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Maly Lubon road tunnel: The case of a full-face excavation of a very large tunnel in the Carpathian Flysch, Poland

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ABSTRACT: The Maly Lubon Tunnel is the longest tunnel in Poland, as part of the expressway S7 Krakow–Rabka Zdroj investment. It is twin tube tunnel 2 km long with sections 220 m^2 large, excavated through Maly Lubon mountain by Astaldi company. The tunnel, still under construction, excavated in the Carpathians, consists of alternative layers of sandstone and shale with high variable degree of weathering. Due to the extreme variability of the rock mass behaviour, it was decided to change the excavation technology from the NATM to the Convergence-Confinement method. The big challenge of this change required a new design for the South portal, in the most weathered and heavily tectonized Flysch, already subject to large displacements during its geological history. Soil-structure interaction analysis were also carried out for a wider tunnel section which included emergency lanes and a vehicular cross passage intersecting with an additional technological tunnel. In order to study the complex geometry of the intersections between tunnels and the mutual effects potentially induced by the excavation of each tunnel, a specific 3D FEM analysis was carried out and provided a valid support to select suitable construction phases during the construction. The aim of the present paper is to describe the problems encountered during the excavation of a large road tunnel in Carpathian Flysch and to outlines the design solutions adopted to overcome the most critical sections of excavation.

1 INTRODUCTION

This paper focus on Maly Lubon Tunnel, the longest tunnel in Poland as a part of S7 from Kraków to Zakopane in the Lubień – Rabka section. Each tunnel, between Naprawa and Skomielna Biala and under the Lubon Maly massif, are about 2 km long.

The original design has been heavily modified both in the tunnel excavation method and in the geometry of south access portal. At construction design stage, it was increased the mining tunnel length of about 87 m in order to reduce the slope height to about 21 m against 36 m according tender design. The proposed design solution follows a detailed evaluation of the topography and geotechnical context of the South portal area, thanks to the results got by additional ground investigations, carried out especially in this zone. This new design methodology does not involve any change of the Tender safety conditions, the main purpose has been only to anticipate the start of the excavation sequences by using full face excavation method and thereby avoid expensive works of soil improvements in extremely heterogeneous soils. The full face excavation method has been judged the most suitable to face the excavation of very large sections in the Carpathian Flysch. In this paper, we describe the main problems encountered in the excavation, highlighting the technical solutions adopted and the calculation methodologies.

2 TUNNEL DESCRIPTION

The tunnel consists of two parallel tubes S7 equipped with 2 lanes (Tajduś *et al* 2013), bands with the width of 2.50 m and 3.00 m and parking bays halfway along the tunnel. Both lines of the

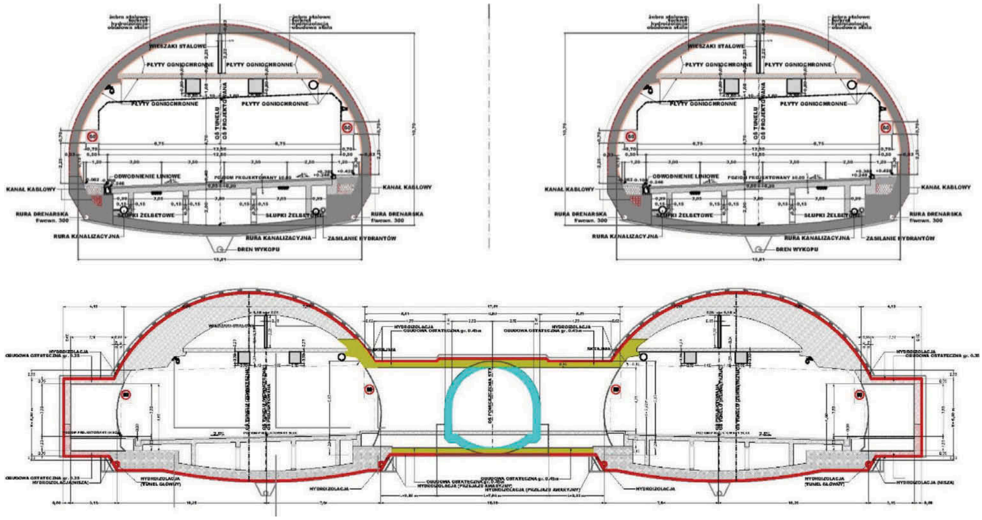

Figure 1. Typical cross section for main tunnel (above) and Lay-Bys (below).

