

FINITE ELEMENT METHOD FOR DESIGNING LARGE SECTION UNDERGROUND WORKS BY SEQUENTIAL EXCAVATION METHOD. STUDY CASE: LUGOJ-DEVA ROAD TUNNEL

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Abstract: Any underground work requires the knowledge and application of appropriate techniques and technologies in all stages of implementation of such a project. An important problem in the design of underground works is the knowledge of the characteristics and behaviour of the massif in which the work will be carried out. It depends on the choice of the excavation solution appropriate to the existing real conditions, which will influence the duration of the work and the costs associated with it. The objective of this paper was to analyse and compare the total deformations of the contour of the underground work, assuming that the work is performed by sequential excavation method (S.E.M.): excavation in the horizontal direction and in the vertical direction. The finite element numerical simulation method was used for the convergence analysis, which showed that the total displacements of the tunnel gallery wall are smaller for the horizontal sequential excavation (SEM) variant, both for the hypothesis of coefficient of pressure in state of rest having the value $K_0 = 0.6$, as well as for the hypothesis in which $K_0 = 2.27$.

Keywords: sequential excavation method (S.E.M.), finite element method, underground work, deformation, convergence-shrinkage, rock-support interaction, coefficient of pressure

1. Introduction

Underground works are a special category in the field of construction, their specificity being given primarily by the fact that they are performed in a natural environment, which is often very little known. Geological and hydrogeological conditions are the determining factors of the degree of difficulty and the cost of carrying out an underground work. At the same time, the underground works require paying special attention to the importance of the study of subsoil reconnaissance, due to the existence of very strong interactions between: geology and geotechnical characteristics of the site on the one hand and the conception and definition of the work to be constructed, respectively the choice and application of the appropriate method of execution, on the other hand. The geomechanical characterization of rock or soil massifs allows obtaining the information necessary to establish their behaviour during the execution and exploitation of the work, defining the execution methods, sizing the works, particular protection measures and / or special consolidation methods that should be considered during the realization of an underground work. An important factor that influences the stability of underground works is the cross section of the work. In the case of large section underground works, such as tunnels, there is a very high risk during construction by sub-excavation, especially in the case of soft or weak ground [1-8]. At the same time, due to the limitation or modification of the characteristics of the underground space, the stability can be negatively influenced due to the interaction between the twin tunnels.

The sequential excavation method (SEM) was first developed in Austria and it is a conceived method to be applied to the construction of tunnels in rock massifs. This method, known in particular as the New Austrian Tunnelling Method (NATM), is based on understanding the behaviour of the rock massif that reacts when its equilibrium is disturbed by excavating an underground work. The method of excavation in successive stages

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allows to make the most of the participation of the land that supports an important part of the geostatic load. Basically, the rock is decompressed during successive excavations, the confinement pressure in the final stage being reduced on average by $1/4$ or $1/3$ of the initial pressure. This important phenomenon underlies NATM. The method consists in the combined support of the tunnel with anchors and projected concrete, namely shotcrete, a support method which, according to all the findings made, has a high efficiency. This method uses a light support to take over the deformations of the rock massif. Through this methodology, instead of simply supporting the rock massif disturbed by the execution of an underground work, it allows the massif to support itself, in other words, the rock itself is the one that participates in the support [9 - 11]. If a controlled convergence is considered, the anchor - shotcrete system and possibly welded panels or metal fibres will cause a redistribution of stresses in the rock massif and therefore a stable equilibrium. The peculiarity of the New Austrian Tunnelling Method (NATM) is the use of a combined support, made of perforated anchors (active or passive) and quick-setting shotcrete, from the combination of the two resulting a light support of the massif, to take over the deformations: active anchors or prestressed anchors - solid or wired rods, their fixing being done by injections of cement milk or synthetic resins; passive anchors (punctuated or distributed); the shotcrete has a role of protection and formation of a thin wall that follows the geometry of the ground, is applied immediately after jowling the side walls and will usually be reinforced with fiberglass or metal fibres and wire mesh [9, 11-13].

For the design of the Niayesh urban road tunnel in a massif with poor characteristics (soft ground) and given the large section of the underground work, Sharifzadeh et al. (2013) considered the sequential excavation method, so that in the construction phase of the tunnel the central diaphragm (DC) method and the side wall displacement (SD) method have been proposed; it has also been shown that surface subsidence and tunnel convergence can be effectively controlled [9]. One of the important factors in carrying out a large section underground work by sequential excavation method is the temporary support [2, 6, 7, 11, 13-16]. The finite element numerical simulation method was used to determine the displacements and loads acting on the temporary support [15], based on the geological parameters of the future tunnel location.

The purpose of this paper is to analyse and compare the total deformations of the twin tunnels on the Lugoj-Deva road tunnel, in the hypothesis of performing the work by sequential excavation method (SEM) - excavation in the horizontal direction and vertical direction. The stability was analysed by the convergence-shrinkage method with the Rocscience software, RocSupport module. The behaviour simulation of the tunnel in three-dimensional space was performed using the Rocscience software, by the finite element method (FEM).

2. Engineering context

The Lugoj - Deva Lot 2 road tunnel, section E2 from km 52 + 880 to km 56 + 220, is part of the Pan-European Corridor IV, which crosses the Romania territory from Nădlac to Constanța. The road tunnel section is located in the western part of the country, in the Banat region and has a total length of 3,214 km. The two road tunnels with double gallery are provided between km 52 + 875 and km 53 + 215: The first tunnel with a length $L = 340.0$ m and km 53 + 640 - km 54 + 502; the second tunnel with a length $L = 744.20$ m, having a longitudinal profile development of 1,084.20 m. The Lot 2 route of the Lugoj-Deva road tunnel has an approximate length of 28,600 km and is oriented in the West-East direction (figure 1). The longitudinal profile of the route is presented in figure 2. In the transversal profile, the road tunnel presents geometric elements corresponding to a design speed of 120 km/h, being in accordance with the provisions of PD 162-2002.



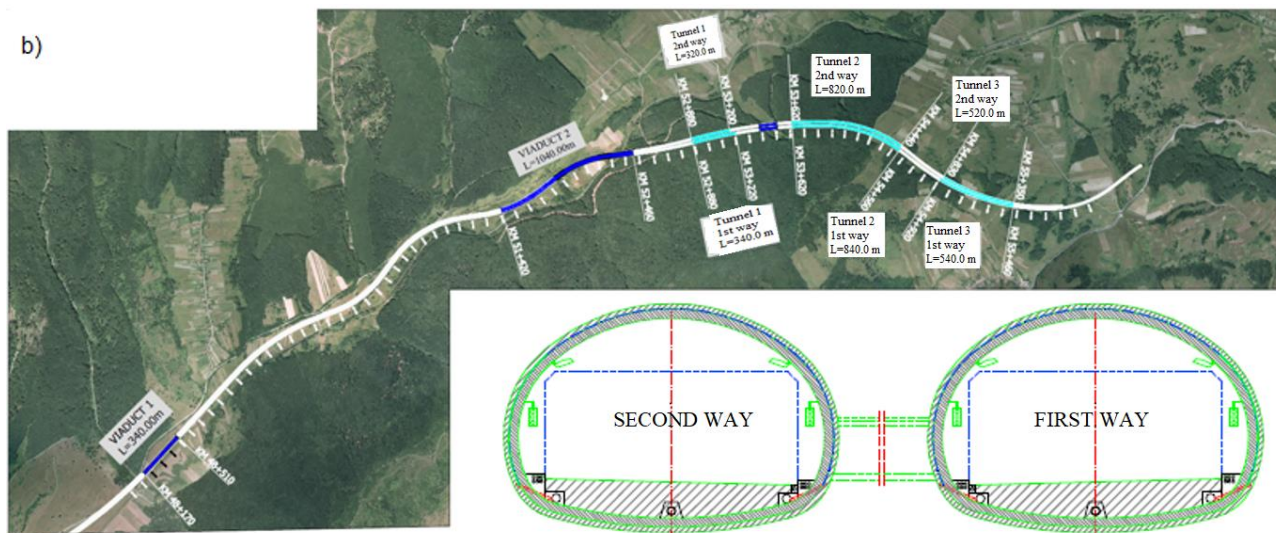


Figure 1. Location of the tunnel a) and its satellite image b)

