

# Design and construction of NATM underground station tunnel by using the forepoling method in difficult conditions for Athens Metro

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**ABSTRACT:** This paper describes the detailed engineering design process in terms of the relative geotechnical interpretation stage, the subsequent multistaged numerical simulation steps by using SOFISTIK software, the critical decision making points and the actual results achieved during the construction of Ag. Savas underground station tunnel for Athens Metro.

## 1 GENERAL DESCRIPTION OF AGIOS SAVAS UNDERGROUND COMPLEX

### 1.1 Project layout

Agios Savas underground complex is a part of the new extension LINE 3 of Athens' metro. The project is considered to be very complicated (Figs 1, 2), since it contains an underground station of 110m in length (1), the main building (2) close to the station tunnel and a cut&cover dome (3) which will be connected with the main building via an underground cross gallery (4). Furthermore, four underground inclined galleries (5) with stairs are foreseen for the transportation between the dome, the station and the main building.

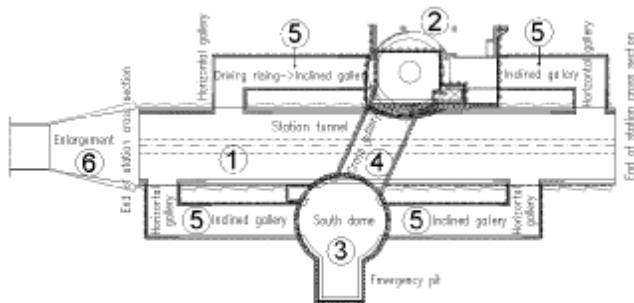


Figure 1. Plan view of Agios Savas underground complex at the final construction stage.

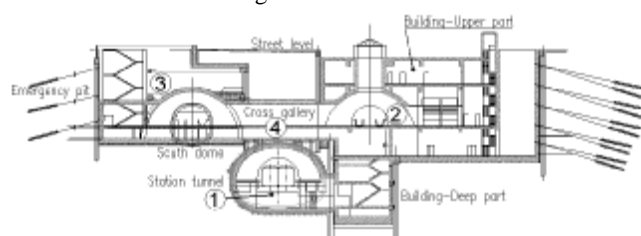


Figure 2. Section of Agios Savas underground complex. Station tunnel will be constructed in advance.

For the excavation and temporary support of the station tunnel, a forepoling method was applied in order to overcome the time schedule delays, which are generally produced by a typical side drifting method.

The piling work in the area of Agios Savas Station building pit began at January 2003 and the contractor J/V AKTOR SA-IMPREGILO SpA will fulfill the civil works until the beginning of 2008.

According to the construction sequence, the following stages are foreseen:

- Installation of the retaining system of the building pit and south dome (concrete piles and prestressed anchors) and partial excavation of the building pit.
- Excavation of the station tunnel with forepoling method and top heading and bench.
- Implementation of the final lining of the station tunnel and excavation of the deep part of the building pit with top-down method.
- Concreting works of the building (up to a certain level) and the dome.
- Construction of the remaining underground parts (inclined galleries and cross gallery).
- Concreting works of the remaining part of the building.

## 2 FOREPOLING METHOD OF THE STATION TUNNEL

### 2.1 Presentation of the temporary support system

The overburden thickness of the station's tunnel is approximately 18m. The geological formation at the depth of the station tunnel consists of conglomerates and marl with small layers of sand and clay. The

hydrogeological conditions are favourable for the project, in this specific area, since the inflow water quantities are rather neglectable.

Excavation is performed by the use of a hydraulic hammer. Old shafts have been discovered during the excavation of the station, since the area was continuously inhabited during the last 3000 years (considered to be a part of the old ancient Athens).

The excavated cross section of the station's tunnel ranges' between  $195\text{m}^2$  (at the minimum excavation) to  $215\text{m}^2$  (at the maximum excavation enlargement). The width of the top heading and bench is 20.2m, the total height of the tunnel is 14.25m (in the maximum enlargement), while the excavated height of the top heading exceeds 8m (Fig. 3).

