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# Numerical Modeling for Engineering Analysis, Designing and Monitoring of Support Systems for Twin-Tube Tunnel

## Modelare numerică pentru analiza inginerească, proiectarea și monitorizarea sistemelor de suport pentru tunelul cu două tuburi

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**Abstract**

In this research work, Rock Mass Rating (RMR) was used for the characterisation of rock mass along the tunnel alignment based on physical, geological and geotechnical data of the project area. The support systems were recommended for all geotechnical units using RMR and tunneling quality index (Q-system) support chart. Furthermore, Various design input parameters such as physical and geotechnical properties, in situ stresses, modulus of deformation of rock mass, support systems recommended by RMR were used as input parameters in Phase2 2D 8.0 software, in order to compare the calculation results with in-situ monitoring using Amberg Tunnel 2.0 software, to validate the numerical models and to check the deformations of the tunnel in the temporary support stage.

**Keywords:** Tunnel, Rock Mass Ckassification, Provisional Support, Deformations, Numerical Modelling.

### 1. Introduction:

The Texanna twin-tube tunnel with 1.80 -km long (figure 1), was built as a road tunnel using the New Austrian Method (NATM) assuming that the excavation technique used is drilling-blasting and / or mechanical excavation, in anticipation of heavy traffic in the framework of the project 'Penetrating Highway connecting Djen Djen Port to the East-West Highway', in a project of 110 km. The tunnel is located in the project route between KP: 24 + 818.545 - KP: 26 + 648.352 for the right tube and between KP: 0 + 711.683 - KP: 2 + 593.879 for the left tube. The rock mass classification was carried out using rock mass rating (RMR) based on geology of project area, bore holes data and physical and strength properties of rock samples collected from site. In the present work, the rock mass rating (RMR) were used as empirical methods for characterization of rock mass based on real-time geological and site geotechnical data and physical and strength properties of rock samples collected from the alignment of tunnel. The rock mass along the tunnel axis was classified into Five geotechnical units (class III, III-A, IV, IV-A, IV-B). The support systems for each geotechnical unit were designed. The rock mass behavior in term of the in situ monitoring of total deformations and effects of provisional support on (arch, bolts and shotcrete) due to excavation of the tunnel profile were investigated and analyzed by comparing with simulated model (Phase2- 2D software).

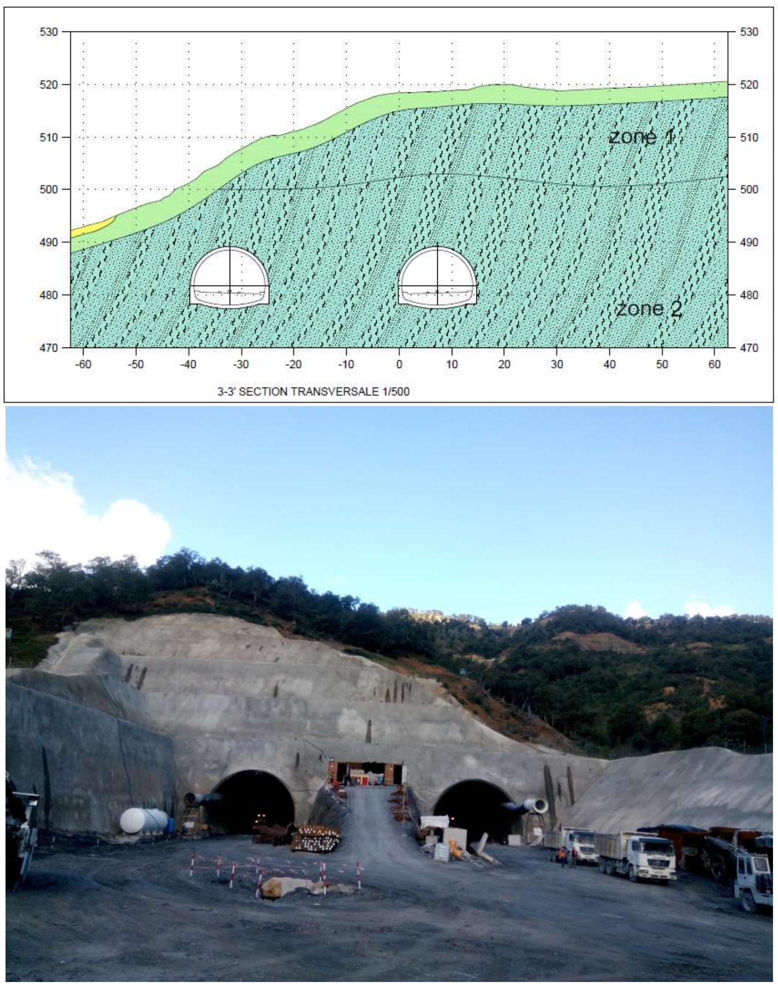


*Fig. 1– Photography and Geological cross section and longitudinal profile of the south portal of the tunnel*

The empirical and numerical design approaches are considered very important in the viable and efficient design of support systems, stability analysis for tunnel, and underground excavations [1]. During stages of excavation projects, the empirical methods like rock mass classification systems are considered to be used for solving engineering problems [2,3]. The empirical methods used defined input parameters in designing of any underground structures, recommendation of support systems, and determination of input parameters for numerical modeling [4]. The empirical methods classified the rock mass quantitatively into different classes having similar characteristics for easily understanding and construction of underground engineering structures [5]. Numerical modeling is gaining more attention in the field of civil and rock engineering for prediction of rock mass response to various excavation activities [6]. Modeling of rock mass is a very difficult job due to the presence of discontinuities, anisotropic, heterogeneous, and non-elastic nature of rock mass, using empirical and numerical methods [7]. The complex nature and different formation make the rock masses a difficult material for empirical and numerical modeling [8]. It is suggested that numerical and empirical methods be used together for the safe, stable and efficient design of tunnels, other underground structures in the rock mass environment and reliable support systems [9,10].

## 2. Geology of project area:

Between the mass of Babors, developed in the west and that of the Kabylian pedestal, which extends eastward over more than 100 PK (Petite Kabylie), there is a region of ridges and wooded hills, still well known, or dominate, under the neogene post-nappes, the Numidian series and the Mauritanian flysch of Guerrouch (formerly Texanna), to the south of the port of Jijel, on the northern edge of Tamesguida mapped by F. Ehrmann 1946. This author was noted in the valley of the DjenDjen wadi, the existence of "green rocks" presented as an ophiolitic complex with intercalations of cornea and glandular gneiss. In 1956, Mr. Durand Delga presents a precise cartography to the sector of Texenna. The metamorphosed "green rocks" are interpreted as a lacolith in the micaschists of the Kabylian pedestal, which is widely carried southward; conception that complete and corrected by Bouillin in 1971 [11]. It then defines the unit of " Sendouah-Tabellout" which stratigraphically comprises from bottom to top the following layers:

- Green rocks probably containing pillow lavas, which may represent an ophiolitic complex;
- Shales and limestones attributed to the Neocomian Jurassic;
- Cretaceous flysch, schisto-sandstone for the most part; this unit is overturned and metamorphosed, it is straddled by the Kabylian pedestal to the north, and faces, to the south, according to a very corrected contact, other series of flyschs carried on the Tellian domain.

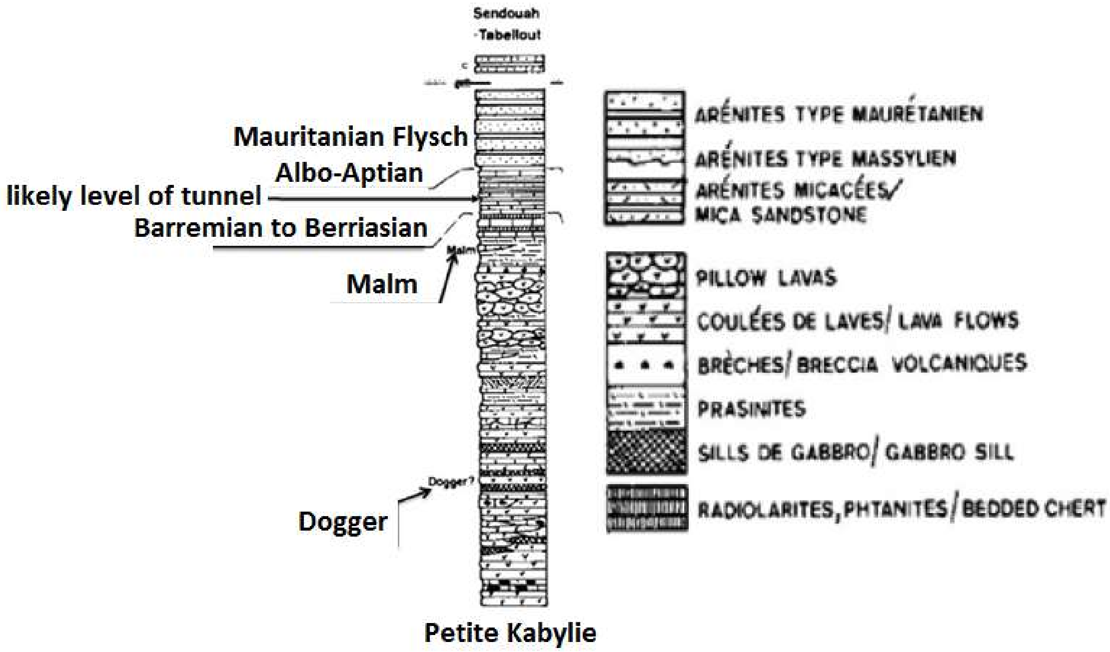


*Fig. 2– Geological and structural maps of the western terminus of Kabylia (after H. Djellit, 1987) [11]*

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### 2.1 Geological conditions along the tunnel:

The alignment is not located in different lithological units as previously indicated but in an old Albo-Alpien Flysch composed entirely of the alternation of mudstone and sandstone. The flysch is composed of a mudstone with a folded, weakly-moderately decomposed, weakly-weak, and fine-grained sandstone character that has a medium-thick, poorly decomposed, moderately solid-solid rock nature. The first part of 10 m on the surface of the flysch unit is very or totally decomposed and has a very low-excessively low rock nature. However, this zone of decomposition does not reach the tunnel dimension and according to sounding studies, an alternation of mudstone and sandstone which includes a nature of weakly decomposed rock, partly weak, generally medium-solid will be observed at the tunnel level. The different geological conditions of rock mass along the alignment of tunnel are shown in Figures (3).

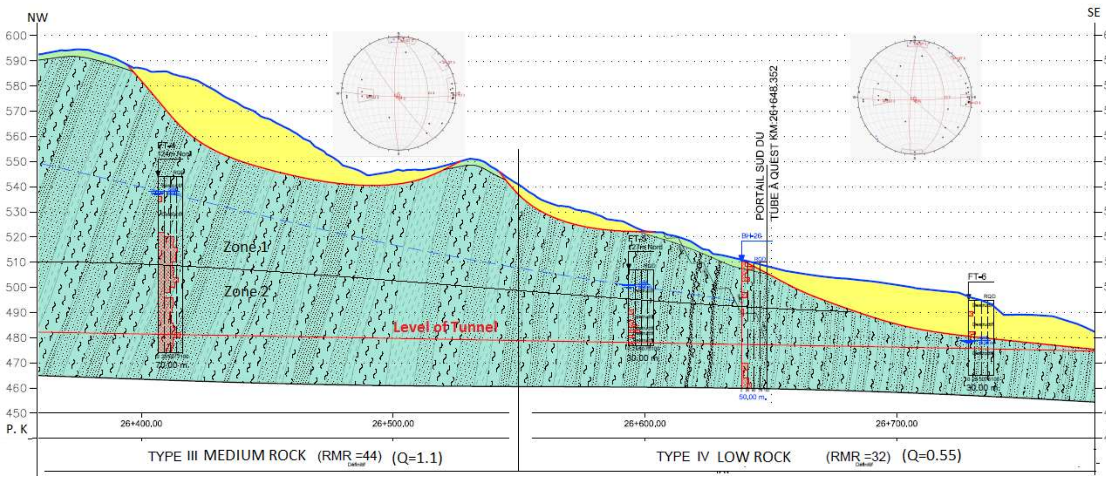


*Fig. 3– Geology and cross-sectional view of tunnel alignment of this study*

The geological conditions of the Texanna Tunnel site are composed by the Mauritanian Flysch, which consists essentially of shale-sandstone alternations with hard quartzite passages, resting on the surface of fractured and weathered shales. All of these formations cover the formation of hard argillite slightly weathered and fractured whose upper part, and in depth it is very hard and little fractured. This argillite is present almost all along the tunnel as shown in Figures (4).

Based on the foregoing considerations, it appears that the study area is located in an area characterized by average class II-a seismicity, according to the RPOA, 2008 (Algerian standard). In the case of well-built tunnels through a host rock of good quality, the seismicity effects are generally low. However, special attention should be given to areas with a host rock of poor quality, particularly at tunnel portal, where the coverage is lower and there is generally lower quality land. Under these conditions, special precautions must be taken in the design and construction phase to counter the seismic effects on the tunnel structure.

*Fig. 4– Photography of tunnel alignment Geology of this study*

## 3. Rock Mass Classification along the alignment of the tunnel:

### 3.1 RMR- system:

The empirical methods classify the rock masses into different categories having less or more similar geological and geotechnical properties on the basis of results obtained from rock mass characterization. The rock mass classification systems are considered very beneficial to use during the initial stages of the project when limited information about rock mass behavior, stresses and hydrological characteristics are available [12, 13]. The rock mass along the tunnel axis were classified into different categories based on Geo-mechanical classification system also called Rock mass rating (RMR- system) [4]. This system utilised the following six parameters for rock mass classification based on quality in to various groups of similar behaviours:

- Uni-axial Compressive strength (UCS);
- Rock Quality designation index (RQD);
- Spacing of discontinuities (DS);
- Condition of discontinuity (DC);
- Ground water condition (GWC);
- Orientation of discontinuities (DO).

