HYDROGEOLOGY, ENGENEERING AND ECOLOGICAL GEOLOGY

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STRESS AND STRAIN ANALYSIS DURING THE SLEME TUNNEL EXCAVATION

(Reviewed by the editorial board member B. Maslov)
A stress and strain analysis by Finite Element Method (FEM) has been carried out during the portal section excavation for the Sleme Tunnel right tube.

A comparison has been made of 2D and 3D model results with on-site measurement results for tunnel convergence and ground surface settlement. Multistage excavations with pipe roof support of the working face were modelled.

The calculation model has shown that safe and cost-efficient excavation technology had been applied in the case under consideration.

Introduction

A tunnel excavation requires detailed consideration based on geotechnical and geophysical investigation results, excavation method, supporting method, geotechnical measurements inside a tunnel, land surveying of the ground surface, and an impact of the tunnel excavation on structures in urban areas. Development of traffic networks entails excavation of tunnels in geologically unfavourable conditions. Due to large radii and relatively small ascents and gradients of the road and railroad routes which cannot follow the ground configuration, it is necessary to drive tunnels even in weak rocks. Tunnel is a civil structure built in unknown material, unlike concrete, wooden or steel structures where the material is known in advance. Tunnels are also built in urban areas, with low overburden, which makes the construction still more complex.

Theoretically, excavations under low overburden are those with the overburden height H being less than the tunnel diameter D (H<D).

When a tunnel is excavated with low overburden in weak and loose rocks, the possibility of working face caving-in increases, thus additional measures need to be taken including supporting and multistage excavation.

Excavations when height H = D are made with particular caution and careful selection of technology, which calls for wide experience. Designers tend to avoid portal zone excavation, which requires high portal cutting or side cut. This approach is easily applied outside urban areas, but when the works are carried out in urban areas there is often no alternative.

For road tunnels with two lanes (width D), the New Austrian Tunnelling Method (NATM) defines height H ≤ 2D with low overburden, and this approach has been accepted in construction practice. Width D ≈10 m is adopted for typical cross-section of a road tunnel in Croatia. Figure 1 shows heights of overburden H = D = 10 m and H = 2D = 20 m.

Figure 1. Tunnel portals [3]

The NATM method can be tailored to frequent changes in geological and geotechnical conditions at the working face when the neighbouring geological formation is integrated with the ring-shaped support structure which, to put it simply, means that the rock itself is made a part of the support structure. The majority of time-related processes of stress redistribution happen in the bearing rock mass arch, a zone around the tunnel opening. The term "rock mass arch activation" means activities undertaken to maintain and/or increase the rock bearing capacity and use it to favourably affect the development of the induced stress state, since the tunnel boring changes the in-situ stress state σ_v^o and σ_h^o into a considerably less favourable induced stress state σ_v and σ_h.

Unlike mechanical excavation of an entire profile in favourable geological formations, boring in rock mass with frequently changing geological and geotechnical conditions along the tunnel line requires that a multistage excavation method be used. Profile development using a multistage excavation method is applied when the excavation with low

overburden is carried out in urban areas, weak rocks/soil, fault zones, and when excavation is carried out for underground rooms with larger cross-sections. A multistage excavation is usually carried out in three stages: top heading (Stage I.) – bench (Stage II.) – invert (Stage III.). The distance between the stages depends on geological conditions at the working face. In weak rocks/soil, the bench and invert are excavated in intervals exceeding one tunnel diameter D, in order to quickly close the support ring and reach the new state of balance preventing occurrence of larger convergences.

The majority of theoretical research is into the tunnel excavation with low overburden focus on excavation using the Tunnel Boring Machines (TBM) in urban areas. The reason is a possibility of forecasting ground settlement as the most frequently encountered side effect of tunnelling within urban areas. This topic is particularly interesting for development and use of the TBMs with large diameters, which thus indirectly influences the direction in which the

research goes. However, in excavation of shorter tunnels, frequent changes of tunnel cross-section and surface areas and side passages, the TBM is still much more expensive than excavation using the NATM.

Authors writing about the NATM focus on the cases of low overburden and use of NATM in weak rock mass and urban areas, when NATM is used instead of the TBM. The Croatian authors have also published works [10, 16] describing examples of tunnel construction (design and construction). Some authors analyse incidents occurring during excavation of tunnels with low overburden using the NATM,

which are frequently due to insufficient knowledge of the method [1].

The highest risk encountered when excavating in weak rocks with low overburden is ground surface settlement, since consequences might sometimes be grave (Figure 2), even disastrous, causing human casualties. Therefore, the excavation methods chosen need to ensure security of work performance and minimum settlement, which depends on geological and geotechnical characteristics of the rock mass, tunnel cross-section and excavation technology.

London (2002)   Munich (1994)
Figure 2. Ground surface caving-in during tunnel excavation in urban areas [1]

High standards of the work carried out along the tunnel line minimize the risks during excavation. However, caving-in happens during the tunnel excavation and accidents in tunnelling practice cannot always be avoided.

A condition analysis needs to be made of endangered structures along the tunnel line which might respond to an underground excavation before the tunnel excavation starts.

In damage classification, it is necessary to differentiate between the damage on load-bearing parts of the structure which could ultimately cause collapse of the structure or a part thereof and the damage on secondary structural members.

