NUMERICAL MODEL OF SCL TUNNELS IN COMPLEX SUBSOIL CONDITIONS

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Abstract

The paper presents numerical analysis of two similar tunnels constructed using Sprayed Concrete Lined (SCL) technology. The structure of interest is a part of the Fővám Square station of the 4th metro line in Budapest. At present that structure is finished. All data from geotechnical monitoring has been gathered and is a solid base for further analysis. Geotechnical conditions examined in site investigation turned out to be highly complex with many fault zones, over consolidated soil and high value of pore water pressure.

To verify numerical model, reliability evaluation was performed. Reliability verification was based on the comparison of results obtained from numerical and data from geotechnical monitoring [1].

Streszczenie

W artykule przedstawiono analizę numeryczną dwóch bliźniaczych tuneli w obudowie z betonu natryskowego (SCL). Konstrukcja poddana analizie jest częścią stacji Fővám, czwartej linii metra w Budapeszcie. Aktualnie konstrukcja jest ukończona. Dane uzyskane w wyniku monitoringu geotechnicznego stanowiły podstawę do dalszej analizy. Warunki geotechniczne uzyskane w wyniku badań polowych i laboratoryjnych zostały sklasyfikowane, jako bardzo skomplikowane z występującymi licznymi strefami uskoków, prekonsolidowanym gruntem oraz dużą wartością ciśnienia wody w porach gruntowych.

W celu weryfikacji sporządzonego modelu numerycznego przeprowadzono ocenę rzetelności w/w modelu na podstawie porównania wyników otrzymanych z modelu numerycznego do danych otrzymanych podczas budowy w wyniku szeroko zakrojonego monitoringu geotechnicznego [1].

Keywords: **Sprayed Concrete Lined tunnels; Numerical Modelling; Fault zone; Complex Geotechnical Conditions.**

1. INTRODUCTION

Currently Budapest has got three metro lines where the first one is the oldest metro line in continental Europe. Since 1972 the fourth line has been planned. 3.7 mln people use public transport in Budapest every day [2]. New metro line will cut travel time for each passenger by around 10 minutes. There will be two main connections that will allow a change of metro line: in Kálvin square to the third line and in Keleti railway station to the second line. An overview of the 4th metro line with connections to the two other metro lines is shown in Figure 1. Almost all kinds of structural works have been finished; only the interior works need to be done. In general, the 4th metro line is divided into 3 construction stages; currently the 1st stage is under construction.

Figure 1.
Section of the 4th metro line in Budapest [3]

