3D analysis of the influence of primary support stiffness on the surface movements during tunnel construction

3D-analize vpliva togosti primarnega podporja na razvoj pomikov površine med gradnjo predora

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Abstract: The usage of underground structures is increasing which is the reason that in many cases construction takes place in heavy and difficult geological geotechnical conditions. The main goal to reduce surface settlement on the minimum is connected with stiffness of the primary lining and effective protection excavation face against deformation process realized in rock pillar ahead the excavation. The construction process was modelled with *PLAXIS 3D tunnel program*. Input parameters were determined by 3D back analyses with Soft-Soil-Creep (SSC) constitutive material model, which takes into account rheological phenomena. 3D development of stress-strain fields during the tunnel excavation was performed to show major stress-strain changes in the surrounding rocks and support elements. The calculations showed that during excavation of the top heading substantial deformations are developed ahead of the top heading. Influence of the support stiffness has strong connection with amount of surface movements above the tunnel.

Izvleček: Uporaba podzemnih prostorov se v svetu povečuje, kar je pogosto razlog, da v mnogih primerih gradnja poteka v zahtevnih geoloških in geotehničnih razmerah. Glavni cilj podpornih ukrepov v takih primerih je, da se doseže zmanjšanje deformacij površine, ki je naseljena, na najnižjo možno raven. To je v direktni povezavi s togostjo primarne obloge in z učinkovito zaščito izkopnega čela pred procesom razvoja deformacij v hribinskem stebru pred njim.

Original scientific paper

Simulacija gradnje predora v obravnavanem primeru je bila izdelana s programskim paketom PLAXIS 3D-predor, ki je ustrezen za tovrstne izračune. Vhodni parametri za izračune so bili določeni s povratnimi 3D-analizami z uporabo materialnega modela Soft-Soil-Creep (SSC), ki omogoča upoštevanje časovno odvisnih procesov v hribinah. 3D-analiza razvoja napetostnih polj in obremenitev podpornega sistema med izkopom predora je bila opravljena tako, da so bile upoštevane različne togosti podpornih elementov. Izračuni so pokazali, da so med izkopom posebej pomembne velikosti deformacij pred izkopnim čelom, ki v skupnem seštevku deformacij, ki se razvijejo v predoru med izkopom in vgradnjo podpornih elementov, lahko močno presegajo dopustne vrednosti. Z izračuni deformacij v 3D-prostoru je bil dokazan realen vpliv togosti primarnega podpornega sistema na razvoj pomikov površine nad predorom.

Key words: 3D Finite Element Analysis, SCC constitutive model, tunnelling, stiffness of primary lining, surface movenets

Ključne besede: 3D-analize z metodo končnih elementov, konstitutivni model SCC, izkop in primarno podpiranje predora, togost primarne obloge, pomiki površine terena

INTRUDUCTION

The influences of tunnelling on surface in areas of low overburden are sometimes still difficult to predict, despite contemporary computer techniques. The cause for that could be in quite changeable physical and mechanical properties of the surrounding rocks mass and soils and also in selected construction method related with stiffness of primary support, which is installed after the excavation. In technical and scientific literature at this area we could find some numerical models, which deals with relations between stresses and strains in the support system and surrounding rock mass for different distances between closed primary support and the face of excavation also for the particular stiffness of the support system.

In fact this is the effect of the support stiffness in function with surrounding rock mass in which the stiff support system has the biggest contribution. Stiff support system is usually combination of steel support together with shotcrete, rock bolts or anchors and also with additional support systems like steel pipe roof, micro piles, temporary invert with elephant foot, etc. In rocks with low bearing capacity, the arrangement of stresses and the amount of deformations which occurred, could be a serious problem regarding to the acceptable surface subsidence. In reality deformations in tunnel shell usually make no problems with stability and excavation process. The technological procedures are schematic shown in the Figure 1.

Regarding to that, this demands detailed analysis of stiff support effect taking in to account development of deformation fields with excavation progress and installation of primary support.