Various physical and geotechnical properties of rock mass along the tunnel alignment were determined by testing the rock samples obtained along the tunnel alignment. The different physical and Mechanical properties of rock mass along the tunnel length are presented in Table (1).

### 3.2 Q- system:

The Rock mass classification systems are considered as an integral part of the designing of underground structure, support systems, stability analysis and in determination of input parameters for numerical modeling within the rock mass environment [14]. Various rock mass classification system has been developed based on civil and mining engineering case studies by different researchers. In this research, RMR and Q systems were used due to its flexibility in terms of input parameters and widespread range for selection of support systems. The Q-system is developed by Bortan in 1974 at Norwegian Geotechnical Institute (NGI) [15]. The Q-system has wide applications in underground excavations and „field mapping, and it depends on the underground opening and its geometry. The value of this system may be different for undisturbed and disturbed rock [16]. This system classifies the rock mass environment into different classes on the basis of:

- the rock quality designation (RQD),
- joint number (Jn),
- joint roughness number (Jr),
- joint alteration (Ja),
- joint water reduction factor (Jw),
- and stress reduction factor (SRF).

The values of this system indicate the quality of rock mass and give description about the stability of an excavation within the rock mass environment. The maximum value of Q-system indicates good quality of rock meaning good stability and the minimum value indicates poor quality of rock meaning poor stability [17, 18]. The RMR and Q classification systems were applied on bore hole data and physical and strength properties determined in laboratory of the collected rock samples along tunnel alignment. Based on the results obtained from RMR and Q system, the rock mass along the tunnel axis was divided into five geotechnical units. The results of RMR and Q classification system are presented in Table (1) and (2).

### 4. Provisional support system:

The fundamental principle of digging a tunnel with the new Austrian method is to transport the rock by itself (the ability to transform a mass of rock that surrounds the profile of a tunnel into a load-carrying element instead of an element that constitutes a load). Allowing the rock to deform slightly (provided that it remains within the permissible safety limits) considerably reduces the loads on the bearing system. The rock released under control transfers the load to the sides and thus uses its transport capacity to the maximum by forming a transport chain around the excavation. Instead of carrying all the load of the rock, the support systems are rather used to control the plastic deformations while preserving the integrity of the transport chain around the excavation and to avoid the excessive relaxations. So the flexibility of the system to the point of adapting to the deformations of the rock is one of the most important criteria of the method. If the rock is too weak to carry its own load, the support used stabilizes the system by providing additional pressure still needed to reach equilibrium after approaching the rock carrying capacity. As a result, limited deformation is allowed before and after the application of the primary support system (provisional). The main feature of NATM is the application support at the exact moment. If the support is applied without allowing any deformation, the support system will be overloaded and will no longer be economical. And if not, deterioration of soil and excessive deformation will occur.

*Table 1*

*Geotechnical Design Parameters and Rock mass classification for twin tube tunnel*

<table>
<tr><td colspan="2">Kilometric point (P.K)</td><td>between the exit gate part- KP: 26 + 550 of the right tube and exit gate - KP 2 + 490.970 of the left tube of the tunnel</td><td>between the KP part: 26 + 230 - KP: 26 + 550 of the right tube and KP: 2 + 191.682 - KP: 2 + 490.970 of the left tube of the tunnel</td></tr>
<tr><td colspan="2">Rock class type</td><td>Low Rock (IV)</td><td>Middle Rock (III)</td></tr>
<tr><td colspan="2">Geological determination</td><td colspan="2">The level of the tunnel is located entirely in the flysch consisting of the alternation of mudstone and sandstone of the old albo-Alpien.</td></tr>
<tr><td colspan="2">Underground water condition</td><td colspan="2">State of groundwater in the form of dripping and leaking</td></tr>
<tr><td colspan="2">UCS, Uniaxial compressive strength (MPa)</td><td>10,0</td><td>13,0</td></tr>
<tr><td colspan="2">GSI, Geological strength Index</td><td>25</td><td>40</td></tr>
<tr><td colspan="2">mi, Material Constant</td><td>10</td><td>10</td></tr>
<tr><td colspan="2">D, Disturbance factor</td><td>0</td><td>0 – 0,8</td></tr>
<tr><td colspan="2">Ei, Elasticity Modulus (GPa)</td><td>6,75</td><td>15,0</td></tr>
<tr><td colspan="2">υ, Poisson’s ratio</td><td>0.34</td><td>0.32</td></tr>
<tr><td colspan="2">$\gamma_n$, Unit weight (kN/m3)</td><td>27</td><td>27</td></tr>
<tr><td colspan="2">H, Effective Rock Height (m)</td><td>60</td><td>130</td></tr>
<tr><td>c, Cohesion (kPa)</td><td>D=0</td><td>139</td><td>345</td></tr>
</table>

| | | | | |
|---|---|---|---|---|
| | D=0.8 | | | 199 |
| **Ø**, Internal friction angle (º) | D=0 | | 32 | 33 |
| | D=0.8 | | | 22 |
| **Em**, Deformation modulus (GPa) | Nicholson & Bieniawski | | 0,49 | 2,2 |
| | Hoek & Diederichs | D=0 | 0,4 | 2,4 |
| | | D=0.8 | | 0,77 |
| **RMR**, Rock Mass Rating | | | 32 | 44 |
| **Q**, Tunneling Quality Index | | | 0,55 | 1,1 |

The elements of the provisional support system consist of the following systems and / or their various combinations depending on the class of rock or geological conditions encountered:

### 4.1 Shotcrete:

The use of shotcrete is essential as a supporting element that prevents the relaxation of the peripheral rock. Shotcrete is the element that provides the greatest support pressure among the support elements. A first-layer shotcrete will be applied in all support systems after excavation against the risk of failure and collapse of the layers. A second layer shotcrete will be applied in all support systems after the location of Steel lattice and steel Retaining.

*Fig. 5– Photography of tunnel shotcrete stage*

### 4.2 Steel lattice:

A steel lattice will be applied between the concrete layers to form the static and constructive reinforcement of the concrete coating. The use of steel lattice is intended to ensure adhesion between rock and shotcrete, stabilization, increase in shear strength and prevention of excessive cracks until the setting of concrete.

*Fig. 6– Photography of tunnel Steel lattice stage*

### 4.3 Steel retaining:

In principle, the steel support provides immediate support before the shotcrete freshly begins to wear and constitutes the reinforcement with the lattice after the concrete has acquired its strength. Steel support is also a support for drilled bolts and provides mental confidence for employees. HEB "180, 220" profiles are used in this project.

*Fig. 7– Photography of tunnel Steel retaining stage*

### 4.4 The pipes or pre-supporting iron bar:

The purpose of the piles support is to provide support by Umbrella effect around the forehead. For this purpose perforated pre- support will be used with Ø 5.0 "and 7.0" injection pipes depending on the rock classes. Their distance is between 20-40 cm depending on the class of rock. In addition, after attaching certain pipes to the hole, making them wait without an injection for a moment ensures the drainage of the groundwater that can come on the face. In the following by the injection these pipes will assume their own function.

*Fig. 8– Photography of tunnel pre-supporting iron bar (Umbrella) stage*

### 4.5 Rock bolts:

Rock bolts will be applied systematically as part of the support type system. Rock bolts are used in all support systems because they increase the quality and strength of the rock mass by increasing shear strength, reducing deformation in the tunnel and preventing rock breakage. The whole procedure will be performed by injection given that it is not a design that aims to support the rock blocks or thin layers. The length of the rock bolts is chosen so that they extend at least ~ 2 m above the plastic zone formed around the tunnel. The diameter of the injection hole will be 1.5 of the diameter of the bolt. The bolts will be installed in radial position on the walls of the tunnel. Bolts of type SN and IBO will be used.

*Fig. 9– Photography of tunnel Rock bolts stage*

### 5. Numerical modeling and Analyzes performed with the Phase 2 2D software:

The numerical analyzes were performed with the Phase 2 2D program (Version 8.0). Is a finite element program developed by the University of Toronto which models the masses of rock and the sustained behaviors of these masses. The program is progressively modeling the underground excavation, providing support with bolts, steel retaining, steel lattice and shotcrete. In addition, the load split between the excavation phases and the material softening can be applied to the model. The designation of support systems based on practice and experience, numerical analyzes were considered as a guide for practical decisions. The support system will have to be revised according to the actual field situation and the geological mapping and the footage results.

### 5.1 Soil and provisional support modeling:

The calculation sections are taken on the part represented by the rock formation between the determined KP (kelometric point). The calculations for these sections are valid for the part represented by the section. The parameters of the rock mass are estimated with these calculation sections according to the recommendations and approaches of the literature. Excavation coordinates are given in the X-Y system that accepts the center of the tunnel in the zero coordinate (O1). These units are given in meters in the program.

*Table 2*

*Provisional Support Systems Offered in Twin-tube Tunnel under the Massif Rocky Classification System and recommendation*

<table>
<tr><td colspan="3">Rock class type</td><td colspan="2">Low Rock (IV)</td><td colspan="2">Middle Rock (III)</td></tr>
<tr><td colspan="3">R, Tunnel radius (m)</td><td colspan="2">7.5</td><td colspan="2">7.5</td></tr>
<tr><td colspan="3">P<sub>0</sub>, In-situ pressure (MPa)</td><td colspan="2">1.62</td><td colspan="2">3.51</td></tr>
<tr><td colspan="3">ESR, Excavation Support Ratio</td><td colspan="2">0.9 - 1.1</td><td colspan="2">0.9 - 1.1</td></tr>
<tr><td colspan="3">Sfr+B, Support class (Grimstad and Barton 1993, Barton 1995 & 2002)</td><td colspan="2">6</td><td colspan="2">6</td></tr>
<tr><td colspan="3">k=0.25+7Eh(0.001+1/z)</td><td colspan="2">0.30</td><td colspan="2">0.39</td></tr>
<tr><td rowspan="2">Excavation / Provisional support</td><td rowspan="2">Advancement of the upper half / lower half.</td><td>Calotte</td><td colspan="2">advance of the upper half by 20 to 25 m maximum</td><td colspan="2">advance of the upper half by 35 to 50 m maximum</td></tr>
<tr><td>Strauss</td><td colspan="2">advancement of the lower half of 2.0 m maximum</td><td colspan="2">advancement of the lower half of 3.0 m maximum</td></tr>
<tr><td rowspan="2">Steel retaining</td><td rowspan="2">Steel S275JR</td><td>Dimension (mm)</td><td colspan="2">HEB 220</td><td colspan="2">HEB 180</td></tr>
<tr><td>Distance between them (m)</td><td colspan="2">0.75</td><td colspan="2">1.25 - 1.50</td></tr>
<tr><td>Shotcrete</td><td>RN-30/40</td><td>Dimension (cm)</td><td colspan="2">35</td><td colspan="2">25</td></tr>
<tr><td>Steel lattice</td><td>Steel FeE400</td><td>Dimension (mm)</td><td colspan="2">( 2x Q589/443) (150x150x6.5)</td><td colspan="2">( 2x Q221/221) (150x150x6.5)</td></tr>
<tr><td>Anchor plate</td><td>Steel FeE26</td><td>Dimension (mm)</td><td colspan="2">200/200/15</td><td colspan="2">200/200/15</td></tr>
<tr><td>Drain pipes</td><td>if necessary</td><td>Dimension (m)</td><td colspan="2">3x 12</td><td colspan="2">/</td></tr>
<tr><td>Injection Pre-supporting iron bar</td><td>ST37</td><td>t=3mm, a=30cm</td><td colspan="2">45x (Ø 7.0, L= 8m)</td><td colspan="2">45x (Ø 5.0, L= 6m)</td></tr>
<tr><td rowspan="4">Rock bolt</td><td rowspan="4">Steel FeE400<br>(PG PULT = 250 KN in St III steel)</td><td rowspan="2">SN, Ø 32 mm</td><td>Lenght</td><td>/</td><td>Lenght</td><td>L=6m</td></tr>
<tr><td>Mumber</td><td>/</td><td>Mumber</td><td>19 - 23</td></tr>
<tr><td rowspan="2">IBO, Ø 32 mm</td><td>Lenght</td><td>L=8m</td><td>Lenght</td><td>/</td></tr>
<tr><td>Mumber</td><td>29 -33</td><td>Mumber</td><td>/</td></tr>
<tr><td colspan="3">Deformation measurements (mm)</td><td colspan="4">Will be performed every 10 meters along the tunnel</td></tr>
</table>

Relevant soil modeling is very difficult in soil excavations given the many uncertainties and complexity. The numerical analyzes are performed according to the elastic-plastic solution. Thus the detailed modeling which includes all the conditions is neither possible nor this modeling is useful. The relaxation of material used in the weak rock masses as indicated above is applied at 0.65 (65%) in the excavation of the upper half and 0.35 (35%) is reflected in the model with the installation of the supports of the upper half and when excavating the lower half. The purpose of this distribution is to determine the rate of load to be carried by the rock and the rate of load to bear by the supports. The linear composite is applied in 3 layers on the model in the excavations of the upper part, the lower part and the slab. In the excavation levels, the first layer of shotcrete lining and the steel retaining (HEB) and the second layer of shotcrete liner and steel lattice are entered into the model. The analyzes are carried out in two stages namely, for earthquake and without earthquake. Simplification of the model may be possible under the following conditions:

- Reduction of three-dimensional conditions to two dimensions,
- Acceptance of the symmetry of the section with the axis,
- Simplification of the soil with simple descriptions,
- Simple and comprehensive description of the progress conditions of the tunnel and the excavation,
- Soil is considered homogeneous and isotropic.