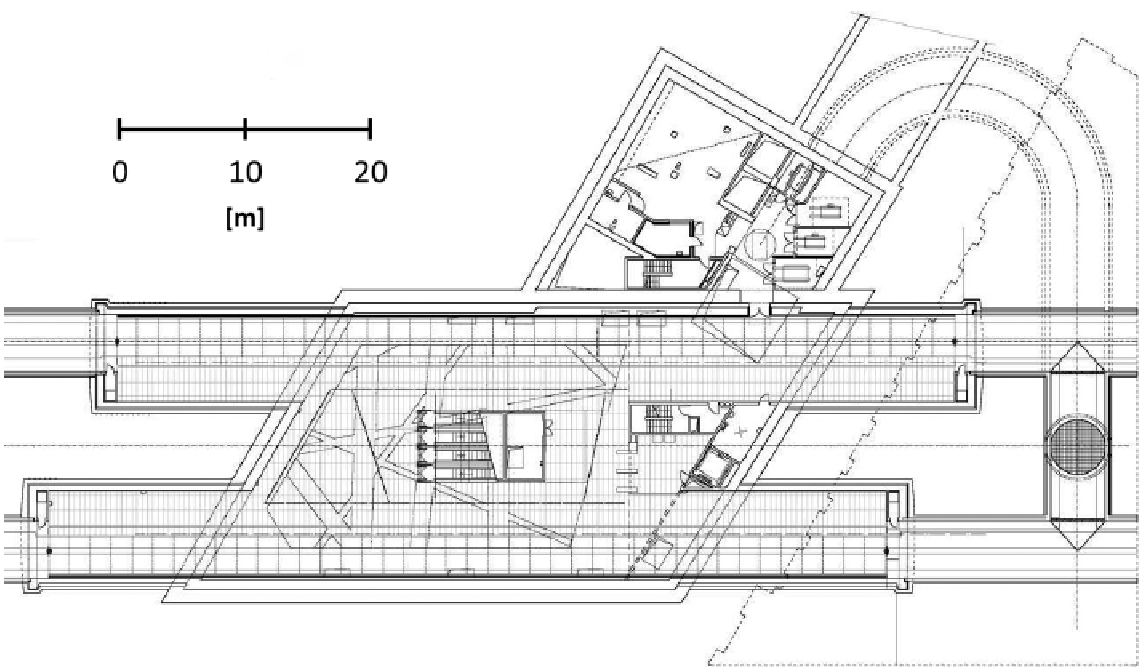

Figure 2.
View of the Fővám Square station [4]

The objects of interest are two SCL Tunnels being a part of the Fővám Square station of the Budapest 4th metro line. SCL Tunnels are a part of TBM tunnels. SCL Tunnels allowed TBM to be safely connected to the station structure. A view of the whole Fővám Square station is shown in Figures 2 and 3.

The Fővám Square station is the deepest metro station in Budapest and it is located in the Pest side of the city. Each direction is operated by different tunnel as can be seen on the view shown in Figure 2. The tunnels were driven with the use of a 5.2 m diameter Tunnel Boring Machine (TBM). Earth Pressure Balance Tunnel Boring Machine (EPBM) was used only during excavation of the tunnels under Danube River. Before TBM could reach the structure of metro station, the SCL Tunnel for each line was constructed. SCL Tunnels were executed with traditional mining technology.

Four separate geotechnical expert opinions were performed for better understanding of geotechnical conditions in the area of the Fővám Square station. These expert opinions allowed to reduce the risk of unexpected problems and mistakes.

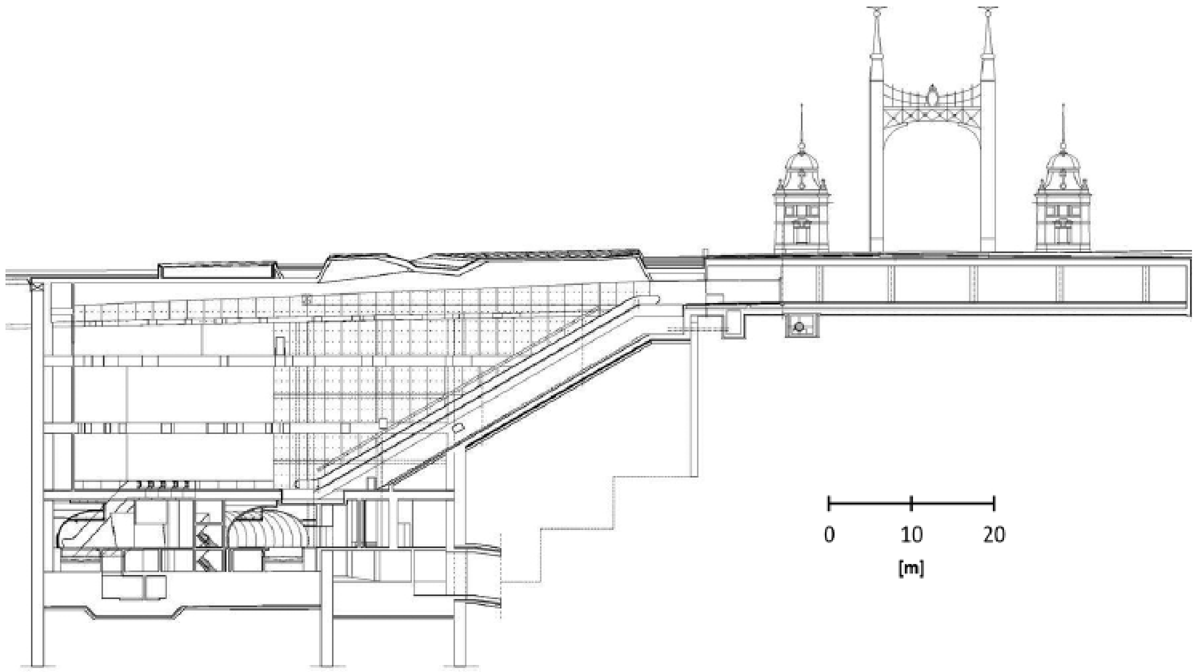

Figure 3.
Cross section of the Fővám Square station [4]