tunnel will be connected to each other by pedestrian cross passages spaced every 172.5 m fulfilling the function of evacuation routes. The shape tunnel is polycentric closed at the bottom by an inverted arch. The section is fully waterproofed: the water captation is achieved through a drainage under the invert at the provisional stage during tunnel excavation and by means of two lateral drainage pipes (placed below the knees of final lining) at final stage. The final lining is executed in reinforced concrete with a constant thickness of 50 cm in case of support class 1 and 2 or variable thickness from 50 to 100 cm in case of support class 3. A full round waterproof system is installed made by a layer of pvc and a layer of geotextile. The design also includes signaling-alarm and teletechnical niches on the outer sides of each pipe of the tunnel spaced every about 172 m. Inside the tunnel, a vehicular cross passage for emergency is designed, as well as technical room ST3. This stretch of tunnel, called Lay-Bys, is characterized by an enlarged excavation section (over 200 m^2) to the presence of the parking lane (L = 2.50 m).

In the southern portal area, there is the technological building ST1 in which the air treatment plants are housed and which contributes to the stability of the retaining structures present on the sides of the portal.

The works started on March 2017 and the end of works it is fixed on end of 2020. The initial value of the tunnel it was estimated about 181.410.923 €.

3 GEOMORPHOLOGY AND CLIMATE OF THE AREA

The area of Maly Lubon Tunnel lies in the sub-province of Outer Western Carpathians, in the mesoregion of Island Beskids. A landscape feature characteristic of the Island Beskids is the occurrence of isolated peaks rising to heights of ca. 400–500 m above the level of the intermontane plain, up to the height of 850–1170 m a. s. l.. Mountain slopes are overgrown with a lower subalpine forest, and the intermontane plains are occupied by crops and residential buildings.

The region has a continental climate with cold winters from December to March. January temperatures average -1°C to -20°C. Summers, which extend from June to August, are usually warm, sunny and less humid than winter. July and August average temperatures range from 16°C to 19°C, though some days the temperature can easily reach even 35°C. The annual rainfall-snow can reach easily as much as 1300 mm/year. This rainy and snowy regime obviously influences the underground water circulation in a sensitive way especially in the first meters of depth.

4 GEOLOGICAL AND GEOTECHNICAL FRAMEWORK

4.1 *Geological framework*

The area of the designed works lies in the zone of the Magura nappe limited from the south with a zone of longitudinal strike-slip faults from the Pieniny rock belt, and from the north bordering on the Silesian nappe (Stupnicka, 2007). In Magura nappe, dominate thick-shoal sandstones with inclusions of conglomerates and shales. In their vicinity, there are belts of shales and thin-shoal sandstones, divided by faults, of the so called Hieroglyphic strata, classified in terms of age as the Middle Eocene. The southern section of the future road intersects with belts of hieroglyphic and sub-Magura strata, whereas the northern section for the designed tunnel, runs through thick-shoal sandstones and shales of the Magura strata, formed in the micaceous facies.

It is not possible to easily find a structure inside the rock mass, since the material occurs in most cases in the form of folded layers and with metrically variable discontinuity planes.

4.2 *Hydrogeological conditions*

In the area of study thicknesses of aquifers are low and do not exceed 5 m. Values of the filtration ratio are varied, depending on formation of deposits. The depth of water table is varied and strictly depends on precipitation inflow and the level of surface water. In the youngest river sediments, the free surface of water is shallow (from 0.2 to 1.0 m b.g.l.) and is closely connected with the level of water in rivers.

