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A POSSIBLE TUNNELS EXECUTION TECHNOLOGY ON SUBSECTION E2 OF THE HIGHWAY A1

Ileana PASCU[1], *Mircea GEORGESCU*[2*]

[1] University of Petroșani, Petroșani, Romania
[2] University of Petroșani, Petroșani, Romania

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Abstract: *The paper aims to establish the design elements of the tunnels to be executed on subsection E2 of the Lugoj-Deva highway section, an integral part of the A1 highway. From the multitude of problems related to the execution of a tunnel, the paper investigates the following aspects: geotechnical investigations of the areas to be crossed by tunnels, analysis of tunnel stability, static checks of the final support/lining and, finally, a possible technology for their execution.*
Keywords: *highway, tunnels, geotechnical investigations, stability, execution technology*

1. Introduction

In the last two to three decades, road traffic has increased, which has required measures to create the conditions for its normal and safe operation. Among these measures, increasing the traffic capacity of road infrastructure to meet new traffic performance is a sine qua non.

It has proven that the most effective solution to meet this goal is the creation of highways which means: safety, increased speed, comfort, time saving, stress reduction, economic growth.

In Romania there is insufficient development of highways because this lack is increasingly felt.

The A1 highway is part of the Pan-European Corridor IV, which crosses the Romanian territory from Nădlac to Constanța (figure 1).

Figure 1. TEN-T Project Map no. 7. The Romanian Nădlac-Constanța segment (A1 Motorway)
(Source: European Commission, 2005)

* Corresponding author: Mircea Georgescu, Prof. PhD. Eng., University of Petroșani, Petroșani, Romania, contact details (University st. no. 20, Petroșani, Romania mirgeorgescu@gmail.com)

The subject of this article is subsection E2 of the A1 highway, an integral part of the Lugoj-Deva highway section, (figure 2) between km 52 + 880 and km 55 + 420, which crosses an irregular hilly area.

Figure 2. Route of the Lugoj-Deva highway section, Lot 2 [1]

2. The solution proposed by the designer

In the *Technical Report* prepared for Subsection E2 - tunnels [2] in order to avoid the realization of large cutting, with negative environmental impact, it was proposed to cross the most critical areas with 3 tunnels, the first having a length of 290 m, and the next two 834 m and 400 m respectively.

The initial solution, with three such tunnels, was modified (according to the conditions of the Environmental Agreement, of 2013) and it was decided to join the last two, eliminating the discovered portion of 440 m between them, finally achieving a single tunnel, from km 53 + 620 to km 55 + 420 (figure 3). The length of the two tunnels will be approximately 2.13 km.

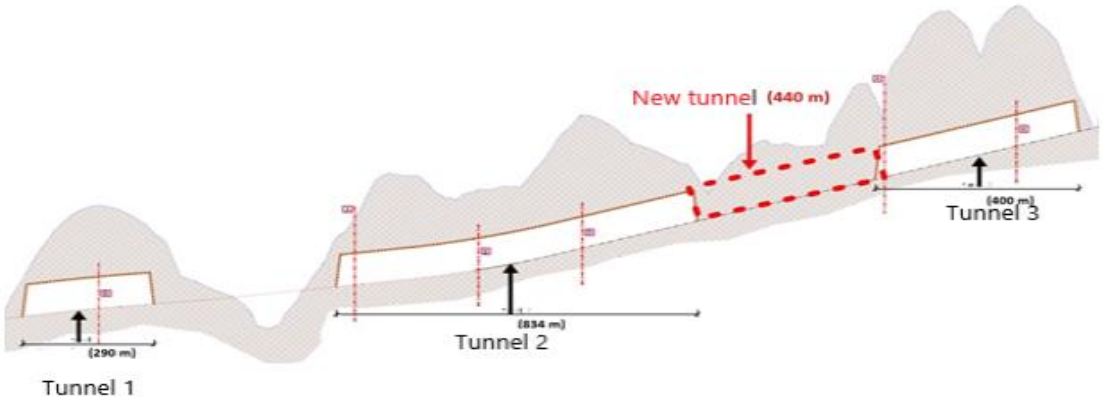


Figure 3. The tunnels on Subsection E2 [3]

The tunnels will consist of two unidirectional galleries provided on each traffic lane with a section of 185m^2 in excavation, and the road structure (figure 4) has the following dimensions:

- Roadway with a width of 2 · 3.75 m = 7.50 m wide;
- Framing strip with a width of 0.50 m
- Tape for emergency parking with a width of 2.50 m;
- Approaches of 0.25 m on the left side and 0.50 m on the right side = 0.75 m;
- Footways with a width of 2 · (min. 0.80 ÷ max. 0.98) m = 1.60 m ÷ 1.96 m resulting in a total highway width of 12.85 m ÷ 13.21 m, for each tunnel.