The tunnels were excavated in tertiary complex soils. Subsoil was divided into the following categories (directly affecting the tunnels) as a result of site investigation [5]:

- Ol3 – formations of the Chattian stage (Oligocene);
- Ol2 – Meso-Oligocene formations:
- Cleaved/weathered zone of Kiscell Clay;
- Fractured, fissured, expanded and intact zone of Kiscell Clay;
- Ol1 – Lower Oligocene formations;
- TM – tectonized zone;
- Material of faults to be taken into consideration in both the Ol2 and Ol3 formations.

The tunnels were constructed using Sprayed Concrete Lined technology. To limit ground deformations, excavation of the tunnel face was divided into several specific sections. Two sections formed one tunnel tube: the first section which created a drilling chamber and the second to reach the final length of the tunnel. Drilling chamber served as ground strengthening with the use of nitrogen freezing. Each tunnel tube section was divided along the tunnel axis into a side drift and an enlargement. In the first step the side drift for the first section of each tube was constructed succeeded by the enlargement. Additionally, the tunnel face was horizontally divided into two parts: a top heading with bench and an invert. A detailed plan of the construction sequence is presented in Figure 4.

After construction of the SCL Tunnels (primary lining), a secondary lining (permanent lining) was constructed. The primary tunnel lining was designed for 2-year service life. Regular cross-section of the North and South Tube is shown in Figure 5. Minimum shotcrete thickness was 0.35 m and the first outer 50 mm were considered as an insulation layer and were not taken into account during structural design. Shotcrete thickness varied from 0.35 to 0.45 m on the sidewalls, on the head wall shotcrete had a thickness of 0.45 m.

During construction of the SCL Tunnels of the Fővám Square Station the following geotechnical monitoring was involved:

- Displacements of diaphragm wall,
- Displacements of soil during consolidation,
- Development of stresses in the tunnels lining,
- Convergence of tunnels,
- Water pressure,
- Water temperature,
- Soil temperature during nitrogen freezing.