Hydrogeological parameters of the designed tunnel, where there are upper sections of watercourses, are low. This applies to both thickness and filtration parameters due to the often clayey nature of colluvium, in which filtration ratios drop to several m/day and, in extreme cases, even to approximately 1 m/day. Supply of water takes place through direct infiltration of precipitation, from side inflows and surface water.

In the cracked series of Carpathian Flysch, formed primarily as sandstones with shale inclusions, the aquifer is mostly composed of complexes of thick-shoal Magura sandstones with inclusions of silt-marl shales. The average thickness of the first (counting from the surface) aquifer is estimated to be about 15 m, whereas the average value of the filtration ratio of about 1.0 m/day. The best filtration parameters ($k = 10^{-5} – 10^{-6}$ m/s) occur down to the depth of 30–40 m b.g.l. Ground water occurs at different depths, from several meters in the bottoms of valleys to a dozen or so meters on slopes.

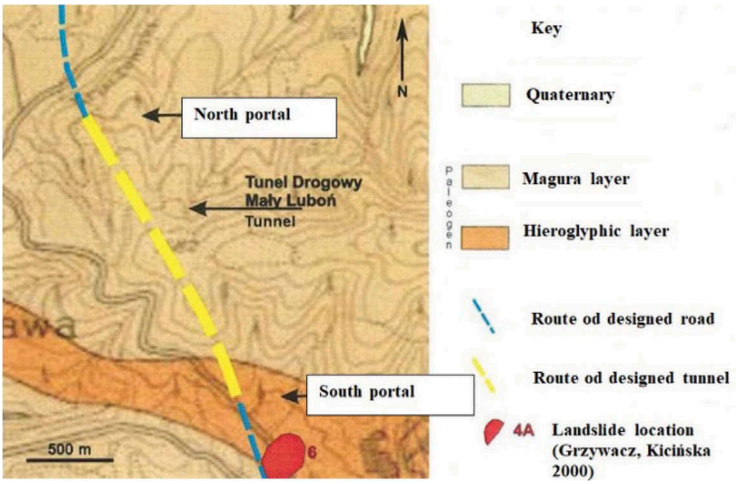

Figure 2. Contemporary geological situation in the mapped area.

4.3 *Stratigraphy and geostructural properties*

Starting from the surface there are the Quaternary deposits which consist of clayey silts with gravels and arenaceous boulders classified as Colluvium. The thickness of these deposits reaches a maximum of 10–12 m when depressions (such as in the area of the south portal) are present.

Below the Colluvium are the oldest formations, respectively the Flysch of the Hieroglyphic (Middle Miocene) and the Magura (Eocene-Oligocene).

These layers are present, below the Colluvium throughout the South area where are strongly disturbed from the tectonic point of view and anisotropic. It is not possible to easily find a structure in the rock mass since the material occurs in most cases in the form of folded layers and with metrically variable discontinuity planes.

Magura layers are marine Flysch sediments of the upper Eocene. The layers are composed of alternating layers of sandstone and slates. Sandstone series in Magura layers are fine or medium grained. Most commonly they take the form of medium to thick banks.

The average spacing can be assumed equal to 0.4 m in shales and 0.8 m in sandstone. Three different areas in the stratification, have been found. Homogeneous areas S1 and S2 are characterized by Magura layers, the homogeneous area S3 by Hieroglyphic layer systems. The table below summarize the geometry of these areas. In some of the boreholes, during examinations of the core, typical features of disturbances (dislocations) have been detected. These area were confirmed by the excavation of the tunnel, although in slightly different positions with respect to predictions. In some cases the excavation is progressing through strongly altered, fractured, deeply weakened and not homogeneous rocky materials, with stratification and faulting features changing in few meters - quite at each step, respect to the axis of the tunnel.

Table 1. Geometric properties of main families of discontinuities for different layers.

Uniform area	boreholes	dip direction (°)	dip angle (°)
S1	BK1–BK8	175	25–60
S2	BK9–BK11	194	9–35
S3	BK12–BK14	194	29–61

Figure 3. Hieroglyfic strata - typical borehole cores.

Figure 4. Magura strata – good core of rock in main tunnel.