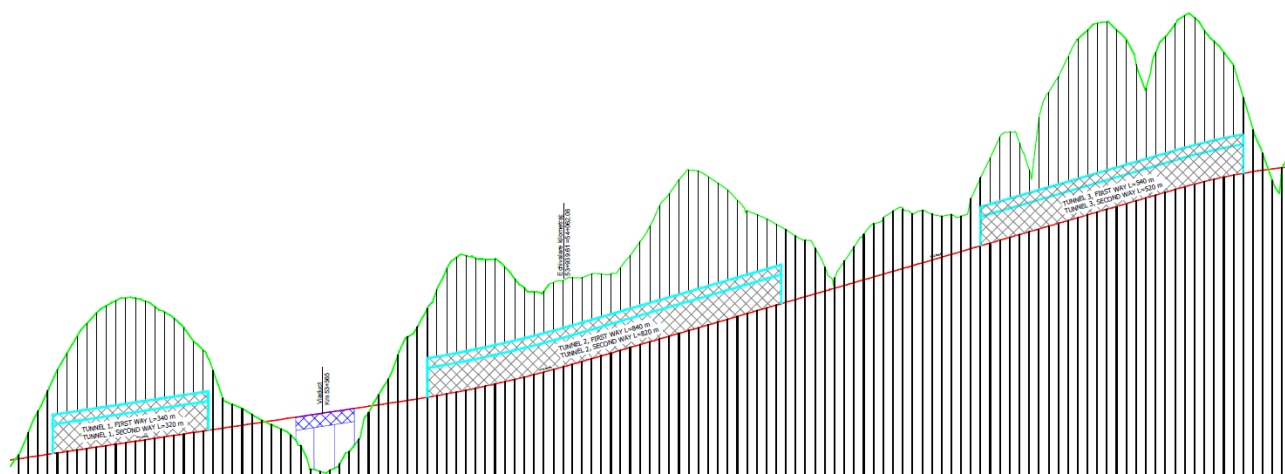


Figure 2. Longitudinal profile

According to PD 162-2002, for the types of tunnels on the road tunnel, the minimum width of the road between the edges will be 11.50 m and the gauge of free vertical passage of 5.00 m (figure 3); in terms of size, the road tunnel has a radius of $R = 6.90$ m, and in terms of the excavated gallery has a width of $D = 16.00$ m and a height of $H = 14.00$ m.

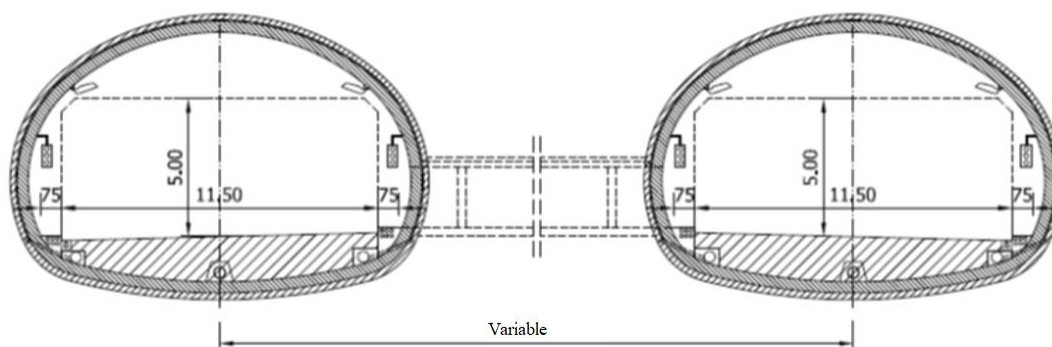


Figure 3. Typical cross section for road tunnels

From a geomorphological point of view, the studied region is represented by a hilly area, with altitudes between 200 and 400 m. Morphologically, this area is part of the general appearance of the hilly terrain and valleys. From a geological point of view, the region in which the researched location belongs to the Pannonian Depression, being framed by the Getic domain with a predominantly mountainous character in its southern

and central part, determined by the crystalline and eruptive formations of Retezat, Tarcu and Poiana Rusca mountains. The elevated areas are bordered in the west, northwest and east parts by intermountain basins with hilly relief, slightly accentuated, corresponding to tertiary sedimentary formations. Most of the geological formations of the Getic canvas belong to the Southern Carpathians. In the northern part are also included the southern endings of the Apuseni Mountains (Drocea and Metaliferi Mountains). The oldest formations belong to the Proterozoic and Paleozoic; they are widespread and are mainly represented by crystalline schists and granitoid rocks, which make up the mountainous areas of Retezat, Tarcu, Poiana Rusca and Semenik. These formations are distributed from the tectonic point of view to the autochthonous Danube and Getic, respectively, the two tectonic units of the area to which we refer. There are tectonic relations between the formations of the two domains, the Getic domain being decked over the Danube one. Epimetamorphic and granitoid crystalline formations, attributed to the Upper Proterozoic - Lower Paleozoic, poorly metamorphosed Paleozoic deposits and Mesozoic sedimentary formations, take part in the composition of the autochthonous Danube, which constitutes Retezat and part of the Tarcu mountains.

The Getic crystalline formations consist of metamorphic schists (prior to the Upper Proterozoic) that make up the southern half of the Poiana Rusca massif, the north-western part of the Tartar and Semenik mountains and epimetamorphic schists (attributed to the Upper Proterozoic and Lower Paleozoic) the northern half of the Poiana Rusca. The mesometamorphic crystalline schist of the Getic domain are well represented in the Poiana Rusca massif, in its southern part; they also appear in the NW part of the Tarcu Mountains and in the N part of the Semenik Mountains.

The Mesozoic sedimentary deposits belong to the Rusca Montană - Lunca Cernii basin (Barremian - Danian) and the Metaliferi mountains (Neocomian, Vraconian-Cenomanian and Turonian-Cognac sedimentary deposits as well as basic Mesozoic eruptive rocks belonging to the initial alpine magmatism). During the Neogene, the sedimentation basins of Lugoj, Caransebeș, Mureș, Strei-Hățeg were formed, by sinking the older formations along some fracture systems. The marine series of the Upper Miocene represents a special importance through the extremely rich fossiliferous deposits from Buituri, Coștei, Lăpugiu-Delnești.

The Badenian formations, transgressively arranged over various pre-Miocene terms, present a wide development on the slopes of the Mureș between the localities of Deva - Lăpugiu - Coștei, consisting of a wide range of lithology (breccias, conglomerates, gravels, sands, marls, clayey marls, coal debris, limestone, gypsum, and pyroclastite). The basal horizon consists of breccias, conglomerates, clayey marls sometimes reddish, over which a psephitic complex follows and often red or grey clays with green spots; the succession continues with the marly facies made up of grey marls, with intercalations of sandstones, sandy clayey marls, with a poor paleontological content. The lagoon facies has the widest extent and is represented by gravels, in which, at various levels, lenses of sandstones and conglomerate banks appear, then an alternation of micaceous grey marls, shale clays with radiolarians, coal clays, sometimes even intercalations of 5- 10 cm of coal, coarse tuff and gypsum.

The Pannonian deposits complete the succession of Neogene sedimentary formations. The two horizons (lower - consisting of sandy clay blue or greenish-gray clays with irregular intercalations of sands, sometimes coarse, with lenses of gravel and fragments of coal, and the upper - consisting of sands with gravels and rare clay horizons), lie discordantly over the Tortonian formations, or over the crystalline schists.

The Quaternary formations consist of: glacial deposits, proluvial deposits, deluvial-proluvial with reddish clays and alluvial deposits, belonging to the terraces and being attributed to the Pleistocene. The Lower Holocene is attributed to the fluvial deposits of the low terrace made of gravel and sand, and the Upper Holocene is attributed to the recent alluvium of meadows, made of sand and gravel.

Neozoic magmatism is represented by tufa sedimentary rocks, tuffs, pyroclastites, lava flows, vein bodies, nekuri and pillars, with varied petrographic composition mainly andesitic. The products of Neozoic volcanism are widespread in the Bulza - Lăpugiu - Sîrbi area where they are mainly represented by pyroclastites and subordinated by bodies and flows of andesitic lavas. These products cross or are arranged over the older antepannonian formations, and are transgressive covered by Pannonian deposits, the main phase of placement being placed in Sarmatian. The oldest Neogene eruptive rocks are represented by rhyolites and rhyolite tuffs, which occur at Pojoga and north of Tomești.

From a hydrogeological point of view, the researched area is part of the Pannonian and Quaternary sedimentary complex in the alluvial plain of the river Bega. Its foundation consists of alternating clays and marls with sand intercalations (Pannonian), being covered by Quaternary deposits represented by fine and coarse alluvium (gravel, sand, clay, dust). The water was intercepted at the contact between the deluvial and eluvial deposits with the basic formations made of clays and marls, at depths between 9.70 m and 12.90 m, the quasi-stabilized level being at depths between 7.60 m and 12.90 m.