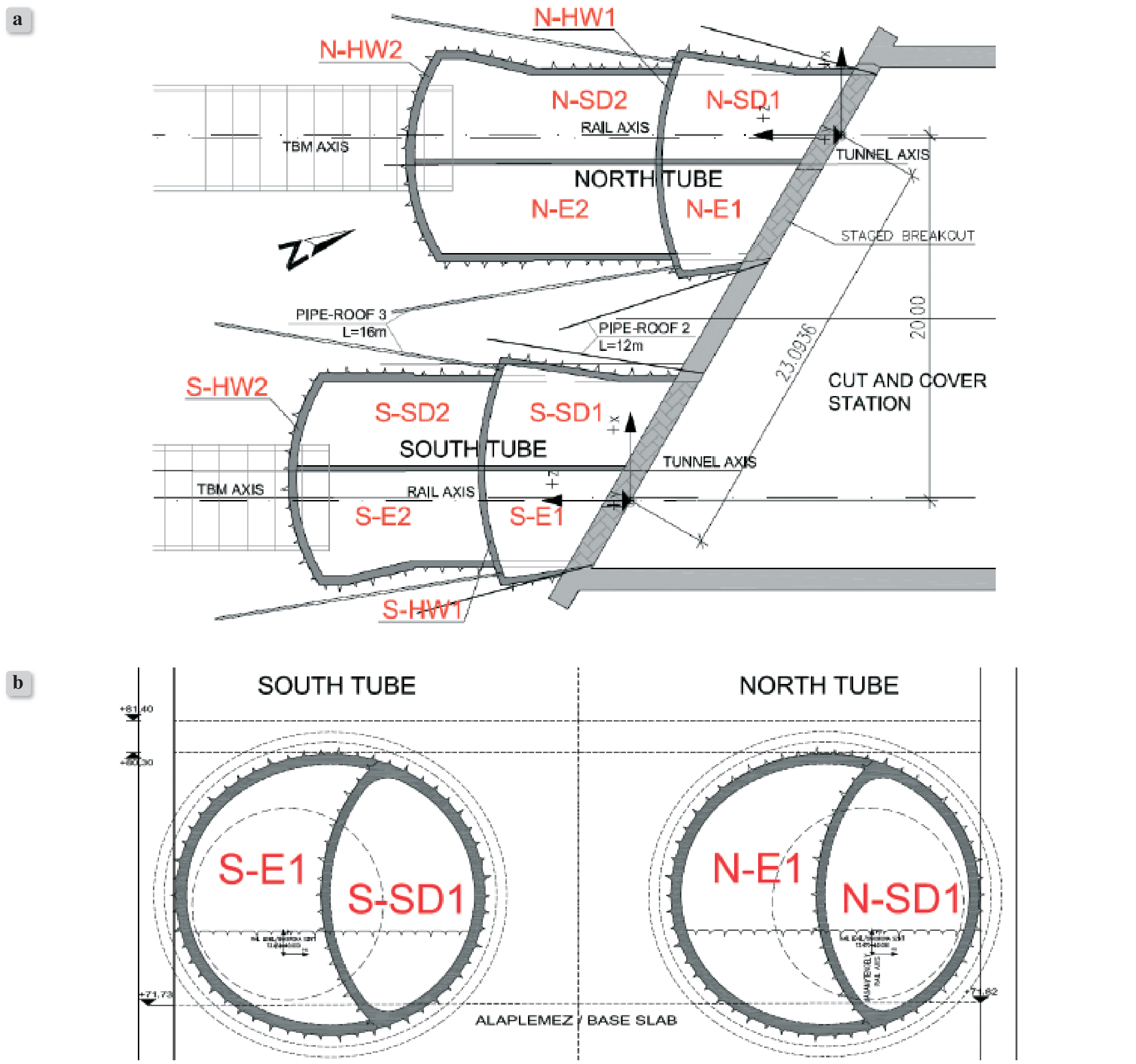

Figure 4.
Construction sequence of the SCL Tunnels: a) View of the construction sequence, b) Cross section of the construction sequence [6]

2. NUMERICAL MODEL

To create finite element model and carry out the analysis Midas GTS 2011 (v1.1) software with student licence was used. The reason to choose that software was the capability to handle easily with complex geotechnical problems and very good technical support.

2.1. General remarks

To create a finite element mesh simple four-node Tetrahedron 1st order elements were used in spatial elements. Three-node 1st order triangular elements were used in plane elements (tunnel lining, interface etc.). These types of elements give the best results in Gauss nodes and in the middle of elements. To provide the best possible quality of mesh the number of elements with small volume was minimized. Quality of the finite element mesh was presented in Figure 6a for the whole generated mesh and in Figure 6b for the tunnels mesh. The mesh was automatically generated by Midas GTS as a tetrahedral solid mesh, with variable sizes in smooth transition. The size of the elements was adjusted to maximal possible level which still provides reliable results, small size elements (approximately 1 m) were used in the areas of stress concentration (especially in the area of tunnel tubes where geometry was complex).