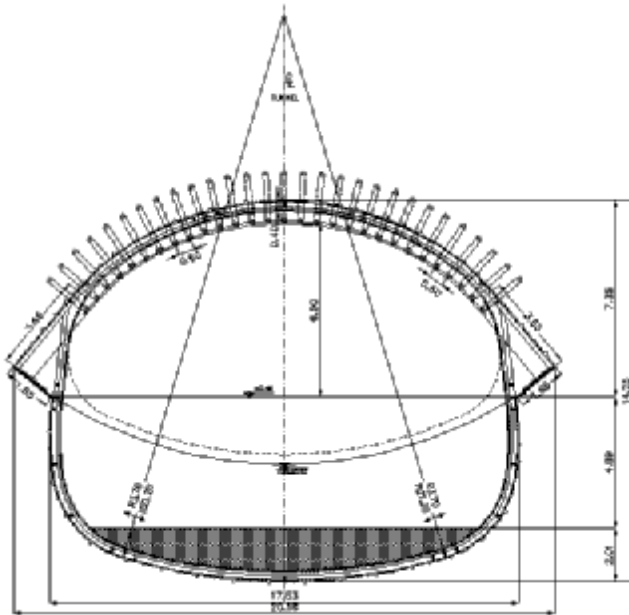


Figure 3. Excavation geometry of the typical cross section (maximum enlargement).

In order to implement the first series of forepoles, it was necessary to enlarge the running tunnel's cross section, thus creating the required space for installing the forepole tubes at the station's cross section (Part 6 in Figure 1). The enlargement must be gradually increasing from the running tunnel's cross-section to the station's cross-section, creating a stable and statically acceptable excavation. A transition part of the running tunnel has already been constructed from the running tunnel to the station. The length of the transition part was determined at 20m (Fig. 4). The excavated cross section of the transition part of the running tunnel ranges between  $91\text{m}^2$  (minimum excavation) to  $213\text{m}^2$  (maximum excavation enlargement).

The excavation and temporary support system is based on the following:

- Top heading round length 1.00m.

- Forepoling tubes St37,  $\Phi 193.7/179.5\text{mm}$ , 12m length, placed at average axial distances of 0.60m per 6 round lengths of top heading.
- Shotcrete shell of 40 cm minimum structural thickness, C25/30. However, in the junction areas of temporary invert and elephant feet the structural thickness requirement increases for providing a better structural system with smooth geometrical corners and for avoiding stress concentrations.
- Steel sets placed at each round length, HEB 180, St37.
- Rock bolts of 6m length, S500, 25mm in diameter, installed at a grid 1.00m x (each top heading round length) (cross x axial spacing) at the sidewalls of bench (except from the final and temporary invert) and pairs of rock bolts for steel sets secure.
- Reinforcement layers T188 mesh type, (external, intermediate and internal) are installed.
- Temporary invert with prefabricated steel reinforcement cages for faster installation and two layers of steel mesh T188 and shotcrete 35 cm thickness, C25/30.
- Reinforced final invert (with two steel meshes T188), shotcrete 40 cm minimum thickness C25/30.



Figure 4. Top heading construction of the transition section of the running tunnel.

Two (2) different support classes have been provisioned. The principal considerations for the design and determination of the main / composite / transitional excavation and temporary support classes pointed to:

- The homogenisation and compatibility between the excavation and temporary support classes relevant to their basic structural components (shotcrete shell, rockbolts grid, temporary and final inverts thickness). The excavation and temporary support classes determined according to this consideration are more flexible, so the

application of the composite and transitional excavation and temporary support classes is facilitated.

- Adjustment according to the ground conditions anticipated during the excavation, even if each excavation and temporary support class has a specific application range.



Figure 5. Installation of the forepoling tubes at the top heading of the Station tunnel.



Figure 6. Top heading excavation of the Station tunnel. Installation of the HEB 180 steel arches.

## 2.2 Special conditions and different support system implemented

Due to the particularities of each different part of the project, a possible alteration of the excavation and temporary support classes proposed was anticipated. Due to this fact, by evaluating the ground profiles, the hydrological regime and the probable difficulties expected to be faced during the excavation, additional ground improvement techniques were recommended (drainage holes, probable application of consolidation grouting) in order to provide the as secure as possible design in case of unfavorable conditions in the excavation face area.

The excavation of the station tunnel is influenced in specific areas by the presence of piles (South Dome and Building Pit) as well as by the presence of the prestressed anchors (Fig. 7). As for the station tunnel's excavation in the area the Deep building Pit piles, there are a lot of construction difficulties to be confronted, which concern mainly the demolition of some of the piles and the connection of some of the remaining ones with the temporary support shell of the station tunnel.

Furthermore, the presence of the deep Building Pit's piles at the north side of the station tunnel, prevents the construction of the typical elephant foot. Due to this reason, station tunnel's north side temporary support shell is connected to Deep Building Pits piles via a special structural connection system with the use of steel dowels, anchored on the existing piles by injection of special adhesives (Fig. 8).