SOME NUMERICAL MODELS WHICH DEALS WITH STIFFNESS OF PRIMARY SUPPORT SYSTEM

The question, which is related with the share of primary state in rock mass, which the support system has to take over, is in professional literature treated in different ways by many authors (Whittaker & Frith, 1990; Kim & Eisenstein, 1998; etc.). But in latest past, the usage of complicated models were less desired, because it took a lot of knowledge and time for engineers to obtain some results in practice and on the other hand the input parameters were complicated and difficult to define with standard laboratories tests (Vižintin, 2009a). This causes some problems at planning and execution of detailed laboratories tests, which also could had limited practicability at wider area of the surrounding rock. This fact is quite related with inhomogeneous and anisotropic behaviour and anomalies, which often occur in surrounding mass (Vižintin, 2009b). Reduction factors, which were used by some authors, show us, that they were fully aware of the complexity of rock mass structures.

Experiences that we have with usage of that numerical models indicated, that they are useful for tunnels or others underground objects with low overburden, which are construct in compact rock mass with high cohesion. In cases,

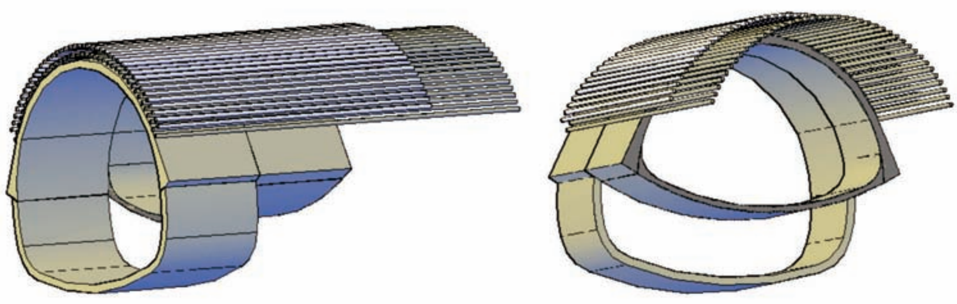


Figure 1. Technological sequences during the tunnel tube excavation.

where the rock mass has low cohesion, the results are questionable and not so reliable. Kim in Eisenstein 2006 suggested, that calculation method developed by Schwartz in Einstein 1980, could be used, when the quotient came to $L_d/R < 1$. In this equation the $L_d = 0.7$ m is the distance between the excavation face and the centre of gravity of the latest closed segment of primary lining and $R = 5.5$ m, which is the equivalent radius of excavation.

In present case is $\lambda_d = 0.7 - 0.57\ (L_d/R) = 0.63$, which corresponds their recommendation.

Technical particularity of construction of the Trojane tunnel at the section of shallow cover class

The construction of the tunnel at the section of shallow cover class was adjusted to unfavourable conditions. The following working phases were performed:

a) An excavation of top heading was carried out in 5 phases with simultaneous protection of working face by wire mesh Q189, 15 cm thick shotcrete. 35 pieces of 15 m long IBO rock bolts with bearing capacity 250 kN and over covering at 5 m to 7 m were also installed. The amount of excavation step was 0.8 m.

b) After excavation of the 3rd phase, the steel support was installed, which included 2 steel segments IPE 180 and 2 steel micro piles at every side of top heading, with length 6 m and 64 mm in diameter.

c) After excavation of the 4th and 5th phase, temporary invert was installed with the thickness of 25 cm and one layer of wire mesh Q283. Also the second layer of wire mesh Q283 and shot concrete was installed, so that final thickness of primary support came to 35 cm.

d) An installation of pipe roof contained 42 pieces of pipes with 114 mm in diameter and over covering of 5 m to 8 m.

e) An excavation at top heading was followed by excavation and installation of primary support at bench and invert. Distance after working face of top heading was 10 m to 20 m with excavation step 0,8 m.

Figure 2 presents the cross section of the tunnel tube with the supported measures.