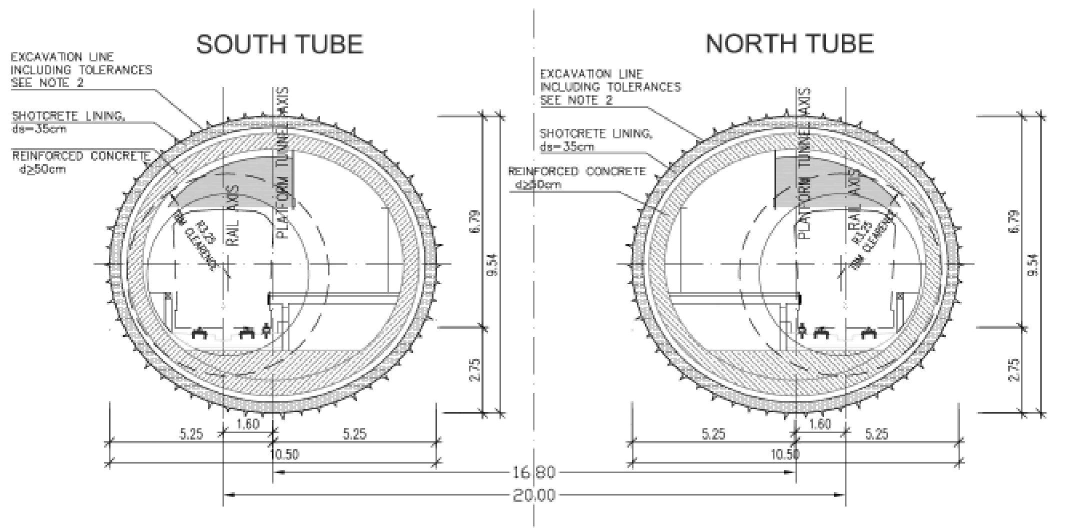

Figure 5.
Regular (minimal) cross-section of South and North Tube [6]

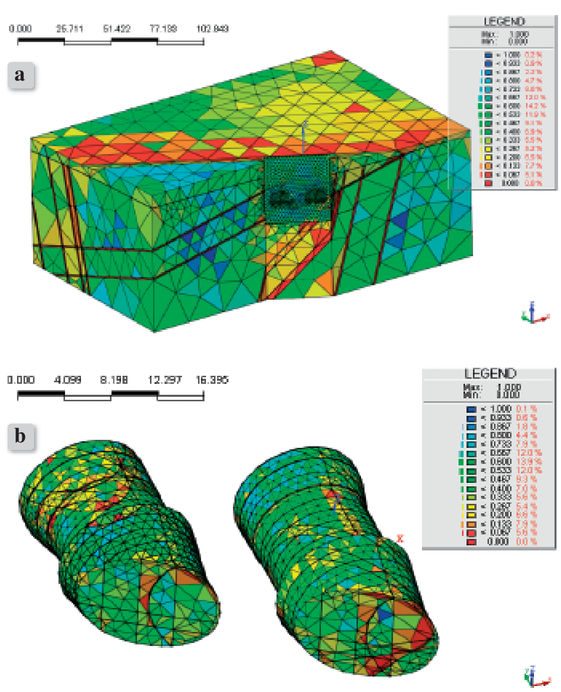

Figure 6.
Quality of generated mesh: a) Mesh of whole model, b) Mesh of SCL Tunnels [1]

Problems with the mesh quality can be observed in fault zones. Elements are relatively elongated in a direction compliant with fault zones. In the area of tunnel tubes the quality is significantly increased by a small size of the elements (approximately 1 m). Towards the boundary the mesh becomes coarse which causes the elements of fault zones to be quite elongated in one direction.

Soil behaviour was modelled using a nonlinear constitutive Mohr-Coulomb model. Two variants of parameters were assumed. Parameters in the first variant were the same as in the structural design project. Parameters in the second variant were determined according to Self Boring Pressuremeter (SBP) test. Parameters used in the structural design were defined on the rail level (32 mBGL), which means that for the majority of soils they were overestimated which do not put calculations on safe side. Most of soil parameters increase with depth which is normal behaviour. A widely used Mohr-Coulomb model does not take into account that behaviour of soils. However, Midas GTS has additional input parameters which include the behaviour of parameters values increasing with depth. The second set of parameters (from SBP) uses the above described feature of the software.

Table 1.
List of Törökbálint sandstone parameters used to create numerical model [1]

Parameter	Symbol	Unit	Design assumptions	SBP results
Modulus of elasticity	E	$\frac{kN}{m^2}$	120000	24000
Poisson;s ratio	ν	–	0.22	0.22
Cohesion	c	$\frac{kN}{m^2}$	687	100
Friction angle	ϕ	°	36	36
Increment of elastic modulus	E_{inc}	$\frac{kN}{m^3}$	–	14330
Increment of cohesion	c_{inc}	$\frac{kN}{m^3}$	–	110
Reference height	y_{ref}	m	–	0
Dilatancy angle	Ψ	°	–	-

2.2. Boundary conditions

To adjust boundary conditions specific model was prepared with fully excavated tunnels without any support (tunnel lining). For the matching initial boundary conditions recommendations from [7] were used.