The prestressed anchors of Building Pit's south side and the cross and the E/M Service openings (connected to the South Dome and Building Pit) are also presented.

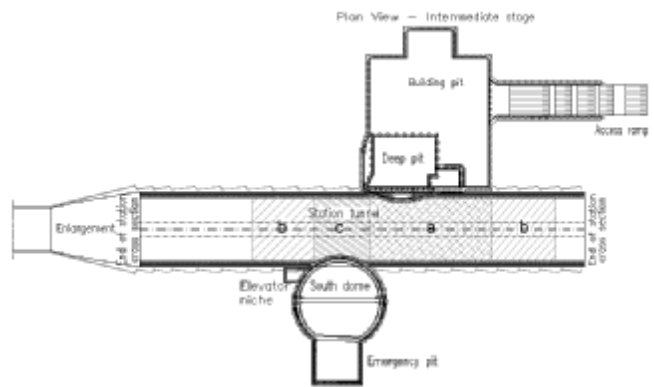


Figure 7. Plan view of the different parts of the station tunnel that are influenced by the neighbouring constructions of a) piles of the building pit, b) anchors of the building pit and c) piles of the south dome.

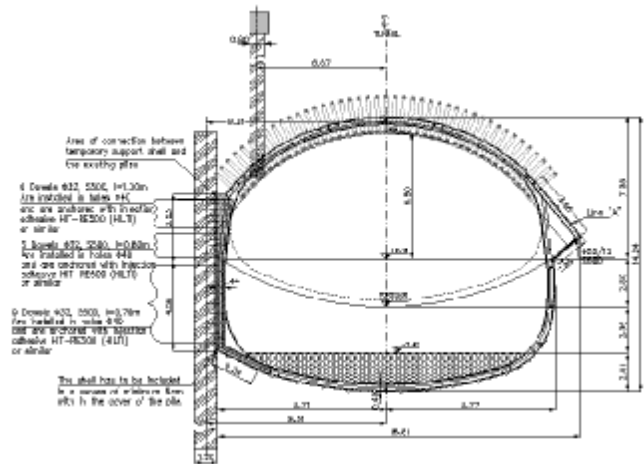


Figure 8. Connection details between temporary support shell and existing piles of the building pit.

According to the above-mentioned reasons and especially due to the presence of the Building Pit's and South Dome's piles at the face and at the crown of the station tunnel's excavation, there is a number of forepole tubes which cannot be normally installed. Instead of those forepole tubes, spiles have been installed, in order to avoid any interruption of the applied presupporting system in the station tunnel's crown.

### 3 CALCULATION PROCEDURE ACCORDING TO 3D MODELS

One of the most important parameters to be taken into consideration, during tunneling in urban environment, is the stability of the face of the station tunnel. By providing sufficient stability on station tunnel's face with the application of efficient temporary support measures, the construction of the project is secured and the surface settlements will be limited to their minimum values; thus the surface constructions (buildings) and relevant installations will be secured by irrecoverable damages.

In the present project detailed analyses have been performed by the use of a powerful computational code SOFiSTiK3D, in order to approach in general the effectiveness of the application of the support measures, the settlements along the station tunnel, the face stability and the rockmass relaxation at the excavation face area in relation to the distance of the face. By taking into account the results of the 3D model, the (stiffness core relaxation  $E$ /in situ ground stiffness  $E_0$ ) coefficient was determined, for each distinguished load case and excavation and temporary support class in the 2D analyses. The convergence of the rockmass as determined according to the 3D analysis was presented in special diagrams. The analytical dimensioning and structural adequacy check of each support class is performed according to the 2D analyses.

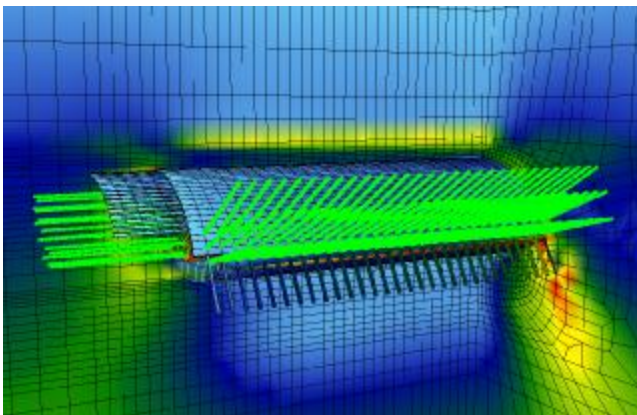


Figure 9. Analytical 3D model of the Station tunnel by taking into account the forepoles, the fiberglass bolts in the tunnel face, the shotcrete lining, the micropiles, the anchors, etc.