Ground surface settlement surveying (levelling, GPS – Geodetic Positioning System) is carried out during the tunnel excavation, and optical 3D measurements are carried out in the tunnel including measurements of excavation outline benchmark displacement (profile convergence).

Ground settlement caused by tunnel excavation at a certain distance gradually increases closer to the working face. To mitigate settlements and stabilise the excavation, it is necessary to improve the rock mass during the excavation process by grouting, pipe roof installation, soil freezing or core reinforcement by fibreglass pipes [12]. This paper discusses use of pipe roof for reinforcement [7].

Figure 3. Excavation protection with paling [3]

The pipe roof method is a modern version of paling (steel rods and breakdown sheets, Figure 3), an old mining method of working face protection against material falling in

and easier installation of the support system members during the rock mass excavation.

A pipe roof method (Figure 4) is a technique where grout-mix filled perforated steel pipes are embedded along the tunnel outline in the excavation progress direction, which creates a grouted zone as a protective arch under which the excavation is carried out [11, 13]. An actual pipe roof installation effect could only be evaluated by back analysis once the geotechnical measurements have been completed, since it depends on the rock type, number of installed pipes, diameter of rock bolt walls and excavation method.

Figure 4. Pipe roof installation [3]

Land Surveying Results

The Sleme Tunnel is situated at the Vrata – Delnice section of the Rijeka– Zagreb Highway. The 835 m long left tube was finished in 1995, and the 858 m long right tube in 2008. The right tube portal excavation was based on the known geology and using the pipe roof support.

The tunnel line passes through the Lower Jurassic carbonate sediments – liassic limestone with dolomite interlayers. The limestones are light to dark grey, often recrystallised, fossil contents are encountered with rarely, typically in about 0.50 m layers fissured and fractured due to subsequent tectonic activity and occasionally hardly noticed. The limestones are highly karstified in the surface zone, with large crack aperture, and karstification of deposits confirmed during the tunnelling. The crack filling material consists of high plasticity clay and calcite up to 5 mm. The clay is also encountered in form of clusters, lenses and interlayers. The surface is mostly covered with fine detritus and clays-sandy material. The rock mass is heterogeneous and it consists of clay, sand and loose karst rocks.

The area in which the Sleme Tunnel was built is tectonically highly fractured and it belongs to the geotectonic unit of Dinaric carbonate platform. Generally, the area is characterized by variable discontinuity orientations on a relatively small space, which is an indication of its being exceptionally tectonically fractured, as was actually confirmed during boring of both tunnel tubes.

The measurements were carried out in the west portal. Because the entire section was at risk of the working face caving in, a pipe roof was installed consisting of ϕ114.3/6.3, 15.0 m long steel pipes filled with cement grout mix as a reinforcement of the top heading arch.

The excavation outline displacements were measured with optical 3D method in three benchmarks: in benchmark 1 in the u top heading apex, and in benchmarks 2 and 3 in the bench sides (Figures 5 and 6).

Figure 5. The Sleme Tunnel portal with marked positions of the right tube measuring points 1, 2, 3, R1 and R2

Settlement was measured using optical methods. Two benchmarks were set at the chainage 34+961.50 of the west portal, 10 m from the excavation start point. The benchmark installed in the tunnel axis is R2, and the benchmark R1 is placed 8.50 m left from the tunnel axis. Positions of benchmarks R1 and R2 are marked in Figures 5 and 6. The measurement results are given in Table 1.

Table 1

Measurement results for profile convergence and settlements during the Sleme Tunnel right tube excavation

Chainage	Overburden (m)	Measurement of	Vertical displacement u_y(mm)	Transverse displacement u_x(mm)	Longitudinal displacement u_z(mm)
34+962.50*	8.5	convergence	-12	3	8
34+975.00*	11.0	convergence	-5	7	3
34+985.00	17.0	convergence	-4	5	3
34+996.00*	21.0	convergence	-2	3	3
34+961.50	8.0	settlement	-17		

