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3D NUMERICAL MODELING OF ROAD TUNNEL STABILITY – THE LALIKI PROJECT

MODELOWANIE 3D DLA OCENY STATECZNOŚCI TUNELU DROGOWEGO W LALIKACH

The paper presents an evaluation of 3D numerical modeling of the first road tunnel built mining method in Poland, on the Polish-Slovakian border. The Laliki tunnel presents a unique case for an assessment of 3D numerical modeling as a tool for tunnel design due to detailed data being available from monitoring during construction. Geotechnical evaluation carried out for the project proved insufficient, which called for an urgent necessity to work out an additional reinforcement of preliminary lining, using forepoling as the arch support. Stability analyses of the tunnel with new lining were carried out on the basis of 3D numerical modeling of displacements and stresses around the tunnel.

Keywords: tunnel stability, geotechnical conditions, numerical modeling

W pracy przedstawiono zachowanie się utworów fliszu karpackiego w rejonie tunelu drogowego wykonanego w ciągu drogi ekspresowej S-69 Bielska-Biała – Zwardoń. Cechą charakterystyczną fliszu jest różnorodność i anizotropia właściwości fizycznych wynikająca z naprzemianległego zalegania słabych łupków i silnych piaskowców. Realizacja tunelu okazała się bardzo trudna z powodu niskiej jakości górotworu, licznych zaburzeń, nieregularności warstw skalnych oraz występowania licznych stref osłabienia.

Praca zawiera wyniki przestrzennych obliczeń numerycznych przeprowadzonych dla oceny skuteczności wzmocnienia górotworu za pomocą dwóch typów obudowy wstępnej tj.: obudowy betonowej wraz z kotwieniem górotworu oraz obudowy betonowej wraz z mikropalami. Dla oceny stateczności tunelu wykonano analizę wyników pomiarów konwergencji.

Analizowany tunel przecina grzbiet niewielkiego wzniesienia i znajduje się ok. 5 km od granicy polsko – słowackiej. Całkowita długość tunelu wynosi 678 m. W odległości 30,7 m od osi tunelu drogowego znajduje się oś tunelu ewakuacyjnego. Maksymalna głębokość wykonywania tunelu wynosi 35 m. Podstawowe dane tunelu drogowego są następujące: żelbetowa konstrukcja tunelu o dwóch warstwach: zewnętrznej (obudowa wstępna) i wewnętrznej (obudowa ostateczna), zewnętrzne gabaryty tunelu drogowego: szerokość 13,48 m, wysokość 9,50 m, szerokość użytkowa to 11,2 m w tym 8,40 m – jezdnia, wysokość nad jezdnią w osi tunelu: 6,55 m, wysokość skrajni drogi: minimum 4,70 m.

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Tunel drogowy, podobnie jak tunel ewakuacyjny, wykonywany był metodą górniczą zgodnie z zasadami Nowej Austriackiej Metody Tunelowania (NATM), a w rejonie portali metodą odkrywkową. Pomiędzy tunelami są cztery przejścia. Oś trasy tunelu ma stały spadek podłużny w kierunku portalu południowego (Karakuş i Fowell, 2004).

Na głębokości drążenia napotkano dużą zmienność litologii warstw skalnych oraz kątów ich zapadania (rys. 2 i 3). Miąższości warstw skalnych łupków ilastych są na ogół nieduże tj. do 10 cm, a sporadycznie w warstwach piaskowca do około 50 cm. Kąt upadu warstw skalnych określono w przedziale 37°÷86°, przy rozciągłości od 50° do 90° (tablica 1). Pod względem tektonicznym masyw fliszowy w którym zlokalizowany jest tunel charakteryzuje się zmiennością strukturalną, a więc i skomplikowaną tektoniką. Występują więc liczne sfałdowania, uskoki oraz spękania i szczeliny o zróżnicowanej charakterystyce.

Ze względu na zmienność warunków geotechnicznych zaprojektowano wstępnie cztery typy obudowy wstępnej (tabela 2): od typu 1 (dla najkorzystniejszych warunków geotechnicznych) do typu 4 (dla najtrudniejszych warunków geotechnicznych). Trudne warunki w trakcie drążenia tunelu oraz oznaki lokalnej niestabilności czoła tunelu sprawiły, że wystąpiła konieczność zaprojektowania nowego typu obudowy tj. typu 5 z wykorzystaniem parasola mikropalowego (rys. 4).

Dla oceny skuteczności tego typu obudowy przeprowadzone zostały przestrzenne obliczenia numeryczne. Analizę przeprowadzono Metodą Różnic Skończonych w programie FLAC 3D. Wymiary modelu wynosiły 60 m w kierunku osi *x*, 80 m w kierunku *y* oraz 45 m w kierunku *z*. Współrzędne wektora normalnego płaszczyzn osłabienia to (–1.468, –5.472, 1.000). Przyjęto, że drążenie tunelu rozpoczyna się w warstwie mocnej, by po 10 m przejść w warstwę słabą. Parametry warstwy mocnej odpowiadały parametrom łupków ilastych (L_{zd}), które mają porównywalne własności do parametrów przyjętych w projekcie wykonawczym. Warstwę słabą przyjęto o parametrach zbliżonych do faktycznie uzyskiwanych własności w trakcie drążenia tunelu (L_{zl}). Modelowanie przeprowadzono w dwóch wariantach:

1. Typ obudowy 4 – (obudowa z betonu natryskowego + obudowa wbijana wyprzedzająca o długości 4 m + kotwy ułożone radialnie o długości 6 m (rys. 5),
2. Typ obudowy 5 – obudowa z betonu natryskowego + obudowa wyprzedzająca w postaci parasola z mikropali o długości 12 m wraz z iniektem wzmacniającym górotwór w bliskim otoczeniu mikropali + kotwy w dolnej części obudowy.

W tabelach 5 i 6 przedstawiono właściwości materiałów wykorzystanych w modelowaniu. Na rysunkach 6 i 7 przedstawiono wartości sił osiowych w kotwach oraz w mikropalach. Z rysunku 6 wynika, że część kotew zabudowanych wokół wyrobiska przenoszą siły ściskające, ze względu na położenie ich zgodnie z występującymi płaszczyznami osłabienia. Kotwy takie, w przypadku nachylenia warstw zbliżonego do pionowego, nie stanowią zatem wzmocnienia górotworu i zabezpieczenia tunelu.

