Tunnelling and reinforcement in heterogeneous ground – A case study

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Abstract

A case study of tunnelling in heterogeneous ground conditions has been analysed. The case involves a tunnel excavated in mixed-face conditions, where the main host material was rock, but for a distance of about 30 m, the tunnel had to be driven through a thick layer of soil, primarily moraine and sandy soil materials. During tunnel drifting, a "chimney" cave developed through the soil layer, resulting in a surface sinkhole. This case was analysed using a three-dimensional numerical model with the FLAC^{3D} software code, in which the soil stratigraphy and tunnel advance were modelled in detail. Tunnel and soil reinforcement in the form of jet grouting of the soil, pipe umbrella arch system, bolting, and shotcreting, was explicitly simulated in the model. The study aimed at comparing model results with observations and measurements of ground behaviour, and to replicate the major deformation pattern observed. The modelling work was based on a previous generic study in which various factors influencing tunnel and ground surface deformations were analysed for different cases of heterogeneous ground conditions. Model calibration was performed through adjusting the soil shear strength. The calibration provided a qualitatively good agreement with observed behaviour. Calculated deformations on the ground surface were in line with measured deformations, and the location of the tunnel collapse predicted by the model. The installed tunnel reinforcement proved to be critical to match with observed behaviour. Without installed pipe umbrella arch system, calculated deformations were overestimated, and exclusion of jet grouting caused collapse of the tunnel. These findings prove that, in particular, jet grouting of the soil layer was necessary for the successful tunnel advance through the soil layer.

1 Introduction

Tunnelling in urban areas has become increasingly challenging because of city development. Urban tunnelling usually implies that neither an optimal tunnel orientation nor an optimal tunnel depth can be achieved due to existing underground facilities or particular design needs (e.g., construction of a new station). A difficulty that often arises in urban areas is that the tunnel has to be excavated through mixed-face conditions with both soil and rock, which can have a major effect on the construction techniques and on the surroundings (see e.g., Clough & Leca 1993; U.S. Department of Transportation 2009; CEDD 2012).

This paper describes a case study of tunnelling in mixed ground conditions in the Stockholm area, and the post-construction numerical analyses. The objective of the study was to increase the understanding and knowledge of ground behaviour around tunnels in mixed ground conditions for future application and implementation. Moreover, an assessment of the performance of the reinforcement measures used in the project, such as compensation grouting, umbrella arch system, shotcrete lining and bolts, was carried out via numerical analysis.

2 Stockholm city-Line: passage under the maria magdalena church

The Stockholm City Line is a 6 km long commuter train tunnel running between Tomteboda and Stockholm South. One part of this project involved tunnelling under the Maria Magdalena church in the southern part of Stockholm, and relatively close to the Stockholm South Station. The passage under the church was technically difficult due to mixed-face conditions, in addition to the thick layer of soil up to the ground surface. The tunnel train track is depicted in Figure 1, where the railway track is marked in red and the area of study is indicated in yellow.

Figure 1 Overview of the Stockholm City-Line tunnel track in the southern end, where the area of interest is surrounded in yellow, with the Maria Magdalena church next to it (Google Earth 2015)

During tunnel drifting, a "chimney" cave developed through the soil layer, resulting in a surface sinkhole with an area of 2 x 2 meters and 1.5 meters depth. This failure mechanism typical of frictional soils — flowing ground — is largely a construction design issue that should be avoided to the extent possible. Support measures like compensation grouting or pipe umbrella systems are well fitted to solve this kind of problems, and were studied in detail in this work to quantify their influence on the stability of the working area.

2.1 Geology

The geology consists of a thick layer of soil overlaying the bedrock. The dominating lithology of the area is "Stockholm granite", which was grouped into different rock types based on extensive rock mass characterisation as part of the City Line design work. The three rock types present in the area of interest were, according the nomenclature adopted for the design of the City Line tunnels: Rock type A ($RMR \geq 70$), Rock type B ($50 \geq RMR > 70$) and Rock type C ($30 \geq RMR > 50$).

The soil layer, in turn, comprised three different types with variable characteristics. The three soil types that were found through geotechnical pre-investigations were: sand (in the upper part, close to the ground surface), esker material (directly under the sand layer) and moraine (located between the layer of esker material and the rock). For practical reasons, only two soil types were included in the numerical model, where esker material and sand were forming one group and the moraine comprised the other type of soil.

Figure 2 illustrates the location of the tunnel with respect to the rock-soil interface, the approximate topography of the area, the situation of the moraine layer and the distribution of the rock types along the model.