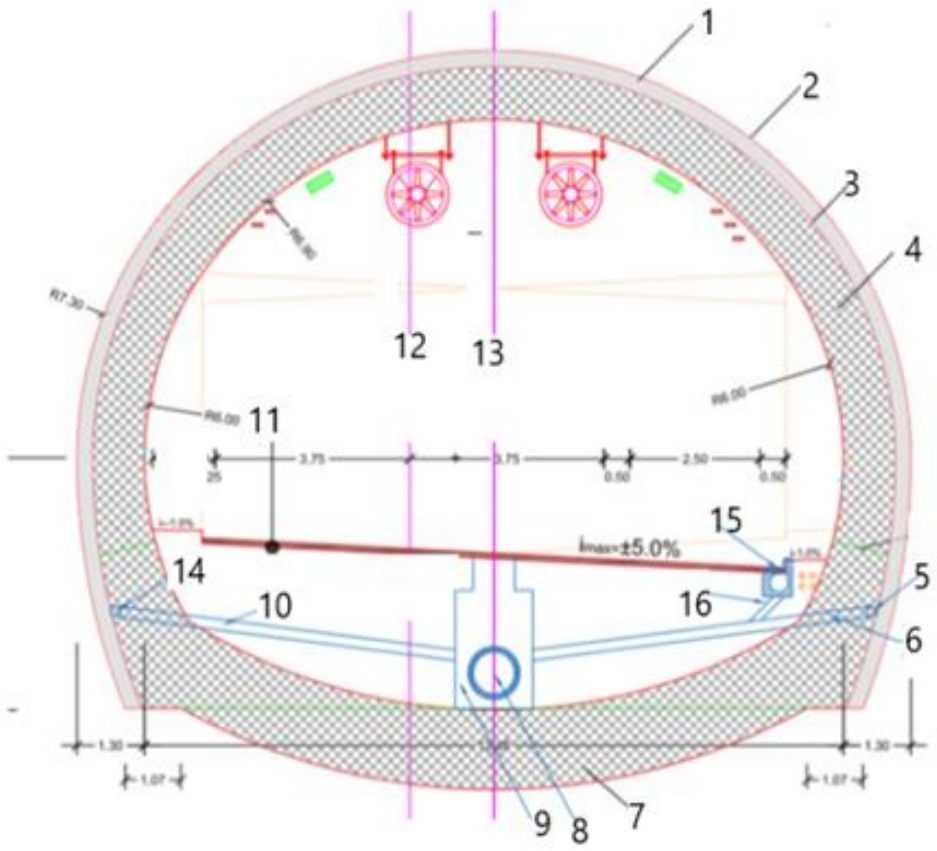


Figure 4. Standard cross profile for tunnel [3]
1-outer lining; 2-excavation limit; 3-intermediate waterproofing; 4-inner lining; 5-drain Ø200mm; 6-tube Ø200mm drain discharge; 7-raft/inverted vault; 8-tube; 9-gutter; 10- tube Ø200mm drain discharge; 11-roadway; 12-road ax; 13-tunnel ax; 14- drain Ø200mm; 15-gutter; 16- tube Ø200mm gutter discharge

The inner perimeter of each tunnel consists of a polycentric arch, which ensures a minimum gauge of 5 m on the asphalt platform. For the long profile, the red line of the platform has different uniform declivity for each tunnel (1-2-2.9%), slopes that follow the upper line of the tunnel cap.

The tunnels will also be provided with emergency evacuation galleries, service niches for the placement of security equipment, electrical installations, ventilation, fire extinguishing and ITS equipment.

3. Field investigations

A series of field investigations were required to obtain the technical documentation.

3.1. Cartographic and topographic investigations

A planimetric and altimetric topographic network was created resulting in the thickening of the national support network in the Stereo 70 reference coordinate system and basic topographic plans were executed at the scale of 1:1000 and detailed at the scale of 1:200.

3.2. Geotechnical investigations

For Subsection E2 between km 52 + 880 - km 56 + 220, were initially executed: 18 drillings with depths between 6.00 m and 33.00 m, 3 dynamic cone penetration drillings made with the super heavy dynamic penetrometer type B (DPSH - Type B) and 1 dynamic cone penetration performed with the light dynamic penetrometer (PDU). In addition, 10 drillings were dug with depths between 10-35 m and geophysical - seismic downhole investigations in 3 boreholes [4, 5, 6, 7].

The geological characteristics of this area are presented in figure 5 and table 1 [8].