**5.2.1 Evaluation of the deformations of the middle rock class (III) of the part between KP: 26 + 230 - KP: 26 + 550 of the right tube and KP: 2 + 191 - KP: 2 + 490 of the left tube of the tunnel:**

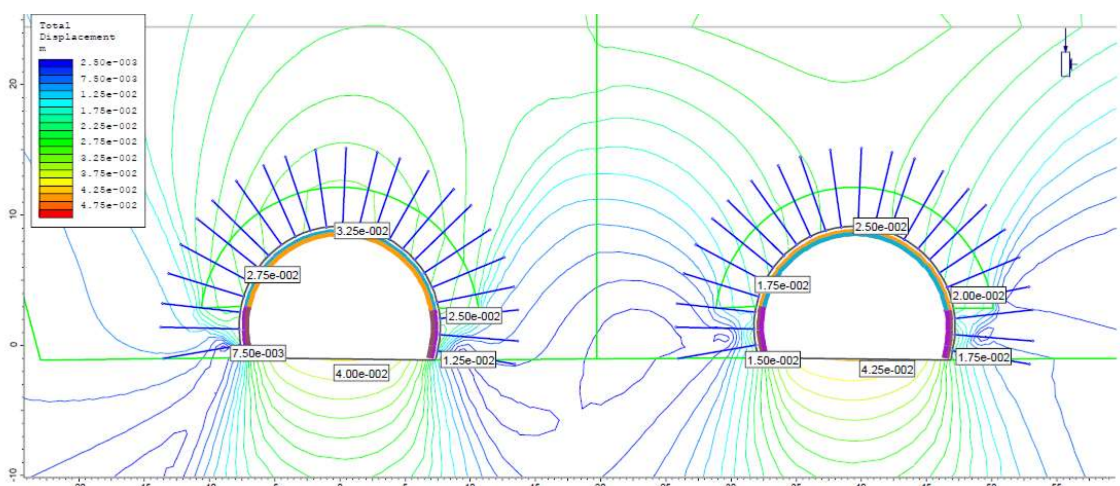


*Fig. 10– (a) Total displacement in Situation without earthquake in class III.*

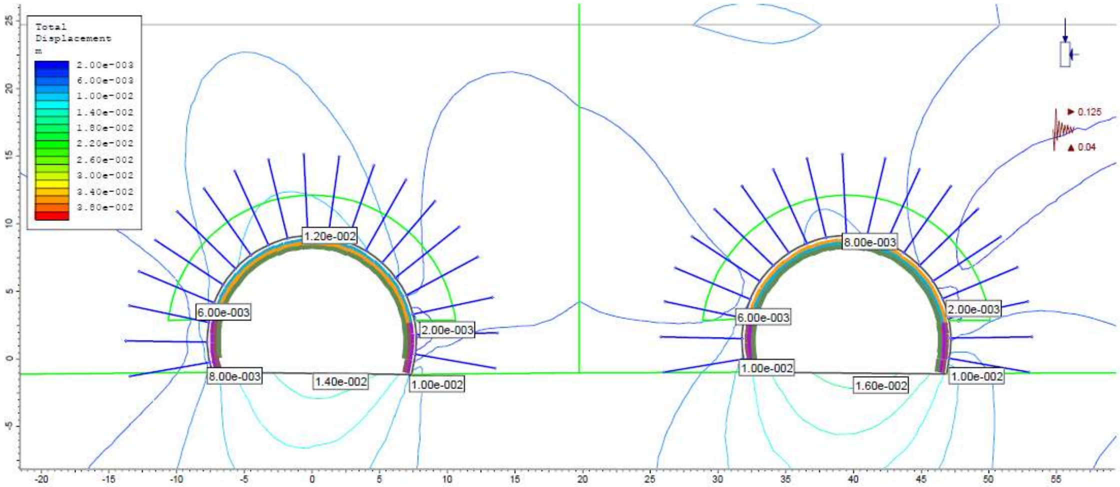


*Fig. 10– (b) Total displacement in Situation after earthquake in class III.*

Examination of displacement formed around the tunnel (figure 10, a) indicates a displacement of 3.25 cm in the ceiling (summit), 2.75 cm and 2.50 cm in the left and right wings of the tunnel, 0.75 cm and 1.25 cm in the lower left and right parts of the tunnel and 4 cm in the bottom of left tunnel tube. In the right tunnel tube, is observed a displacement of 2.5 cm in the tunnel ceiling, 1.75 cm and 2.0 cm in the left and right wings, 1.50 cm and 1.75 cm in the left and right lower halves and 4.25 on the bottom. When the earthquake was applied the examination of the deformations around the tunnel (Figure 10, b) shows in the left tunnel tube, a displacement of 1.2 cm in the tunnel ceiling, 0.6 cm and 0.2 cm in the left and right wings, 0.8 cm and 1.0 cm in the left and right lower halves and 1.4 on the bottom. In the right tunnel tube, a displacement of 0.8 cm in the ceiling of the tunnel, 0.6 cm and 0.2 cm in the left and right wings, 1.0 cm in the left and right lower halves and 1.6 on the bottom. According to the results, it appears that the provisional support system consisting of steel lattice, steel retaining, bolts and shotcrete is able to carry the loads from the tunnel.

**5.2.2 Evaluation of the deformations of the low rock class (IV) of the part between KP: 26 + 550 and the exit of the right tube & KP: 2 + 490.970 and the exit of the left tube of the tunnel:**

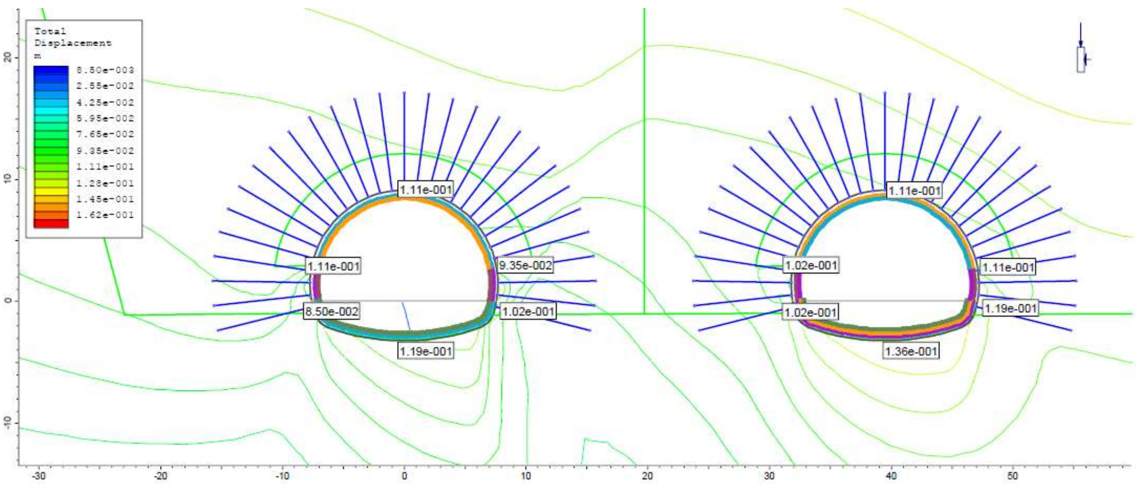


*Fig. 11– (a) Total displacement in Situation without earthquake in class IV.*

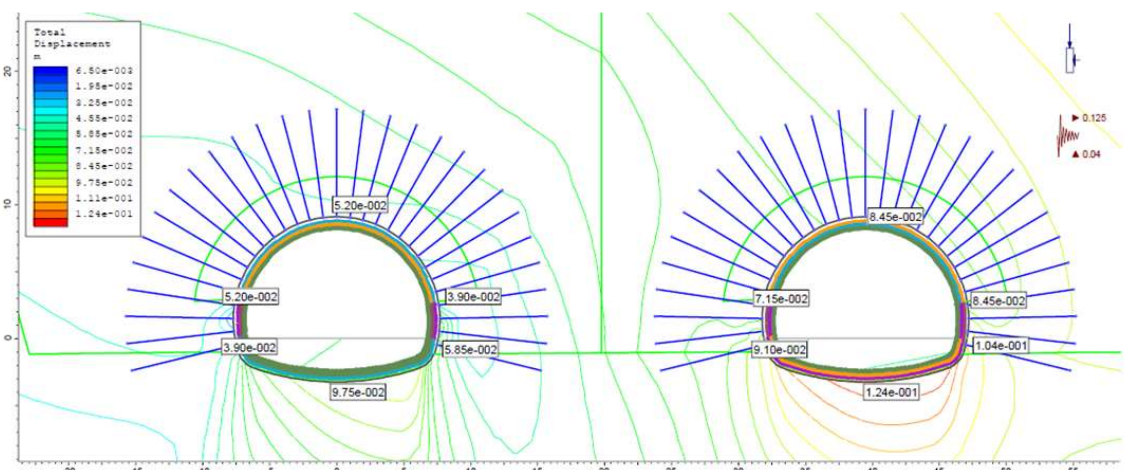


*Fig. 11– (b) Total displacement in Situation without earthquake in class IV.*

The examination of the deformations around the tunnel (figure 11, a) indicates a displacement of 11.1 cm in the ceiling, of 11.1 cm and 9.35 cm in the left and right wings of the tunnel, of 8.50 cm and 10.2 cm in the lower left and right parts of the tunnel and of 11.9 cm in the bottom of the left tunnel tube. In the right tunnel tube, there is a displacement of 11.1 cm in the ceiling of the tunnel, of 10.2 cm and 11.1 cm in the left and right wings, of 10.2 cm and 11.9 cm in the left and right lower halves and of 13.6 cm on the bottom. When the earthquake was applied the examination of the deformations around the tunnel (Figure 11, b) shows in the left tunnel tube, there is a displacement of 5.2 cm in the tunnel ceiling, 5.2 cm and 3.9 cm in the left and right wings, 3.9 cm and 5.85 cm in the left and right lower halves and 9.75 on the bottom. In the right tunnel tube, there is a displacement of 8.45 cm in the tunnel ceiling, 7.15 cm and 8.45 cm in the left and right wings, 9.1 cm and 10.4 cm in the left and right lower halves and 12.4 on the bottom. According to the results, it appears that the provisional support system consisting of steel lattice, steel retaining, bolts and shotcrete is able to carry the loads from the tunnel.

## 6. Monitoring of the underground deformation:

Tunnel ground deformation monitoring is the main means for selecting the appropriate methods of excavation and retaining from among those provided in the design to ensure the safety of the tunnel construction (including the safety of personnel in the tunnel and the safety of structures on the ground surface). The construction of the system is planned for the continuation of the stop of the deformations and movements of the ground likely to occur after the construction of the elements of primary support in this system. In this case, it is accepted that there will be no load transfer on the coating concrete as the pressure from the ground is supported by the provisional support system. As a result, a separate analysis was not performed for the coating concrete. The monitoring program includes the specification of the measurement procedure, the location of the monitoring devices and the monitoring schedule. Attention is given to the fact that monitoring results are often affected by instrumentation, installation and environmental effects. The type of instrumentation chosen must ensure the following:

- A feasible installation procedure,
- Sustainability during the monitoring period,
- Protection against damage during construction,
- Simple processing of measurements (acquisition and transmission of data),
- Precision is required.

In general, close readings of excavation activities are taken daily; the frequency is reduced with the distance to the forehead and the decrease of the displacement rates. Shorter monitoring intervals may be required due to the specific project requirements. Monitoring sections in tunnels and shafts are usually located at distances of 5 to 20 m depending on the conditions and requirements limits. A possible concept showing minimum reading frequencies and ranges for surface and underground monitoring for a summit-wings-bottom sequence as shown in figure (12).

*Fig. 12– Photography of the tunnel underground monitoring*

In general, there are types of failure that cannot be detected in time by deformations monitoring, it is recommended to use additional monitoring of absolute displacements, but in a small extent. Thus the presence of an emergency surveillance system in case of adverse field conditions is ensured. In the case of block rock mass tunnels, the characteristic hazards are the detachments caused by the discontinuity of the blocks, therefore the observations must concentrate on the soil structure, the location and the orientation of the discontinuity with respect to the alignment of the tunnel. In the case of tunnels with moderate to high overload in the bedrock or foliar mass, the characteristic risks are; the orientation of the stratification or foliation, the displacement of the pavement, the displacements of the soil and the structure of the soil. consequently the Observation focused on; visual inspections, laboratory tests, absolute displacement monitoring.

### 6.1 Monitoring methods and requirements:

Measurements are performed using a total station and objectives. Precise prism lenses as well as bi-reflex lenses (reflectors) are used and their spatial position in the global coordinate system or project is determined. Discrete three-dimensional displacement measurements are performed by repeated measurements (usually on a daily basis). Since full monitoring cannot usually be performed from one position, an interconnected observation pattern is required, which is established using identical reference points. Stable reference points are differentiated from points that always move. Points with a defined maximum displacement rate (usually <1mm / month) can be used as reference points.

The principle of "free parking determined" is used to obtain the position of the instrument. The absolute position of all coordinate components of the marked measuring points shall be determined with an accuracy of +/- 1 mm (standard deviation) with respect to adjacent measuring sections over the entire observation period. The following sources of error should be avoided:

- Observations near the tunnel wall (minimum wall distance of 0.5 m to 1 m),
- Measurement errors due to refraction (for example. observation through or near heat sources),
- Position of the instrument near the side walls,
- Observations in asymmetrical connection,
- Measurements in a very dusty environment or when there is a lot of vibration (i.e. Caused by machines).