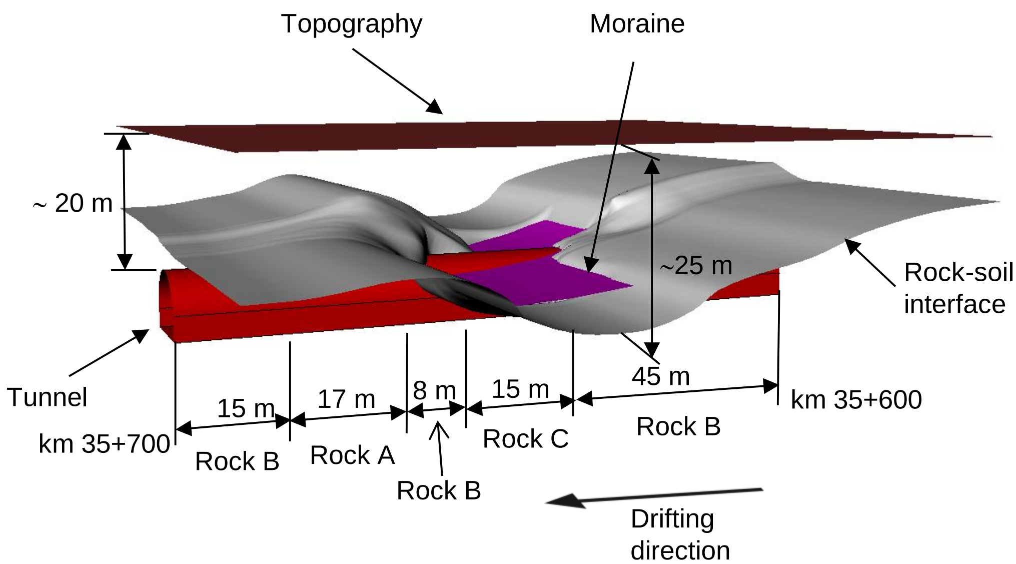

Figure 2 Rock-soil interface (in grey), tunnel contour location (red), topography (brown) and moraine layer (magenta) in the area of the tunnel passage under the Maria Magdalena church from km 35+600 to km 35+700

2.2 Tunnelling procedure

This study focuses on a passage with a total length of 30 meters with mixed face conditions, excavated by the "drill-and-blast" method. The target of the study was the passage located between the km 35+633 and 35+663, for which the excavation was performed in three different phases of 10 meters each (phase A, phase B and phase C). A detailed description of the tunnelling procedure was given by Willer (2014), in which the collapse of the tunnel was also described. An explanatory scheme of the excavation sequence and reinforcements is shown in Figure 3.

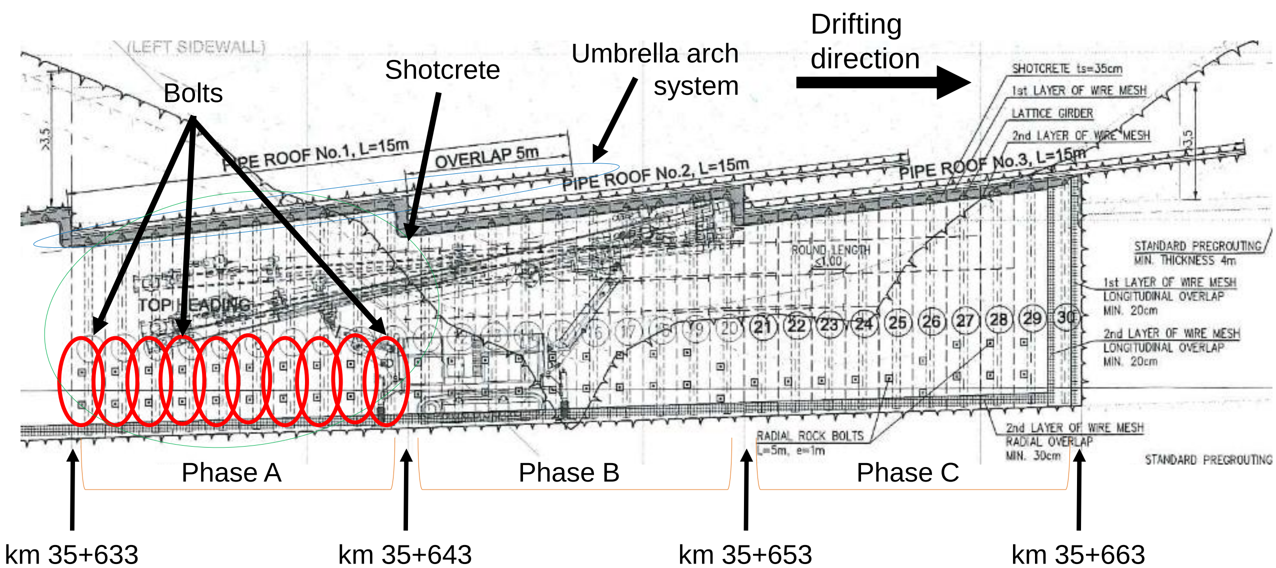

Figure 3 Longitudinal profile of the analysed section with the excavation sequence and reinforcement solution between km 35+633 and 35+663 (modified after Willer 2014)

The first step, and prior to the actual excavation, consisted of compensation grouting performed from the tunnel face. The tunnel was thereafter excavated with a split face and 1 meter round length, where the heading was first excavated for the total length of 30 meters, followed by the excavation of the bench.

The excavation and support sequence in phase A and phase B were as follows:

1. Symmetrical pipe umbrella arch, comprised by 60 pipes, each one of 15 meters length, with 7° outwards in the longitudinal axis and a covered angle of the cross section of 150°. The c/c distance was about 30 cm.
2. Conical excavation with round length of 1 meter, with 7° outward, with the maximal diameter at every 10 meter.
3. Systematic bolting, in the lower part of the arch, comprising 4 bolts (2 on each side) per excavated meter, with 5 meters length each and a c/c distance of 1 meter.
4. Shotcreting in the whole 180° arch, 35 cm thickness.