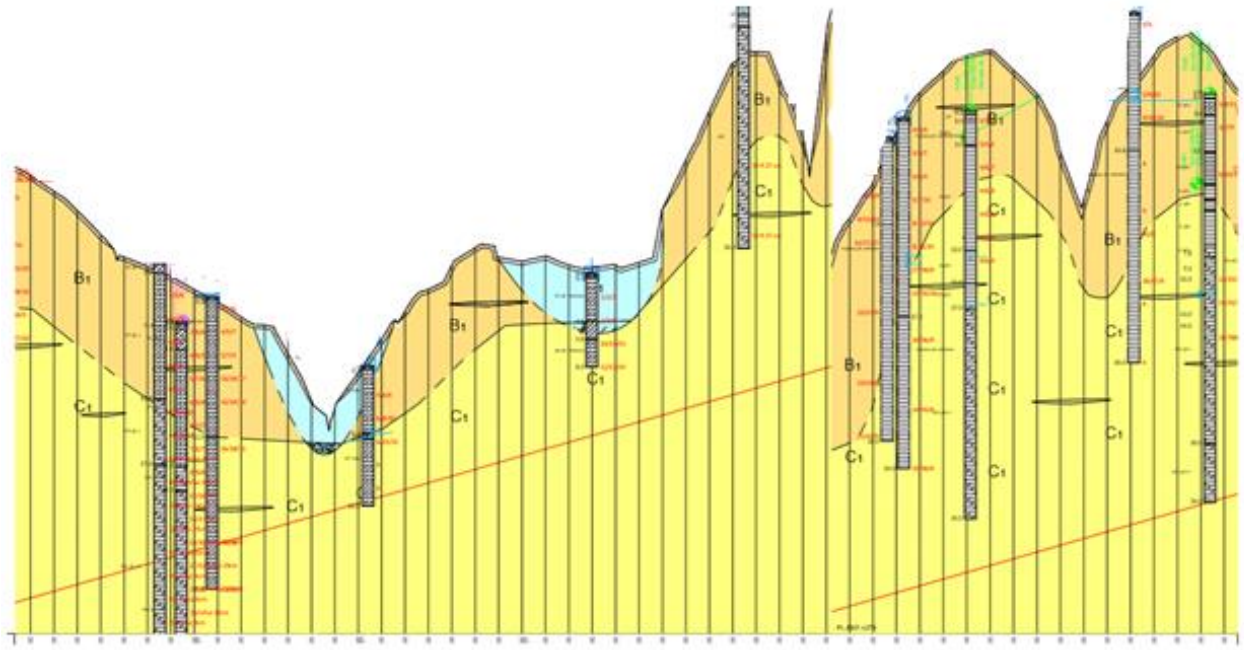


Figure 5. The horizons encountered between km 54+360 and km 55+380

Tabel 1. Geological formations on Subsection E2

Type of formation	Type of deposit	Horizon	Grading characteristics
Covert	Bed deposits 0.30-8.00m	A1	cohesive soils: lam, clays, dusty clays
		A2	poorly cohesive soils: sandy powders, dusty sands, clayey sands, clayey sandy powders
		A3	non-cohesive soils: gravel sands, gravel sands and boulders, sand gravels, small gravel dusty sands, dusty sand gravels
	Adobe / residual soil deposits 0.30-16.00m	B1	cohesive soils: lam, clays, dusty clays
		B2	poorly cohesive soils: sandy powders, dusty sands, clayey sands, clayey sandy powders.
Basic	7.00 – >33.00 m	C1	cohesive soils: lam, clays, dusty clays
		C2	poorly cohesive soils: sandy powders, clayey sandy powders, dusty sands, clayey sands
		C3	non-cohesive soils: gravel sands, gravel sands and boulders, sand gravels, small gravel dusty sands, dusty sand gravels
Difficult terrain	-	-	cohesive soils with low consistency (Ic <0.5) and soils with high heaves and shrinkage - SHHS (active soils)

The geotechnical risk class for the areas crossed by tunnels were determined according to the norm NP 074/2007 and is presented in table 2.

Table 2. Establishment of the category and geotechnical risk for the tunnel area [3]

Factors	Category	Scores
Terrain condition	Difficult terrain	6
Groundwater	With normal or exceptional dewatering	2 - 4
Classification of the construction according to the category importance	Special	5
Affecting adjacent buildings	Moderate risk	3
Seismic area	a_g = 0.08 g	1
Total		**17- 19**

With a total score of 17-19 points and taking into account the complexity and purpose of the works to be executed, the work is classified in ***geotechnical category 3 with major geotechnical risk.***

3.3. Laboratory tests

Based on the geotechnical investigations performed, the subsoil of the area affected by the construction of tunnels (galleries) can be considered, in general, homogeneous. There is an upper complex, with the average thickness of 13-14 m, consisting of dusty clays and clay powders, and a lower one, which extends to the depth of the galleries, represented mainly by dusty clays and sandy marls, preconsolidated.