3. Numerical simulation method

3.1. Pre-dimensioning of the tunnel support by the convergence- shrinkage analytical method

Digging a tunnel changes the stress and deformation state of the massif in which the underground work is performed [8, 10, 12, 14, 15, 17]. This new state of tension and deformation is manifested with a different intensity from the initial state, much higher and with the development of radial and tangential stresses, whose value depends on the size of the underground excavation, the depth at which it is executed and the physical – mechanical characteristics of rocks massif. However, the response of rock massif in which the underground work is carried out also depends on the method of excavation and support of the underground work. The excavation process takes place in time and space, and the redistribution of the initial stress state in the massif and the rock-support interaction are also phenomena that evolve as the excavation progresses.

The computation of the supports for the studied tunnel was performed by the Duncan Fama method based on the Mohr-Coulomb failure criterion. The geomechanical parameters of the rock are shown in Table 1.

Table 1. Geomechanical parameters of the rock considered in computation

Considered parameter	Value
Modulus of elasticity, E (MPa)	2,000
Uniaxial compressive strength, σ_{rc} (MPa)	1.2
Internal friction angle, φ (degrees)	15
Coefficient of Poisson, μ	0.37

The convergence-shrinkage method was performed with the Rocscience software, the RocSupport calculation module. The main support of the tunnel gallery consisted of the following (figures 4 - 5):

- EXX Swellex anchors located at a distance of 1.00 m x 1.00 m;
- 150 mm thickness of shotcrete with uniaxial compressive strength (UCS) at 28 days of 35 MPa.

The obtained results for fixing the support at a distance of 2.00 m behind the front of the tunnel gallery are presented graphically and as a value in section 4.

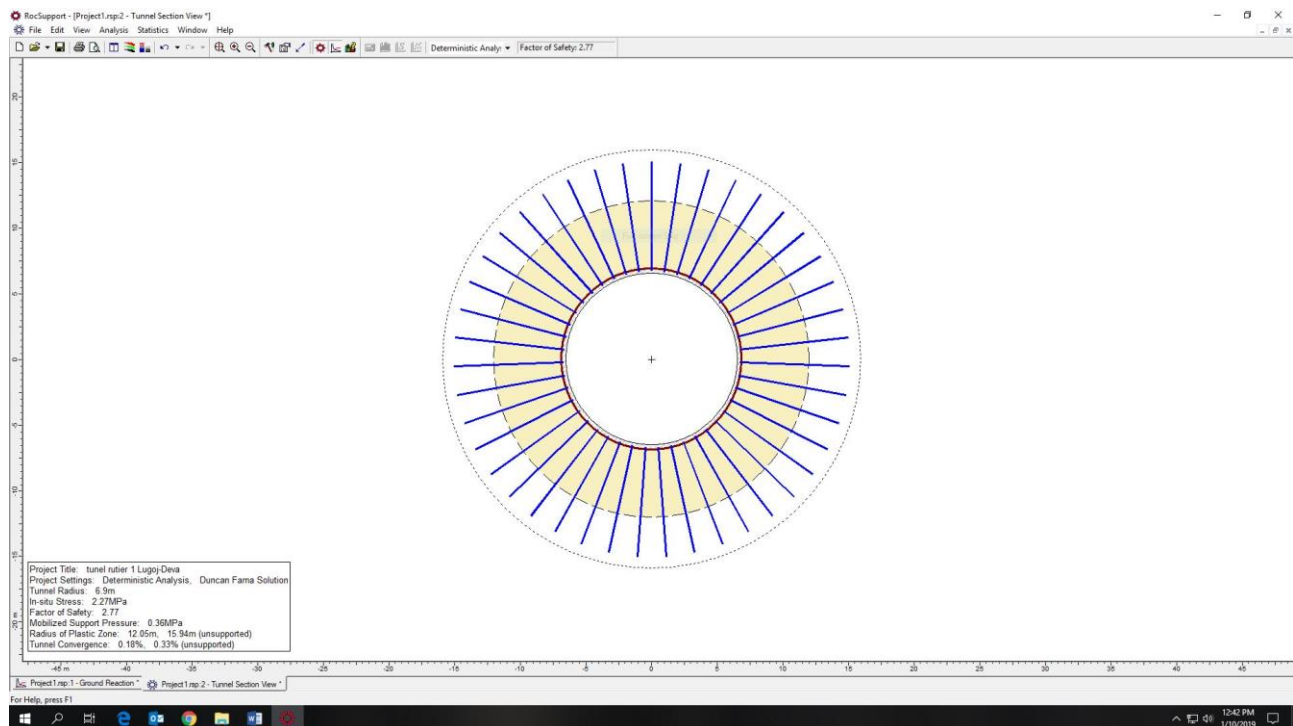


Figure 4. Tunnel section according to the convergence-shrinkage diagram, safety factor 2.77 and $K_0 = 2.27$

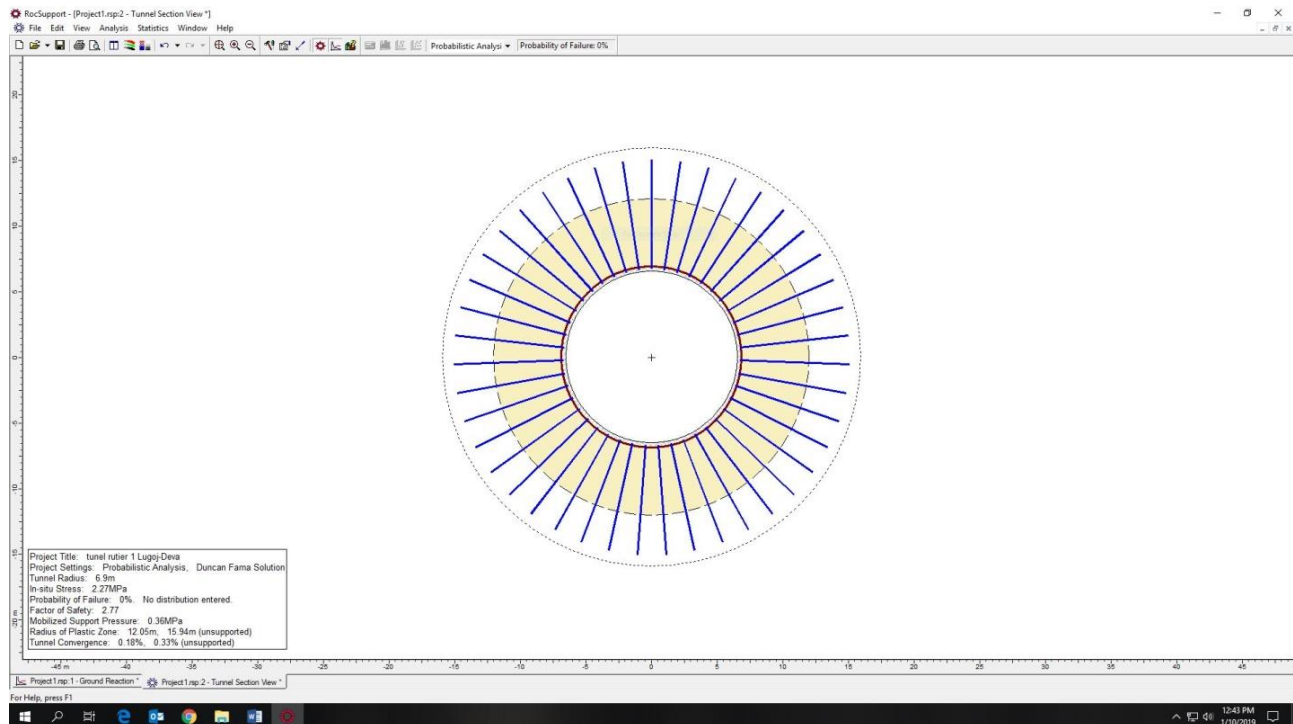


Figure 5. Tunnel section according to the convergence-shrinkage diagram, probability of failure 0%, safety factor 2.77 and $K_0 = 2.27$

3.2. Sequential excavation method

Establishing an optimal, safe, substantiated scientifically solution is one of the important issues in designing and carrying out underground work; this solution must also be economically appropriate. For large cross-sections of underground works, such as tunnels, the optimal solution for carrying out these works is to divide the area of the work into sections that are excavated separately, figure 6 [1].