The surveyor must record and submit the following items after each measurement action:

- Measurement sequence system (relative to the measurement section or along the tunnel),
- Unmeasured points and reason indication (destroyed, not visible, etc.),
- Significant displacements (measurement error, rapid increase in displacements),
- Readings to zero,
- Monitoring conditions (air quality, vibration, limited visibility, sources of heat, etc.).

The geometric definition of the sections is shown on the drawings. The purpose of these sections is to measure convergences in the tunnel during construction. In general, the convergence sections will be composed by 5 points distributed as shown above, one in the summit, two in the forward section (calotte), in the gables at a height of 1.50m from the base excavation and the other two, at the stross section, at a height of 1.50m from the tunnel bottom, also in the gables.

**6.2 Deformations diagrams from monitoring results:**

A lateral and longitudinal displacement of 40-55 mm / month has been observed in some sections of the middle rock (Class III) left tube calotte (Figure 13). Also in the right tube calotte, lateral and longitudinal movements of 30-40 mm / 2 months (Figures 14) and lateral displacements of 90 mm / 2 months (Figures 14a, c), which forces us to reinforce immediately by bolts IBO / L = 8m (3 top, 2 wall right side, 2 wall left side), on the other hand, the remains sections are stable. Consequently; deformations were stopped and class III-A was created. In along the low rock tunnel (Class IV) (Figures 15, 16); in the left tube; a maximum settlement of 80 to 120 mm / 4 months was observed (Figure 15 a, b, c), lateral and longitudinal displacements of 40- 100 mm / year (Figures 15). In the right tube; maximum deformations of 20 to 40 mm / 2 months have been observed (figure 16 a), deformations up to 70 mm / 2 months (Figure 16 b), maximum settlements of 60 to 80 mm / year (Figure 16c, d), deformations of 60-150 mm / 10 months (Figures 16 e, f, g), which forces us to reinforce immediately by bolts IBO / L = 16m (6 top) and sometimes bolts IBO / L = 12m (2 lateral right, 2 lateral left), on the other hand, the remains sections are stable. Consequently; deformations were stopped and classes IV-A, IV-B were created.

Real deformations are more than numerical modeling when we compare the monitoring results with numerical modeling results, we can say that it is logical, because the software cannot really simulate at one hundred percent construction tunneling phenomenon without making uncertainties between real and digital data. In conclusion, maximum attention must be given to deformations when the implementation of tunnel digging and the setting up of provisional support.

### 6.2.1 Middle rock (Class III):

**Left tube:**

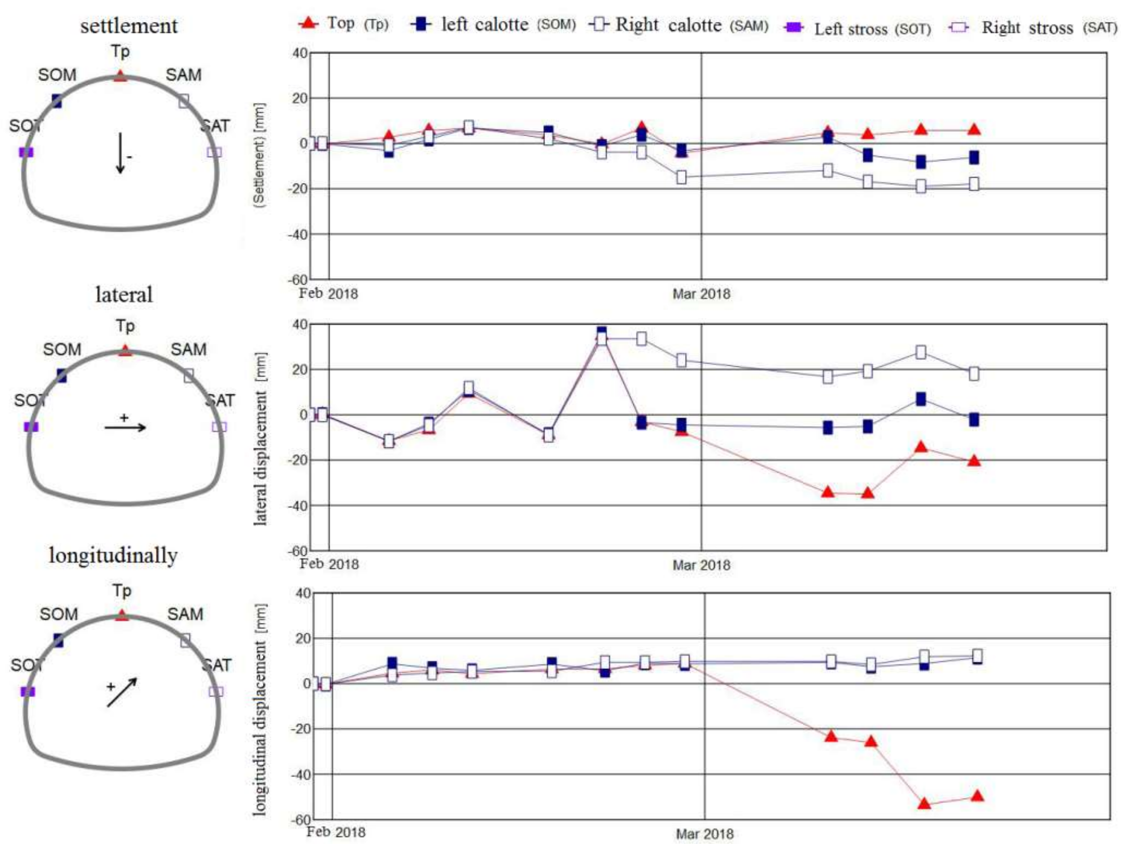


*Fig. 13– Left tube Tunnel Cross section KP - 2,485.000 / First measure: 30.01.2018, Last measure: 21.03.2018*

**Right tube:**

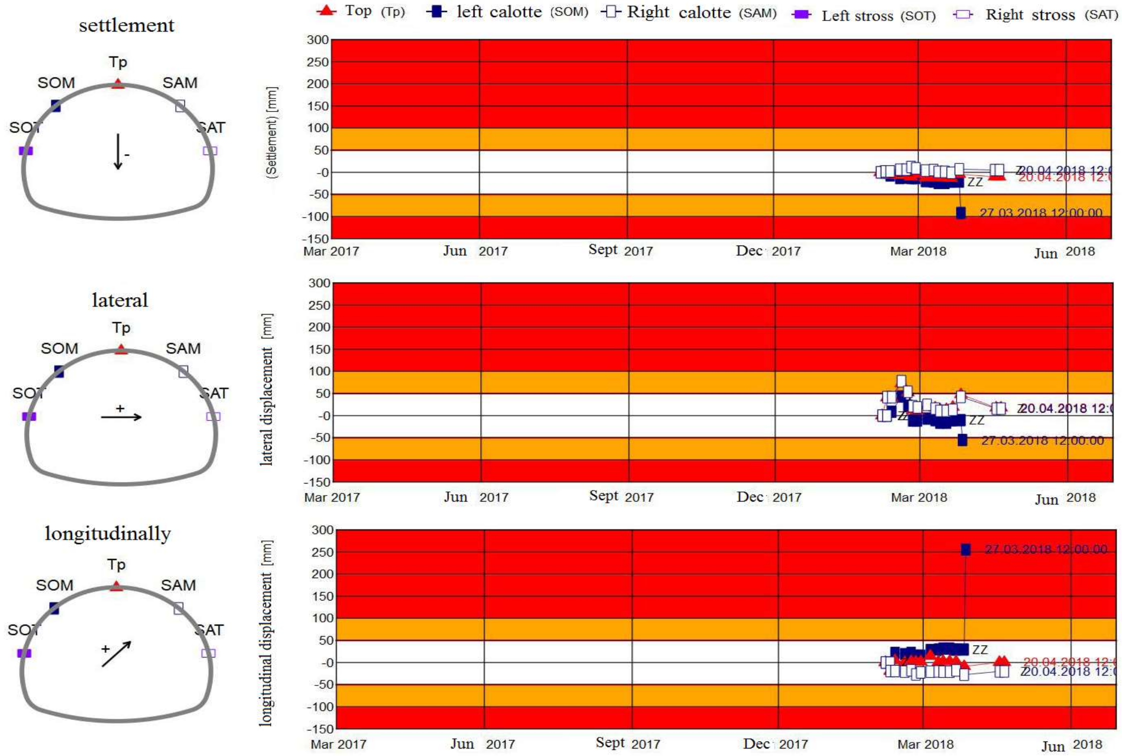


*Fig. 14– (a) Right tube Tunnel Cross section KP 26,512.000/ First measure: 5.02.2018, Last measure: 20.04.2018.*

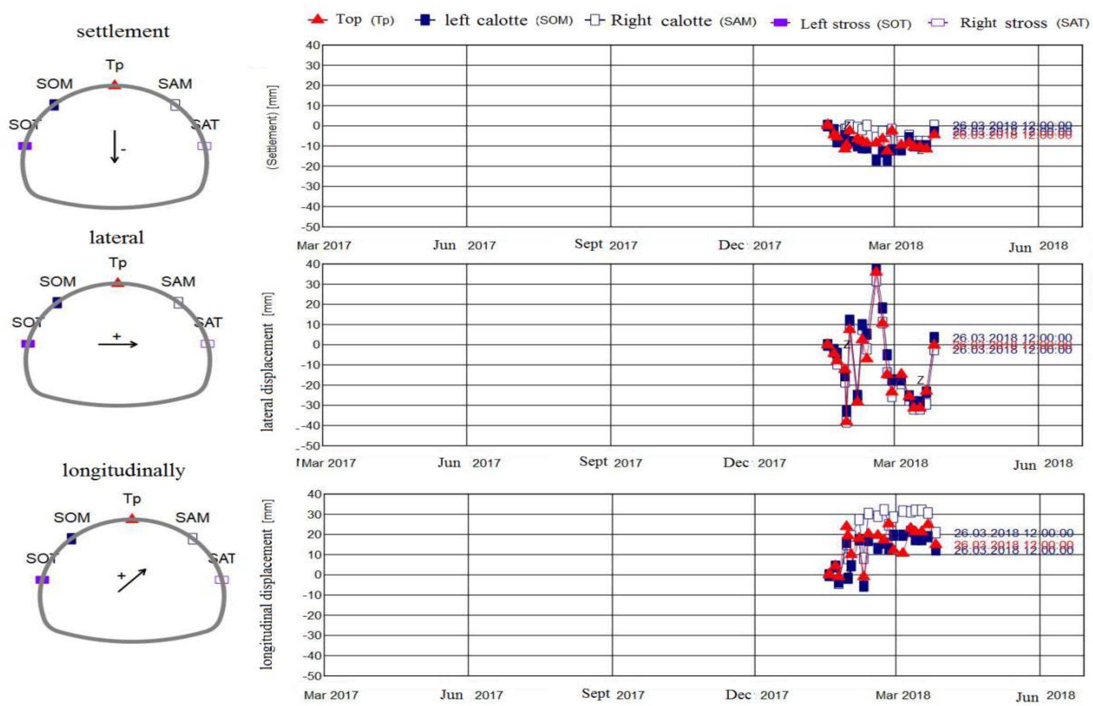


*Fig. 14– (b) Right tube Tunnel Cross section KP 26,527.000/ First measure: 17.01.2018, Last measure: 26.03.2018*

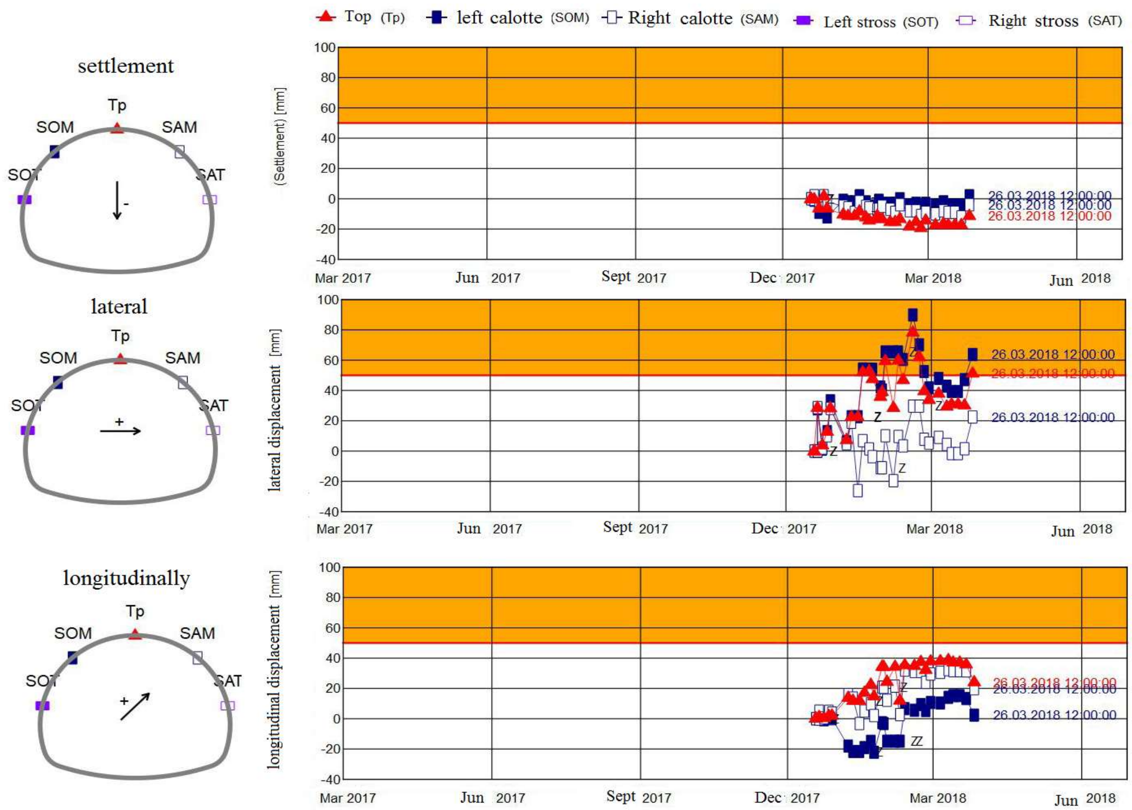


*Fig. 14– (c) Right tube Tunnel Cross section KP 26,543.000 / First measure: 18.12.2017, Last measure 3.04.2018*