The excavation and support sequence in phase C differs from the previous ones and was as follows:

1. Asymmetrical pipe umbrella arch, comprised by 50 pipes, each one of 15 meters length, with 7° outward in the longitudinal axis and covering an angle of the cross section of 125°. The c/c distance of the pipes was about 30 cm.

2. Conical excavation with round length of 1 meter, with 7° outward, with the maximal diameter at the end of phase C.
3. Systematic bolting, in the lower part of the arch, comprising 5 bolts (2 on the left side and 3 on the right side) per excavated meter, with 5 meters length each and a c/c distance of 1 meter.
4. Shotcreting in the whole 180° arch, 35 cm thickness.

3 Model setup

The 3D numerical modelling code *FLAC^{3D}* (Itasca 2013) was used to model the ground surface settlements produced by tunnel excavation, and to assess the performance of the support systems that were used in this case study. It should be noted that the actual chimney cave and the soil flow was not simulated explicitly. The use of a continuum approach with *FLAC^{3D}* was considered appropriate given the characteristics of the rock mass and soil layers present in this study.

3.1 Geometry and boundary conditions

The model mesh was constructed based on the conical shape of the tunnel excavation. The separation layers for the different materials were created in the CAD program *Rhinoceros* (McNeel 2015), and imported to *FLAC^{3D}* as delimiting surfaces. The numerical model comprises 1,785,000 zones and was divided in regions with different zone sizes. The finest zone size elements are located adjacent to the excavation (0.25 m x 0.12 m x 0.20 m), with zones sizes increasing gradually towards the model boundaries. The geometry of the *FLAC^{3D}* model is shown in Figure 4.

Stresses were initialized in the model in each element. The boundary conditions applied to the model consisted of roller boundaries for the vertical sides of the model, pinned boundary condition for the bottom of the model, and a free surface for the ground surface.

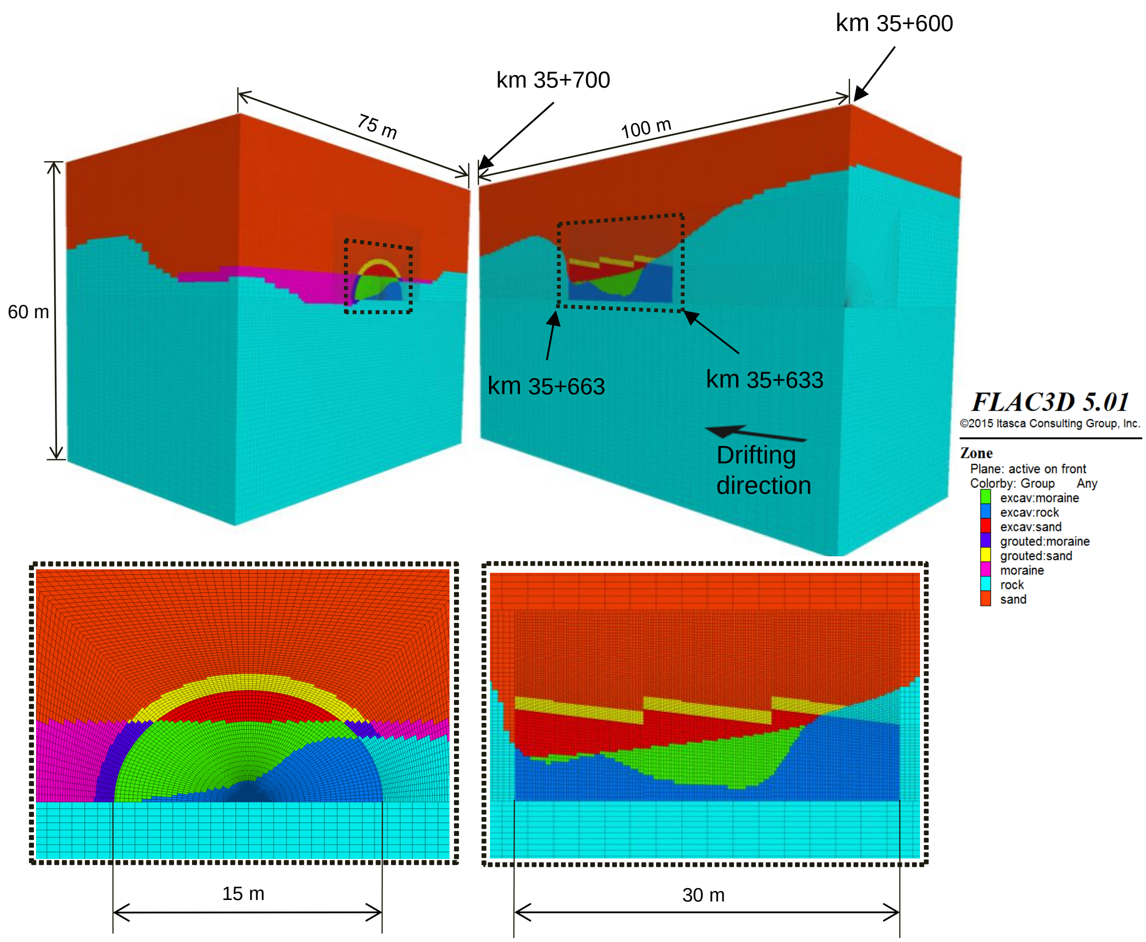

Figure 4 **Finite difference mesh in the numerical model used to analyse the passage under the Maria Magdalena church, showing longitudinal and transversal sections in the middle of the model, including the excavation area**