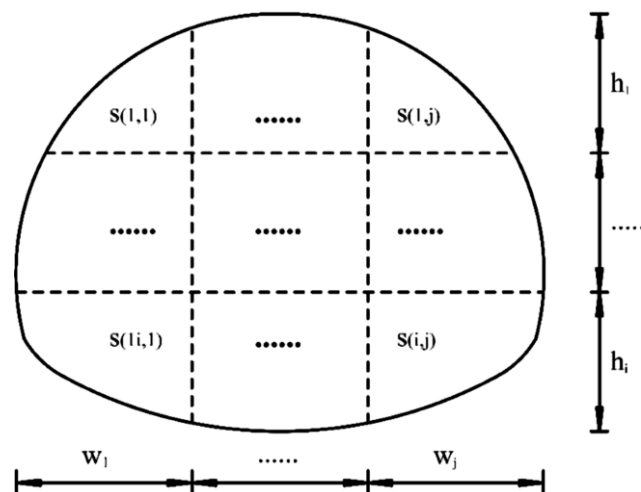


Figure 6. Sequential partition of planar excavation (according to Wu and Huang, 2020)

The sequential excavation method (S.E.M.) is a method that offers flexibility in the geometry and size of the tunnel cross section. In general, the cross section has an ovoid shape for a uniform redistribution of stress state of the massif around the new created gallery [1, 2, 6, 7, 13, 16]. By adjusting the construction stages, mainly the length of the excavation step, the type of support and the time period until the mounting of the support, the sequential excavation method allows the realization of tunnels in rock and soil massif. Depending on the size of the gallery section and the quality of the rocks or soil in the massif, the excavated section of the tunnel can be divided into several galleries [18], figure 7.

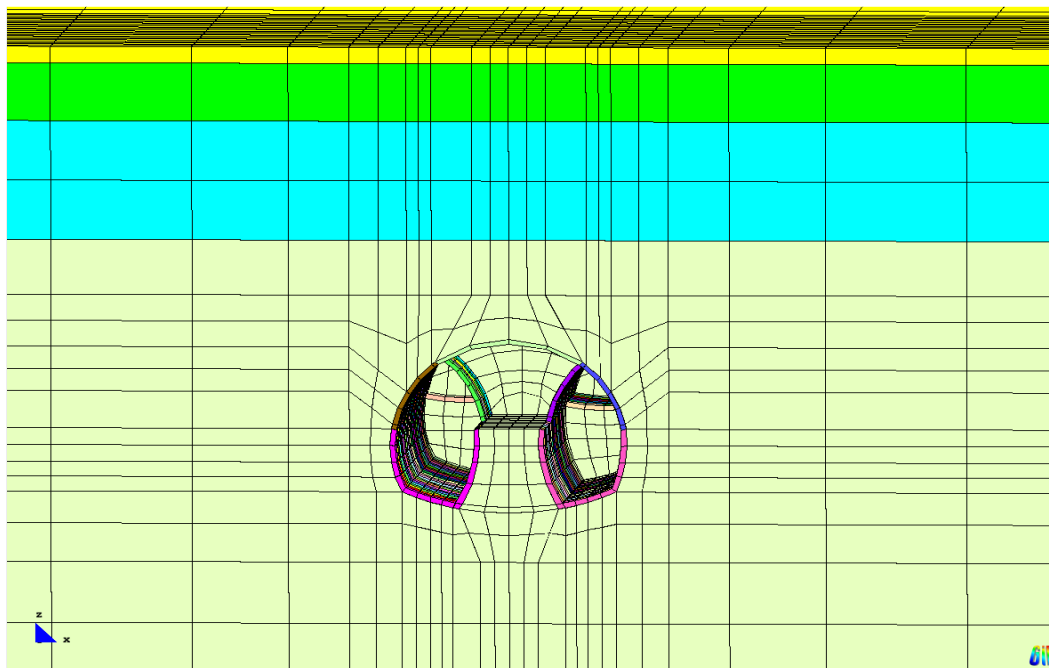


Figure 7. Sequential excavation in the vertical direction (Svoboda & Masin, 2010)

The sequential excavation method (S.E.M.) involves the following: classification of rocks or soils in the massif, the type of excavation and the supports based on field investigations; defining the excavation and the types of support (maximum length of unsupported excavation; methods of supporting the gallery (shotcrete and reinforcement with anchors and bolts); dividing the cross section of the tunnel into multiple areas as needed; umbrella-type supports in the portal area; if necessary, the operation of supplementing or, as the case may be, thickening the support can be performed locally); instrumentation and monitoring; measures to improve the rock or soil massif in front of the unit face.

The important element in the support is the shotcrete or the projected concrete. Guniting, as part of the initial support system, contributes significantly to the mobilization of rock around the underground work. This mobilization can be achieved by controlling the deformations that occur in rocks. Shotcrete is an active support, involving the rock in the process of taking over the pressure that manifests itself on the contour [19]. The shotcrete is capable of filling cracks and allows for continuous support of the tunnel gallery. The instrumentation elements of the sequential excavation method consist in monitoring the deformations of the tunnel and the area around it, allowing the evaluation of the design hypotheses and the adjustment of the tunnel realization process. The shape of the tunnel cross section must be designed in accordance with the principles of the sequential excavation method (SEM). Thus, as far as possible, the vault effect should be created around the excavation to self-support the excavated gallery. Thus, the profile of the tunnel will be curvilinear at both the vault and the hearth (if the tunnel is made in soil massif). Straight or broken lines for the tunnel walls in cross section shall be avoided. Therefore, the geometry of the excavation cross section will be able to take over and redistribute the stress state from the rock/soil massif around the tunnel gallery, minimizing the action of stresses loading on the tunnel supports. The shape of the inverted vault will depend on the geomechanical conditions of the massif in which the tunnel is made. In competent rock formations, the inverted vault will be flat (straight), while in rocks of medium strength (or altered) and in soils the inverted vault of the tunnel will be circular (curvilinear) to facilitate the closure of the annular section and ensure the stability.

3.3. Tunnel modelling by the finite element method

Modelling by numerical analysis of the tunnel was performed using Rocscience software. The type of analysis was the deformation in plan, by the finite element method (FEM). The simulation of the behaviour of the tunnel gallery in three-dimensional space (3D) was performed by establishing several excavation stages in the plan. Each excavation stage was assigned a decreasing value of the modulus of elasticity (Young), relative to the value of the modulus of elasticity in situ (Core Replacement Technique - Material Softening). The Mohr - Coulomb failure criterion was used for the design, for a plastic material. The mechanical characteristics of the massif in which the tunnel was designed are shown centrally in Table 2.

Table 2. The mechanical characteristics of the massif in which the tunnel was designed

Type material	Volumetric weight (MN/m ³)	Young modulus (MPa)	Poisson's coefficient	Internal angle of friction Φ' (degrees)	Cohesion, c' (MPa)
Overlying rock	0.027	2000	0.37	10	0.05
Base rock	0.027	2500	0.37	15	0.08

Due to the fact that the thickness of the overlying rocks located above the keystone of the tunnel vault is relatively small, and the sedimentary rocks have low geomechanical characteristics, it means that the natural equilibrium vault of the excavated gallery cannot be formed. In order to support the vault of the tunnel gallery and to redistribute the stresses from the massif, prior to the beginning of the excavation, on the contour of the vault of the tunnel gallery it was considered necessary to apply an "umbrella" type support (table 3).

Table 3. Characteristics of the "umbrella" type support

"Umbrella" type support	Diameter (mm)	Length (m)	Tensile strength (MPa)	Young modulus (MPa)
Anchors	19	17	0.1	200000

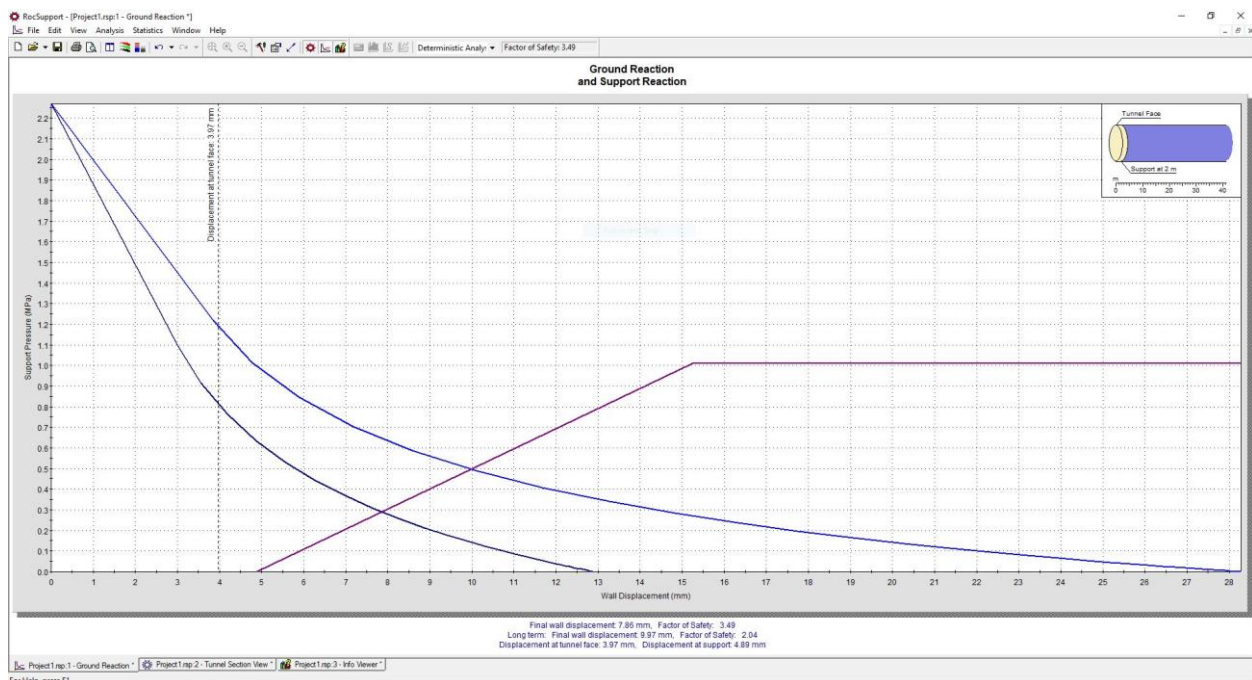
A reverse vault (reinforced concrete tunnel invert) with the characteristics presented in table 4 was designed in the floor of the tunnel gallery.