* Numerical calculations for 10 m and 20 m high overburden

For comparison, the largest measured horizontal (transverse) displacement of the left tube at a 25 m distance from the entrance [8], in identical rock mass, was 20.42 mm, which is considerably more than the displacement in the right tube (3–7 mm). A lot of problems were encountered during the left tube excavation work, and a top heading arch was reinforced using the paling (Figure 3). Minor convergences in the right tube are caused by installation of the pipe roof.
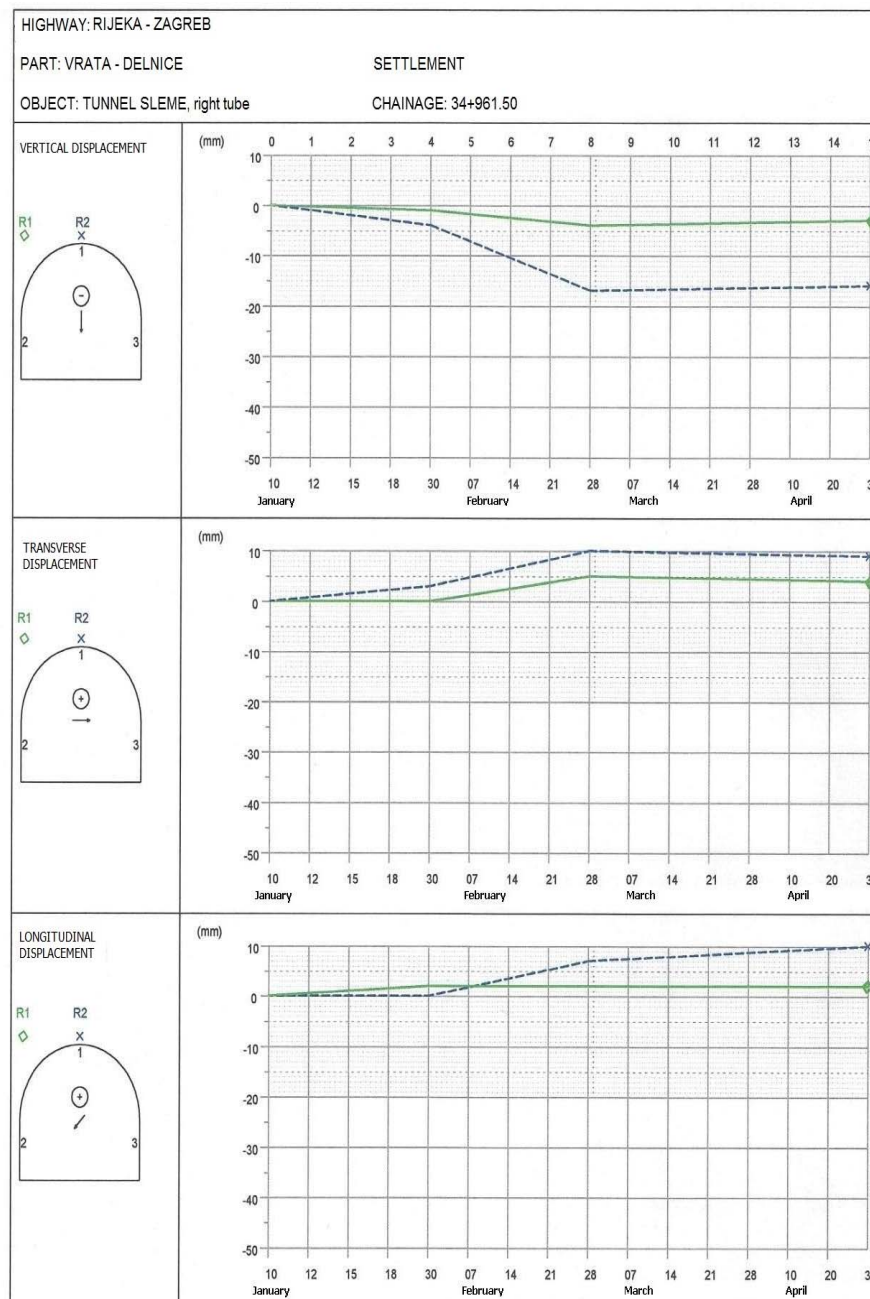

Figure 6. Settlements measured at chainage 34+961.50

3. Excavation Numerical Modelling

An important role in selecting an excavation method is played by numerical modelling of excavation in rock mass based on geotechnical data for the constructed tunnels. [2]. Some of the problems encountered in numerical modelling of weak rocks are described in the paper [6]. Numerical analysis, together with geotechnical measurements, ensures safer and more cost-efficient design of tunnels and other underground structures [10]. Actual mechanical properties of a rock mass are very hard to determine [9] and even the best field investigations could render incomplete data. The tunnel construction is faced with a problem of considerably varying physical and mechanical characteristics of the rock mass, which results in sudden changes in geological conditions that have a major impact on the tunnel excavation. Stress variations within the rock mass/soil quite often result in working face caving in when reactive

pressure with which the primary support acts on the rock mass is not sufficient. The lower overburden causes less favourable stress state due to the heterogeneous nature of the rock mass, lower self-supporting capacity and surface impacts (water courses, structure load acting on the surface, dynamic load of roads).

The stress and strain analysis with the FEM method uses the following software: Sage Crisp 4 for 2D models [15] and Plaxis 3D Tunnel for 3D models [14].

Calculations were made for 10 and 20 m high overburden, for three-stage excavation without the pipe roof (Case a), three-stage excavation with pipe roof in top heading arch – design concept (Case b), and two-stage excavation (top heading+bench, invert) with use of pipe roof and 2 m invert excavation stage (Case c). Ultimately, the two-stage excavation with reinforced pipe roof and 4 m invert excavation stage (Case d) was carried out.
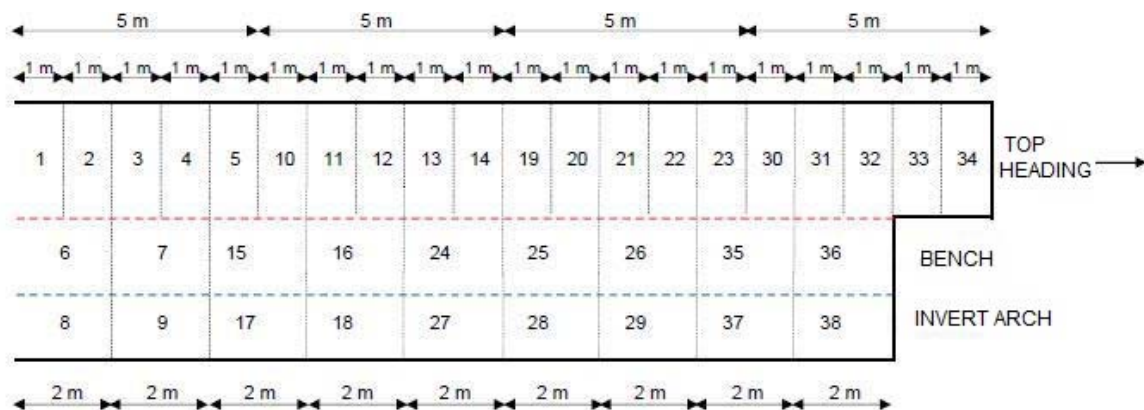