Mapy wektorów przemieszczeń (rys. 8 i 9) wskazują, że w przypadku zastosowania obudowy typu 4, wartości przemieszczeń mogą sięgać nawet powierzchni terenu i będą one największe w czole tunelu. W przypadku obudowy typu 5 największe przemieszczenia będą występowały również w czole tunelu i będą głównie wynikiem relaksacji naprężeń i uwarstwieniem górotworu. Wartości tych przemieszczeń będą jednak o 30% niższe. Obudowa typu 5 z wykorzystaniem parasola mikropalowego przenosi więc obciążenia górotworu i umożliwia bezpieczne drążenie tunelu w strefach osłabionej jakości górotworu.

Jednym z podstawowych pomiarów stosowanych do oceny stateczności tunelu jest pomiar konwergencji. Pomiary tego rodzaju wskazują na skuteczność stosowanej obudowy w określonych warunkach geotechnicznych. W przypadku tunelu w Lalikach, pomiary konwergencji były prowadzone w 46 przekrojach w odstępach do 20 metrów od siebie a ich zakres obejmował pomiar przemieszczeń w trzech kierunkach na 7 punktach pomiarowych (rys. 11). Maksymalne wartości konwergencji na długości tunelu przedstawiono na rysunku 12. Wynika z niego, że największe przemieszczenia kontury tunelu w obudowie wstępnej osiągające wartość 140÷20 mm występowały na odcinkach tunelu: 200÷240 m oraz 530÷580 m.

W podsumowaniu artykułu stwierdza się, że tunel w Lalikach był realizowany w bardzo trudnych i zmiennych warunkach. Litologia, nachylenie warstw czy nieciągłości zmieniały się nawet po kolejnym zabiorze. Nachylenie warstw skalnych wraz z licznymi płaszczyznami nieciągłości spowodowało konieczność zastosowania dodatkowego typu obudowy z wykorzystaniem parasola mikropalowego. Obliczenie numeryczne pozwoliły na stwierdzenie, że ten typ obudowy pozwala na zmniejszenie przemieszczeń górotworu, które w przypadku zastosowania standardowych kotew mogły sięgać nawet powierzchni terenu. Zastosowanie iniektowanych mikropali tworzących sztuczne sklepienie nad tunelem pozwoliło więc na bezpieczne drążenie tunelu oraz zwiększyło bezpieczeństwo w trakcie jego użytkowania.

Słowa kluczowe: stateczność tunelu, warunki geotechniczne, modelowanie numeryczne

1. Introduction

The very first tunnel ever constructed in Poland provided formidable challenges for designers and contractors, due to complex geotechnical conditions appearing in the mountain region of Southern Poland, where the tunnel was constructed in Carpathian flysch characterized by heterogeneity and anisotropy in physical properties due to alternate bedding with weak slates and strong sandstones (Thiel et al., 1995). Existing discontinuities, also including fractures, resulted from tectonic processes and weathering. Tunneling proved to be very difficult due to a low quality of rock mass, alternate and inclined deposition of geological strata, as well as numerous zones of weakness.

The paper presents the properties of Carpathian flysch formations in the area of the road tunnel driven in the course of S-69 express highway in the section between Bielsko-Biala and Zwardon. In the analysis, 3D numerical modeling was applied in order to examine the interaction between rock mass and tunnel construction. Such an analysis allowed for the evaluation of the road tunnel stability in highly complicated or differentional geotechnical conditions (Siemińska-Lewandowska & Mitew-Czajewska 2009; Shasenko et al. 2009).

2. Overall characteristics of the roadway tunnel in Laliki

The analyzed tunnel cuts through the ridge of elevated ground located approx. 5 km from the western part of the Polish – Slovak border. The total length of the tunnel reaches 678.00 m. The axis of an evacuation tunnel is positioned in the distance of 30.7 m from the axis of the roadway tunnel. The maximum depth of tunnel is 35 m.

Below are presented the essential technical specifications of the analyzed road tunnel:
- tunnel construction – reinforced concrete with two layers: outer layer (preliminary lining) and inner layer (principal lining);
- overall dimensions of roadway tunnel: width 13.48 m, height 9.50 m;
- operational width of typical cross section: 11.2 m, including 8.40-meter wide roadway (driving lanes in the tunnel: 2 × 3.5 m; bands: 2 × 0.7 m);
- height of the crest in the tunnel axis: 6.55 m;
- vertical roadway gauge; minimum 4.70 m.

The roadway tunnel, just as the evacuation tunnel, was driven with the application of the principles of the New Austrian Tunneling Method (NATM), whereas in the area of portals – with the application of open-cast method. The major principles of NATM are (Karakuş & Fowell, 2004):
- the inherent strength of the soil or rock around the tunnel domain should be preserved and deliberately mobilised to the maximum extent possible,
- the mobilisation can be achieved by controlled deformation of the ground; excessive deformation which will result in loss of strength or high surface settlements must be avoided,
- initial and primary support systems consisting of systematic rock bolting or anchoring and thin semi-flexible sprayed concrete lining are used to achieve the above purpose. Permanent support works are usually carried out at a later stage,
- the closure of the ring should be adjusted with an appropriate timing that can vary depending on the soil or rock conditions,

- laboratory tests and monitoring of the deformation of supports and ground should be carried out,
- those who are involved in the execution, design and supervising of NATM construction must understand and accept the NATM approach and react co-operatively on resolving any problems,
- the length of the unsupported span should be left as short as possible.

2.1. Geotechnical conditions obtained before the design

There are four passages between the tunnels. The tunnel route axis demonstrates an unremitting southbound downwards grade. The overburden of flysch strata in the area of the tunnel in Laliki mainly consists of precipitates of silty clay and silt with variable degree of plasticity. Below the quaternary formations there appear detrital formations of flysch strata, created as eluvium of shale, clay and sandstone (Dziewański et al., 2001).

Immediately below the weathered layer, there are tertiary formations consisting essentially of packets of compact shale, interbedded with slates of medium and thick lamination. Among the laminations, also occur arenaceous shales, as well as fine-grained and medium-grained sandstones (Łukaszewicz, 2007).

At the depth of tunneling, considerable alternation of strata lithology and subsidence angles was encountered. The rock strata thickness is generally not large, i.e. up to 10 cm, however occasionally it reaches approx. 50 cm in sandstone strata. The determined dip angle of rock strata ranges between 37°÷86°, with bed's strikes ranging between 50° and 90°.

As far as tectonics is concerned, the flysch massif, in which the analyzed tunnel is located, is characterized by considerable structural alternation (i.e. also complex tectonics). Such conditions are related to the overlapping borderlines of Magura nappe and Silesian nappe. The dislocation of Magura nappe resulted in the appearance of scales and disturbance in regular subsidence of strata in the form of smaller or larger fold faults and dislocations of strata. Apart from inter-layer discontinuities, another distinguishing feature of the massif in the analyzed region is the occurrence of fracture and fault fissures of variable characteristics. The analyzed region is not characterized by complex water conditions, i.e. no water-bearing horizons have been found and increased water inflows appeared only as a result of massive precipitations.