4.4 *Geotechnical characteristics of soil/rock mass*

For the geotechnical characterization of the area interested by the excavation of the Mały Luboń tunnel, test results provided in the Tender documents were interpreted and integrated with the results obtained by additional tests. For the measurement of strength properties have been used SPT, DPH and DS tests for colluvial and soft argillite/shale Flysch and PLT/UCS/Brazilian tests for rocky soils. Strain properties have been estimated by means of seismic refraction and MASW profiles. In the table below are the mechanical parameters for the different geotechnical units.

Table 2. Geomechanical parameters assumed for different layers.

Unit	γ (kN/m^3)	GSI	σ'$_f$	c' (kPa)	φ (°)	E (MPa)
Colluvium	21	-	-	10–15	20–25	10
Hieroglyphic	23	15–35	0.5–15	20–120	24–26	100–300
Magura	23	45–55	15–35	300–430	44–52	1900–6600

The geotechnical profile along the tunnel was designed with the most relevant geotechnical units taken in account in the numerical analysis. The overburden above the tunnel goes from 15–25 m (on the portals) to 100 m, at the maximum overburden.

4.5 *Water table conditions*

In the boreholes, the piezometric level varies between 5.20 and 8.80 m below ground level, more or less at the colluvium/weathered shale transition level. It most likely represents a suspended

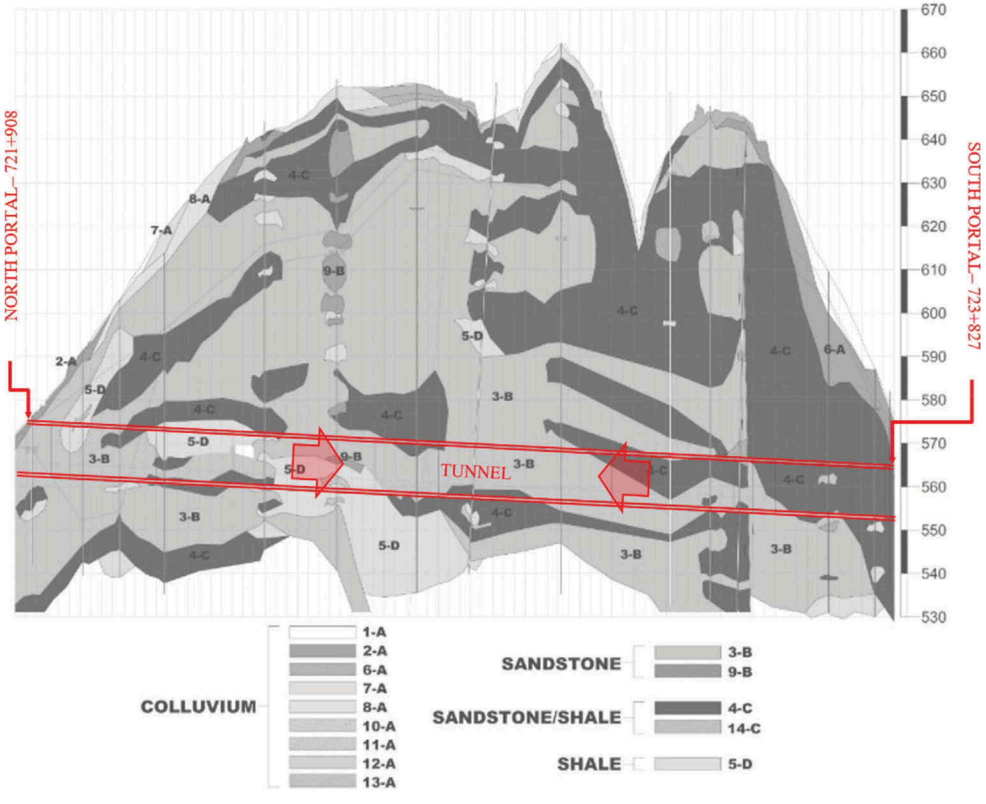

Figure 5. Geotechnical profile.

aquifer that does not affect the weathered rock material below the colluvium. This water, not at all easy to manage due to the extreme variability of the structure and the structure of the discontinuities, has been controlled with draining pipes arranged both along slopes/retaining walls at the portals and on the mining tunnel. In mining tunnel stretches, except in some cases, the flow rates flowing from the excavation face were of the order of few l/s.