Figure 7. Three-stage profile excavation for 3D calculation

The computational model of three-stage excavation using NATM method – top heading, bench, invert consists of six phases (Figure 7):
1. Top heading excavation
2. Shotcrete placement into the top heading
3. Bench excavation
4. Shotcrete placement into the bench
5. Invert excavation (excavation stage 2 m)
6. Shotcrete placement into the invert.

First, the top heading excavation and supporting with shotcrete on a 5 m long section were simulated, with 1 m cycle step (excavation and support). Then, excavation and shotcrete supporting of the bench on a 4 m long section was simulated. The cycle step was 2 m. Next, the excavation and shotcrete supporting of the invert on a 4 m long section was simulated, with 2 m cycle step. A new top heading, bench and invert excavation cycle followed until the top heading length reached 20 m, and bench and invert length 18 m. The total number of excavation cycles was 38, and the total number of 3D calculation stages 76. A model was created for a half of the cross-section excavation, since the tunnel cross-section is symmetrical related to the axis (Figure 8).

Since 3D calculation can show results for several cross-sections with co-ordinates corresponding to the excavation stages, the cross-sections z = 0 (initial plane) are shown which present the calculation in the excavated and supported part of the tunnel located 20 m from the working face (z = –20 m). Figure 8 shows position of these planes.

The computational model of two-stage excavation using the pipe roof consists of four phases (Figure 9):
1. Top heading and bench excavation
2. Shotcrete placement into the top heading and bench
3. Invert excavation (excavation stage 4 m)
4. Shotcrete placement into the invert.

A 15-node wedge finite element mesh was used for 3D model, while triangular and square finite elements were used for 2D model. In order to eliminate impact of boundary conditions, width of the coverage area selected for calculations is four times tunnel opening to the left and right from the tunnel axis. In the tunnel axis direction (longitudinal direction), the coverage area width in 3D analysis was 50 m.

The adopted physical and mechanical characteristics of the portal zone rock mass [4, 5], pipe roof and primary support are given in Table 2. The Mohr–Coulomb elastic–ideally plastic model was selected for the rock mass, and an isotropic- elastic model for the primary support.

The pipe roof consists of 29 steel pipes, ϕ114.3/6.3, 15 m long, filled with cement grout mix. The shotcrete thickness is 20 cm, and rock mass reinforcement using pipe roof in the top heading arch is 50 cm, which approximately corresponds to the as-built situation. Since there is no general rule for calculation of the pipe roof reinforcement, an approximate method was used to calculate modulus of elasticity acc. to [6], Table 3, where the weak rock mass was replaced with a system comprising steel pipes + grout mix. Bulk density of the zone reinforced in this way is 19.2 kN/m^3.

The calculation results are tabulated and shown in a diagram (stresses, vertical displacements, ground settlements). Figure 10 shows the pipe roof reinforcement for two-stage excavation of the whole profile (2D calculation), effective vertical stresses (a) and vertical displacements (b) for a 10 m overburden.

The 2D calculation results, i.e. ground surface vertical displacements – settlements with 10 m high overburden after excavation of all stages, are shown in Figure 11.