Table 4. Characteristics of the tunnel floor support

Reinforced concrete reverse vault (tunnel invert)	Compressive strength (MPa)	Thickness (m)	Poisson's coefficient	Young modulus (MPa)
Concrete	40	1.20	0.15	35000
Reinforcement	400	-	0.25	200000

4. Results and discussions

The analytical method for approximating the stability of the tunnel gallery was performed by the convergence-shrinkage method. The calculation of the elastic supports for the studied tunnel was made by the Duncan Fama method, which is based on the Mohr - Coulomb failure criterion. The main support of the tunnel gallery consisted of EXX Swellex anchors and shotcrete. The obtained results for fixing the support at a distance of 2.00 m behind the front of the tunnel gallery, in the short and long term, are presented in Figure 8 and Table 5.



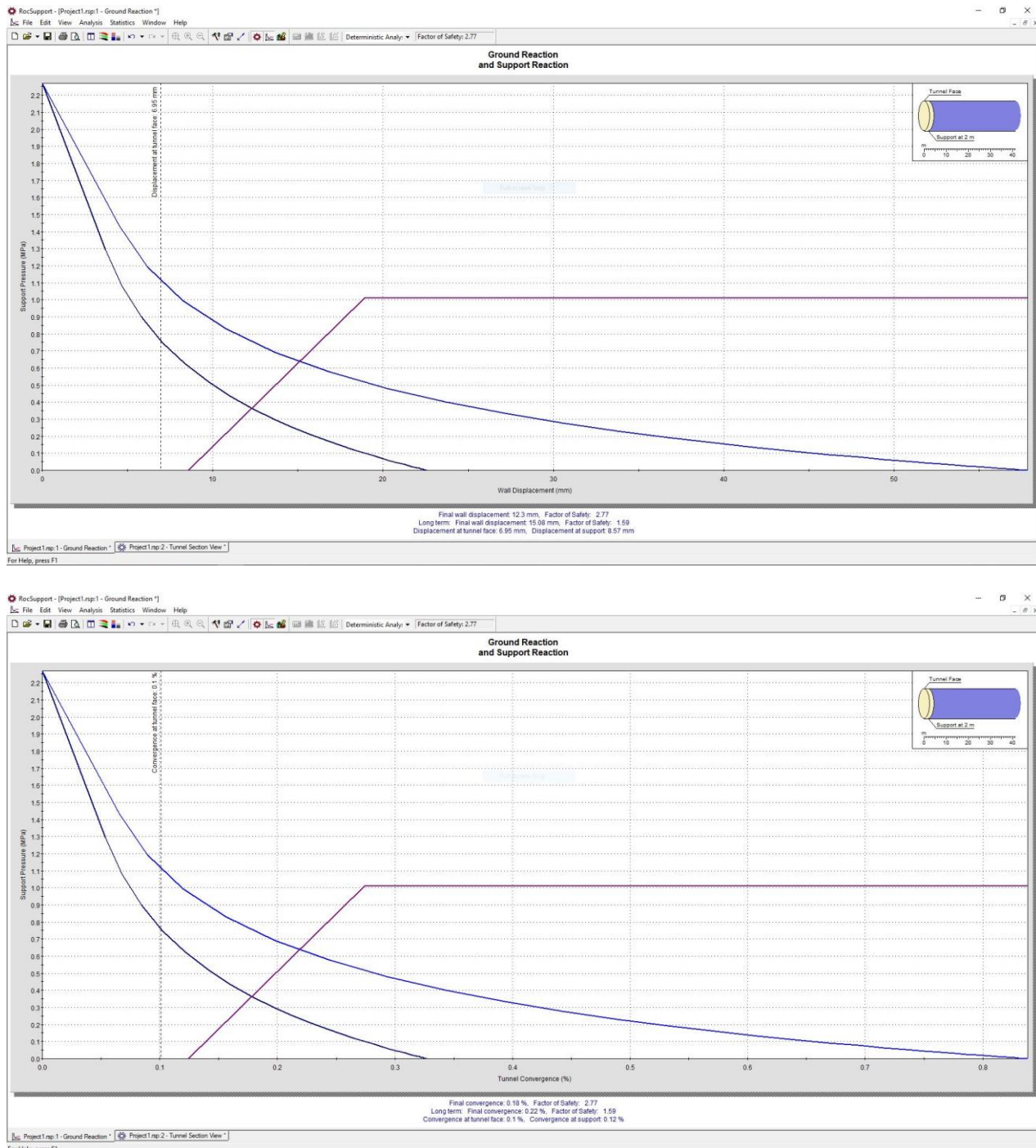


Figure 8. Diagrams of the rock massif - support system interaction

Table 5. Obtained results by convergence - shrinkage method of the short-term and long-term:

Determined parameter		Value
Short term		
Short-term stability factor (F.S.)		2.77
Shrinkage pressure mobilized in the short term, MPa		0.36
Radius of the plastic zone r_p , m	without support	15.94
	with support	12.05
Convergence of unsupported tunnel gallery, %	without support	0.33
	with support in the short term	0.18
Failure probability of tunnel wall, %		0
Final displacement of the gallery wall in the short term u_p , mm		12.30
Extrusion at the front of the tunnel, mm		6.95
Support displacement, mm		8.57

Long-term	
Long-term stability factor (F.S.)	1.59
Shrinkage pressure mobilized in the long term, MPa	0.65
Final displacement of the gallery wall in the long term u_p , mm	15.08
Convergence of tunnel with long-term support, %	0.22
Convergence at the tunnel front, %	0.1
Convergence to support,%	0.12

Tunnel modelling by numerical analysis, finite element method (F.E.M.) was performed in the hypothesis of the coefficient of pressure in state of rest $K_0 = 0.6$ and $K_0 = 2.27$. The results of the tunnel modelling by finite element method (FEM) are shown in Figures 9-16 and Table 6.