Changes of stress strain relations in rock-support system around the tunnel`s working face

Excavation in low bearing and tectonic damaged rocks like tectonic clays, clay grain and gravel stone, which are present at the section of Trojane was connected with large amount of stress strain changes in front of the working

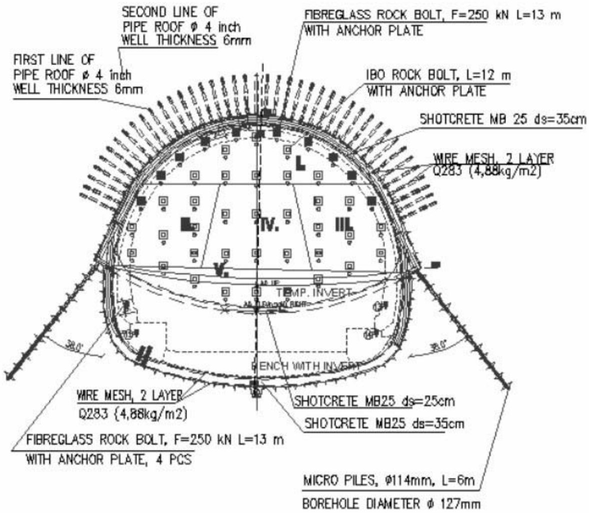


Figure 2. Cross section of tunnel tube with supported measures

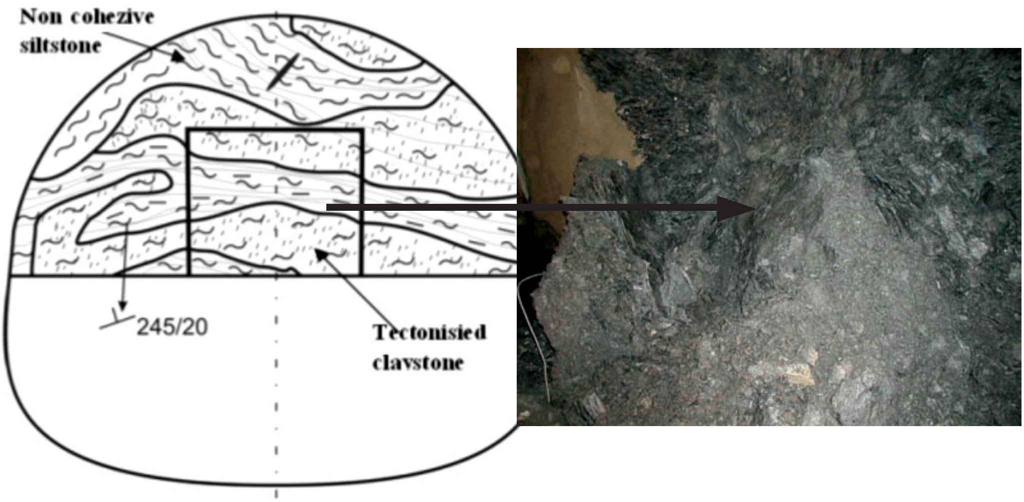


Figure 3. Typical geological cross section of the Trojane tunnel (East left tube).

face and wider area around the tunnel. In mostly clayey and relatively soft rocks, the influence was 3D or even 4D in front of the working face, where D stands for the diameter of the tunnel tube. This phenomenon was extremely unfavourable because of influence to the time dependent subsidence at the wider area. From the so far completed analysis and engineering interpretations, it could be estimated that large amounts of clayey components essentially influence the time dependent deformations. That was significant for the right judgment of possible deformations in a long time period.

Very heterogeneous geological structure and primary damaged rocks were often indicated in the differential subsidence of tunnel sidewalls. That also unfavourably affected the governing of the amount of deformations in the tunnel roof and on the influential section at surface.

BACK GEOSTATIC ANALYSIS OF STRESS STRAIN FIELDS

Modern computer techniques based on numerical methods make quick and qualitative assessment of the changes, which are the result of excavation and installation of the support system. Anyhow, input parameters, which we obtain from standard laboratory and in situ tests, are not always quite realistic, comparing results from analysis obtained with those parameters to actual influences that developed between excavation and support measures. These statements were confirmed during construction of the Trojane tunnel, especially on the sections with clayey material.

Absolute values of the deformations calculated with analysis for estimation were quite smaller than those that actually occurred during the construction and later on. It is interesting that values of the differential deformations were more in accordance with the measured values.