Geotechnical prospecting in the planned tunnel strip allowed on the basis of surface boreholes to distinguish nine sections with different structures and rock mass quality (Table 1).

The evaluation of rock medium was carried out mainly for strong rocks, i.e. sandstones and shales. Unfortunately, the evaluation failed to determine the properties of crumbled and damaged rocks. Furthermore, project-related calculations were based on probable lithology, which was the case in the sections referred to as numbers 2, 4, 6 and 8 in Table 1. In the case of the aforementioned sections, no data related to lithology and rock properties were available, or the body of available data was scarce. What undoubtedly causes considerable difficulties in unambiguous determination of rock properties is the angle of strata dip ranging between 30÷90°, where approx. 30% of the tunnel strip was to be characterized by the angle of dip ranging between 75÷90°. Hence the completed geological prospection in extremely unfavorable cases is limited merely to some dozens of centimeters in the premises of a drilled prospecting hole. As a matter of fact, what is obviously important for a tunnel designer is a prospection for a total width of a heading, i.e. 13.48 m in the analyzed case.

TABLE 1

Geological and geotechnical prospection along the roadway tunnel in Laliki

Parameter \ Section		Number of section								
		1	2	3	4	5	6	7	8	9
		Length of section [m]								
		0÷35	35÷80	80÷167	167÷225	225÷264	264÷453	453÷510	510÷638	638÷678
Geology	lithology	Sh+S	Sh+S	Sh+S	Sh+S	Sh+S	Sh+S	Sh+S	Sh+S	Sh+S
	degree of weathering	W2-W5		W3-W5		W3-W5	W2-W5	W2-W5		W2-W5
Discontinuities	strata orientation – dip	45-90°		55-85°		30-90°	37-62°	35-90°		37-90°
	strata orientation – bed's strike	WSW-ENE		WSW-ENE		WSW-ENE	WSW-ENE	NE-SW		WSW-ENE
	thickness of strata [m]	< 4,0		< 1,0		< 2,0		< 4,3		< 2,5
	degree of weathering	W2-W5		W3-W5		W3-W5	W2-W5	W2-W5	weakness zone	W2-W5
	orientation of fissures	30-60; +BS				45-90; +BS		35-80; BS		60-70; BS
	interval of fissures [mm]	-		-		-	-	-		< 200
	divergence of fissures [mm]	< 5		< 5		< 5	< 5	< 5		< 5
	fissure filling	silty, calcite,				calcite.	calcite.	calcite.		silty, calcite.
Rock mass quality	RQD [%]	0-46		0-25		0-90	0-100	0-45		0-100
	GSI	22-25		22-25		22-25	25-67	25-33		22-25
	RMR	17-20		17-20		17-20	20-62	20-27		17-20
Type of rock mass		IV-1	IV-1	IV-1	IV-1	IV-1	IV-1 IV-2 IV-3	IV-2	IV-1	IV-1

Sh+S – Shale and sandstone rocks – probable prospection

Preliminary lining in the roadway tunnel in Laliki was designed in four different variants depending on local properties of rock mass (Table 2, Table 3). In this case the length of sections obtained on the basis of geophysical measurements and boundaries of sections are discontinuity zone and lithology changes.

TABLE 2

Predicted length of section with type of lining

Type of lining	Length of section [m]					
	0-111	**111-161**	**161-298**	**298-511**	**511-561**	**561-678**
	Percente of type lining in section					
Type 1	0	0	0	20	0	5
Type 2	10	15	5	30	5	10
Type 3	30	25	35	10	15	25
Type 4	60	60	60	40	80	60

In general, the designers provided four options of lining scheme to be applied: from type 1 (the most favorable geotechnical conditions) to type 4 (the least favorable conditions). The scheme of type-4 lining is presented in Fig. 1.

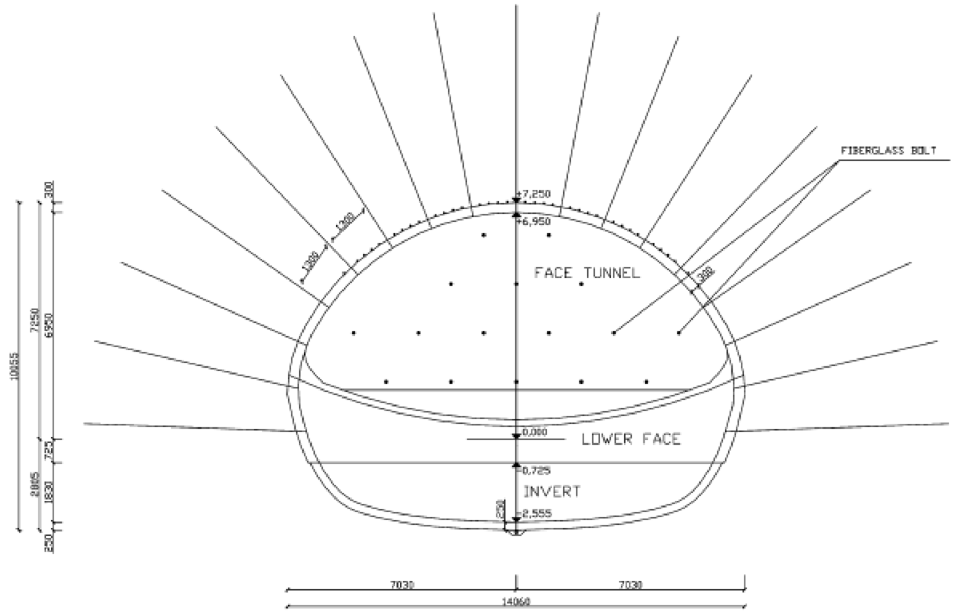


Fig. 1. Cross section of the tunnel in Laliki – type 4

TABLE 3

Characteristics of selected preliminary lining types

Type of protection	Type of lining			
	1	**2**	**3**	**4**
Shotcrete (C20/25)	180 mm	200 mm	250 mm	300 mm
Lattice girder	70/20/30	70/20/30	95/20/30	95/20/30
Steel nets	1×6/150/150	1×6/150/150	2×6/150/150	2×6/150/150
Adhesive bonded anchor bolts	4 m	4 m	4 m	4 m
Self drilling anchor bolts	–	6 m	6 m	6 m

Based on geotechnical and engineering evaluation of rock mass properties, the decisions as to the selection of roof and wall protection of the tunnel were taken on the spot on a daily basis during the construction period.