The 3D calculation results of the ground surface settlement with visible effect of the excavation pipe roof reinforcement are given in Figure 12.
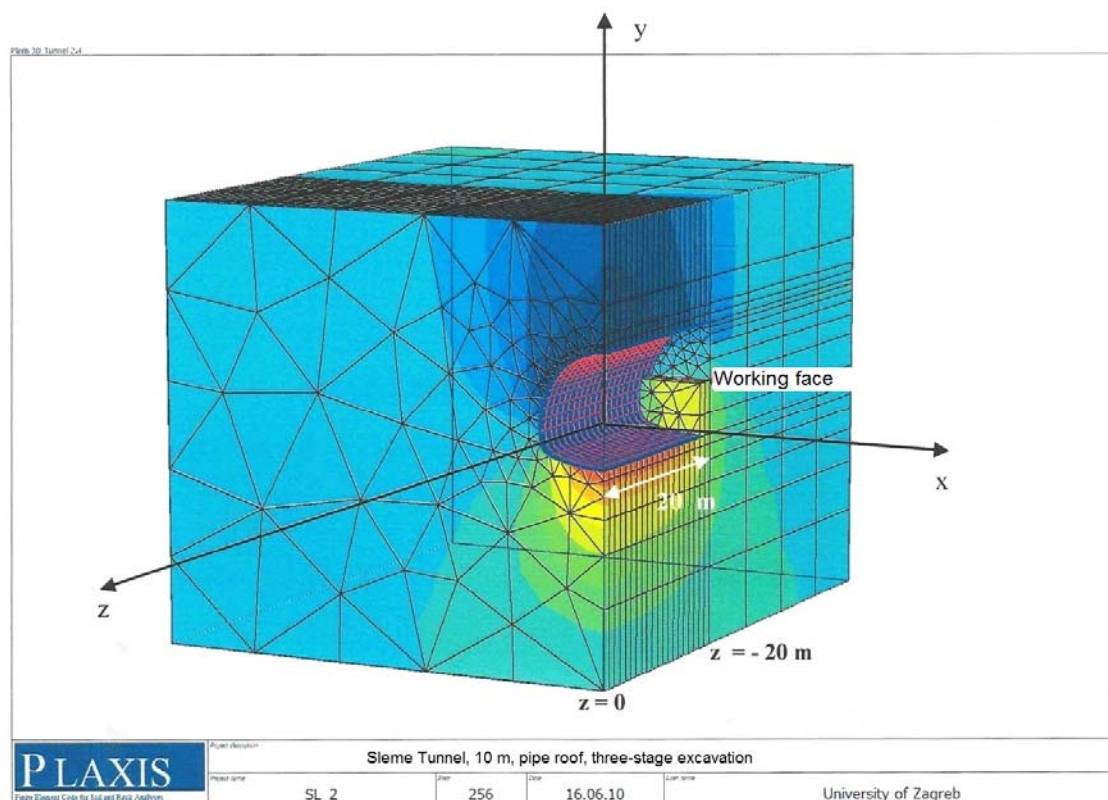
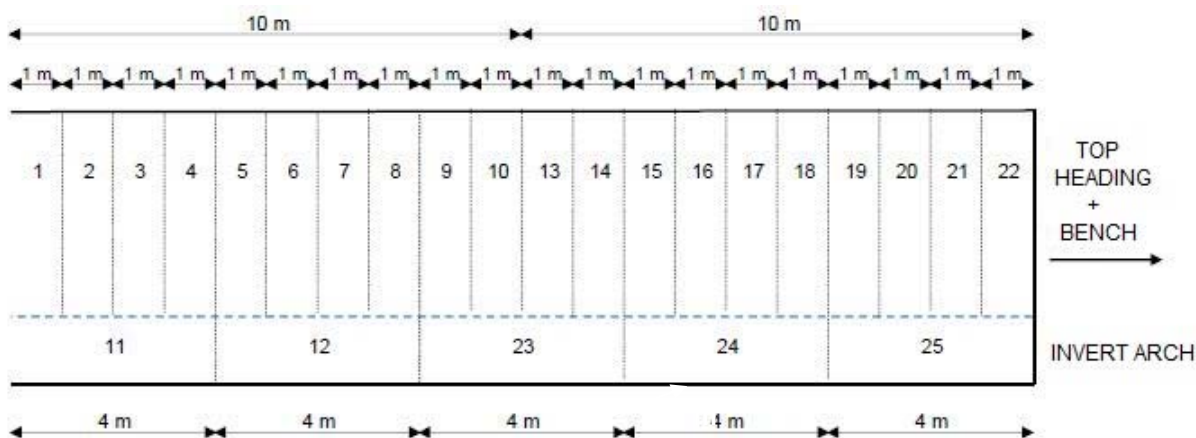**Figure 8. Position of plane z = 0 (excavation of whole profile) and z = –20 m (working face)**

Figure 9. Two-stage profile excavation for 3D calculation

Table 2

Physical and mechanical characteristics of the rock mass, pipe roof and primary support

	Rock mass – Portal zone	Pipe roof – reinforced rock mass	Primary support – shotcrete
Modulus of elasticity E (MPa)	300	19,900	3,000
Poisson coefficient ν (-)	0.25	0.25	0.20
Bulk density ρ (kN/m³)	26.5	19.2	25.0
Angle of internal friction φ (°)	28	-	-
Cohesion c (kPa)	40	-	-

Table 3

Calculation of the modulus of elasticity of the pipe roof reinforcement

Element	Surface area (m²)	Modulus of elasticity (MPa)	Product
	a	b	a · b
Pipes	0.06	210 E+03	12.60 E+06
Grout mix	5.94	18 E+03	106.92 E+06
Sum:	6.00		119.52 E+06
Modulus of elasticity of rock reinforcement with pipe roof mass		19,900	