The reasons for increased deformations are:

a) The fact that in two-dimensional analysis the three-dimensional effect is not considered which is essential for this kind of rocks.

b) The fact that input parameters, especially deformational, which we obtain from standard
laboratory and in situ tests are not always quite realistic.

The application of the two-dimensional geostatic analysis for realistic assessment of deformational fields demands much lover values of the input parameters. This contradicts the correct scientific and engineering work. Figure 4 presents in schematic form the influence of excavation on the surface above the tunnel.

Influence of support system stiffness and rock pillar in front of the excavation face

Influence of the supporting system stiffness on amount and time depended behaviour of deformations is essential for an adequate planning of construction. Increased stiffness of supporting system could be achieved in several ways:

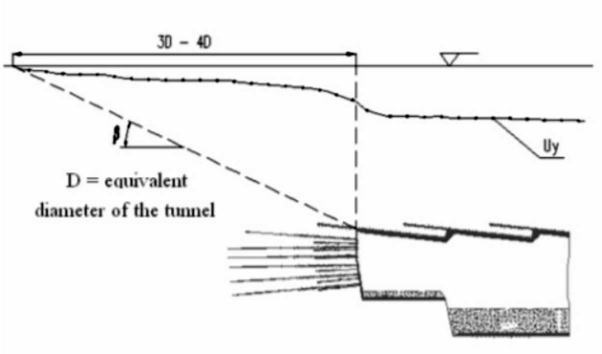


Figure 4. Angle of influence and deformations on the surface due to tunnel excavation

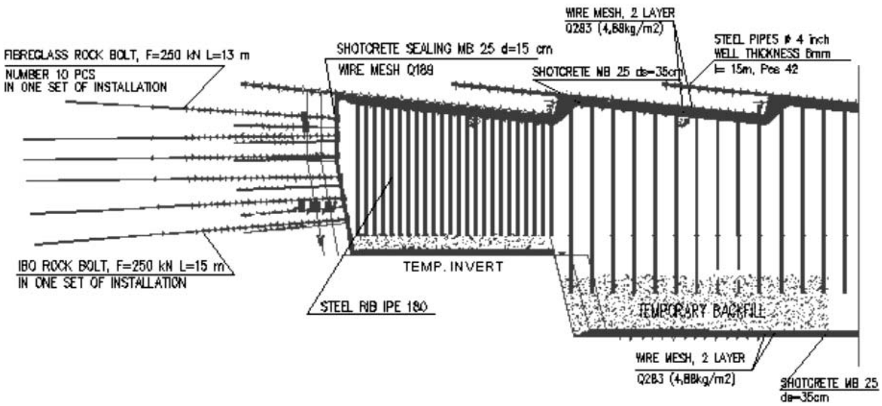


Figure 5. Installation of support system, which ensures quick undertaking of additional loads due to excavation tunnel.

a) by installing the supporting system, which ensures a quick undertaking of additional loads, which are the consequence of the excavation;
b) by excavating and supporting its sections, e.g. excavation with side gallery, where balance is reached in smaller cross sections;
c) by installing the auxiliary support elements with low deformability for increasing stiffness of rock pillar in front of the excavation face;
d) by combining it all.

The question of the intensity of increasing the stiffness of rock pillar in front of the working face by installing the auxiliary supporting elements, which allow normal evolution of the technological process of excavation and support, remains. This understanding is essential for normal operation with fewer interruptions and more continuous construction work.

Results of back geotechnical analysis

3D analysis of stress strain relations were made with software PLAXIS Tunnel 3D, Version 2.0. Geotechnical input parameters for calculation of stresses and strains in supporting system and surrounding rock were partially determent in previous investigations. The values are presented in Table 1.

The simulation of excavation and primary supporting were carried out in such manner that first, the excavation and supporting in top heading in the length of 10 m were performed.

This was followed by the excavation and installation of supporting elements at bench, invert and top heading for 10 m more. Excavation and supporting step was 0.8 m. Deformations were in amount of 200–250 mm in the tunnel and 150–200 mm on the surface. These values are similar to those measured at many points. A part of these measurements is presented in Figure 6, where we could see irregular subsidence, which is a consequence of the extremely unfavourable geologic and geotechnical conditions on this section.