3. Geological conditions during tunneling

During the construction process, in the tunnel in Laliki and in its premises, systematic observations were carried out, including geological cross sections of working face, pressure measurements in the contact between the lining and rock mass, measurements of stress in concrete and measurements of tunnel convergence.

As part of geological observations, at each advance of the tunnel (i.e. every 0.6 m÷1.5 m), a cross section was determined not only with marked lithology, but also with marked course and character of discontinuities, distance between discontinuities, water conditions. What resulted from the data obtained systematically is the confirmation of predictions formulated on the basis of exploratory bore-holes drilled for the project purposes. In other words, the forecast suggesting considerable strata inclination in the cross section of the driven tunnel has been largely validated. The only difference between the forecast and the obtained data is that the predicted strata inclination was to range between 45÷90°, whereas the actual conditions indicate a constant strata inclination ranging between 75÷85° (Fig. 2).

Fig. 2. Tunnel face with typical distribution of strata

In addition, the analysis of particular cross sections, even those located in the close vicinity from one another, implies that the variability of geological conditions is extremely high, which mainly relates to the distribution of the zones of discontinuity, faults and – only to a slightly smaller extent – to lithology. A thorough analysis of each and every advance seems to indicate that the number of sets of discontinuities, as well as the orientation of dip and the dip proper change almost at every advance of the tunnel. Therefore, a possibility of foreseeing and evaluating conditions even up to only 10 prospective meters fails to be fully effective.

The above statement confirms the changes of percentage of particular lithological strata in the tunnel strip in 5-meter sections (Fig. 3). The chart below indicates that 53.5% of the tunnel was driven in shale rocks, in which the percentage of sandstone never exceeds 15% (Thiel et al., 1995). At the same time, only 13% of the tunnel was driven in rock mass consisting of maximum 30% of sandstone strata.

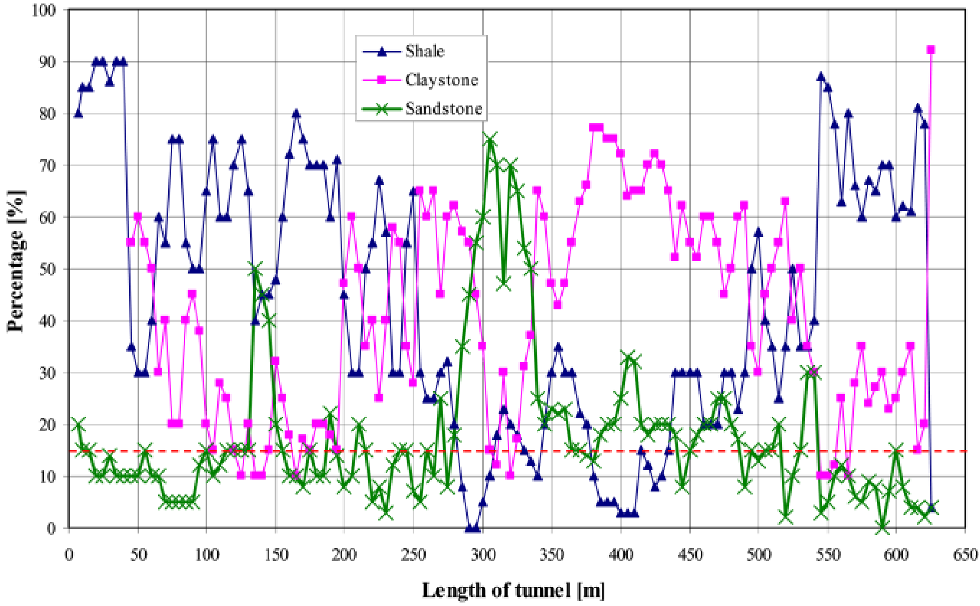


Fig. 3. Change of lithology in tunnel length

The situation described above also resulted in the maladjustment of preliminary lining, designed for the worst conditions (Table 4 – type lining 5). Despite anchoring in advance in the tunnel's roof, fragments of roof strata kept falling off.

In order to protect and increase stability of the driven tunnel, another type of preliminary lining – referred to as 'type 5' in the project (Fig. 4) – was suggested. This was necessary due to symptoms of instability in the face and the roof of the tunnel. The elements of reinforcement were:

- 300 mm thick shotcrete;
- lattice girder 180/30/20;
- steel nets 2×8/150/150 (two layers steel nets where bars are 8 mm and the distance between bars are 150 mm);
- forepoling (ϕ89/5 with the length of up to 20 m filled with cement grout);
- 6-meter long self-drilling anchor bolts.

Forepoling is relatively frequently applied for tunneling in difficult geotechnical conditions (Ocak, 2008; Shin et al., 2008). Finally, the type of lining was determined based on the rock mass quality RQD, RMR (Bieniawski, 1989), classification QTS (Quality Testing Systém) suggested by O. Tesař in 1979 (Aldorf et al., 1984).

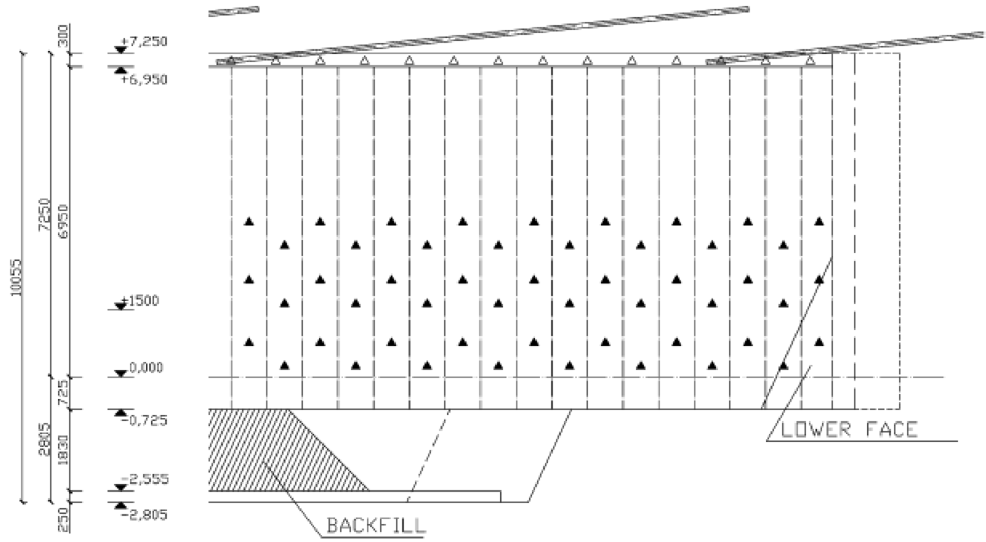


Fig. 4. Scheme of lining type 5 (roof protected with forepoling)