3.2 Initial stress

The initial stress conditions in the rock mass were taken from measurements performed for the Stockholm City-Line at Södermalm (described in Perman & Sjöberg 2007), and given as follows:

$$\sigma_H = 2.0 + 0.125z \quad (1)$$

$$\sigma_h = 1.0 + 0.100z \quad (2)$$

$$\sigma_v = 0.0265z \quad (3)$$

where:

σH = maximum horizontal principal stress in MPa; orientation 160° from Geographic North,

σh = minimum horizontal principal stress in MPa,

σv = vertical stress in MPa,

z = depth from ground surface in meters.

For the soil layer, the stress state was assumed to be lithostatic and given by:

$$\sigma_H = \sigma_h = K_0 \sigma_v \tag{4}$$

where:

K_0 = initial stress ratio (dimensionless)

3.3 Material properties

The rock and soil materials in the *FLAC^3D^* model were represented using a linear elastic-perfectly plastic Mohr-Coulomb constitutive model with tensile strength cut-off. The assumption that the compensation grouting would only affect the soil layers within a radial distance of 1 meter from the tunnel contour was made. The grouted soil material was thus simulated as a soil with increased cohesive and tensile strength, with properties representing a poor quality concrete (as this was the target strength for the conducted grouting). The material properties for the rock and soil layers (cf. Section 2.1) are shown in Table 1.

Table 1 Rock mass and soil properties used in *FLAC^3D^*. (Handboken Bygg: Geoteknik 1984; Lindfors 2008, Larsson 2008; Larsson et al. 2007; PLAXIS Material models manual 2015, Trafikverket 2014)

	Density ρ (kg/m^3)	Young's modulus E_m (MPa)	Poisson's ratio ν_m (-)	Cohesion c (MPa)	Friction angle ϕ (°)	Tensile strength σ_t (MPa)	Dilation angle ψ (°)	K_0 (-)
Rock type A	2650	69000	0.25	6.6	58.3	2.4	7	-
Rock type B	2650	46000	0.25	2.5	58.9	0.5	7	-
Rock type C	2650	11000	0.25	1.0	51.9	0.08	0	-
Sand	1600	20	0.35	0.0	35	0	5	0.43
Moraine	2000	100	0.35	0.0	45	0	15	0.30
Grouted soil	2000	200	0.25	0.1	35	0.2	5	0.43

3.4 Ground support properties

The shotcrete was represented as a liner element and the fully-grouted bolts were modelled as cable elements (only accounting for axial forces) in *FLAC^3D^*. In order to consider the bending resistance provided by the pipe umbrella system, the pipes were represented using pile structural elements. It was assumed that only the pipes were filled with grout, not the area between the pipe and the soil (Volkmann & Schubert 2009). Figure 5 represents the structural elements that were included in the excavation sequence, with the material properties for the support elements shown in Table 2. The interaction parameters between the structural elements and the surrounding material are given in Table 3 through Table 5.

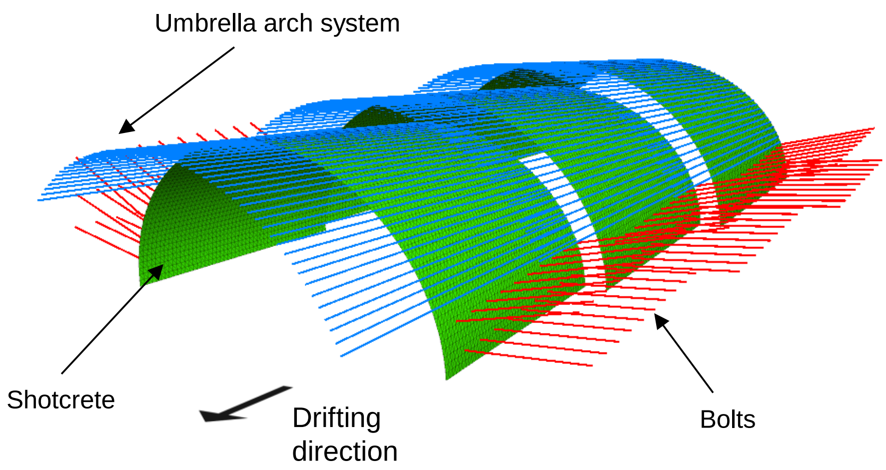

Figure 5 Structural elements in $FLAC^{3D}$ model after the completion of the 30 meter tunnel excavation

Table 2 Properties for rock bolts, pipe elements and shotcrete used in $FLAC^{3D}$. (Rosengren 2004; Malmgren 2005; Holmberg 2014)

	Density ρ (kg/m^3)	Young's modulus E_m (GPa)	Poisson's ratio ν (-)	Diameter d (mm)	Thick-ness t (mm)	Length l (m)	Characteristic compressive strength f_{ck} (MPa)	Characteristic tensile strength f_y (MPa)
Bolt	7800	200	0.3	20	-	5	246	246
Pipe element	7800	200	0.3	160	10	15	-	-
Shotcrete	2300	16	0.25	-	350	-	12	3.9