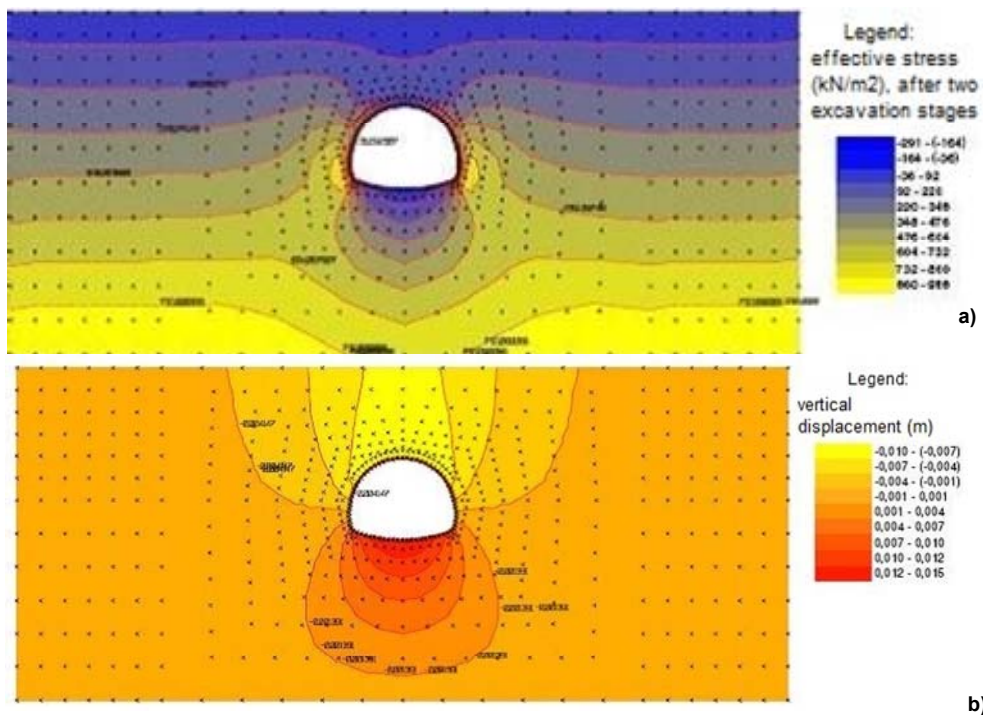**Figure 10. Two-stage excavation with pipe roof reinforcement, 10 m overburden:**
a) effective stress, b) vertical displacement
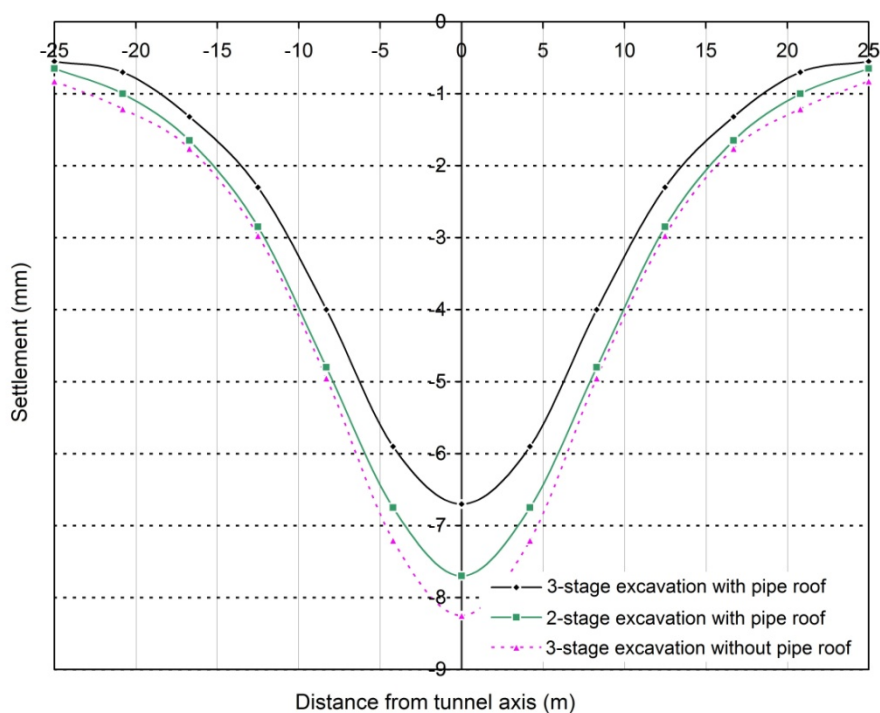

Distance from tunnel axis (m)

Figure 11. Ground surface settlement after excavation of all stages, 2D calculation
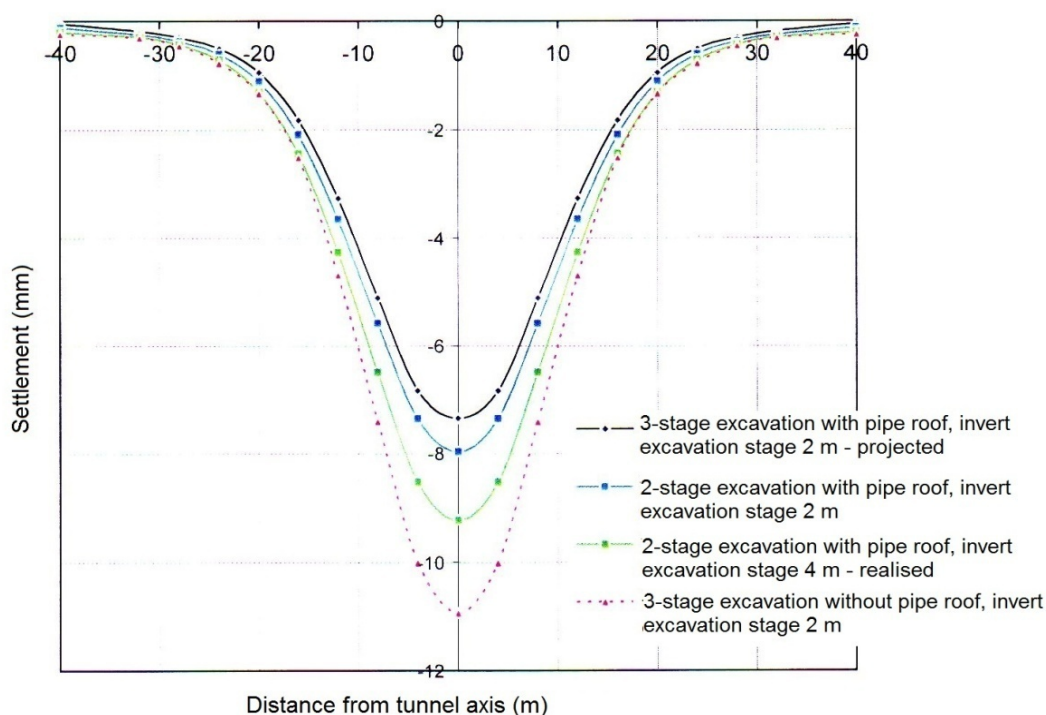
Figure 12. Ground surface settlement after excavation of all stages, 3D calculation