TABLE 4

Rock mass quality and type lining

Rock mass quality	RQD	RMR	QTS	Type of lining
Very poor	0-25%	< 20	< 38	5
Poor	25-50%	21-40	38-52	4
Fair	50-75%	41-60	52-68	3
Good	75-90%	61-80	68-84	2
Very good	90-100%	81-100	> 84	1

5. Analysis of numerical computation results

During tunnel drilling in weak rock with "type 4" preliminary lining, a problem with face and roof stability occurred. To ensure the tunnel stability in weakness zones, "type 5" liner was proposed. In order to analyze the tunnel stability in the case of applying a preliminary lining technically marked as "type 4" and "type 5", 3D numerical modeling was carried out with the assumption that the rock mass surrounding the tunnel contains some planes of weakness.

The analysis was carried out according to Finite Difference Method (FDM) with the FLAC3D v3.1 software (Itasca, 2006). The dimensions of the model were 60 m in the direction of axis x, 80 m in the direction y and 45 m in the direction z. Co-ordinates of normal vector of weakening planes are as follows: (–1.468, –5.472, 1). It was assumed that tunneling began in a strong zone (with high strength parameters), which transferred into a weak zone after 10 m. The parameters of the strong zone correspond to the parameters of argillaceous schist (L_{zd}), which exhibit properties comparable to the parameters assumed in the executive project, whereas the assumed parameters of the weak zone were approximately the same as the actual properties obtained during tunneling (L_{zl}). Modeling was carried out in two variants:

1. Type-4 lining (shotcrete lining + 4-meter long forepoling + 6-meter long radial roof bolting)
2. Type-5 lining (shotcrete lining + advancing lining in the form of umbrella with 12-meter long pipes and injection reinforcing rock mass in the vicinity of the pipes + bolts in lower part of lining)

The beginning of the model corresponds to the 106th meter of the tunnel, whereas the end of the model – to the 186th meter. Tables 5 and 6 present the properties of the materials obtained from laboratory tests, except for Young's modulus and Poisson's ration based on back analysis.

TABLE 5

Rock mass properties

Parametr	Strong zone (Lzd)	Weak zone (Lzl)	Rock mass with umbrella arch (Lzp)
Young's modulus [MPa]	160	160	2500
Poisson's ratio [-]	0.3	0.3	0.25
Friction angle [°]	28	22	30
Cohesion [kPa]	50	18	50
Tensile strength [kPa]	20	6	25
Unit weight [kN/m^3]	22	22	24
Joint friction angle [°]	16	12	-
Joint cohesion [kPa]	20	6	-
Joint tensile strength [kPa]	10	3	-

TABLE 6

Pipes and bolts properties

Parameter	Value		
	Micro pipes	Bonded anchor bolts	Self drilling anchor bolts
Young's modulus [GPa]	210	210	210
Poisson's ratio [-]	0.3	0.3	0.3
Polar moment of inertia [m^4]	2.34e-6	6.04e-8	6.04e-8
Second moment with respect to structural element y-axis [m^4]	1.17e-6	3.02e-8	3.02e-8
Second moment with respect to structural element z-axis [m^4]	1.17e-6	3.02e-8	3.02e-8
Cross-sectional area [m^2]	1.32e-3	6.158e-4	6.158e-4
Perimeter [m]	0.28	0.088	0.088
Shear coupling spring stiffness per unit length, [GPa]	2	2	2
Shear coupling spring cohesion per unit length [N/m]	150	100	100
Shear coupling spring friction angle [°]	0	0	0

Advance was made in 1.0-meter sections for type-4 lining and in 0.8-meter sections for type-5 lining. At the distance of one advance, an upper shotcrete-made lining was installed, whereas the lower lining was fixed at the distance of 5 m. The computations were carried out for a 30-meter long section of the tunnel, i.e. for 10 m in strong strata and 20 m in weak strata.

The rock mass was modeled as a Coulomb-Mohr continuous medium with planes of weakening (ubiquitous-joint model). After model reconstruction and setting particular rock mass properties with boundary conditions (the base of the model was fixed in the z-direction and roller boundaries were imposed on the sides of the model), the model was analyzed and in-situ state of stress was obtained. Then the tunneling commenced, starting from the strong strata. The modeled sequence of works in tunneling with type-4 lining was as follows (Fig. 5):

- setting the advancing lining at every 2 m (i.e. at every second advance),
- making the 1-meter advance (by means of eliminating the materials marked as 'm6', 'm8' and 'm4'),
- setting the upper part of lining made of shotcrete at the distance of 1 m behind the face ('top_shotc' group – Fig. 5),
- setting the 6-meter long bolts arranged radially – 1.5 m behind the face,
- setting the lower part of lining made of shotcrete at the distance of 5 m behind the face ('mid_1_shotc' group – Fig. 5),
- setting the filling layer at the distance of 7 m behind the face ('mid_filling' group – Fig. 5).

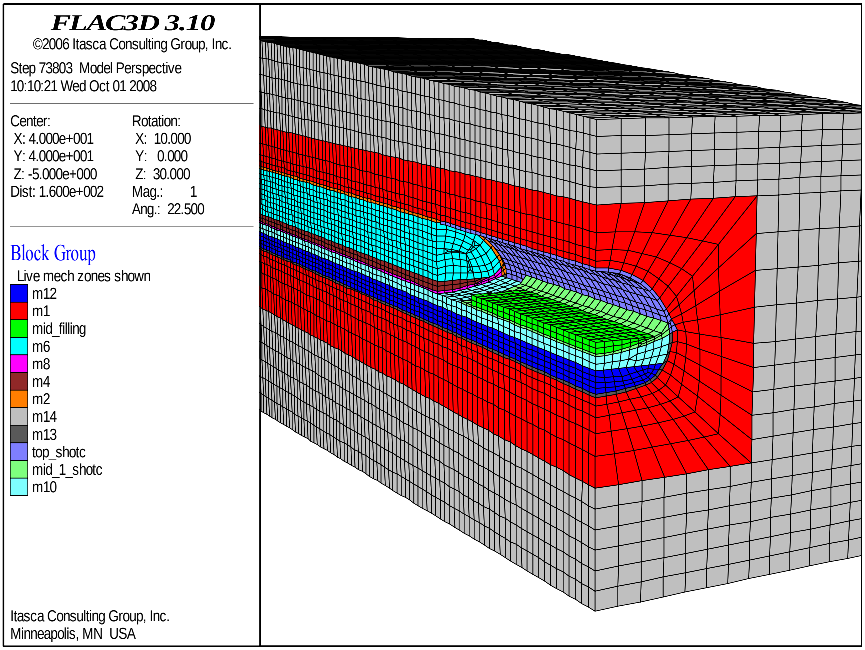


Fig. 5. Right side of the model after tunneling 28 m of the heading – type-4 lining, general view