The way of excavating and supporting was the same at this section, but over covering of the pipe roof in the sections of bigger deformations was greater, even in the range of 8 m. This caused the stiffness to increase in the roof section of top heading together with primary support, which contained 2 steel segments IPE 180, 2 micro piles, wire mesh and shotcrete of final thickness of 35 cm.

The results of calculation are presented in figure 8a and figure 8b in form of deformation field. In calculation was also included passive resistance of rock bolts and shot concrete lining in working face in amount of 250 kN/m^2. Comparison between deformation fields obtained with consideration of single pipe roof and over covered pipe roof shows us that influence of increasing stiffness in soft ground is important. Deformations on surface are 20–30% smaller when double pipe roof is used. This was also established during the construction, where in sections with over covered pipe roof deformations were smaller.

Figure 8a and Figure 8b shows differences in deformation fields for normal over covered and fully over covered pipe roof.

Time dependent processes

Development of deformations at the time of excavation and primary supporting of the tunnel is distinct in carbonic rock, since evolution of deformations is completed only in time period of 180 days or more.

Table 1. Input parameters for calculation with SSC model.

MODEL PARAMETERS		Clay schist and mudstone
Unit weight	γ /(kN/m^3)	24
Cohesion	c/(kN/m^2)	28
Angle of the internal friction	φ/°	26
Modified swelling index	$\kappa^\bullet$	0.035
Modified compression index	$\lambda^\bullet$	0.04
Modified creep index	$\mu^\bullet$	0.0003

Table 2. Strength of support elements with different stiffness
Across section; I Moment of Inertia; E Young Modulus

	A [m˘2]	I [m^4]	E [Kn/m^2]	EA [Kn/m]	EI [Knm^2/m]
PIPE ROOF 2X	1,080	0,071411	876.000.000,00	**10.480.000,00**	**406.134,55**
PIPE ROOF 1X	0,180	0,004574	439.000.000,00	**4.520.000,00**	**31.704,91**
SHELL (Es.c. = 3000MPa)	0,35000	0,003572917	3.000.000	1.050.000,00	10.718,75
IPE 180	0,00390	0,000013200	210.000.000	2.047.500,00	6.930,00
SHELL+IPE180 (EB.B. = 3000MPa)				**3.097.500,00**	**17.648,75**
SHELL (Es.c. = 7000MPa)	0,35000	0,003572917	7.000.000	2.450.000,00	25.010,42
IPE 180	0,00390	0,000013200	210.000.000	2.047.500,00	6.930,00
SHELL+IPE180 (EB.B. = 7000MPa)				**4.497.500,00**	**31.940,42**
SHELL (Es.c. = 7000MPa)	0,35000	0,003572917	15.000.000	5.250.000,00	53.593,75
IPE 180	0,00390	0,000013200	210.000.000	2.047.500,00	6.930,00
SHELL+IPE180 (EB.B. = 15000MPa)				**7.297.500,00**	**60.523,75**
SHELL (Es.c = 3000MPa)	0,35000	0,003572917	3.000.000	1.050.000,00	10.718,75
TH 21	0,00234	0,000003390	210.000.000	491.400,00	711,90
SHELL+ TH 21 (EB.B. = 3000MPa)				**1.541.400,00**	**11.430,65**
SHELL (Es.c. = 7000MPa)	0,35000	0,003572917	7.000.000	2.450.000,00	25.010,42
TH 21	0,00234	0,000003390	210.000.000	491.400,00	711,90
SHELL+ TH 21 (EB.B. = 7000MPa)				**2.941.400,00**	**25.722,32**
SHELL (Es.c = 15000MPa)	0,35000	0,003572917	15.000.000	5.250.000,00	53.593,75
TH 21	0,00234	0,000003390	210.000.000	491.400,00	711,90
SHELL+ TH 21 (EB.B. = 15000MPa)				**5.741.400,00**	**54.305,65**