### 6.2.2 Low rock (Class IV):

### Left tube:

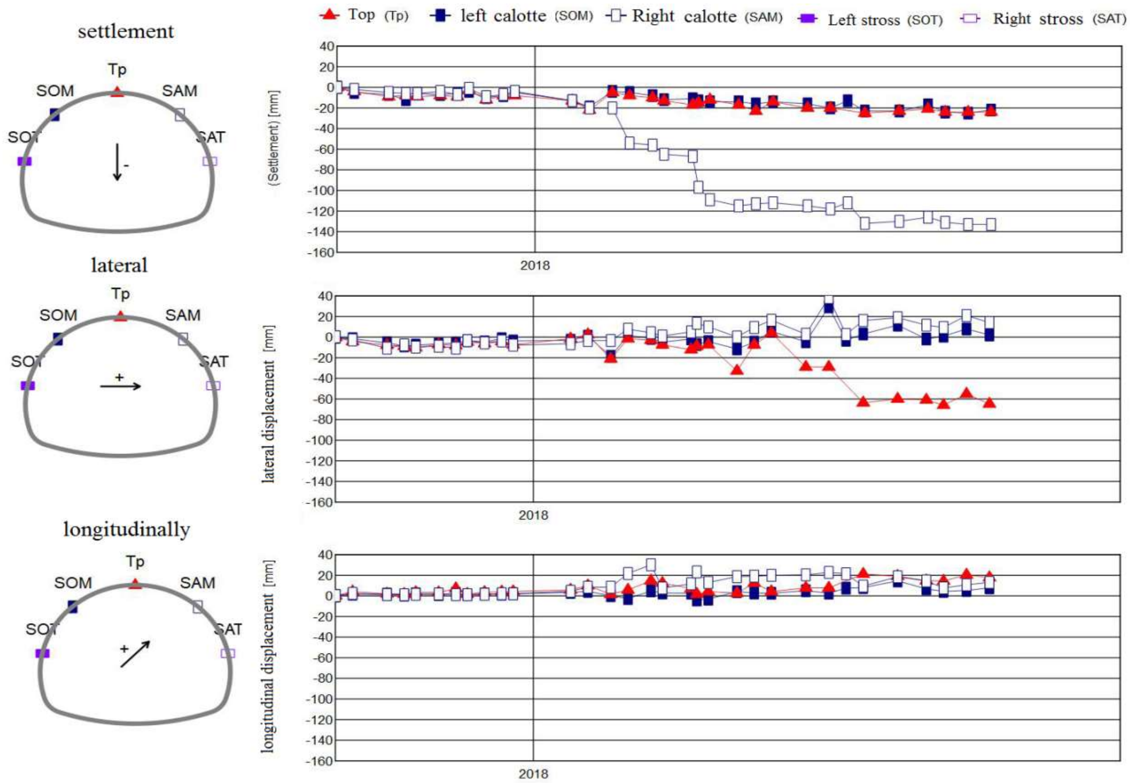


*Fig. 15– (a) Left tube Tunnel Cross section KP 2,501.000/ First measure: 15.05.-2017, Last measure: 7.04.2018*

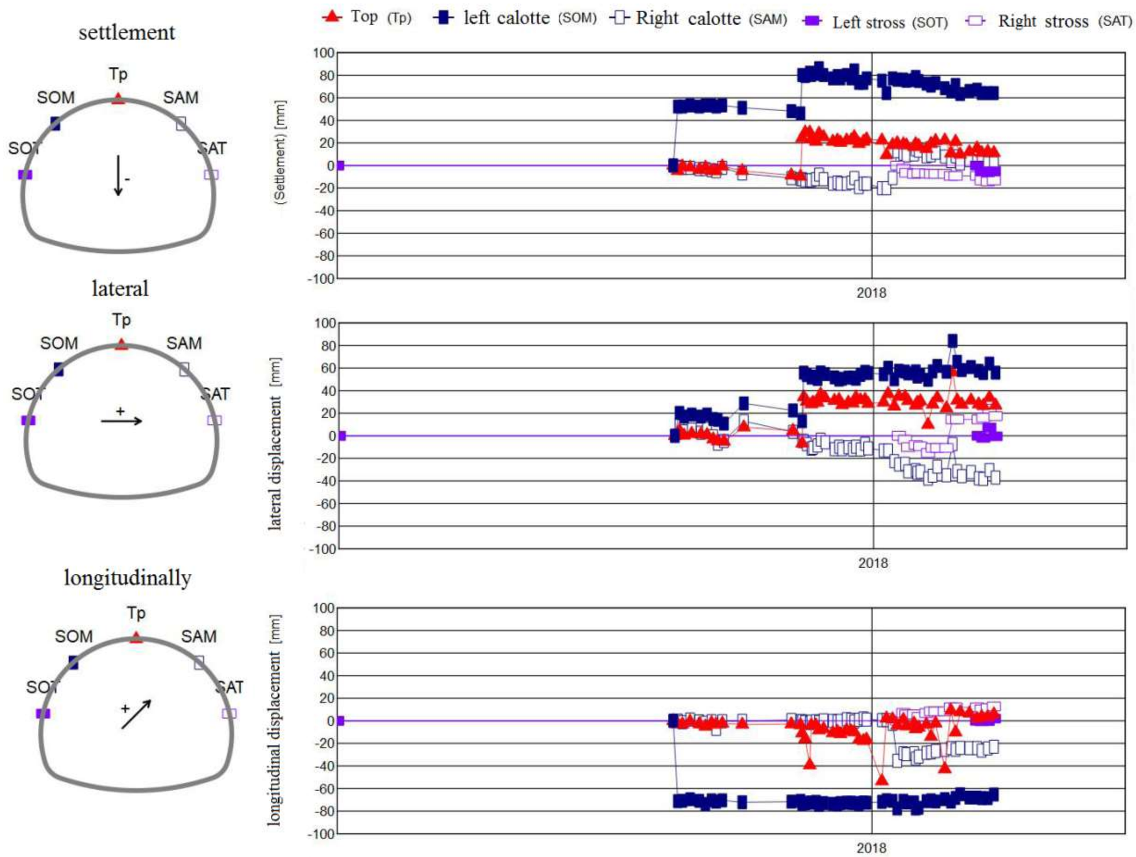


*Fig. 15– (b) Left tube Tunnel Cross section KP 2,516.000/ First measure: 17.01.2017, Last measure: 21.03.2018*

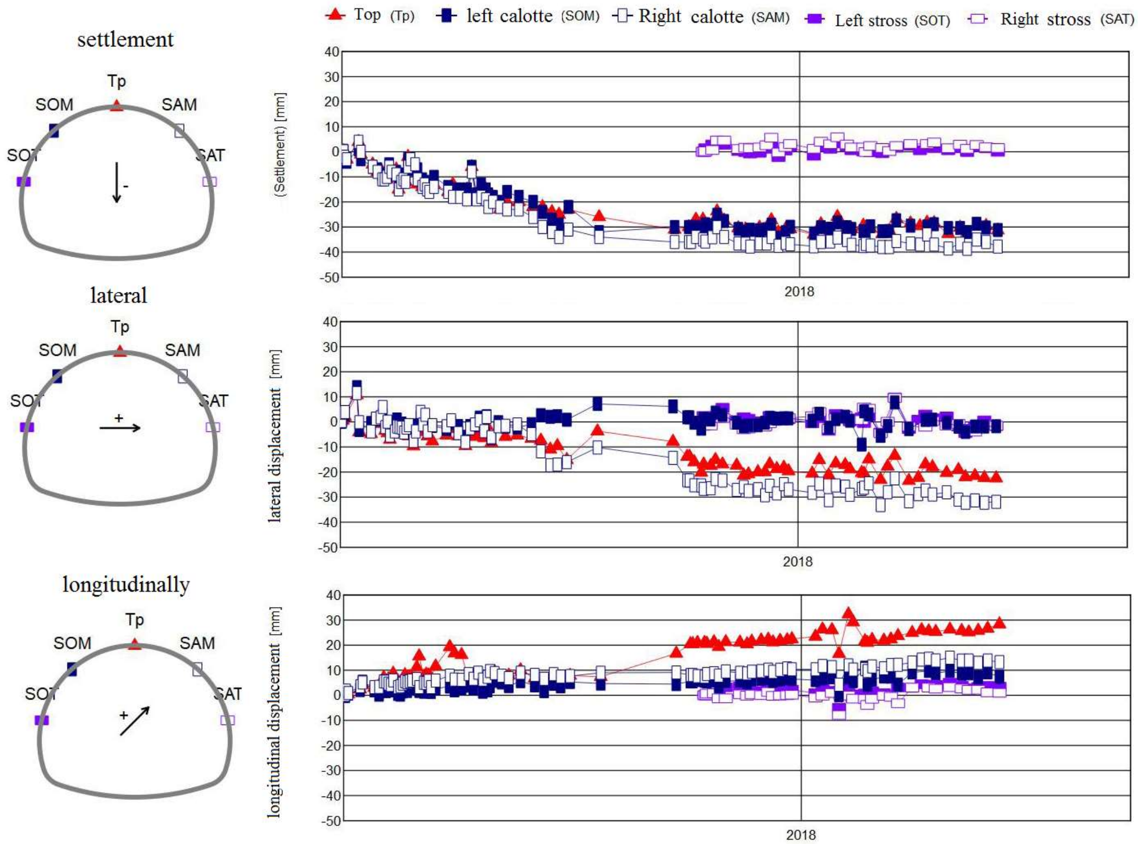


*Fig. 15– (c) Left tube Tunnel Cross section KP 2,541.000/ First measure: 21.06.2017, Last measure: 26.03.2018*

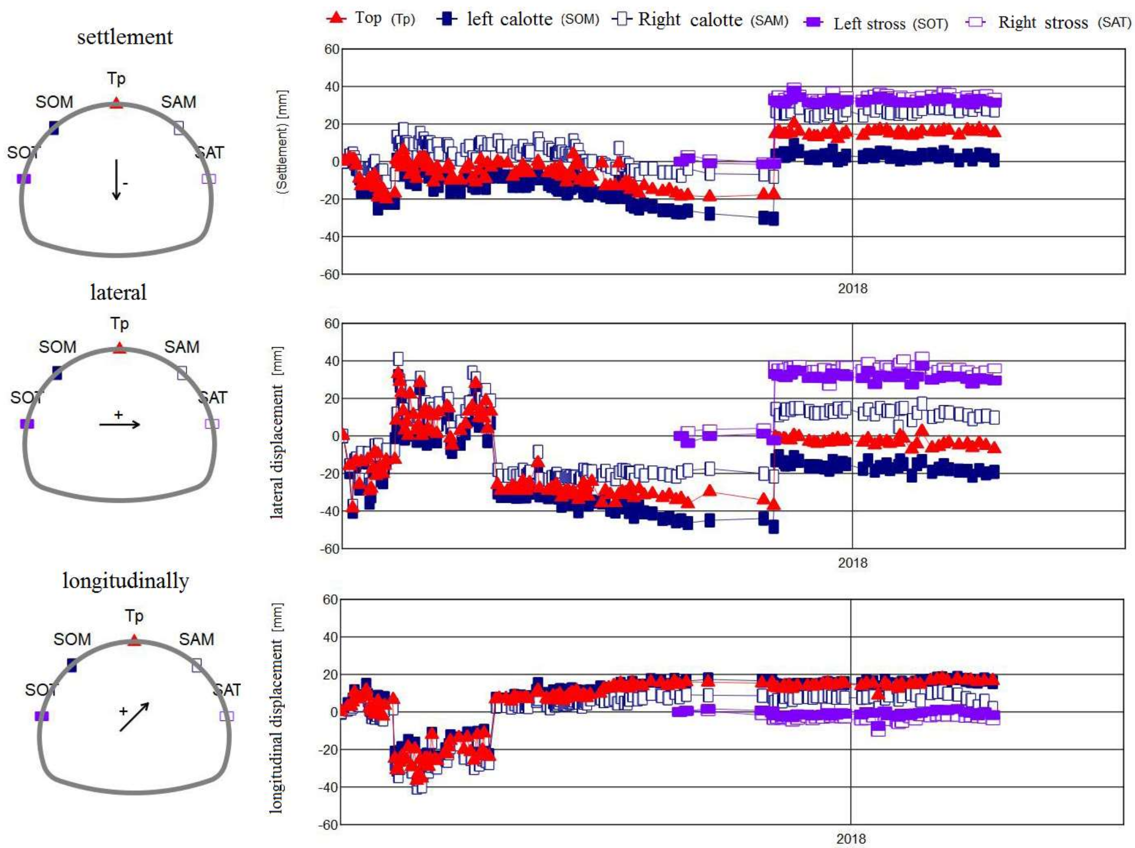


*Fig. 15– (d) Left tube Tunnel Cross section KP 2,555.000/ First measure: 4.03.2017, Last measure: 26.03.2018*