The modeled sequence of works in tunneling with type-5 lining was as follows:

- setting the 12-meter long advancing lining in the form of an umbrella, at every 8 m (i.e. at every 10th advance)
- making an advance of 0.8 m,

- setting the upper part of lining with shotcrete at the distance of 0.8 m behind the face,
- setting 6-meter long bolts in lateral parts of lining walls, at the distance of 1.2 m behind the face,
- setting the lower part of lining made of shotcrete at the distance of 5 m behind the face,
- setting the filling layer at the distance of 8.2 m behind the face.

5.1. Axial forces and moments of deflection in bolts and advancing lining

Figures 6 and 7 present axial forces in bolts for type-4 and type-5 lining respectively. Due to the occurrence of weakening planes, some bolts do not fulfill their function and are even subject to compression – red color marks the occurrence of compression forces in bolts. What results from the calculations is that the bolts, whose axis overlaps with the planes of weakening, fail to provide satisfactory protection. The values of loading for particular bolts, obtained numerically, reach approximately 66 kN.

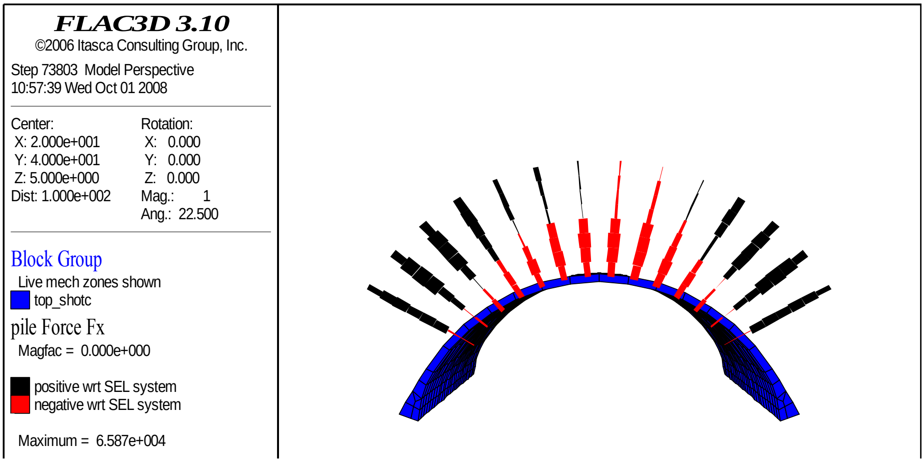


Fig. 6. Axial forces in bolts at the distance of 19.5 m from the beginning of the model – type-4 lining

Therefore, in the case of roofs, in which stratification is approximately vertical, as well as with numerous planes of discontinuity parallel to bedding, protecting the roof in the face with bolts without any additional reinforcement proves ineffective.

5.2. Stability Evaluation of the Tunnel

Fig. 8 and 9 illustrate displacement map with velocity vectors for type-4 lining and type-5 respectively. For type-4 lining large displacements values are from the tunnel face to the ground surface which can cause discontinuous deformations.

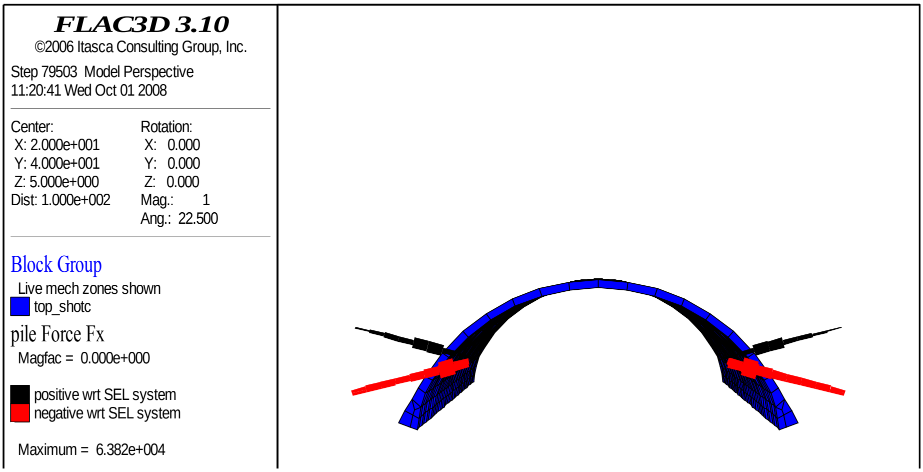


Fig. 7. Axial forces in bolts at the distance of 19.5 m from the beginning of the model – type-5 lining

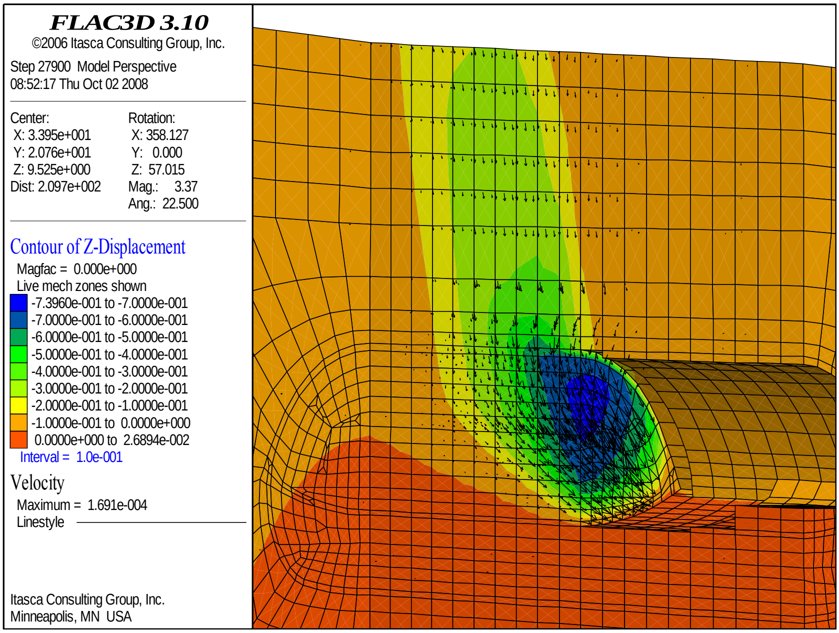


Fig. 8. Displacement map after reducing strength parameters of a weak soil – type-4 lining