Firstly, in the whole model standard boundary conditions were used. Vertical displacement was restrained on the horizontal bottom surface (Z-axis in local coordination system). On the vertical surfaces around the model horizontal displacement in a local perpendicular direction was restrained (X-axis and Y-axis in local coordination system).

The SCL Tunnels are not individual structures. They are a part of the whole metro station structure, which necessitates to implement additional boundary conditions. Typical boundary conditions do not include real behaviour of diaphragm walls of the metro station because stiffness of the whole station is not included. To include this fact, western part of the diaphragm wall with specific boundary conditions was modelled. Nodes which represent slab above and under the tunnels were additionally restrained in Z direction; finally those nodes had 5 degrees of freedom. Stiffness of the whole metro station structure is big enough to minimize the influence of lifting of the diaphragm wall inducted by relief after tunnel excavation. Lifting of the diaphragm wall could be observed in the numerical model after tunnel excavation where specific boundary conditions were not included in the diaphragm wall zone.

2.3. Lining model

Essentially, lining can be modelled as a two dimensional (2D) or three dimensional (3D) structure. 2D model of lining has an advantage of low complexity and shorter computation time. The 2D mesh model of tunnel lining was used for numerical analyses and the sample view of it is shown in Figure 7.

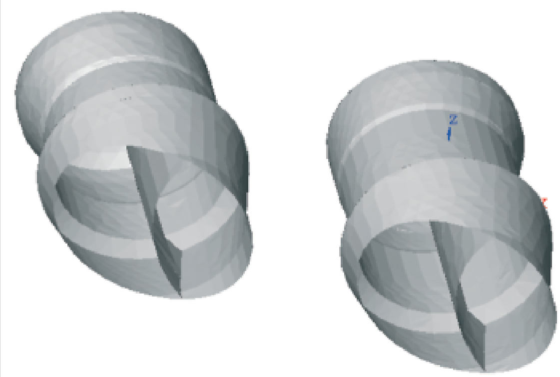

Figure 7.
2-dimensional mesh model of tunnel lining [1]

Shotcrete material is a complex and difficult to describe. During numerical modelling long-term and early-age behaviour of shotcrete should be taken into account. Suitable model is a visco-elastic "Kelvin" creep model (stress independent) which presents the most accurate description of shotcrete behaviour [7]. Due to the lack of this constitutive material model in the used numerical software (Midas GTS), behaviour of 2D lining elements was modelled by using the simplest constitutive model – linear-elastic model. Linear-elastic model with age-dependent stiffness was taken into account in a specific construction stage.

Table 2.
Characteristic of shotcrete parameters for 2D model of tunnel lining [1]

Parameter	Symbol	Unit	Young shotcrete	Hard shotcrete
Unit weight	γ	$\frac{kN}{m^3}$	24	24
Young' modulus	E	$\frac{MN}{m^2}$	6500	16500
Poisson's ratio	ν	–	0.2	0.2

2.4. Fault zones

In the surrounding area of the tunnel tubes a large number of fault zones occurs. Fault zones have significant influence on the behaviour of the structure and occurrence of specific stresses, which act on the structure and generate unwanted displacements. Generally, fault zones can be modelled in two ways as:

– Thin layer of soil with specific material parameters,
– Interface which gives possibility to slip two soil layers in contact.

A proper way of fault zones modelling depends mostly on the type of analysis. If it is necessary to model stress and strain behaviour, fault zones modelled as an interface is a good way to solve the problem. If seepage analysis is performed, especially when water level is above the ground, the model with thin layer needs to be considered. This allows water to propagate into a fault zone which can cause problems during excavation. Another thing is that especially in fault zones which occur in soil (not rock), soil adjacent to the slip surface can have different properties than the remainder of the soil. In this case it is reasonable to use the model with thin layer of soil.

The model of thin layer with different material properties representing a fault zone was applied in the numerical model. Only fault zones close to the tunnels were modelled because the influence of other faults was marginal.

2.5. Soil strengthening

To reduce complexity of the numerical model the treatment methods like freezing and grouting were included. In the zones around the tunnels these treatments were included as local material parameters change of finite elements. It was not necessary to model the geometry of the improved zones, because the real zone of the influence depends on many variables.