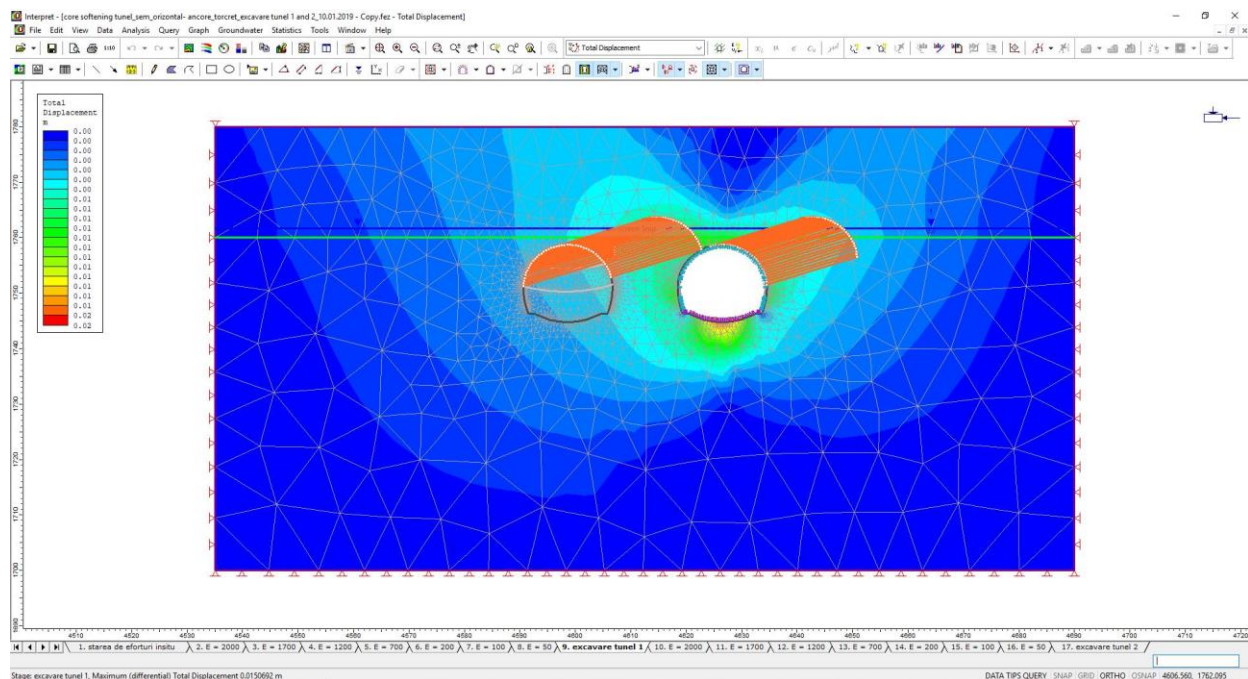


Figure 9. The total displacement $\Delta = 0.0150692$ m at the end of the excavation of gallery no. 1 in the horizontal direction, $K_0 = 2.27$

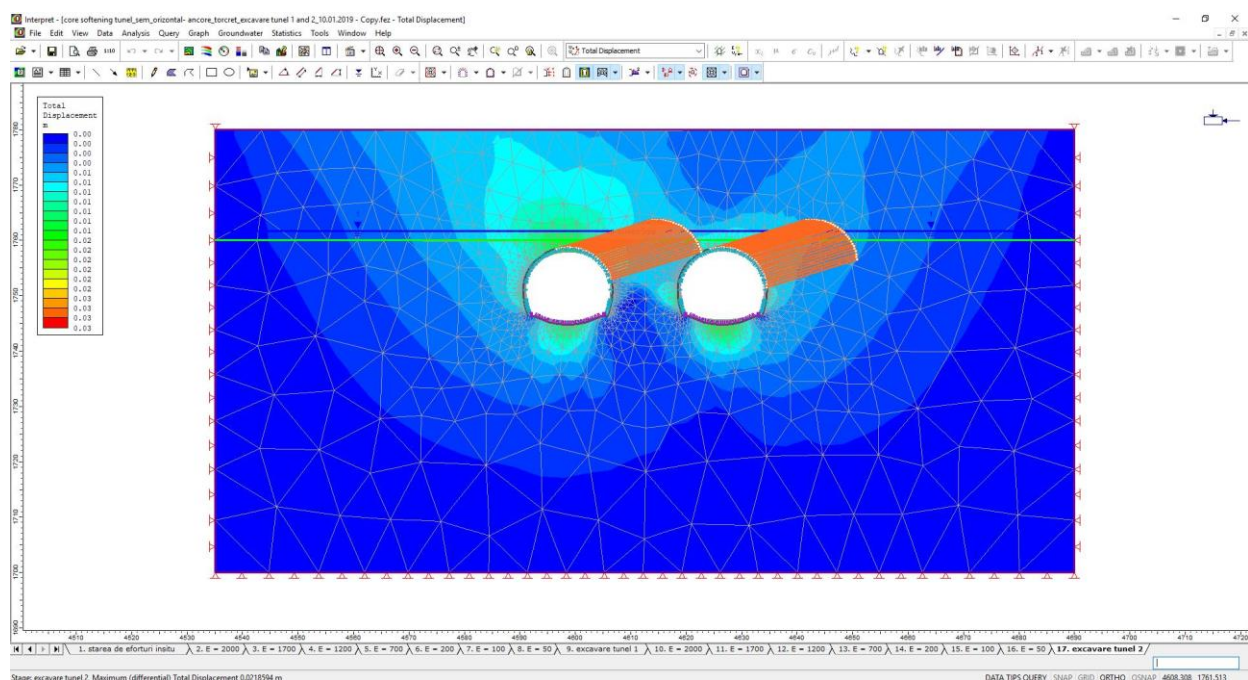


Figure 10. The total displacement $\Delta = 0.0218594$ m at the end of the excavation of gallery no. 2 in the horizontal direction, $K_0 = 2.27$

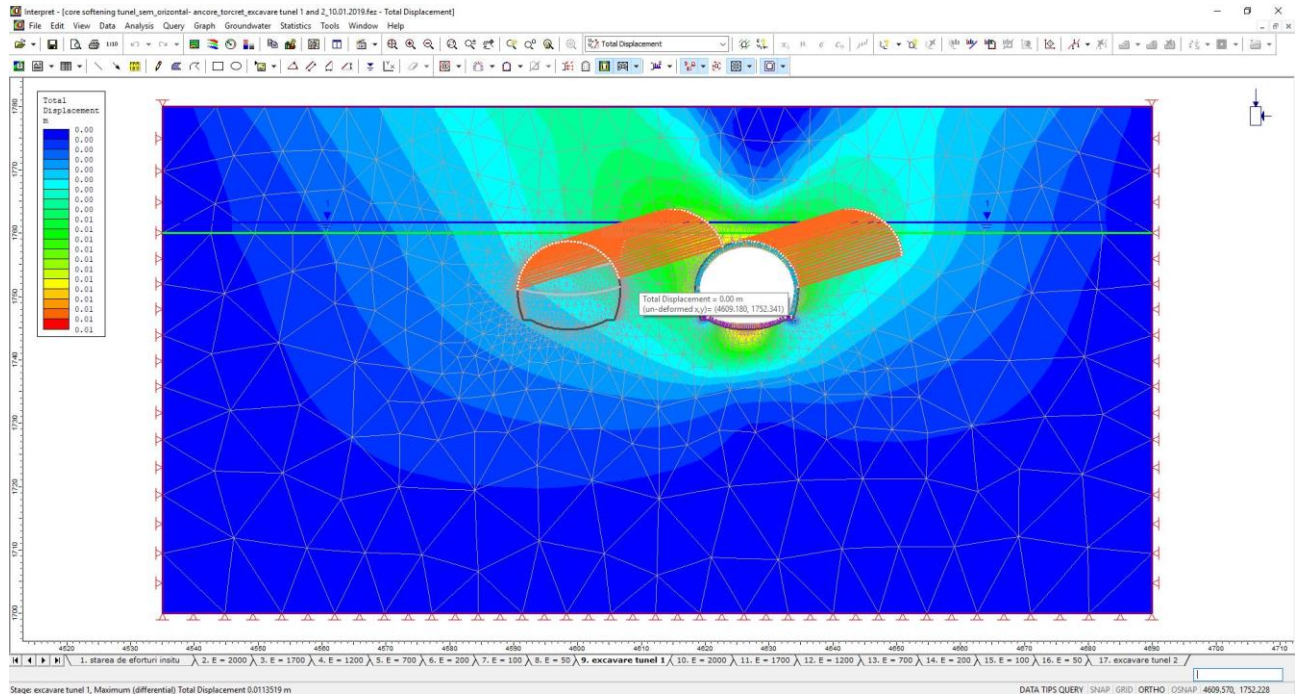


Figure 11. The total displacement $\Delta = 0.0113519$ m at the end of the excavation of gallery no. 1 in the horizontal direction, $K_0 = 0.6$