The 3D analyses in combination with the specific 2D analyses have been performed in order to ensure the requirements relevant to:

- Maximum surface settlements.
- Safety factor in the station tunnel's face.
- Safe construction sequence.

### 4 SETTLEMENT CALCULATION vs ACTUAL SETTLEMENT MEASUREMENTS

The surface settlements as well as the convergence and the adequacy of the temporary shell of the station tunnel were calculated according to detailed 3D analysis with sophisticated models (Fig. 9).

Figure 10 presents the surface settlements calculations, after 30m of top heading and bench excavation. During the construction of the Station tunnel and according to the measured values acquired for the top heading excavation, the predicted settlement values were significantly confirmed in a substantial part of the underground station.

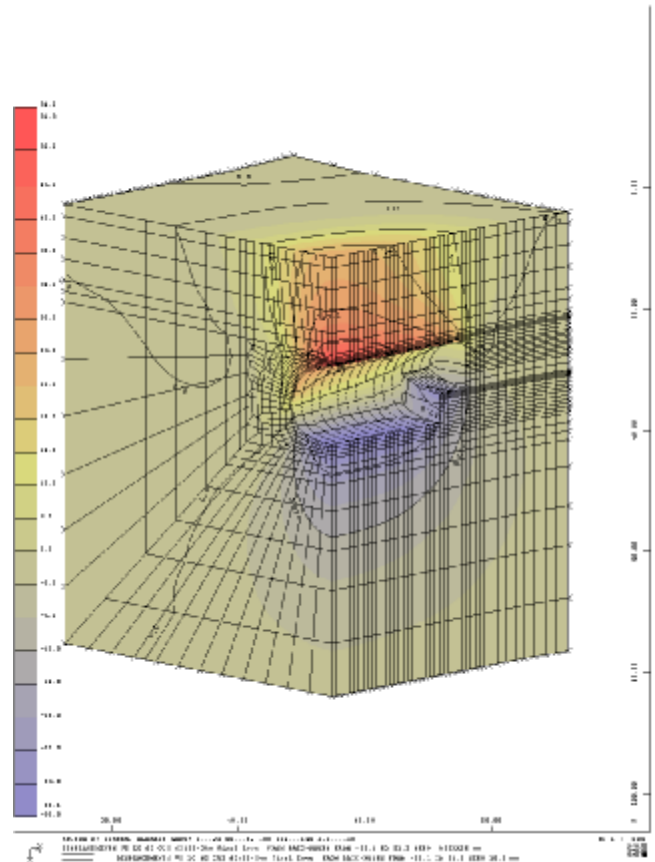


Figure 10. Surface settlement calculations after 30m of excavation and temporary support of the station tunnel.

The adequacy of the shotcrete lining, as well as, the settlements and the behavior of the surrounding buildings were intensively monitored by a dense



geotechnical system. Figures 11 & 12 present partial plots of the daily-evaluated data.

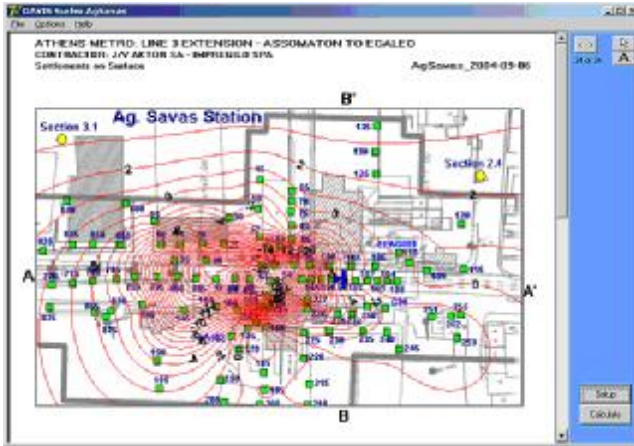


Figure 11. Contour plots of the surface settlements measurements after 50m of excavation and temporary support of the station tunnel.

According to the results of the 3D analysis concerning the excavation and temporary support classes, important conclusions have been arisen regarding the station tunnel's face stability and the surface settlements calculations.