For type-5 lining large displacements are only on the face of tunnel. This displacements can be caused by stress relaxation and rock lamination. It can be seen that in weak zones type-4 lining cannot protect enough tunnel face from rock slide which can go up to the surface. In that

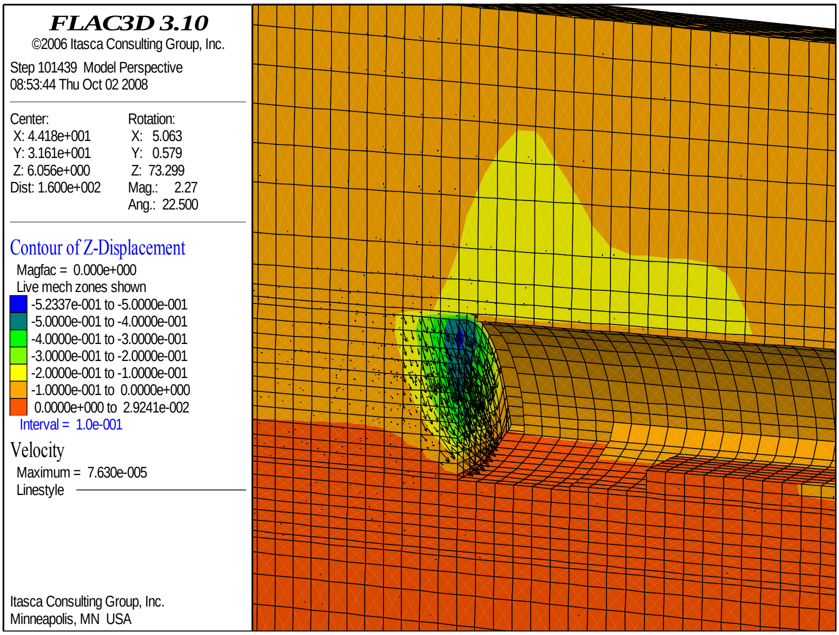


Fig. 9. Dispalcement map after reducing strength parameters of a weak soil – type-5 lining

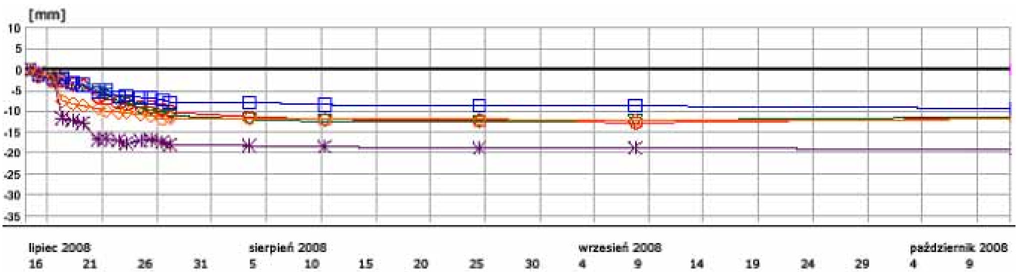
case, type-5 lining was proposed. As Fig. 9 shows, this type of lining can allow to carry loads from upper rock mass and allow to bore tunnel through the weak zones.

6. An analysis of the convergence research

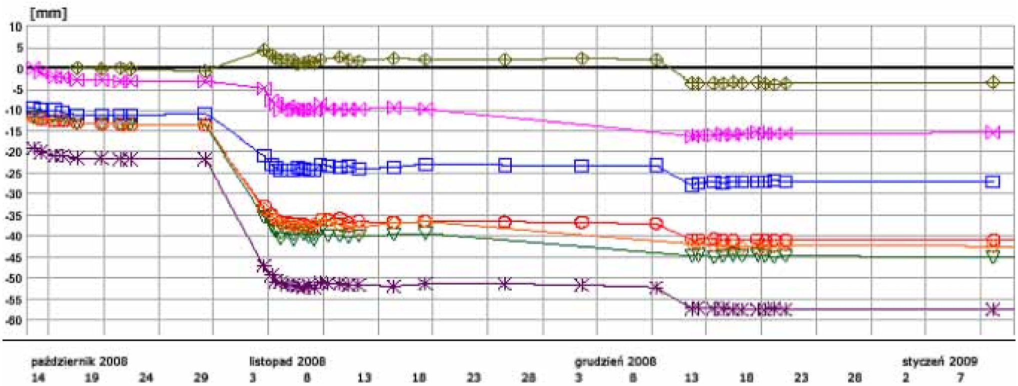
One of the basic measurements used for the evaluation of the stability of a tunnel is convergence measurement. Measurements of this kind indicate the effectiveness of the lining used for particular geotechnical conditions occurring in a given geological setting and the range of the laminary movement that can occur in a given location. In the case of the Laliki tunnel, the convergence measurements were carried out by the surveyors' teams of the contractor in 46 cross sections at intervals of up to 20 meters from one another. The base measurements were generally realized after up to two weeks from the realization of a given tunnel stretch.

As the tunnel was constructed in two stages, at each measuring position there were benchmarks in the calotte of the tunnel and, additionally, after extracting the lower layer two benchmarks in the lying walls. The sample of typical measurements shows for point tunnel 174,4 m (Fig. 10).

A vertical convergence analysis over time for the benchmarks located at 174.7 m (the tunnel stretch realized in laminated mudstone layers) indicates that from the beginning of the measurements to the stabilization of the deformation after the calotte had been made approximately 2 weeks elapsed (Fig. 10). At that time particular points subsided by 8-20 mm. Those values remained unchanged for another three months. The tunneling in the footwall part caused a significant increase in the subsidence, especially at the measurement points in the right wall. The displacement was also varied, as in the right wall it was almost 60 mm while in the left side of the roof it was 27 mm. Stable displacements after, the footwall was driven, were achieved after more than two months.



a)



b)

Fig. 10. The subsidence of the measurement points at 174.7 m of the tunnel a) after the top heading had been made, b) after the driving of the invert

Another figure, i.e. Figure 11, illustrates the displacement of the measurement points in the same section (174.4 m) as of 6 July 2009. The scheme indicates that the displacement of particular measurement points occurred in all the three planes with great intensity. The displacement in the lateral plane of the tunnel reached 85 mm, while in the longitudinal plane it was approximately 50 mm. It can be noted that the roof benchmarks moved mainly towards the excavation. A different situation occurred for side wall benchmarks. The displacement occurred mainly in the lateral plane and the character of the displacement was different for the right and the left wall. It might have been caused by, among other factors, the stratification similar to vertical stratification and the variable lithology.