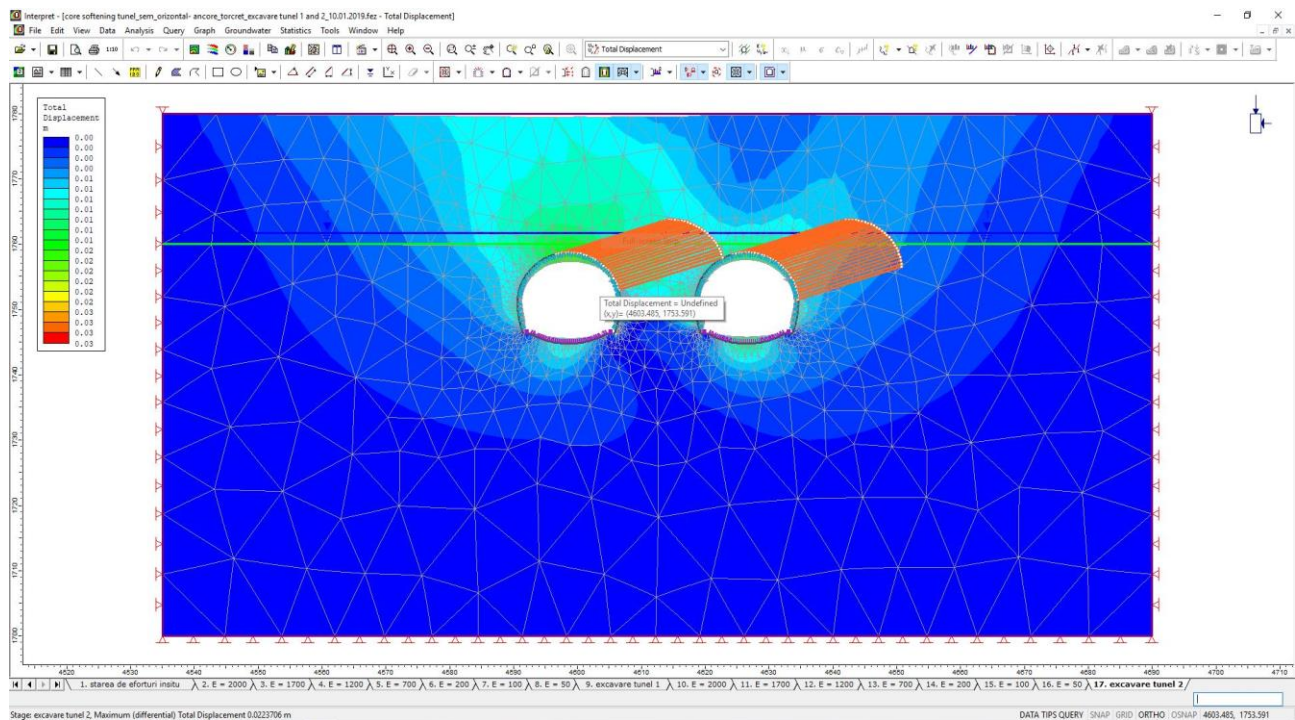


Figure 12. The total displacement $\Delta = 0.0223706$ m at the end of the excavation of gallery no. 2 in the horizontal direction, $K_0 = 0.6$

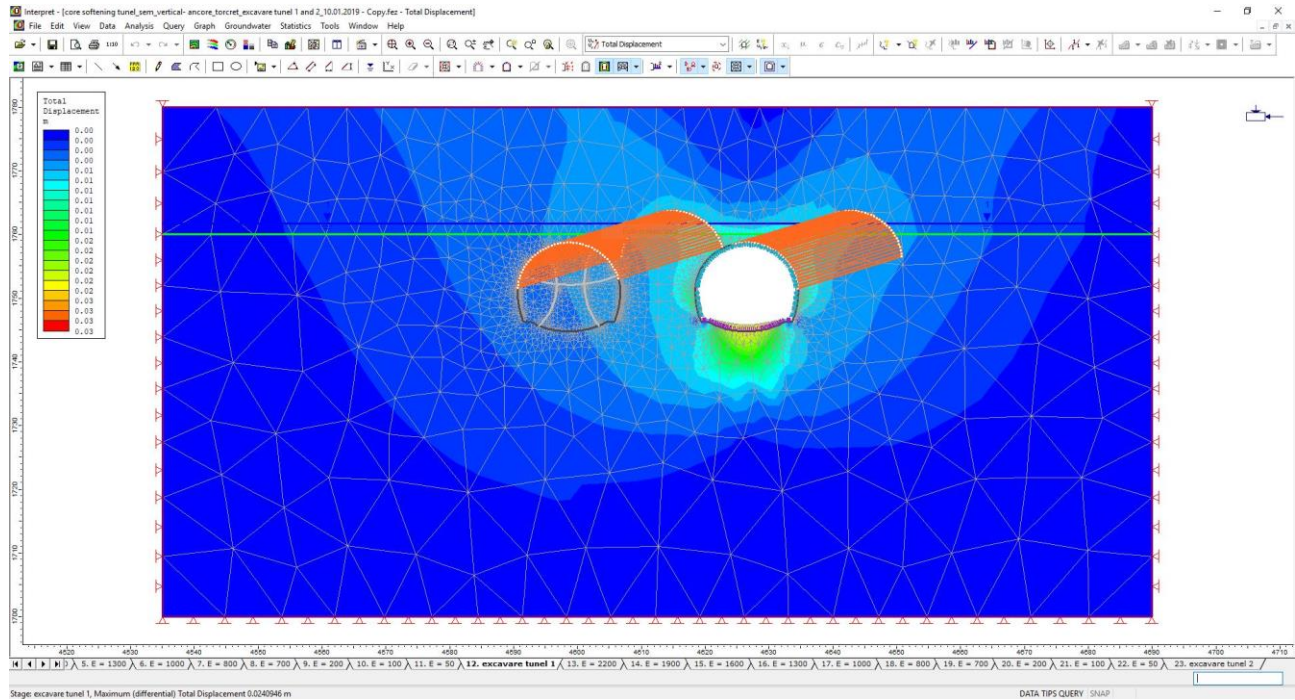


Figure 13. The total displacement $\Delta = 0.0240946$ m at the end of the excavation of gallery no. 1 in the vertical direction, $K_0 = 2.27$

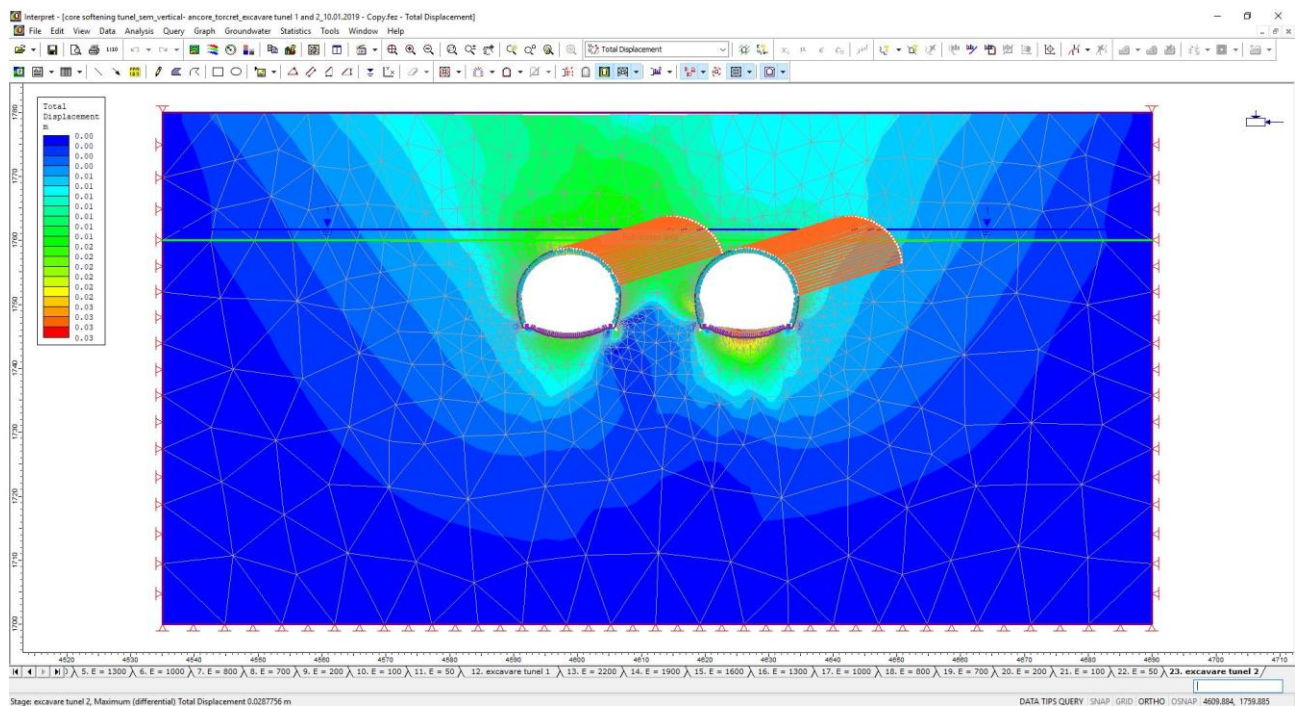


Figure 14. The total displacement $\Delta = 0.0287756$ m at the end of the excavation of gallery no. 2 in the vertical direction, $K_0 = 2.27$