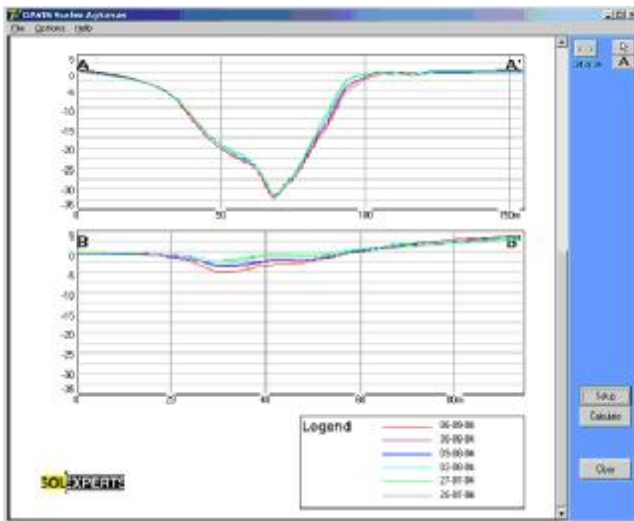


Figure 12. Plot curves of the surface settlements measurements after 50m of excavation and temporary support, in the longitudinal section AA and the transverse section BB of the Station tunnel.

The most important temporary support measures for the stability of the station tunnel's face and the restriction of the surface settlements are the fiberglass bolts and the forepoles (steel tubes). It's a fact that the fiberglass bolts combined with the forepoles reduce not only the extent of the plasticity zone, in front of the station tunnel's face, but also the magnitude of the surface settlements.

The proper cooperation of the fiberglass bolts and the forepoling umbrella vanishes the displacements

of the face, the magnitude of the surface settlements and decreases the probability for chimney development. From the above mentioned, the most important aid is the determination of the appropriate number of the fiberglass bolts applied in the station tunnel's face and the appropriate number of forepoles applied in the station tunnel's crown.

## 5 CONCLUSIONS

The station tunnel design and construction is based on the application of NATM tunnelling, in two discrete stages (top heading and bench) and on the systematic use of grouted forepoling umbrellas. Special problems encountered during the design refer to the close proximity of the station tunnel to the adjacent building pit and the auxiliary galleries and shafts of the station complex, as well as the structural and geometrical efficient transition from the smaller running tunnel section to the required profile of station tunnel.

The tunnel alignment underpasses a significant Athens avenue with heavy traffic and the influence zone of the tunnel includes several commercial buildings and manufacturing sites. There are very few station tunnels in urban areas under such low cover (approx. 18m) worldwide to be designed and constructed by this method (without any type of side drifting) under such strict construction sequence, time, traffic, settlement and geotechnical constraints.

The choice of the forepoling method has proven to be successful for the project, since the time needed for the construction of the top heading was 5.5 months and the total estimated time for the full cross section of the station will be less than 10 months, despite the specific modifications of the forepoling method that was necessary in different parts of the project.

The safe construction of the station tunnel via forepoling method and the limitation of the surface settlements could be provided by the control of the station tunnel's face stability in combination with an adequate shotcrete shell and an immediate construction of the temporary invert (in the present design this construction sequence was applied). Guidelines relevant to the better use and cooperation of the applied support measures in the station tunnel's face are also provided and concern the following:

- The maximum forces on the forepoles/spiles applied in the station tunnel's crown, depend on the round length, their foundation conditions and mainly on the deformability and strength of the station tunnel's face.
- By supporting the station tunnel's face with fiberglass bolts, its strength is improved, so the opening, the deflection (sagging) and the bending moment of the forepoles/spiles is decreased.

- The stresses of the forepoles/spiles can be restricted by increasing their quantity or by applying additional support to the station tunnel's face.
- The fiberglass bolts grid may in each case become denser according to the support requirements met during the excavation, but the immediate increase of the forepoles / spiles is not always possible.
- The in situ recognition of the geomechanical behavior of the weak formation is of great importance and also critical for the normal construction process and the restriction of the surface settlements.

## REFERENCES

- Omikron Kappa Consulting Ltd and Ingenieurburo EDR GmbH, March 2003, Agios Savas Emergency exit and South Dome Pit, Temporary Retaining Structures.
- Omikron Kappa Consulting Ltd and Ingenieurburo EDR GmbH, October 2003, Agios Savas Building Pit, Temporary Retaining Structures.
- Omikron Kappa Consulting Ltd, February 2004, Excavation and temporary support of the transition part of the running tunnel, KP 2+180.32 to KP 2+200.32.
- Omikron Kappa Consulting Ltd, March 2004, Excavation and temporary support of the Station tunnel, KP 2+200.32 to KP 2+312.19