Table 3 Properties for bolt-grout interface used in $FLAC^{3D}$. (Rosengren 2004; Itasca 2010)

	Shear modu-lus G (GPa)	Stiffness (GPa/m)	Thickness t (mm)	Compressive strength σ_c (MPa)	Cohesion c (KN/m)	Friction angle ϕ (°)
Bolt-grout interface	9.0	8.15	10	20.0	565	40

Table 4 Properties for pipe-soil interface used in $FLAC^{3D}$. (Itasca 2012)

	Shear stiffness k_s (GPa)	Normal stiffness k_n (GPa)	Cohesion c (MPa)	Friction angle ϕ (°)
Pipe-soil interface	200	2000	0.0	35

Table 5 Properties for shotcrete-rock interface used in $FLAC^{3D}$. (Malmgren 2005; Itasca 2010)

	Shear stiffness k_s (GPa/m)	Normal stiffness k_n (GPa/m)	Adhesion (MPa)	Cohesion c (MPa)	Friction angle ϕ (°)
Shotcrete-rock interface	0.1	1.0	0.6	0.5	40

4 Model calibration

The calibration of a numerical model against measurements and observations is a key aspect to ensure that results are representative and realistic. Therefore, the model was first calibrated using monitoring data from ground surface settlements at 8 different points, obtained from levelling pins located at the ground surface, more or less in the centre of the cross-section of the tunnel, as shown in Figure 6.

Based on the associated generic study on tunnelling in heterogeneous ground, as part of this overall project (Eriksson et al. 2016), the following assumptions were made: (1) the initial stress state in the rock mass would not influence the main behaviour of the model for mixed ground conditions, (2) experiences from the construction site showed that the soil layer was above the groundwater level and thus unsaturated (Stille 2015), (3) the rock would not be affected significantly by the pore pressures, and (4) the soil properties would have a major influence on the deformations when tunnelling. Therefore, the calibration of the model was done by varying the cohesion of the frictional soil and the moraine.

For calibration purposes, the cohesion of the soil layers in the model was varied between 0 and 5 kPa, showing large sensitivity to these values. The best-fit model achieved for 2 kPa cohesion (checked against measured settlements, see Figure 7) was used for investigating two extra cases, aiming to check the effectiveness of the umbrella arch system and the compensation grouting. The "chimney" cave that developed through the soil layer when drifting was not simulated explicitly, however, the model showed that the maximum displacements occurred in km 35 + 649, coincident with the location where the cave started to develop.

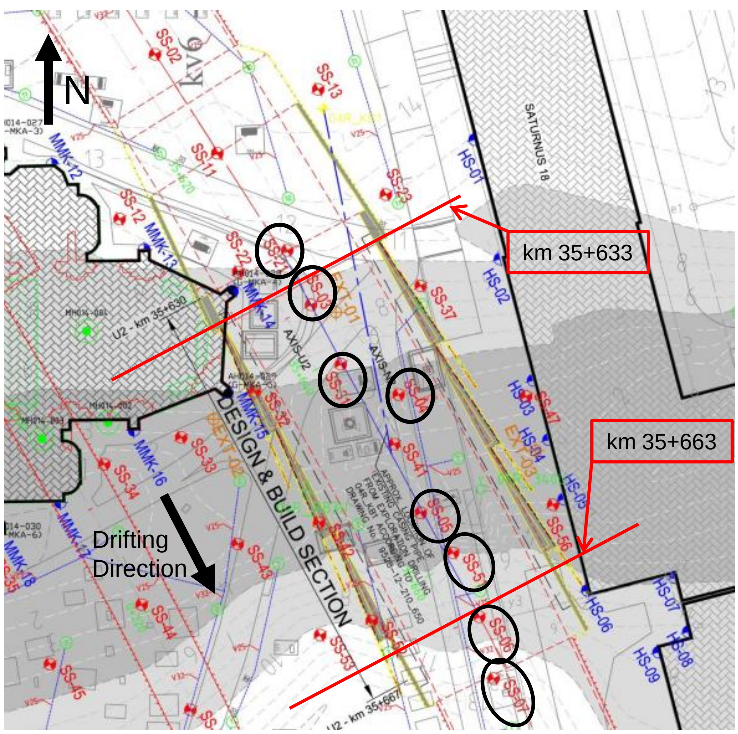

Figure 6 Overview of the Maria Magdalena church area, where the zone with available ground surface measured displacements marked in red and the location of measurement pin marked in black (modified after Trafikverket 2015)