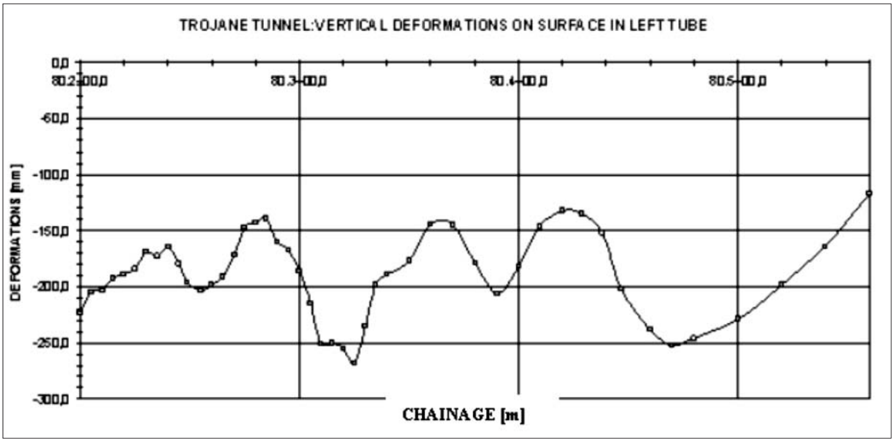
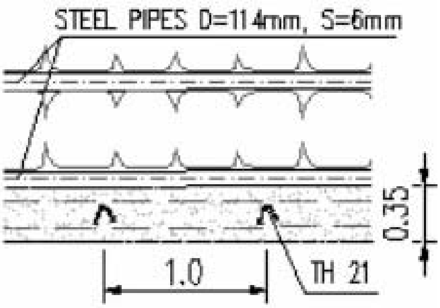


Figure 6. Measured subsidence on the surface along the longitudinal axe of the tunnel tube.

Figure 7a. Standard support elements with steel pipes, TH21, wire mesh and shotcrete



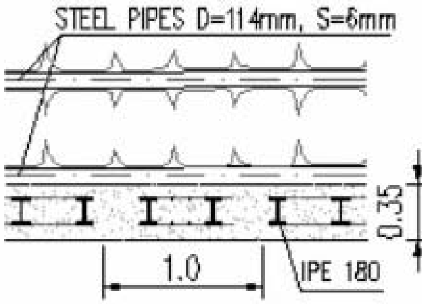


Figure 7b. Support elements with double steel pipes, TH21, wire meshes and shotcrete

Figure 7c. Support elements with double steel pipes, double IPE180 steel rib, wire meshes and shotcrete