5 EXCAVATION METHODOLOGY OF THE TUNNEL

The Maly Lubon tunnel is the largest and longest tunnel in Poland and it is the first one excavated with the full face method in this Country. The excavation method of the tunnel depends on the geomechanical properties of rock mass. There are defined five types of excavation sections called 1, 2, 3, NP, SP. For the sections 1 and 2, excavation can be performed with the use of explosives. For sections 3, NP and SP excavation has to be performed using mechanical means. In the part of tunnel where good rock mass exist, section type 1 or 2 are adopted. Temporary support lining consists of double steel ribs IPE180 at steps from 1.5 m to 2.00 meters and shotcrete 25 cm thick reinforced with steel welded mesh. The final lining has a thickness of 50 cm constant at the crown and in the tunnel invert. The section type 2 differs from the section type 1 in arrangement of radial rock bolts and the step of the steel ribs.

The section type 3 is characterized by forepoling crown support injected with cement grout. The truncated cone configuration is created to protect the excavation in advances. The pipes are arranged in an umbrella configuration around the excavation shape with n°45 steel tube, 15.00 m long, to be repeated after 8.65 meters of excavation steps.

The stabilization of the core face of the tunnel is performed by fiberglass pipes (n° = 46 – 100) 15.00 meters long. As for the forepoling, the fiberglass pipes have a minimum overlap of 6,00 m.

The primary lining consists of double ribs IPE 180 at step of 0.75–1 meter and shotcrete 25 cm thick reinforced with welded mesh. The final lining has a variable thickness for the crown from 50 to 100 cm and constant thickness of 50 cm for the invert arch. Before the final lining concreting, complete waterproofing of the section is provided.

The excavation is performed using mechanical means with step 1.00 m long. The excavation has a minimum area of about 172 m^2 at the first excavation and a maximum area of about 190 m^2 at the last excavation step.

6 MAIN ASPECTS AND PROBLEMS OF TUNNEL AND PORTALS DESIGN

One of the most relevant problems that have occurred during the execution of the tunnel was the soil displacements on South Portal slopes. In this case different geological and hydrogeological condition were encored compared to the design forecasts. In order to define a new geotechnical model the monitoring data were analysed. The monitoring instruments installed in that area were inclinometers, piezometers and targets. Furthermore, during the excavation of the slopes shallow landslides were observed as well. When the Berliner wall was completed and the excavation level reached the work tunnel level, the monitoring data

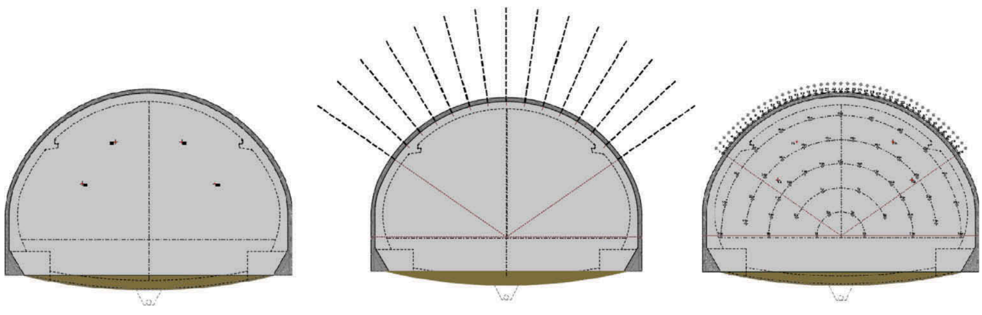

Figure 6. Excavation section type 1 (on the left), 2 (on the middle) and 3 (on the right).