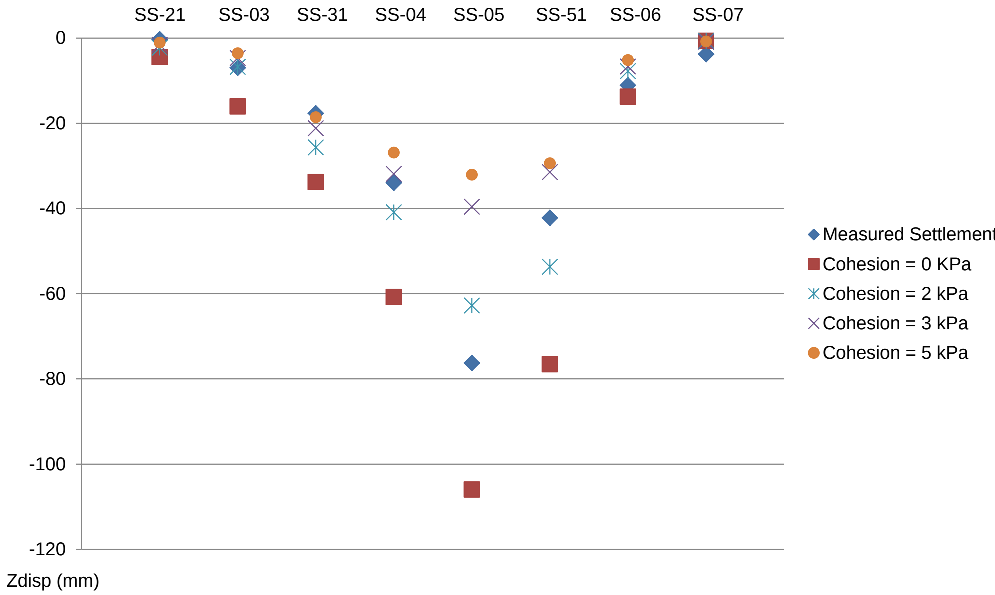

Figure 7 Calculated ground surface settlements for the 8 calibration points for the calibration models, together with the actual surface settlements

5 Evaluation of support measures

The ground surface settlements for the best-fit model, with and without umbrella arch system, are shown in Figure 8, together with measured settlements. These results indicate that much larger surface settlements develop if the umbrella arch system is not included in the model. The simulated results regarding the best-fit model without compensation grouting showed that large-scale collapse occurred in the model, starting from the vicinity of the tunnel and propagating upwards up to the ground surface.

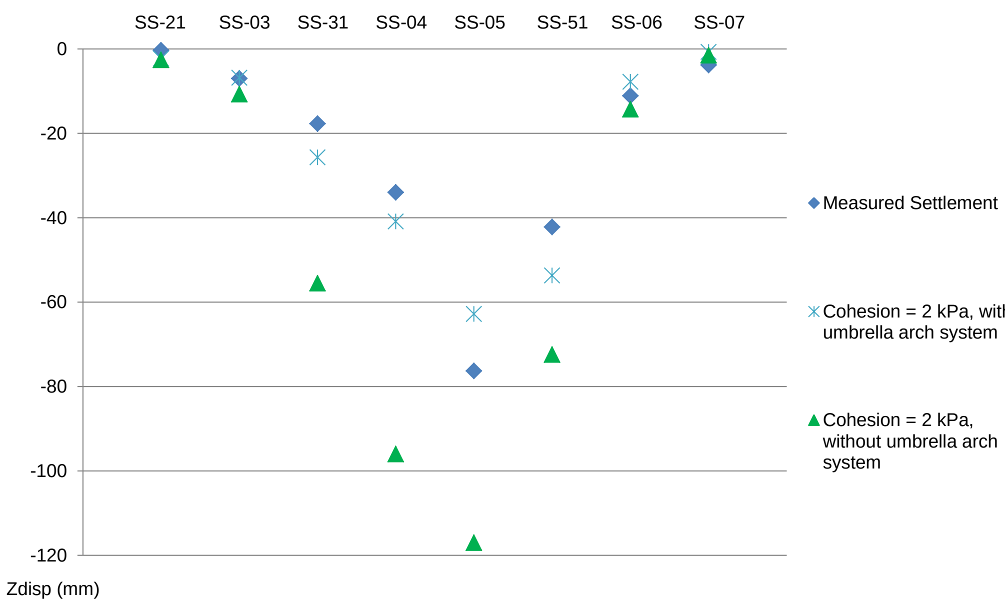

Figure 8 Calculated ground surface settlements for the 8 calibration points for the best-fit model with and without umbrella arch system, together with the actual surface settlement results

A comparison between the best-fit models with and without umbrella arch system was done with focus on ground support elements, in order to prove its effectiveness. This evaluation zeroed in on displacements and acting moments in the liner on the one hand, and forces developed in the bolts, on the other hand.

The influence of the umbrella arch system on liner displacements is obvious (see Figure 9), where this type of support helps to minimize the liner deformations, mainly in the crown of the tunnel. Additionally, the acting moments in the liner are somewhat redistributed (see Figure 10), indicating that the pile elements in the *FLAC*^{3D} model (umbrella arch system) contribute in such a way that the moment acting along the longitudinal axis of the liner is reduced. The influence of the umbrella arch system on the bolt elements is however minor (see Figure 11), since the cable elements used in the *FLAC*^{3D} model only account for axial forces.

6 Discussion

Based on the results obtained from the performed analyses, the calibration of the model against monitored surface settlements was in good agreement with the actual behaviour. Furthermore, the location of the collapse that occurred in the tunnel was captured by the numerical model as well. The results further showed that the ground surface settlements were extremely sensitive to variations of the shear strength of the soil layers, especially for the cases with low cohesion, where a change from 2.0 to 0.0 kPa lead to increased ground surface settlements of up to 100%. The ground support measures also influenced the results significantly. On the one hand, the model without the umbrella arch system yielded up to 140 % larger surface settlements compared to the case with the umbrella arch included. On the other hand, the model without compensation grouting resulted in large-scale collapse of the model.