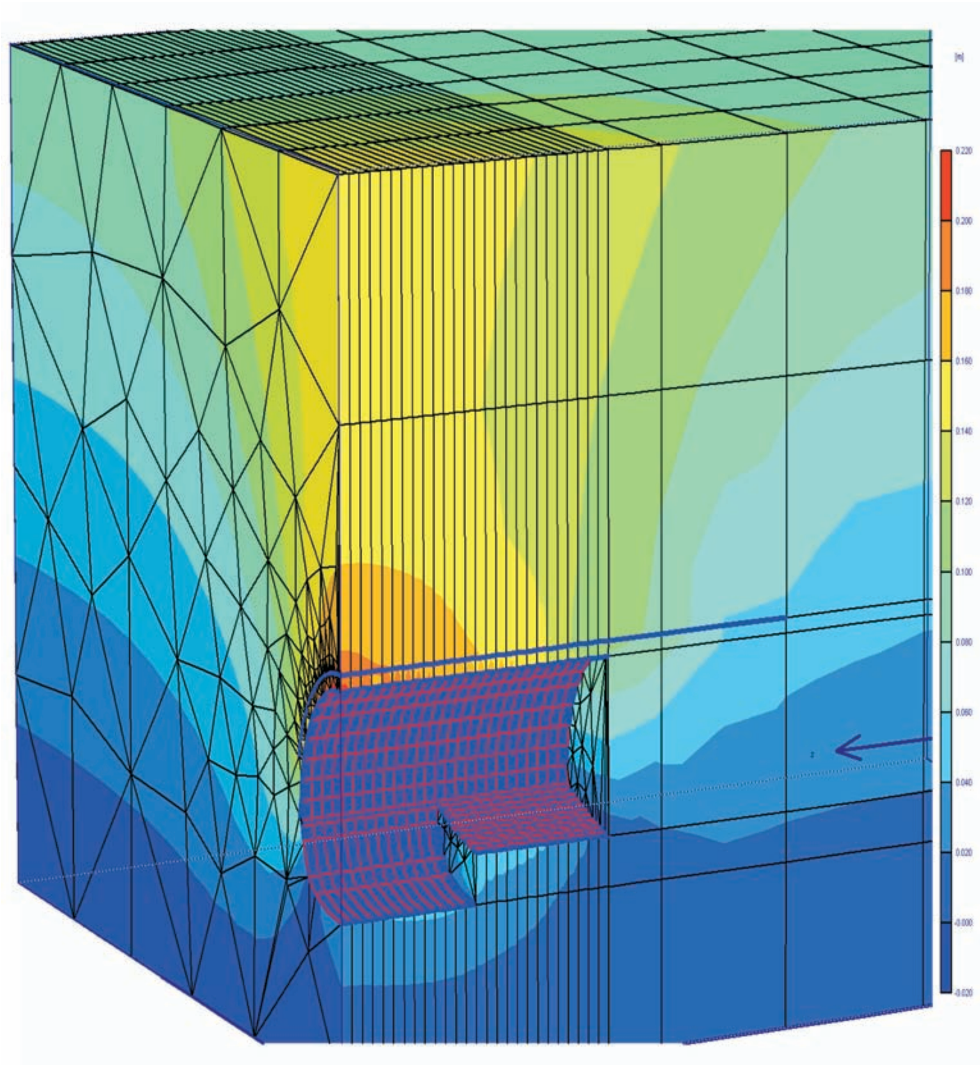


Figure 8a. Deformation fields for normally over covered pipe roof

This was also established in analysis with finite element method, where soft soil creep model was used.

In calculation the hardening of shotcrete was simulated, which gives us increased stiffness of support. During the construction, the support stiffness was also increased with installation of stiff steel segments (2 IPE 180 on section of excavation 0.8 m). The weakness of this combined steel and shotcrete system is that quality of filling in the empty spaces around the steel segments is not easily achieved.