Figure 7. Front face with sandstone and shale (on the left), shale (on the right).

showed an increasing of the deformation values both for the inclinometers dates and targets data. The first additional works performed on the slope, were the draining trenches in order to decrees the load of the suspended water table. Furthermore, even when the anchors on the Berliner wall were completed the horizontal deformation of the wall, measured on the optical target, reached maximum values up to 25–30 cm. These deformations of frontal Berliner wall were manifested as a result of a deep movement that also affected the future excavation section of at least one of the two tunnels. It was then necessary to take safety measures through the design of new additional works before star the tunnel excavation. There were designed and realized two important works that allowed the stop pf the deformation on the Berliner wall and on the slopes:

– reshaping on the slopes in order to remove gravity load on the head of the possible landslide;
– in situ casting canopy tunnel in order to load the foot of the possible landslide;

Through this two additional works, it was possible to stop the deformation and proceed with the tunnel excavation.

During the excavation of the tunnel the major difficulties encountered were related to the high heterogeneity of the excavated material. The geotechnics hypothesis adopted during the design stage were not fully confirmed during the construction stages. The front face showed in most of the logging present very difficult condition in term of discontinues and fracturing status due to the different litotypes observed and tectonic disturbance. Most of the front face consists in Flysch formation constituted by very thick sandstone beds with shale bench thin stratificated. Due to the high rock mass discontinuities, during the excavation of the tunnel especially on first meters, problems of face extrusion and extra excavation occurred. The worst condition of the front face was a front face totally composed of shale. In this condition the support class applied was the type 3 where fiberglass elements at the face were performed. In this material longitudinal strains were observed in the temporary lining during the excavation. Although the steel ribs were fixed at the foot by the invert, maximum displacements of 60–70 mm were measured in the upper part. The displacements have been exhausted once poured moreover in a very short time, the final RC lining.

7 MAIN TUNNEL ANALYSIS IN LAY-BYS STRECTH

Given the complex geometry of the intersections between the tunnels to be built in the Lay-Bys area, and the mutual effects induced by the excavation of each tunnel on the adjacent one, it was necessary to carry out three-dimensional numerical analyzes, the results of which provided a valid support in the choice of suitable executive phases.

In this case, the central tunnels (vehicular by-pass and ST3 technological tunnel), although much smaller than those of the lateral Lay-Bys, will be excavated in the portion of rock mass located between the two pipes of the main tunnel, in an area already disturbed and plasticized by the excavation of lateral main tunnel, also due to the reduced existing distance (about 14 m).

The FEM calculation code used for the numerical analyses is MIDAS GTS-NX 3D,version 2018 release 1.1, which allows to evaluate the stress-strain behavior of the soil-structures and their interaction, by simulating all construction phases of the tunnel.

In the analysis solid-tetrahedrycal elements are used made of 4 nodes classified as 3D sol-id shape. To the faces of the model the followings boundary conditions have been imposed:

– Upper surface: free;
– External side surfaces: fixed displacements in horizontal direction;
– Lower surface: fixed displacements in vertical and horizontal directions.

The model has been extended sufficiently so that to be able to hold negligible, in the zones of interest, the trouble effects due to the boundary conditions used. In this case we have laterally extended the model to a variable distance between. The model has been created using:

– About 460000 3D element (rock mass continuous space with properties of Magura strata);
– About 120000 2D elements (equivalent temporary supports and final lining as shell elements);
– About 2000 1D elements (rock bolts as beam elements).

For definition of the behavior of soil, a Mohr-Coulomb model has been used in FEM analysis, with plasticity associated with development of irreversible strains. A perfectly-plastic behavior is assumed for strain-stress calculation. The construction sequence must be the following one:

– Excavation of first lay-bys tunnel installing steel ribs and shotcrete;
– Rock bolts installation on cross passage area (rock bolts include both elements in the soil and for future support of cutted steel ribs at the beginning of the excavation of the vehicular cross passage);
– Pouring invert, knee and crown parts of first lay-bys tunnel except for the segment at the vehicular cross passage part;
– Implementation of steps 1 to 3 also for the second lay-bys tunnel;
– Vehicular cross-passage excavation by mean of steel ribs, steel wire mesh and shot-crete;
– Rock bolts installation at the technological tunnel ST3 intersection (rock bolts include both elements in the soil and for future support of cutted steel ribs at the be-ginning of the excavation of the ST3);
– Technological tunnel ST3 excavation installing steel ribs, steel wire mesh and shot-crete;
– Pouring of invert and knee parts of cross passage;
– Excavation of the niche on the opposite side of the vehicular cross-passage;
– Pouring of final lining for cross passage, ST3 tunnel, lay-bys tunnel crown and niches in one step.