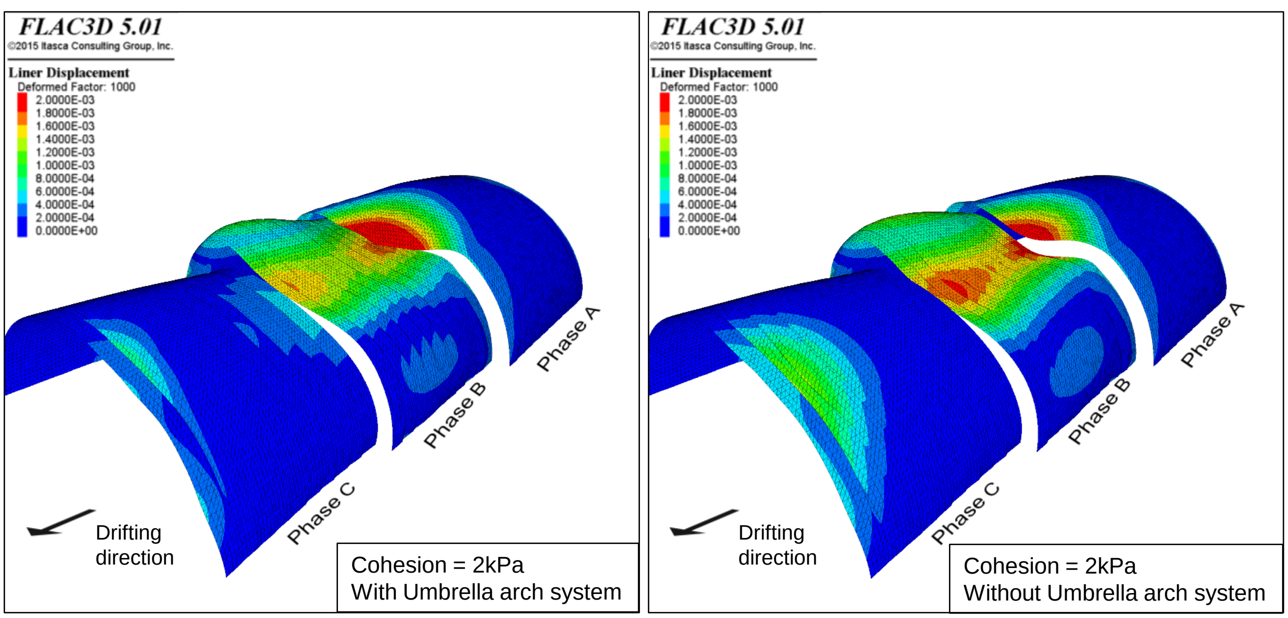

Figure 9 Calculated liner displacements for the numerical models after the excavation of the passage, where the values are expressed in meters

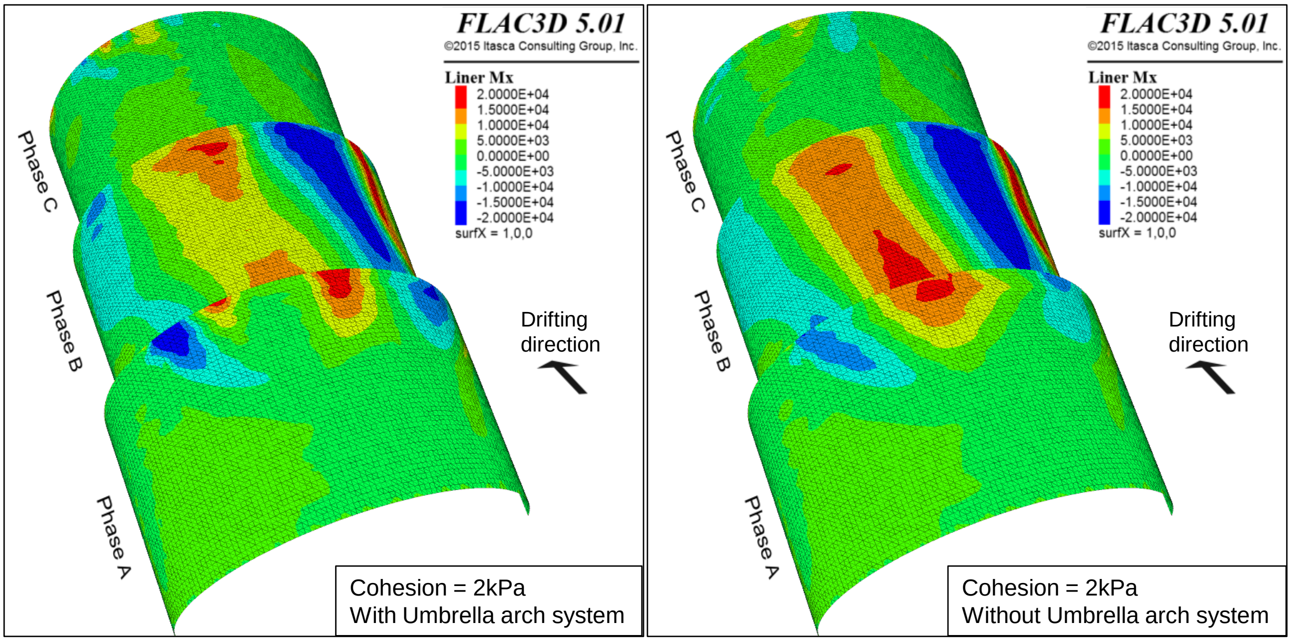

Figure 10 **Calculated liner acting moments along the longitudinal axis for the numerical models after the excavation of the passage, where the values are expressed in Nm**