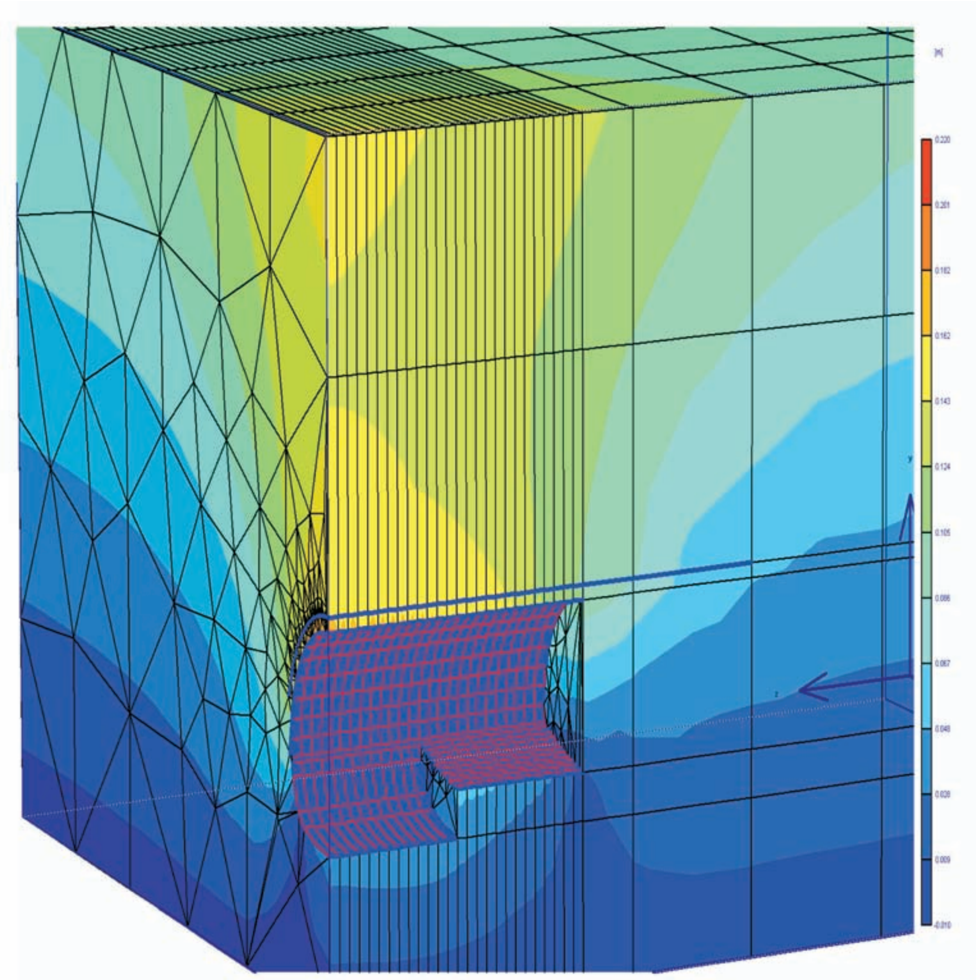


Figure 8b. Deformation fields for fully over covered pipe roof

Conclusion

- Tunnel construction in specific conditions such as in low bearing time dependant rocks under habited area depends of the technological procedure and primary support lining, which were used.
- The area of the surface displacements caused by tunnel excavation is closed to geological and geotechnical conditions and type of surface relief with dipping of the natural slopes and local stabilities.
- The results of presented numerical analyses of the surface displacement caused by tunnel construction were usable. Time dependant dis-

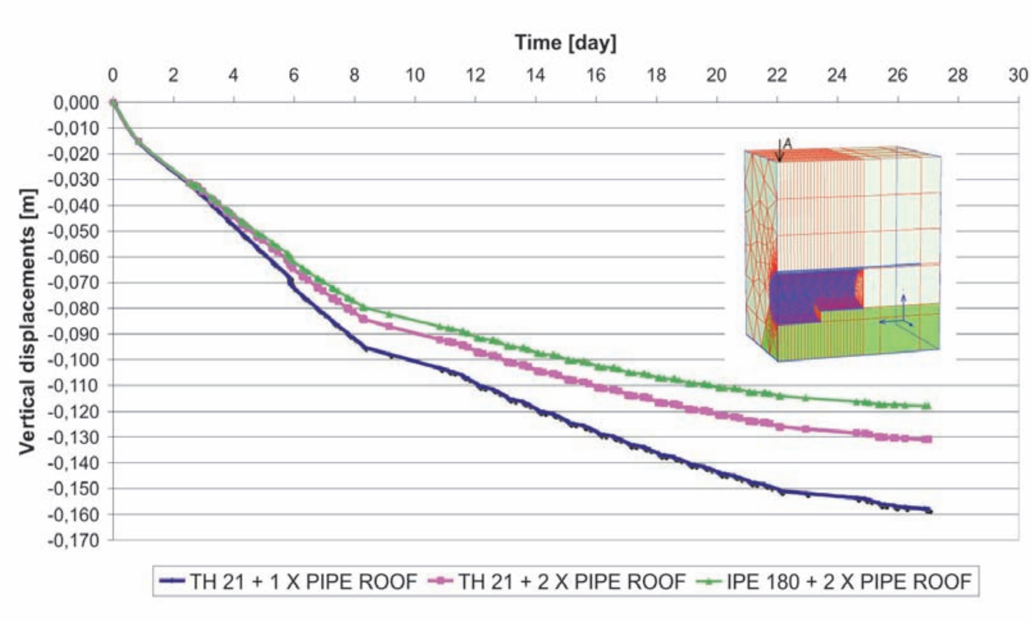


Figure 9. Time dependent surface displacements calculated for different stiffness of the primary lining

placements which were presented help us to make danger volume assessment of objects on the surface.

- Comparison between calculated and measured displacement show us, that this method can be used in similar geological and geotechnical circumstances.

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