The analyzes results show that the most critical phases are those that involving the excavation of the central by-pass and ST3 tunnels, as a consequence of a stress redistribution on the Lay-Bys tunnel as the excavation proceeds. The maximum stresses on the temporary lay-bys support occur precisely in these phases: this circumstance led to use a more robust steel ribs (2IPE220/ 0.75 m) for a total length of 12 m with midpoint on the axis of the vehicular by-pass.

In addition to this intervention, it was necessary, in order to reduce the stress values acting, to provide the invert and knees concreting for the entire length of two lay-bys tunnels and spring line/crown parts for the stretches not interested by vehicular by-pass. The temporary supports of by-pass tunnel has been calculated to bear the high forces generated by ST3 tunnel excavation, parallel to the main tunnel. The results of the numerical analyzes have finally allowed to verify all the tunnel linings structures both in temporary and final conditions and in exceptional conditions (in case of fire) in compliance with the requirements of the current reference standards. In the figures below are presented the main results of 3D FEM analysis.

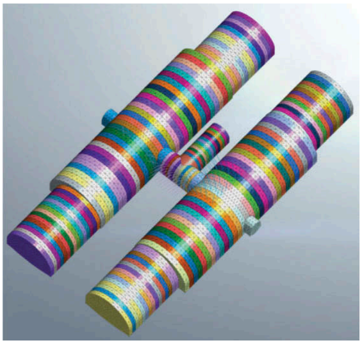

Figure 8. FEM model used to analyzed stress-strain state of Lay-Bys – ST3 stretch.

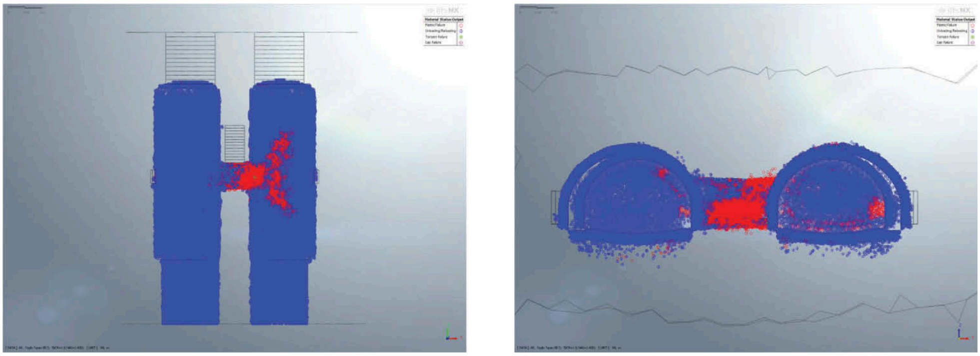

Figure 9. Lay-Bys: plastic zones (red points) at the end of cross passage exacavation.

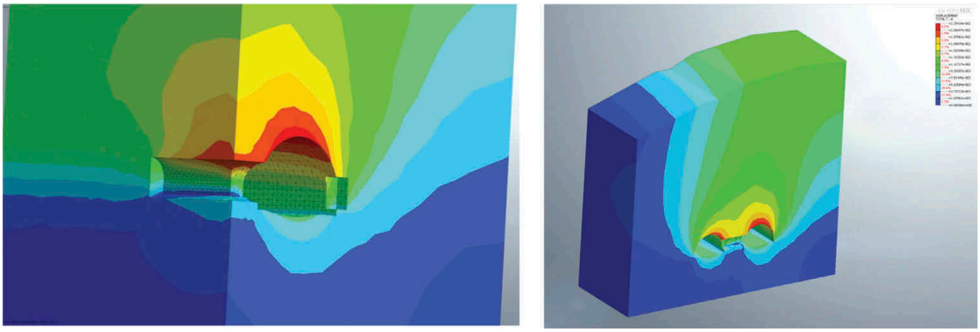

Figure 10. Maximum total displacement on tunnels ate the end of technological tunnel ST3 excavation (max value 25 mm).