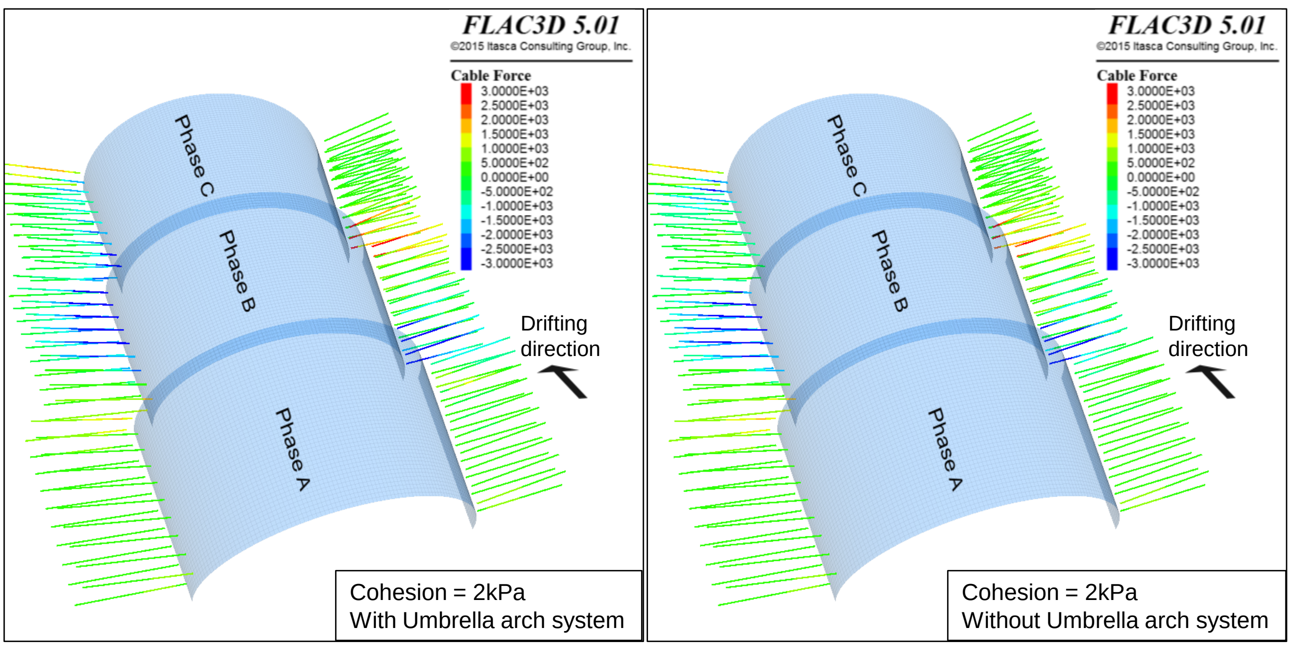

Figure 11 **Calculated axial forces acting on bolts for the numerical models after the excavation of the passage, where the values are expressed in N**

It should be pointed out that the assumption of the jet grouting acting as an element that mainly increases the cohesion of the soil layer surrounding the tunnel was made. However, it is possible (or perhaps even likely) that some areas or volumes do not have enough grouted soil, and therefore have not acquired the properties that were expected in the design stage.

The work that has been presented in this paper can be used as a guide for required pre-investigations in feasibility studies and early design stages, as well as a basis for a numerical modelling methodology. In cases where mixed ground conditions are present, the soil characteristics "dominate" the tunnel behaviour. Thus, a detailed characterisation of rock properties and rock stresses are not necessary. Instead, efforts should be focused on: (i) determining the extent and location of soil layers, and (ii) determining the strength and stiffness properties of the soils. While determining the soil (and rock) stratigraphy is often part of pre-investigations, soil characteristics are often assumed or estimated conservatively. A more detailed investigation of soil properties, including e.g. triaxial tests, and for relevant stress conditions is recommended for this types of cases. A sensitivity analysis of the soil properties can be also beneficial, since these properties are critical with respect to the general behaviour on the model.

7 Conclusions

The use of a three-dimensional model, with a realistic representation of both the actual geological conditions and the excavation sequence, allowed obtaining a good match between model results and monitored deformations on the ground surface. The *FLAC^{3D}* model results also proved the effectiveness of both the pipe umbrella arch system and compensation grouting in preventing collapse of the tunnel during drifting, where the latter showed to have an even more critical contribution to the stability than the pipe umbrella arch system. Apart from this, the pipe umbrella arch system provided a large reduction of the surface settlements.

A three-dimensional numerical model is considered to be an adequate tool to obtain reliable results and good understanding regarding deformation patterns, general tunnel stability and performance of the ground support measures. Furthermore, this type of model can be used to investigate different design solutions and check their influence on neighbouring structures in an urban scenario.

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