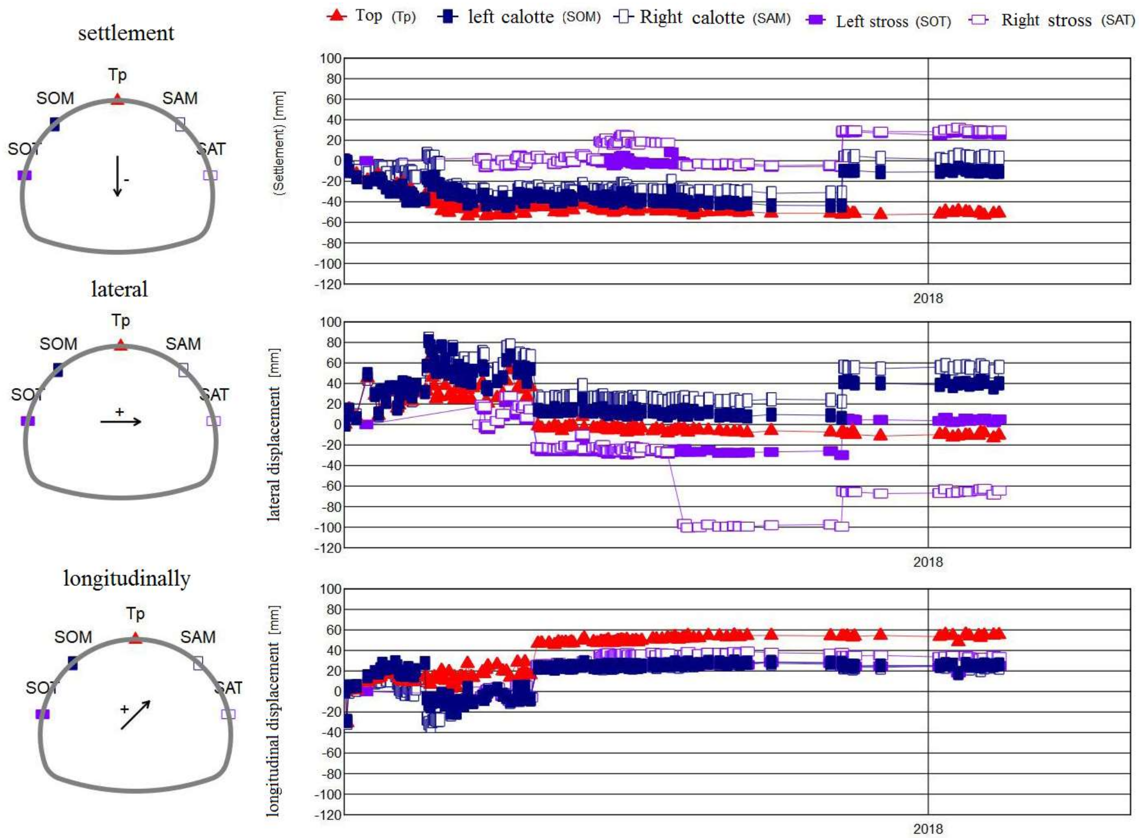


*Fig. 15– (e) Left tube Tunnel Cross section KP 2,566.000/ First measure: 19.02.2017, Last measure: 8.02.2018*

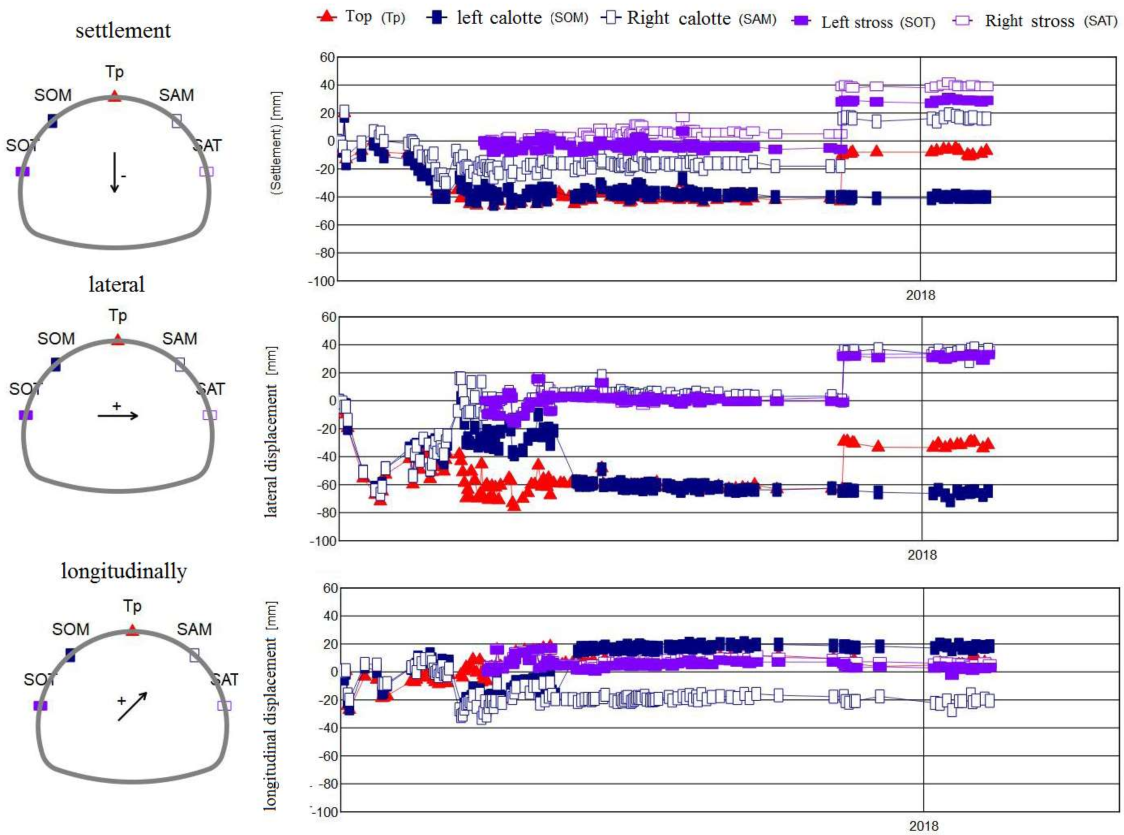


*Fig. 15– (f) Left tube Tunnel Cross section KP 2,577.000/ First measure: 26.01.2017, Last measure: 8.02.2018*

**Right tube:**

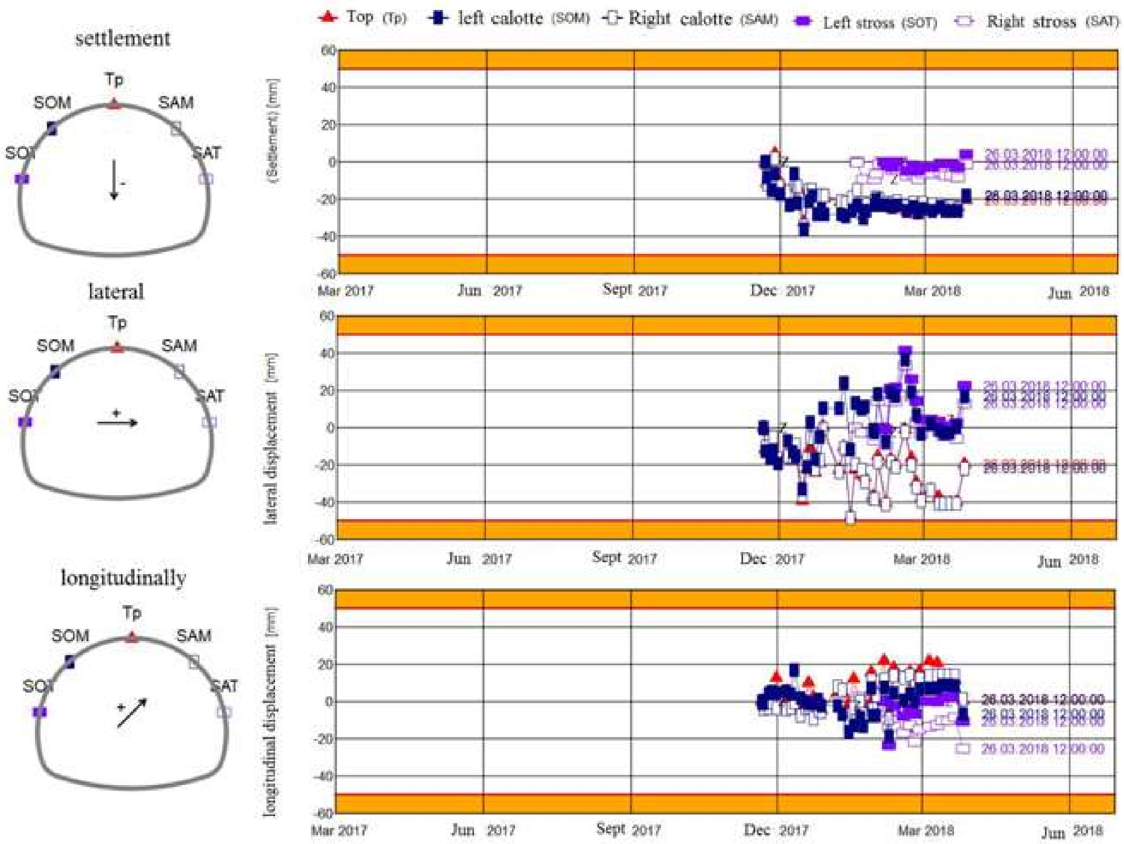


*Fig. 16– (a) Right tube Tunnel Cross section KP 26,562.000/ First measure: 21.11.2017, Last measure: 26.03.2018*

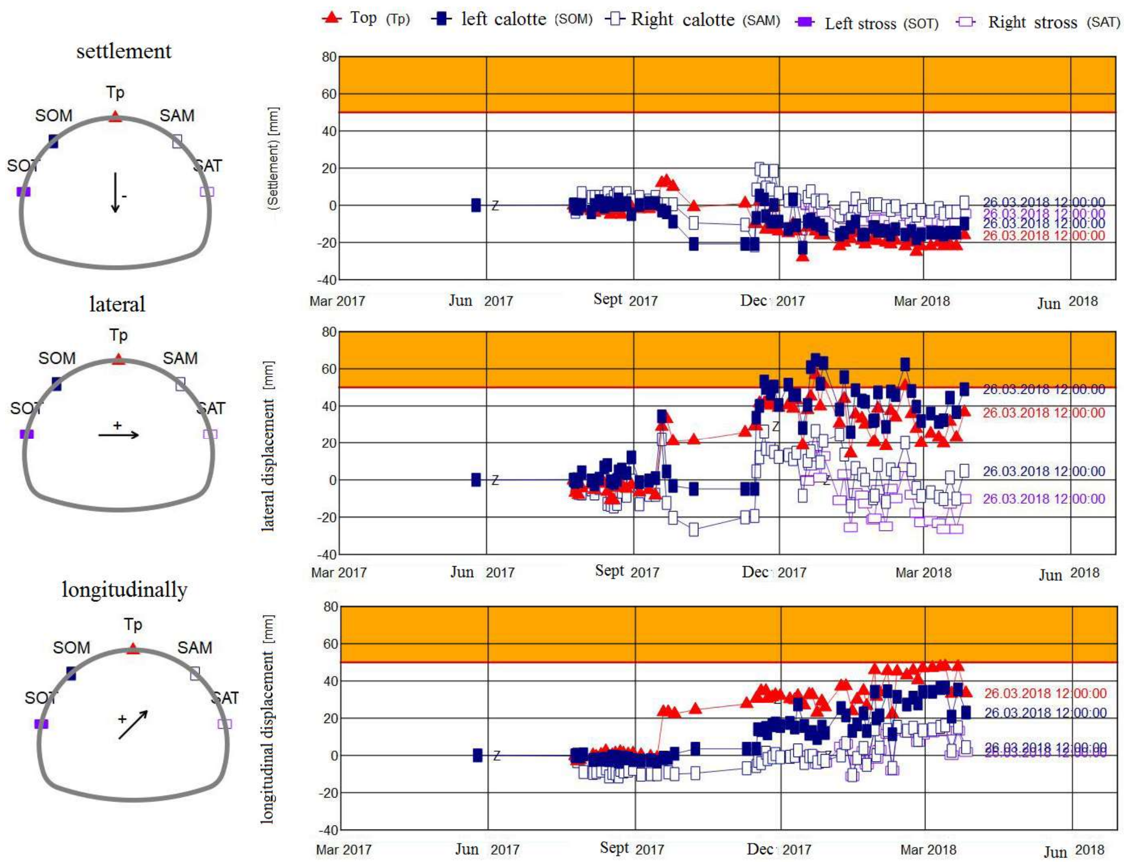


*Fig. 16– (b) Right tube Tunnel Cross section KP 26,576.000/ First measure: 15.05.-2017, Last measure: 26.03.2018*