The value of maximum convergence in all length of tunnel shows that the highest values were in section 200÷240 m and 530÷580 m, where convergence was 140÷200 mm (Fig. 12).

a)

b)

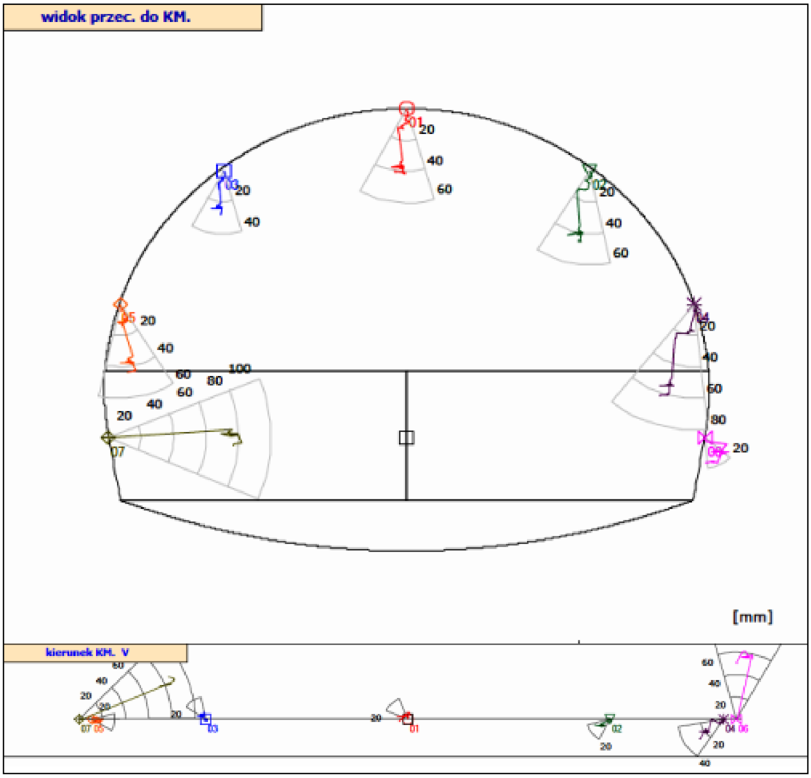


Fig. 11. The displacement of the measurement points at 174.7 m of the tunnel
a) a cross section, b) a longitudinal section – a view from above

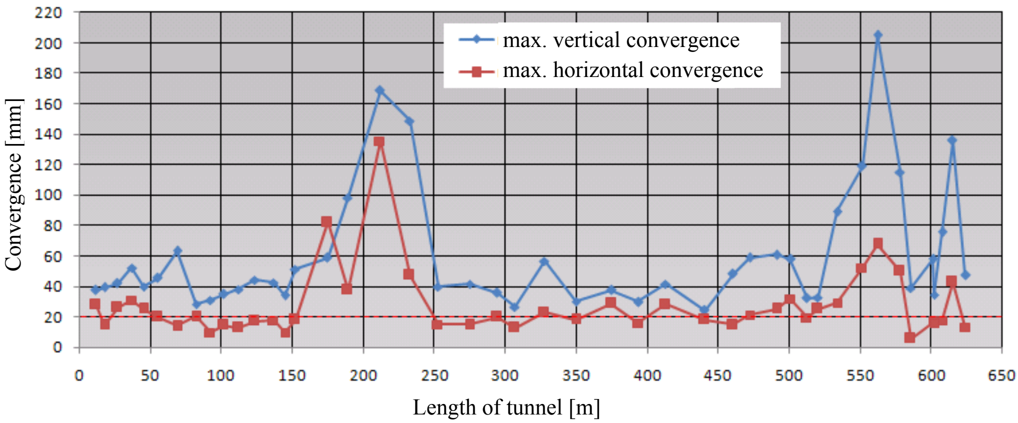


Fig. 12. Maximum convergence in the length of tunnel

The results from the analysis of the tunnel contour displacements show that the amount of maximum displacements is varied. The biggest deformations in the contour of the primary lining occur where there was laminated shale. The displacement of the measurement points in this area reaches almost 160÷200 mm. At the tunnel stretch driven in sandstone the registered values were half of it. The character of this phenomenon was similar in each case. After the crown face is driven and the measurements start, the subsidence stabilizes after 2-3 weeks. Then only slight changes occur until the lower face and invert are driven.

7. Conclusions

On the basis of the above analysis, the following conclusions can be drawn:

1. Tunneling in Laliki was carried out in highly variable geotechnical and geological conditions. Lithology, dip of strata and position of discontinuities changed even within the length of a single advance during tunneling.
2. Comparison of parameters assumed for the project of tunnel lining with data obtained during tunneling indicates that the prospection assumed in the project was nothing more than sheer estimation. For instance, estimated percentage of sandstone was to reach 60%, whereas it occurred to be marginal and did not exceed 15%.
3. Strata inclination along all the tunnel section ranged between 75÷90°. In view of the fact that strata thickness usually does not exceed several or dozen centimeters, and frequently there appear discontinuity zones between them, the effectiveness of tunnel roof protection in the face with the use of bolts proved insufficient.
4. In order to evaluate the effectiveness of new type of preliminary lining, 3D modeling was carried out, which additionally allowed for the increase of safety.
5. 3D modeling results indicate that, in the case of rock mass with lower mechanical parameters, dislocations and loading of type-4 lining are larger than in the instance of type-5 lining. Increase of stress in type-4 lining perimeter is 30% higher than in type-5 lining and exceeds the value of 4.0 MPa. In addition, anchoring bolts in rock mass with weaker parameters proved completely ineffective in some positions, as it overlaps with zones of discontinuity.
6. In the sections of a tunnel driven in rock mass with poor geotechnical properties, protecting the tunnel roof with the so-called umbrella arch support (with steel pipes reinforced with cement grout injections) allowed for safe tunneling without the risk of local falling in the face. Such a solution also guarantees tunnel stability during its further usage.
7. Type-4 lining did not protect sufficiently tunnel face from rock falls which could reach up to the surface. In that case, type-5 lining was proposed. As Fig. 12 shows, this type of lining allowed to carry loads from upper rock mass and allowed to bore the tunnel through the weak zones.

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