2.6. Construction stages

The whole construction process was divided into several construction stages according to the structural design. To reduce the time of calculations, a number of construction stages was reduced to a minimal level.

Initial ground stresses were consider in the first stage, after that stage displacements values were set to zero. Advance length of each excavation step was around 1.8 m and depended on the stage position relative to the geometry of the tunnel. Normally, excavation progress depends on monitoring data acquired during excavation. To ensure the required level of accuracy, every step of the tunnel face excavation was vertically divided into a side drift and an enlargement and horizontally divided into a lower and a higher part of the face. Additionally, each tube was divided into two sections similar to the structural design. Every step was divided into two construction phase: excavation and shotcreting. A detailed plan of construction stages used in numerical modelling was similar to the real plan of construction stages. Additionally, temporary backfilling of lower excavated part (invert) was taken into consideration after shotcrete gained sufficient strength.

2.7. Analysis type

Soil behaviour was modelled using a nonlinear constitutive model, in consequence a nonlinear type of analysis was required. The analysis type was carefully chosen and all the parameters (iteration scheme, convergence criterion, load increment) were adjusted to individual requirements. A specific analysis type was limited by huge amount of finite elements and complex geometry.

In this paper a secant stiffness method was applied, because of much shorter computation time in comparison to the other methods.

In the numerical model convergence criterion of the force norm was used with the value set to 0.02. The reason to set this value is that the numerical model is highly complex which causes long time of calculation. The above value significantly decreases the number of FEM runs to reach equilibrium and provides satisfactory accuracy.

Automatic load increment was chosen in the analysis. Load increments are automatically calculated by solver based on the input of the initial load factor. If convergence equilibrium is not reached initial load factor is reduced by 25%. The efficiency of computations increases when convergence equilibrium is reached.

3. RELIABILTY OF NUMERICAL MODEL

Reliability of the numerical model was checked by comparison of specific results of FEM with the data from geotechnical monitoring. Comparison of displacement curves on inclinometers together with horizontal and vertical stresses that occur in soil was made.

Comparison of displacements from numerical model to the data from geotechnical monitoring is shown in Figures 8 and 9. It can be seen that values obtained from numerical model are much higher than from geotechnical monitoring which results in overestimation of lining structure (thickness and reinforcement).

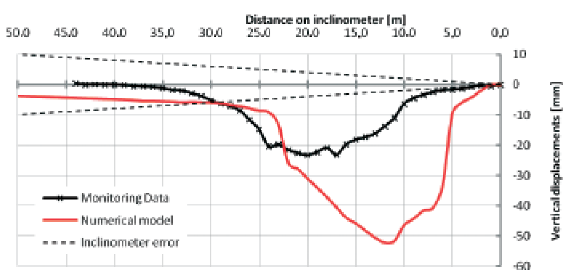

Figure 8.
Comparison of displacements obtained from inclinometer and FEM (North Tube) [1]

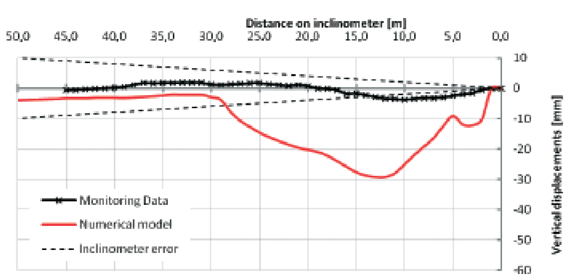

Figure 9.
Comparison of displacements obtained from inclinometer and FEM (South Tube) [1]

Difference between displacements obtained from the numerical model and geotechnical monitoring data for the South Tube are much higher than for the North Tube.