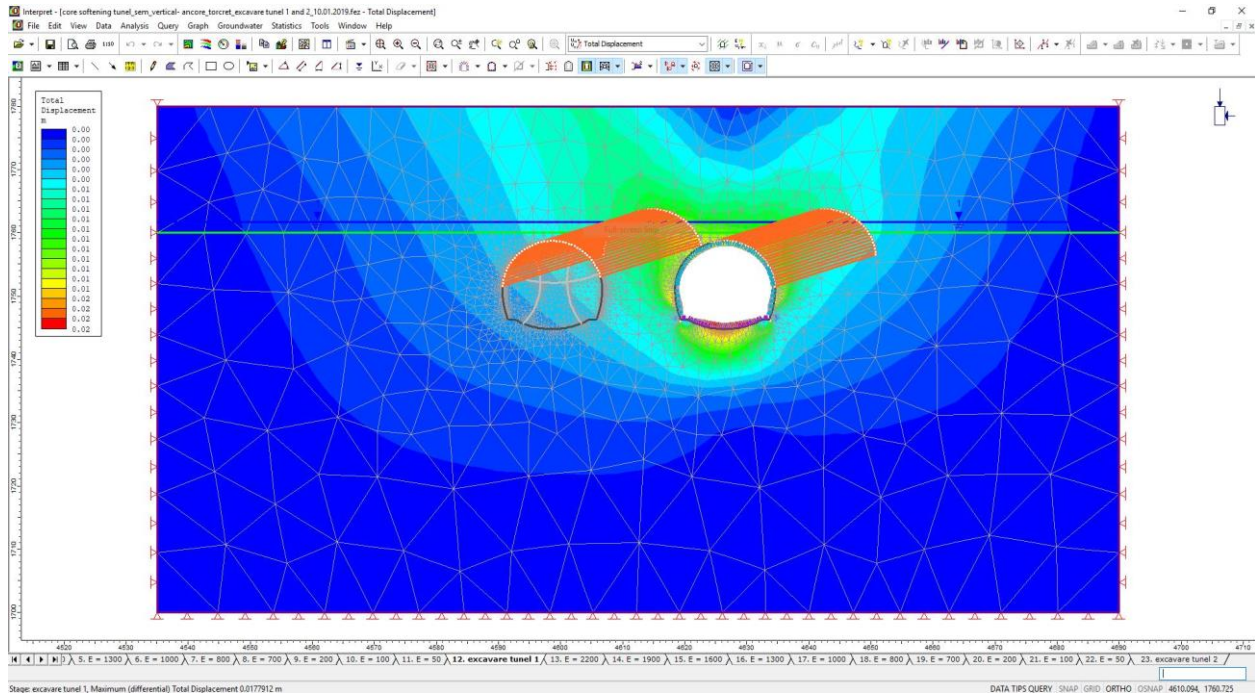


Figure 15. The total displacement $\Delta = 0.0177912$ m at the end of the excavation of gallery no. 1 in the vertical direction, $K_0 = 0.6$

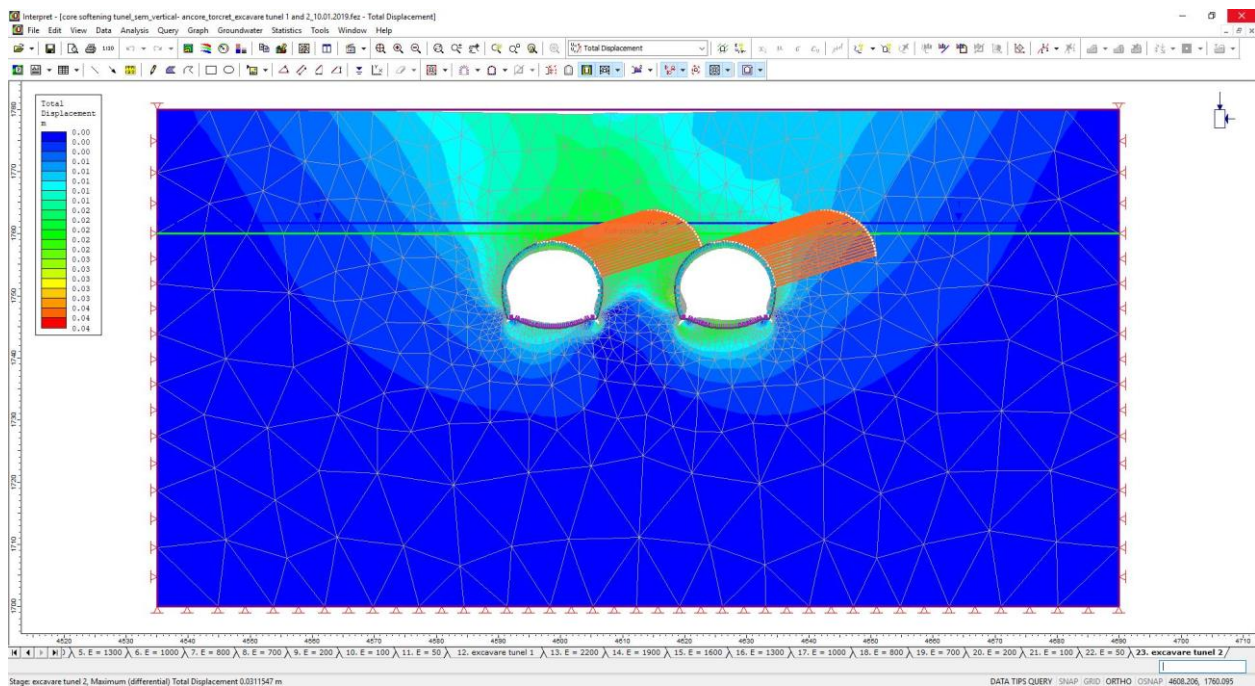


Figure 16. The total displacement $\Delta = 0.0311547$ m at the end of the excavation of gallery no. 2 in the vertical direction, $K_0 = 0.6$

Table 6. Total displacement of the tunnel gallery wall

S.E.M. in horizontal direction	Coefficient of pressure in state of rest	
	$K_0 = 0.6$	$K_0 = 2.27$
Gallery no. 1 – total displacement, Δ (m)	0.0113519	0.0150692
Gallery no. 1 – total displacement, Δ (m)	0.0223706	0.0218594
S.E.M. in vertical direction	Coefficient of pressure in state of rest	
	$K_0 = 0.6$	$K_0 = 2.27$
Gallery no. 1 – total displacement, Δ (m)	0.0240946	0.0177912
Gallery no. 1 – total displacement, Δ (m)	0.0287756	0.0311547

By analysing the obtained values, it is found that the total displacements around the tunnel gallery are smaller for the S.E.M. horizontal direction, both for the hypothesis of the coefficient of pressure in state of rest having the value $K_0 = 0.6$, and for the hypothesis in which $K_0 = 2.27$.

5. Conclusions

In order to limit the very large deformations in the rock mass around the tunnel, it was considered that the best solution under the given conditions is to use the sequential excavation method. The instrumentation elements of the sequential excavation method consist in monitoring the deformations of the tunnel and the area around it, allowing the evaluation of the design hypotheses and the adjustment of the execution process of the tunnel.

The shape of the tunnel cross section must be designed in accordance with the principles of the sequential excavation method (SEM). Thus, the aim is to create an arch or vault effect around the excavation to support the excavated gallery. Thus, the cross section of the tunnel will be curvilinear at both the vault and the floor (if the tunnel is executed in soils). Straight or broken lines for the tunnel walls in cross section shall be avoided. In this way, the geometry of the excavation cross section will be able to take over and redistribute the stress state from the rock massif around the tunnel gallery, minimizing the action of loading the efforts on the supports of the tunnel. The shape of the inverted vault will depend on the geomechanical conditions of the rock massif in which the tunnel is made. In competent rock formations, the inverted vault (tunnel floor) will have a flat (straight) shape, while in rocks with medium strength (or weathered) rocks and in soils the inverted vault of the tunnel will be curvilinear to facilitate the closure of the annular section and ensure stability.

It is proposed that the main support of the tunnel gallery to be done with 5.00 m long anchors, with the distance between the anchors of 1.00 - 1.20 m, reinforced concrete support with a thickness of over 15.00 cm and light metal support at a distance of 1.50 m. The main support of the tunnel gallery will be fixed at approximately 0.50 - 1.00 m behind the face of the excavated gallery.

The key support element is the support in shotcrete or projected concrete, because it is able to fill the free spaces and cracks on the contour of the work; this type of support is a continuous support of the tunnel gallery. The peculiarity of the support made of shotcrete or sprayed concrete consists in the participation of the rocks themselves to achieve the load-bearing capacity. Thus, the rocks, from the object of support, become themselves a mean of support, practically the rocks are self-supporting. In the new formed shotcrete - rock system, the last one has a decisive role in supporting the underground works.

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