Use of pipe roof mitigates convergences in the tunnel and settlements. In real conditions, impact of the underground opening on the settlement equals radius of plastification of the rock material which depends on equivalent radius of the opening. With increase in overburden, the pipe roof effect on the settlement decreases, so when the overburden is higher, the pipe roof acts so that it reduces convergences in the top heading.

Values of vertical displacements of the top heading apex and ground surface after whole profile excavation

(*20 m from the working face in plane z = 0) obtained by calculations are given in Table 4. Calculation results are given for 3-stage excavation (top heading, bench, invert) using NATM method: without pipe roof (Case a) and with pipe roof reinforcement (Case b, Figure 7 – design concept); and for 2-stage excavation (top heading + bench, invert) with pipe roof and 2 m invert excavation stage (Case c) and 4 m (Case d, Figure 9 – construction).

Table 4

Vertical displacements u_y in characteristic points

Overburden 10 m	Calculation	Vertical displacements u_y (mm)			
		No pipe roof	Pipe roof		
		3-stage	3-stage	2-stage	2-stage
Point		Case a	Case b	Case c	Case d
Ground surface	2D	- 7.5	- 6.7	- 7.7	
Top heading apex	2D	- 11.2	- 8.6	-9.7	
				Invert excavation stage 2 m	Invert excavation stage 4 m
Ground surface	3D*	- 8.5	- 7.3	- 7.9	-9.2
Top heading apex	3D*	- 11.9	- 9.4	- 10.0	- 11.4
Overburden 20 m					
Ground surface	2D	- 9.0	- 7.8	- 9.7	
Top heading apex	2D	-19.2	- 13.7	- 16.0	
				Invert excavation stage 2 m	Invert excavation stage 4 m
Ground surface	3D*	- 9.5	- 8.7	- 9.0	-9.9
Top heading apex	3D*	- 18.8	- 15.6	- 15.7.0	-17.0

(* displacement for z = 0; 20 m from working face)

Difference in total vertical displacements for two-stage excavation with 2 m and 4 m invert excavation stage is 1.3 mm for settlement, and 1.4 mm in the top heading apex. The measurement and calculation results show that an optimum invert excavation stage for the contractor would be 4 m. Also, the shorter invert excavation stage results in less intensive overall convergence.

In addition to vertical displacement u_y and excavation cross-section convergences u_x, the 3D stress and strain analysis calculates displacements in working face u_z (pre-convergence, Figure 13) which are particularly important for excavation in weak rock/soil since they ensure timely response. Provided ther are continuous geotechnical

measurements, and sufficiently small strains, a multistage excavation could be done in a smaller number of stages.

Results of 2D and 3D numerical calculations are compared with the results of measurements at chainages 34+962.50 and 34+975.00 for 10 m high overburden and actual two-stage excavation with pipe roof and 4 m invert excavation stage (Table 5.).

The results of 3D calculation for 10 m high overburden correspond very well with measured convergences. The difference in vertical displacements u_y is -0.6 mm, and difference in transverse u_x and longitudinal u_z direction is only 2 mm, which is negligible.
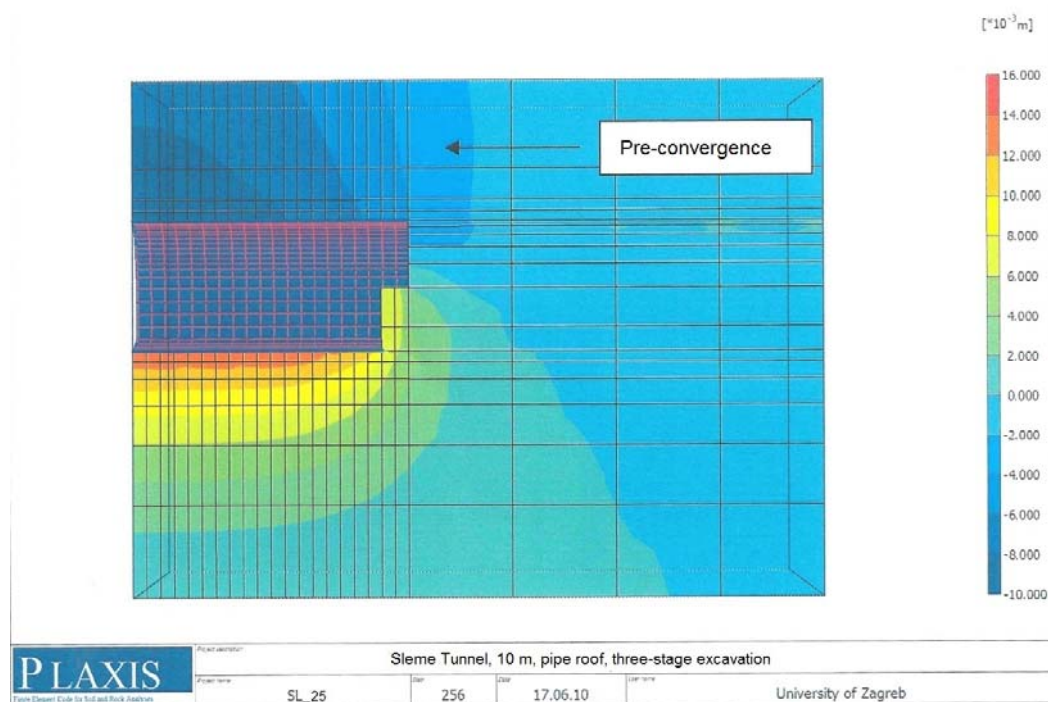
Figure 13. Vertical displacements u_y in longitudinal section, overburden height 10 m, pipe roof