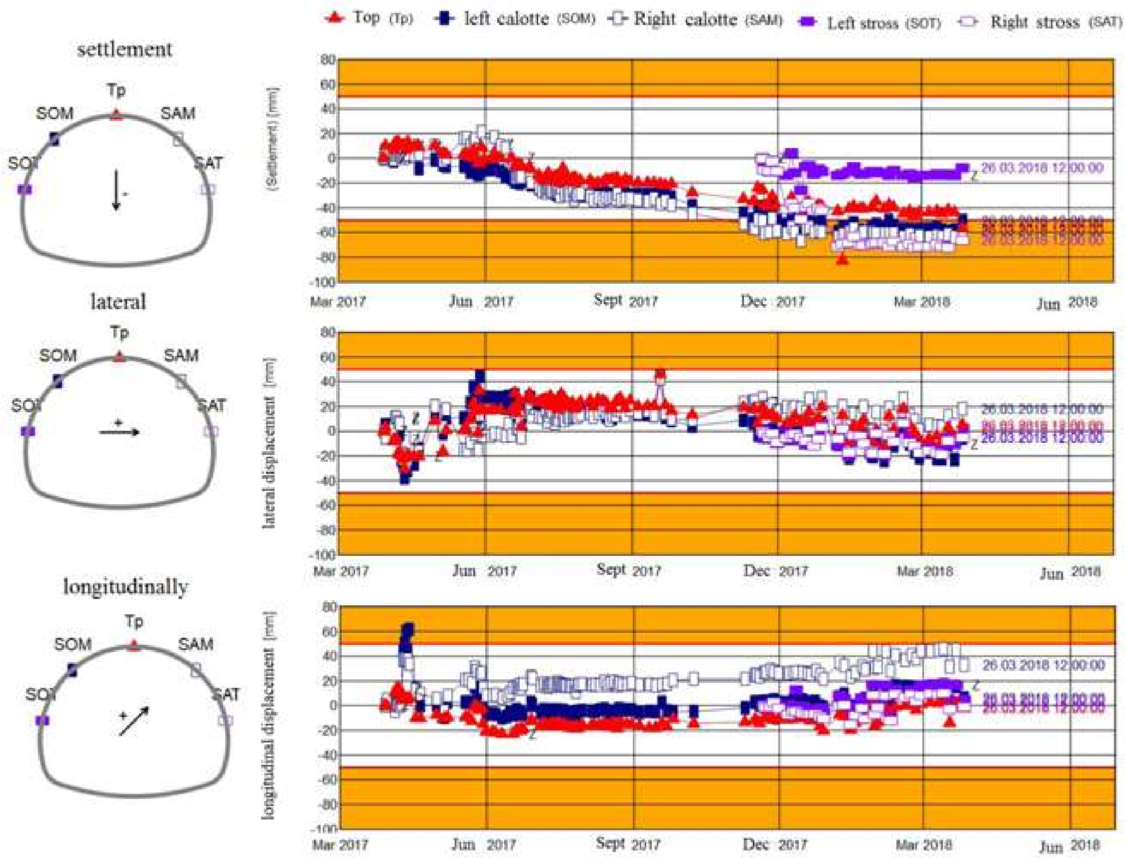

*Fig. 16– (c) Right tube Tunnel Cross section KP 26,588.000/ First measure: 15.05.-2017, Last measure: 26.03.2018*

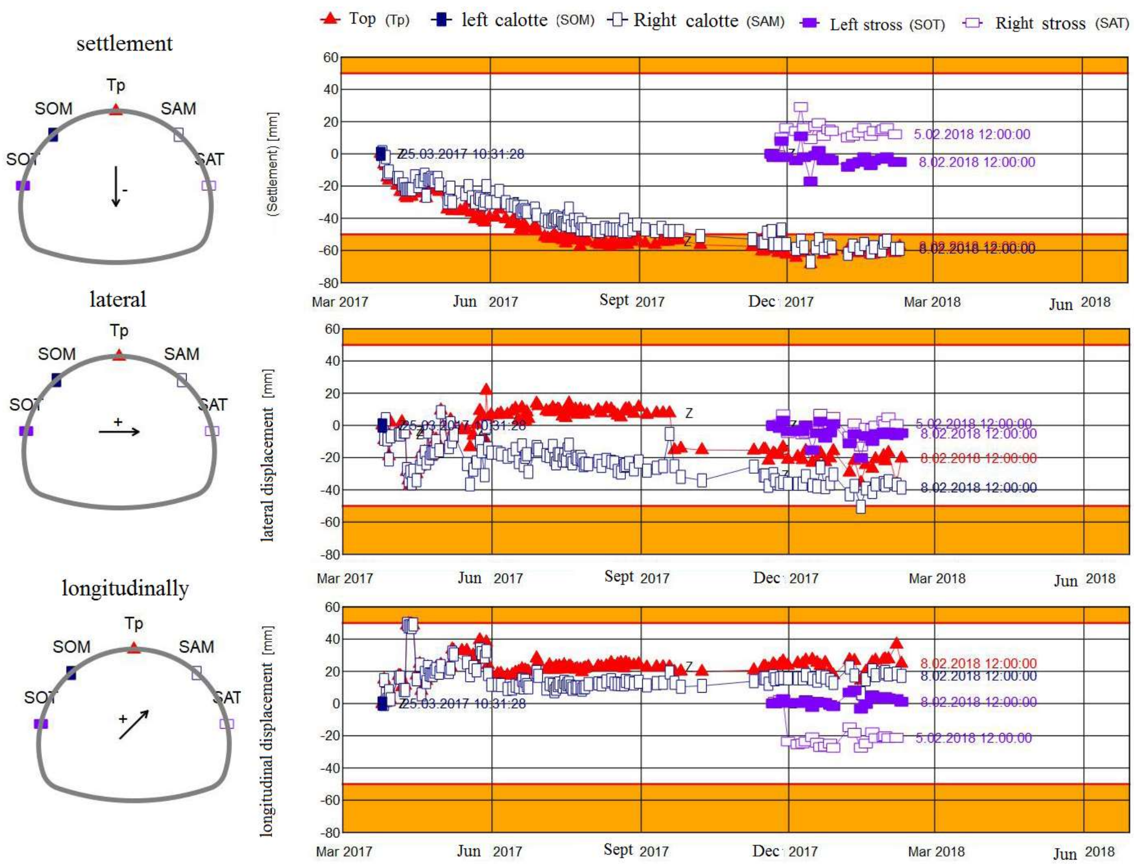


*Fig. 16– (d) Right tube Tunnel Cross section KP 26,603.000/ First measure: 15.05.-2017, Last measure: 8.02.2018*

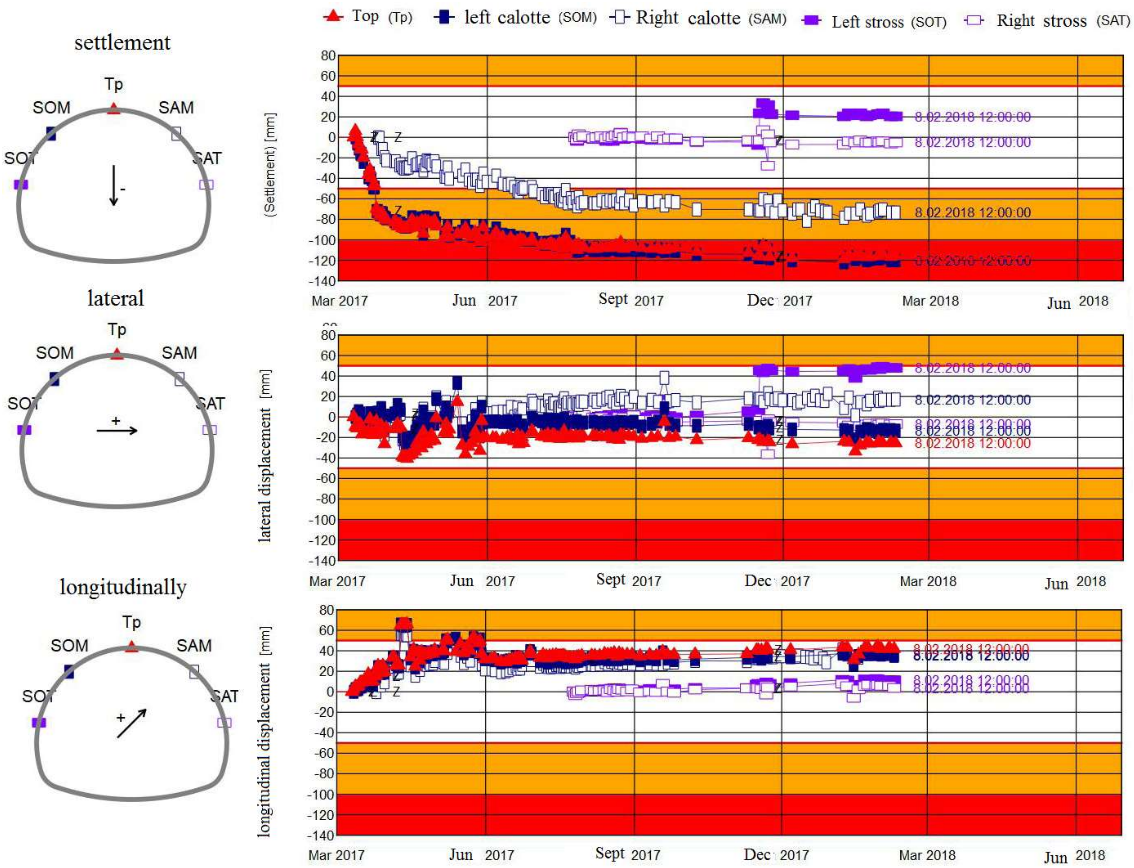


*Fig. 16– (e) Right tube Tunnel Cross section KP 26,616.000/ First measure: 15.05.2017, Last measure: 8.02.2018*

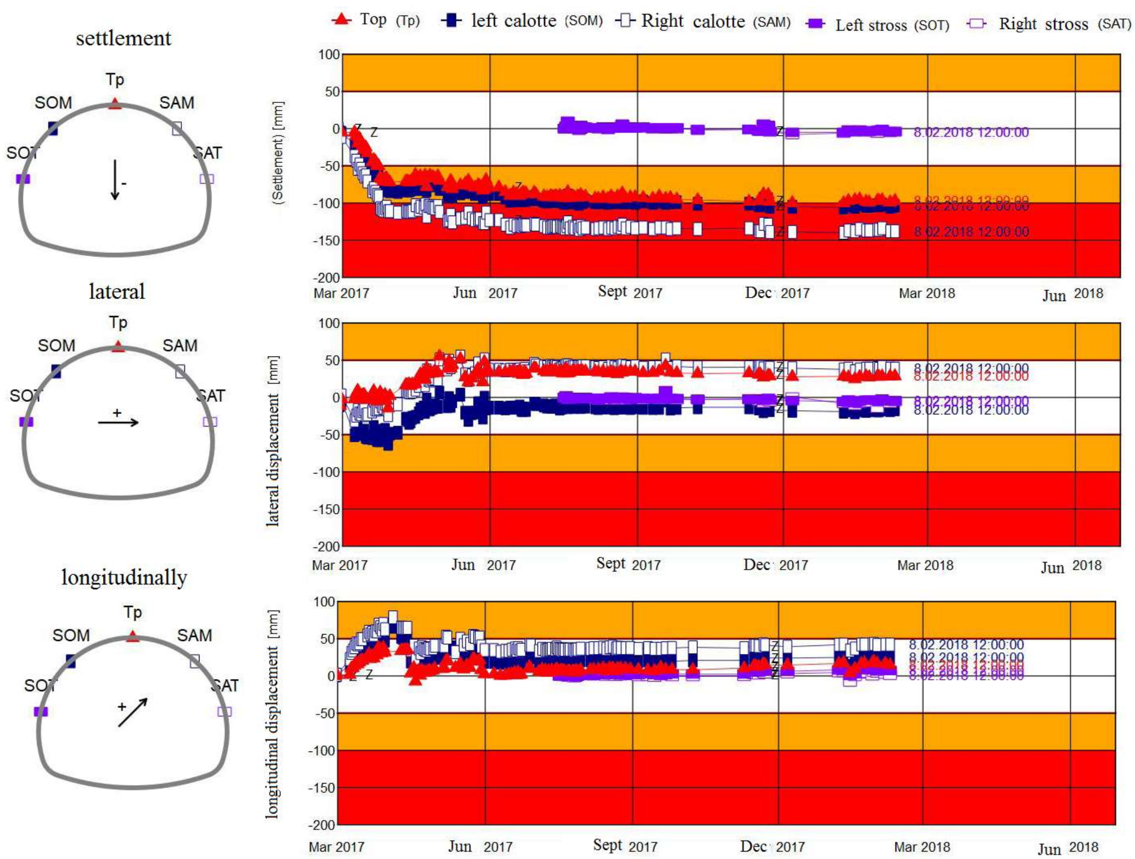


*Fig. 16– (f) Right tube Tunnel Cross section KP 26,626.000/ First measure: 15.05.2017, Last measure: 8.02.2018*

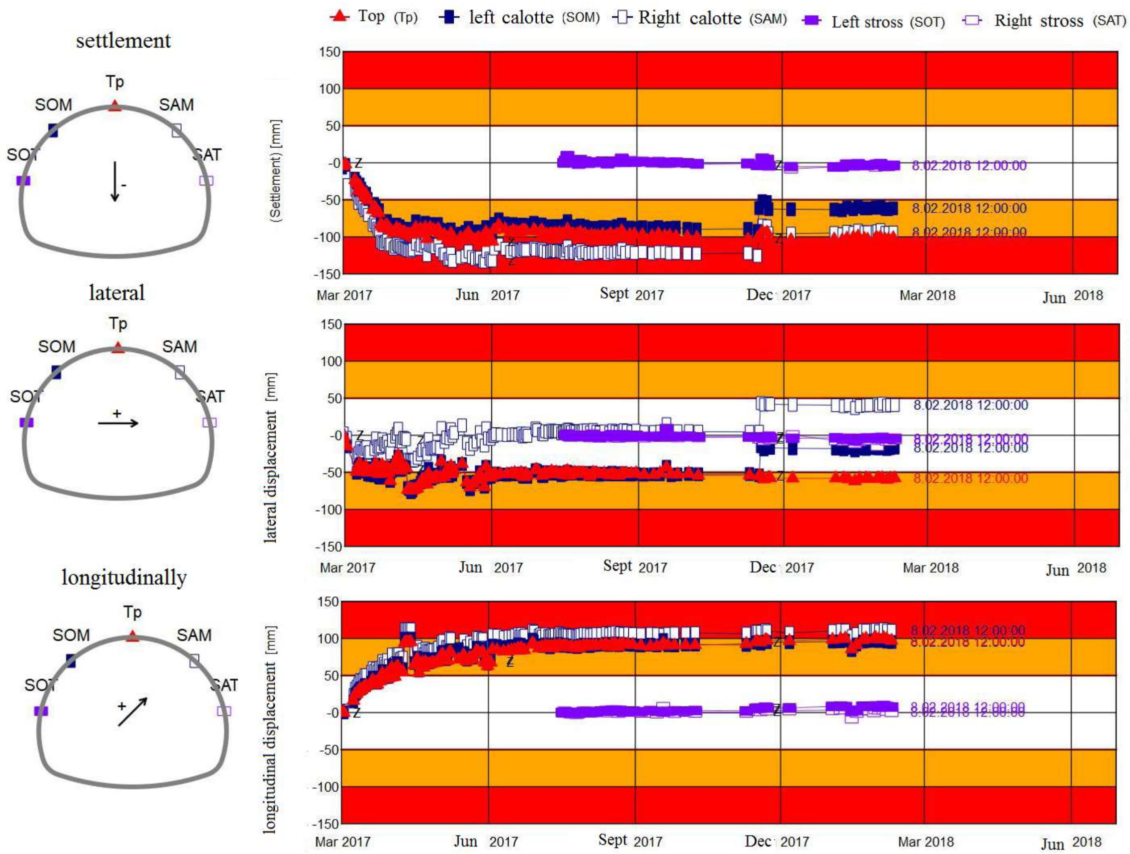


*Fig. 16– (g) Right tube Tunnel Cross section KP 26,635.000/ First measure: 15.05.2017, Last measure: 8.02.2018*

