis by the relaxation factor $\alpha$ had to be made. Due to the stiffness conditioning of the frozen soil body a higher amount of three-dimensional stress redistribution was motivated – therefore, for the current tunnelling analysis with $\alpha = 0.2$ a rather low value was determined, effecting an 80% relaxation of the excavation sections prior to the actual excavation step.

### 4.4 Simulation of the cross cutting process

The cross cutting process between existing and new tunnel tube was performed in a step-wise manner, in particular required because the crown segments of the existing tunnel are not force-locked. Due to the complex constraints, dimensional reduction to a plane computational model was no longer considered feasible for a reliable prognosis of the structural response.

Aiming at the best compromise between efficiency and required accuracy, a novel strategy was adopted. The approach combined the benefits of a fast and clearly arranged two-dimensional analysis for the tunnelling simulation as described above with the capability of a three-dimensional model simulating the cross cut installation process. The connecting link between the two analyses was a procedure that not only generated the three-dimensional model from the two-dimensional one by a special extrusion technique but also mapped the results of the two-dimensional analysis to the three-dimensional model – so generating a suitable primary state for the three-dimensional model that ‘inherited’ the loading history induced by the tunnelling process.
The so-obtained pre-strained three-dimensional model then served as a starting point for the actual staged cross cut simulation.

5 RESULTS OF ANALYSIS AND MEASUREMENT

The tunnel driving process was accompanied by an extensive measurement and monitoring program. The settlements on the ground surface were permanently controlled with geodesic measurements and a special hydraulic hose leveling system that was installed in the cellars of the city hall. In addition, the deformations of the shotcrete outer lining were subject of regular convergence and geodesic measurements. Furthermore, the development of the normal forces in the shotcrete outer lining was controlled using strain gauges.

Comparing the measured data with the predictions of the finite element analysis, a good correspondence could be identified. This confirms validity of the choice of modelling parameters, particularly regarding the relaxation factor $\alpha$, the “Granular Hardening” material parameters and modelling of the volumetric soil expansion during the freezing process and the contraction when defrosting.

In accordance with the results of the analysis, the soil freezing, the tunnelling process and the installation of the cross cuts caused extraordinary little surface settlements, which verified the chosen tunnel face support and the other applied construction methods. At the same time, the frozen soil body above the crown of the new tunnels caused extraordinary high stresses in the shotcrete outer lining. Due to the stiffness conditioning of the frozen soil body, the resulting forces at the working face were concentrated in a shallow vault in the soil region above the crown. With growing stiffness of the shotcrete outer lining and because of the distinctive creeping behaviour of the frozen soil, the major part of the forces within the surrounding soil
skeleton shifted towards the shotcrete outer lining. This effect was increased by the defrosting process of the frozen soil body and the significant interference to the affected soil skeleton.

Figure 8: Measured and computed settlements and normal forces

Settlements caused by drainage of the lower tertiary groundwater level were measured smaller than 3-4 mm. Due to the mining of the pilot tunnel and the drilling of the icing lances displacement of 2-3 mm occurred.

The expansion of the soil body while freezing was restricted to the affected clay layer. The sand was able to drain the water and did not experience volumetric expansion. The existing compressive stress state in the affected soil skeleton had a positive effect on the limitation of the heaving. The result of the soil freezing was a small elevation of 3-4 mm on the surface.

Only 3-4 mm settlement resulted out of the actual tunnel driving. The defrosting process of the frozen soil body had an additional effect of 3mm sinking. The cross cutting process between existing and new tunnel tube was realized with almost no deformations.

Due to the innovative combination of the described measures the lasting surface settlements could be minimized to approximately 7 mm.

6 CONCLUSION

• According to the department “U-Bahn-Bau” of the City of Munich, extending the platforms at the Marienplatz Underground station was one of the most complicated construction projects in the history of the Munich underground system.

• With an innovative combination of adequate construction methods involving subsurface freezing as face support, the construction task was successfully realized with a minimum of surface settlements

• To predict the tunnel induced deformations and settlements, the tunneling process was simulated by means of two-dimensional continuum finite element models of representative cross-sections under plane strain conditions. The well established “α”- or “stiffness reduction” method was used to account for the three-dimensional stress redistribution effects at the working face.
• The realistic simulation of the mechanical behaviour of the frozen soil with respect to soil heaving and the generation of constraint stresses in the soil skeleton was crucial for the quality of the analysis results.

• A special combined mesh generation and mapping technique was applied to create a three-dimensional model that inherited the load history induced by the tunnelling process and thus allowed for simulation of the cross cut installation process.

• Comparing the measured data with the predictions of the finite element analysis, a good correspondence could be identified and confirmed validity of the choice of modelling parameters.

REFERENCES