Table 5

Comparison of measurement results and calculated displacements

Overburden 10 m	Vertical displacement u_y (mm)	Transverse displacement u_x (mm)	Longitudinal displacement u_z (mm)
Measured	-12.0(100%)	7.0 (100%)	8.0 (100%)
2D	-9.7 (81%)	-	-
3D	-11.4 (95%)	9.0 (128%)	10.0 (125%)

With such convergences (maximum 12 mm), a 0.20 m thick support system is sufficiently yielding not to cause the support failure, which complies with the NATM principles. The same applies to 20 m high overburden. The highest calculated convergence in a tunnel for two-stage excavation is 17 mm, which is in good correspondence with measured convergences. Displacement calculations and measurements indicate that it is possible to make a more cost-efficient tunnel excavation under the pipe roof protection in a smaller number of stages, i.e. in two instead of three design stages, which is a common approach to excavation of portal sections in weak and weathered rocks.

Conclusions

Excavation of a tunnel with low overburden is a complex design 3D problem. In such conditions, the rock mass is very heterogeneous and has variable physical and mechanical characteristics, it is prone to surface impacts and thus potentially unstable. The practice shows that even when the rock mass is well investigated, it is still not always possible to forecast all the geological "pitfalls" of the underground. Sudden and unpredicted changes in rock mass characteristics could, in worst case scenario, cause tunnel caving in and damage to the structures on the surface.

Numerical modelling enables predicting of rock mass reaction to the tunnel profile excavation. An advantage of the 3D FEM analysis is that it makes possible not only modelling of an excavation by stages, but also by change in excavation stage length in a particular phase (top heading, bench or invert). The excavation stage is commonly determined empirically, according to some of the engineering rock mass classifications or by in situ variation of excavation stage with permanent geotechnical measurements. Based on its long-term experience, contractor had decided not to carry out the three-stage excavation but rather to make a joint excavation of the top heading and bench during the first stage and to carry out the invert excavation subsequently (second stage). The 3D calculation has confirmed that the excavation stage of 4.0 m for the invert (the envisaged stage was 2.0 m) is a boundary stage where strains are such that they do not cause tunnel caving in. An analysis of an optimum stage for excavation progress shows that there are cases when the stages in certain phases can be longer than designed. A contractor has to decide whether to assume certain geotechnical risks. An increase in an excavation stage shortens the construction time and reduces the project costs. Quality forecasts are possible only with numerical simulations of the 3D model of the tunnel excavation, along with experience, intuition and field measurements. This approach results not only in project cost reduction but also in improvement of work performance safety.

Data on geotechnical measurements, which are usually kept unused on the archives, need to be used in the stress and strain analysis so that the empirical knowledge could be complemented with the results of numerical calculations. The analysis is intended to determine the degree to which the numerical calculation results correspond with the field measurement results. The back analysis of the as-built status of the tunnel is particularly important for verification of design parameters and excavation method.

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АНАЛІЗ СТРЕССУ ТА ДЕФОРМАЦІЙ ПІД ЧАС РОЗКОПОК ТУНЕЛЮ СЛЕМЕ

Проведено аналіз стресу і деформації методом скінченних елементів (МСЕ) під час розкопок розділу портал на правій трубі тунелю Слеме.

Порівняльний аналіз виконано на основі 2D і 3D моделювання з результатами вимірювань на місцевості для конвергенції тунелю і поверхні наземних будов. Багатоступінчасті розкопки з підтримкою труби на даху робочої поверхні були змодельовані.

Розрахункова модель показала, що безпечна і економічно ефективна технологія розкопки були застосовані в розглянутому випадку.

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АНАЛИЗ СТРЕССА И ДЕФОРМАЦИЙ ПРИ РАСКОПКАХ ТОННЕЛЯ СЛЕМЕ

Проведен анализ стресса и деформации методом конечных элементов (МКЭ) при раскопках раздела портал на правой трубе тоннеля Слеме.

Сравнительный анализ выполнен на основе 2D и 3D моделирования с результатами измерений на местности для конвергенции тоннеля и поверхности наземных строений. Многоступенчатые раскопки с поддержкой трубы на крыше рабочей поверхности были смоделированы.

Расчетная модель показала, что безопасная и экономически эффективная технология раскопки были применены в рассматриваемом случае.