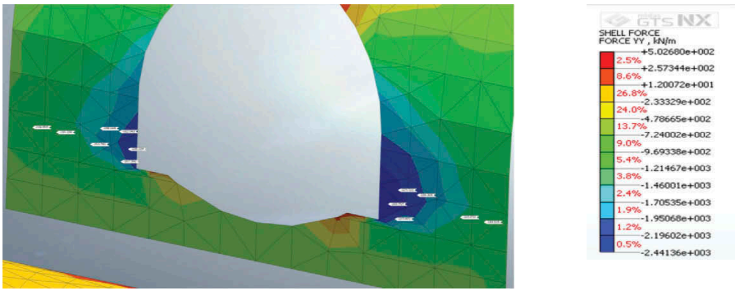

Figure 11. Vehicular Cross Passage – Lay Bis tunnel intersection: axial force concentration on springlines.

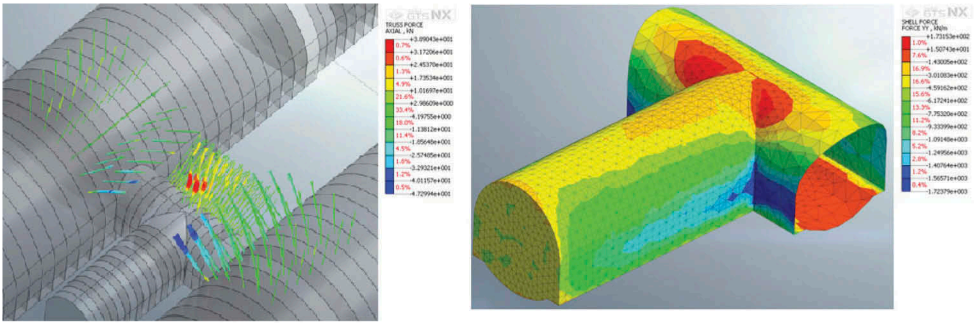

Figure 12. Vehicular Cross Passage – Lay Bys tunnel intersection: axial force on nails (on the left) axial force final lining (on the right).

8 CONCLUSIONS

The aim of this paper is to present the design of the Tunnel Maly Lubon still under construction by Astaldi Company. The main aspect described refer to the extreme variability of the rock mass behaviour, the big challenge of new design for the South portal and the soil-structure interaction analysis carried out through 3D FEM analysis. The main difficulties for the excavation of the tunnel regarding the extremely variability/anisotropy (Margielewski 2002) of the rock mass were faced with the support class variation. In the worst difficult material, the most heavy support class was applied, with fiberglass elements and umbrella steel pipes. The appropriate support class applied during the excavation made it possible to have convergences and forces in temporary support and final RC lining within the design values.

During the execution of the south portal area, big deformations due to different geological conditions occurred during the slope excavation made necessary the execution of additional works in order to increase the safety condition and in order to allow the excavation of the tunnel. Significant contribution has been given from different additional works as new drainage system on the slope that allowed to reduce the water load on the frontal Berliner wall; reinforced concrete canopy tunnel to load the foot of the landslide; reshaping of the slopes to unload the top of the landslide. After the additional works the deformations stopped and it was possible to start tunnel excavation.

All tunnel significant stretches have been designed by means of plane and three-dimensional finished models. In particular, in order to design the execution phases for the excavation of the Lay-bys (wider tunnel) at the intersection between the vehicular cross passage and technological tunnel, it was necessary to carry out 3D numerical analyses. The results of the numerical analysis allowed to verify all the tunnel linings structures both in temporary and final conditions, allowing to analyze the most critical areas, or those of intersection between the various tunnels, and to adopt executive steps suitable to perform the excavation in safe conditions.

9 ACKNOWLEDGEMENTS

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