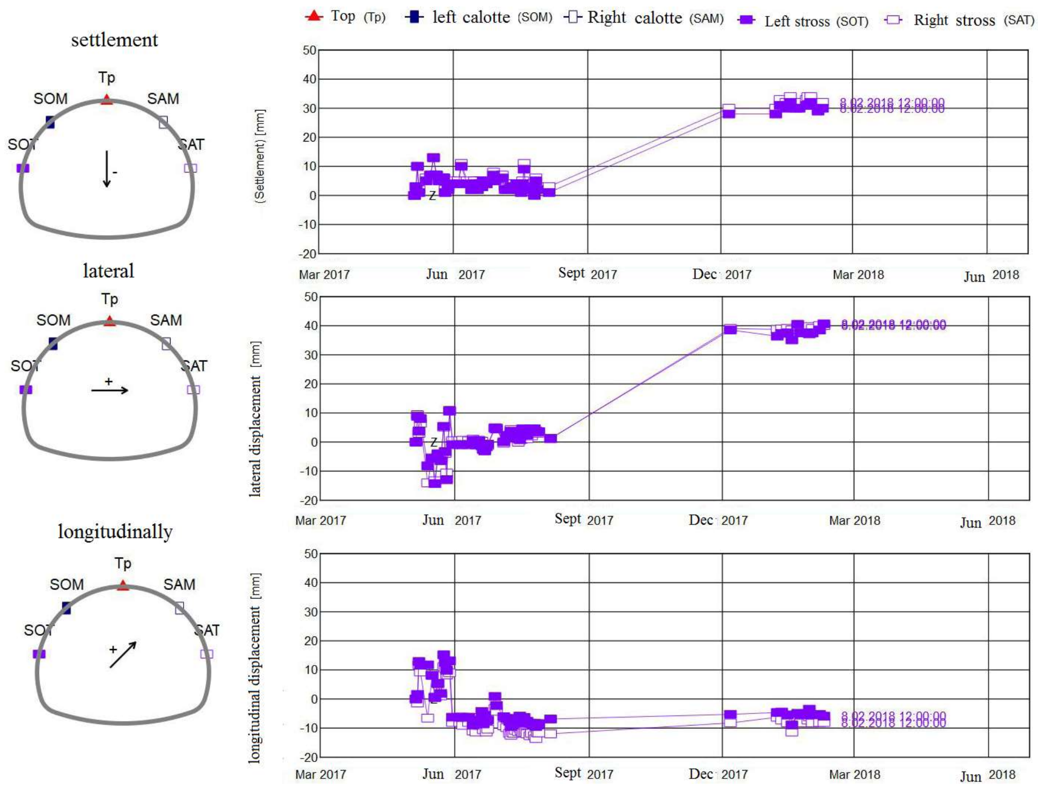


*Fig. 16– (h) Right tube Tunnel Cross section KP 26,641.000/ First measure: 15.05.2017, Last measure: 8.02.2018*

## 7. Conclusion:

The characterization of the rock mass and the site is very essential for tunnel design. Effective characterization provides reliable design input parameters for classification systems. The construction of any engineering structure in the rock mass causes the redistribution of stresses in situ which is not evaluated by empirical methods, it evaluates only the quality of the rock mass. Therefore, it is very necessary to evaluate/predict the quality of rock mass and in turn the "RMR- Q - systems" value with more precision. Moreover, the empirical methods do not analyze either the performance of the support systems, the distribution of the constraints around the opening and the deformation around the tunnel while it is used for the determination of the input parameters for numerical methods. for this purpose; The artificial intelligence used to deal with such nonlinear relations problems of engineering and it can also be used to confirm and improve the design solutions in any engineering projects. Numerical modeling in rock and civil engineering is used as a tool that facilitates the site engineers to evaluate the rock mass behavior and its effects on engineering structures and support systems. This Method resolved complex engineering problem utilising Plane Strain Two Dimension (2D) Analysis, Axisymmetric 2D Analysis and Three Dimension (3D) Analysis.

It is necessary to create a coating system (shotcrete, steel lattice, HEB and anchor bolts) with able to operate with the environment and able to provide bearing capacity immediately after excavation in order to meet the requirements, measure and evaluate continuously the deformations and surface movements inside the tunnel during excavation activities. On the other hand, deformations formed and possible structural damage must be measured and monitored. Measurements made are evaluated with the geomechanical conditions and necessary modifications after the geotechnical measurements required in the tunnel must be made and evaluated during construction to perform some revisions in the support systems (thickness of shotcrete, coating interval, bolt density etc.) and production parameters by the following recommendations:

- Benefit the most of the natural resistance of the rock mass, to this end insert support systems at the most opportune moment,
- Use flexible support systems that can accommodate rock deformations and support to ensure full contact between the support system and the excavation surface,
- Quickly avoid excessive relaxation of deformities using provisional support,
- Control excavation and support systems with permanent deformation footage, carry out a progressive excavation or move to other classes of support if necessary,
- Ensure the total functioning of the support system, particularly in low rocks,
- Provide flexibility for rock classes and support systems specified based on observations and measurements made during excavation.

**Acknowledgements:**

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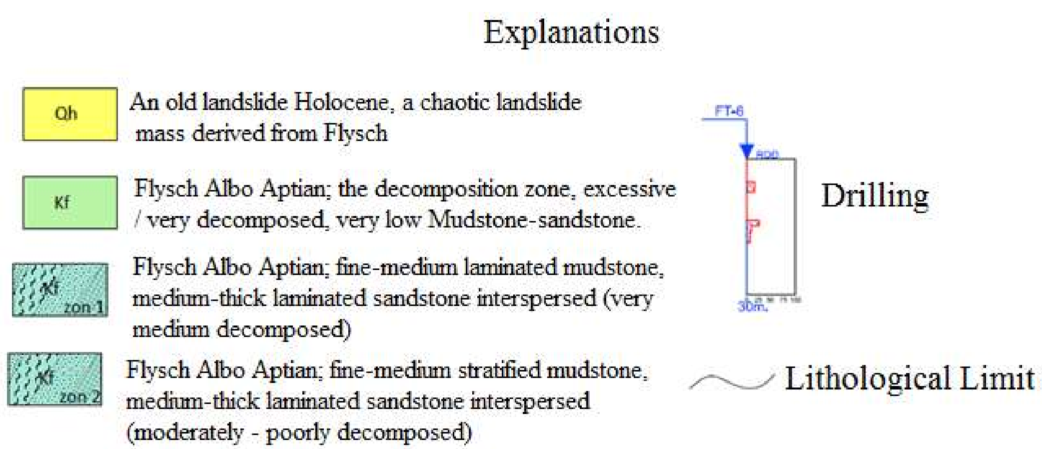
**Appendix A.**



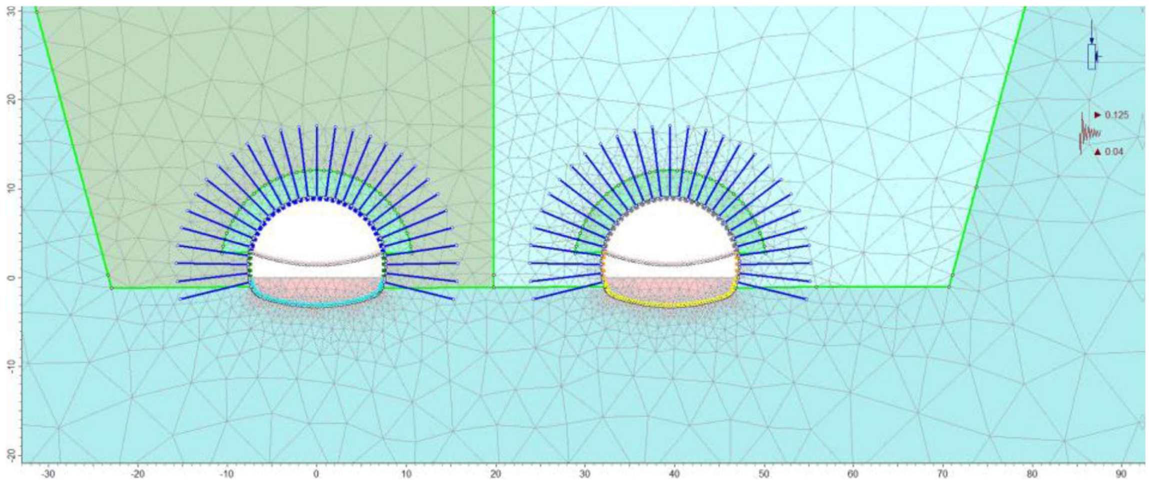


*Fig. 17– (a) The model created with the program Phase2 2D between KP: 26 + 230 and KP: 26 + 550 of the right tube and KP: 2 + 191.682 and KP: 2 + 490.970 of the left tube, (Middle Rock III)*

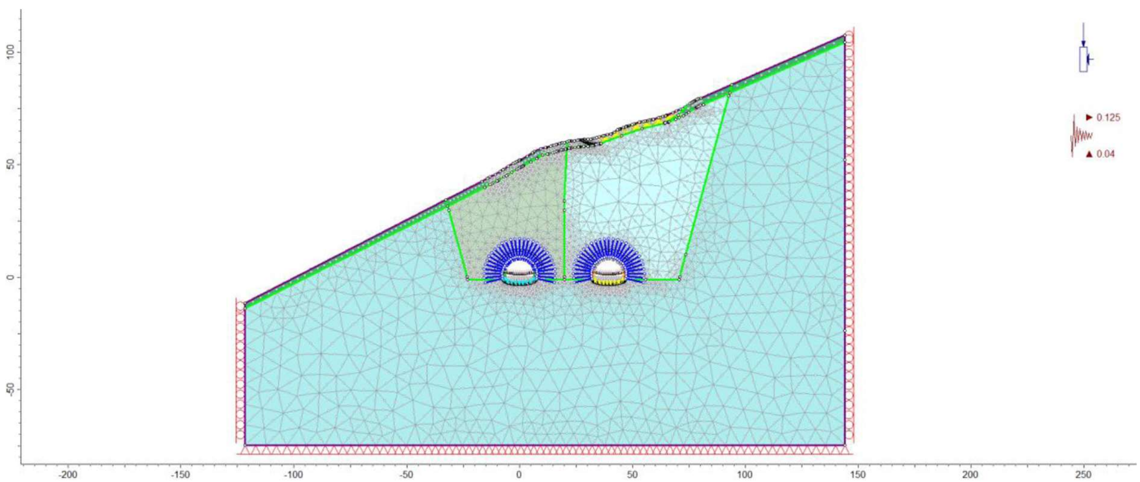


*Fig. 17– (b) The model created with the program Phase2 2D between KP: 26 + 550 and the output of the right tube & KP: 2 + 490.970 and the output of the left tube (Low Rock IV)*

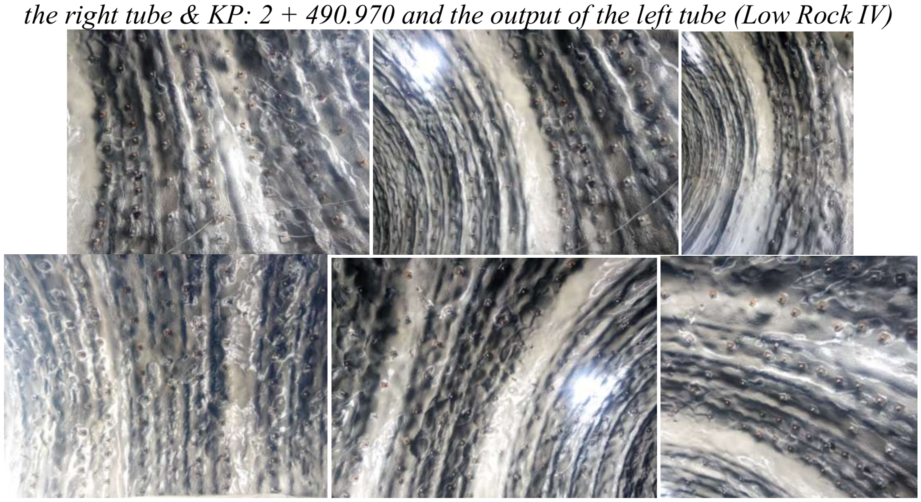


*Fig. 17– Photography's of reinforced zone in the low rock zone IV*

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