Vertical and horizontal soil stresses obtained from the numerical model and geotechnical monitoring (Self Boring Pressuremeter) are presented in Figures 10 and 11. It can be noticed that vertical stresses from

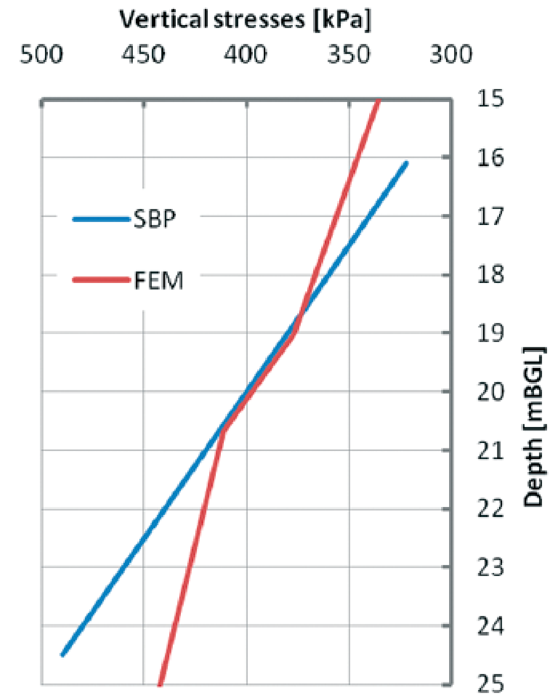

Figure10.
Comparison of vertical stresses obtained from SBP and FEM [1]

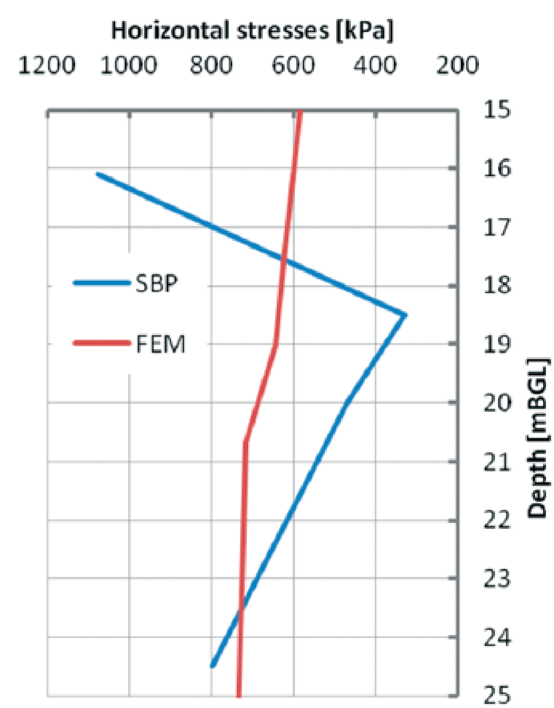

Figure11.
Comparison of horizontal stresses obtained from SBP and FEM [1]

both sources have comparable values. Horizontal stresses obtained by the Self Boring Pressuremeter are much higher than those by the numerical model. Stresses presented by Self Boring Pressuremeter have much larger variability at the depth.

4. CONCLUSIONS

Sprayed concrete lined (SCL) tunnels are sophisticated structures which require comprehensive approach. Issues become more complicated if geotechnical conditions are complex as in case of this paper. In this paper a possibility of numerical modelling of complex geotechnical conditions and receiving reliable results was widely discussed. Numerical modelling gives possibility to take into account all significant factors. Using one value of parameter to describe the whole layer of soil in simple structures is a satisfactory solution. However, when dealing with complex geotechnical conditions and when performing back analysis this solution is not enough to receive reliable results. The best estimation of soil parameters was provided by Self Boring Pressuremeter. These parameters have been used to create a numerical model – they take into account change of the modulus of elasticity and cohesion with depth. All specific conditions encountered during site investigation need to be taken into consideration with caution, i.e. way of modelling the fault zones (use of thin layer instead of interface where estimation of parameters can cause problems is the simplest and satisfactory solution). The numerical model of SCL Tunnels and complex geotechnical conditions led to a very long time of computations, especially including all significant construction stages. The approximate computation time of one FEM run was around 20 h. However, in close future this time will be significantly reduced with an increase of CPU processing power. The structure and soil behaviour obtained from the numerical model is similar to the behaviour presented by geotechnical monitoring which proves reliability of the numerical model. Concluding, the created model can be used in a structural design. However, the numerical model has several simplifying assumptions which can explain the difference with the observed response. The change of K_0 value with depth is only one among all possible reasons.

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