It is noted that the clays are characterized by a high activity index, due to the presence of minerals with risk of swelling (montmorillonite) in a variable percentage up to a maximum of 69%, with an average of 46%.

During the surveys, the presence of aquifers itself was not revealed, a modest aquifer appears a little above the contact between the upper and lower clays, characterized in the complex by low permeability. However, it cannot be ruled out that there will be isolated bodies of water along the galleries and minimal hydraulic circulation through a largely sandy area.

The main geotechnical characteristics of the terrain in the tunnel areas are given in table 3 which can be considered as an estimate of the majority average values within each layer.

It is specified that at the present stage laboratory samples are available on undisturbed specimens only for the surface complex of silty clays, and for pre-consolidated clays present at great depth it could not be possible to take undisturbed specimens. For these, stability calculations are based on data from the literature and previous experiences.

Regarding the deformation modules, the results from the edometric tests, and the seismic samples in down-hole drilling were taken into account.

Table 3. Geotechnical characteristics of the land in the tunnel areas [9]

Geotechnical characteristic	**U.M.**	**Type of terrain/soil**	
		Upper dusty clays	**Lower clay and marls**
Volumetric weight, γ_v	kN/m^3	19	20
Cohesion, c	kPa	15	40
Inner sliding angle, φ	grade	20	25
Modulus of deformation, E	MPa	10	30

4. Tunnel stability analysis

Stability analyzes were performed with the MEF Plaxis® 2D v.2015 program, produced and distributed by Plaxis BV, Delft (Netherlands), one of the best known programs for solving complex geotechnical problems [3].

4.1. Criteria and calculation hypotheses

The calculation method used is the one at limit states recommended by a series of Eurocodes, but also by the Romanian National Standards [9].

According to them, the calculations were performed at the *operating limit state* (SLE) and the *ultimate limit state* (SLU) in static conditions, as well as at the *ultimate limit state* (SLU) in earthquake conditions.

Regarding the static SLU checks, both the ultimate geotechnical and static limit states were taken into account. The conditions for the following combinations of partial coefficients were analyzed:

- SLU group 1: A1 + M1 + R1
- SLU group 2: A2 + M2 + R1

where: A - partial coefficients for actions or their effects; M - partial coefficients for geotechnical parameters (of the soil); R - partial coefficients for resistances.

It is stated that, according to EC7, the group 1 stress were obtained by applying the partial factor 1.35 directly to the stress obtained in the SLE analyzes.

Regarding the analysis at the ultimate limit state at the earthquake, the calculation was performed by the classic pseudostatic approach, applying to the models a horizontal acceleration of $\pm 0.10g$.

The respective analyzes were performed by applying to the soil resistance parameters the partial coefficients M2, and the partial coefficients per share (A) were always kept at 1.0.

4.2. Calculation method

The analysis of the interaction between the tunnel support and the surrounding ground was performed in the condition of flat deformation state, which in this case, given the linear development of the tunnels, the condition fully satisfies the situation in the terrain.

With the help of the **MEF program (Plaxis® 2D v.2015**) [10] the calculation models with finite elements in plane deformation (2D) were made, in the hypothesis of the elasto-plastic behavior of the ground in which they dig the tunnels, taking into account the stress-strain characteristics.

The discretization of the 2D models was achieved by finite elements of triangular surface with 15 nodes depending on the geometry of the problem to be analyzed and the geotechnical model of the basement defined by the investigations performed in the field.

The supports/linings of the tunnels in the two temporary phases/lining and final support/lining were diagrammed as one-dimensional linear elements, beam type. In the models, in order to highlight the interaction between support/lining and ground, interface elements corresponding to the contact between materials were introduced.

The calculation models used were locked below the vertical and horizontal translation along the two vertical sides, having adequate dimensions to avoid that those solutions are significantly influenced by contour effects [2].

4.2.1. Sections analyzed

Two cross-sections were analyzed, representative of the entire route of the galleries / tunnels, namely:

• section 1, in which the thickness of the rocks covering the tunnel is approximately equal to its hoeing diameter (16-18 m);

• section 2, in which the thickness of the rocks covering the tunnel is equal to the maximum value that is encountered along its route (35 m).

In the 2D models the dimensions of the analyzed areas are 150 m×63 m, for section 1 and 150 × 80 m, for section 2.

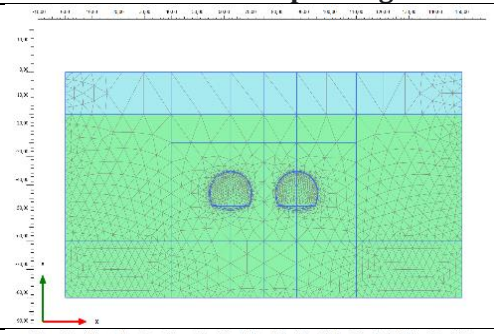
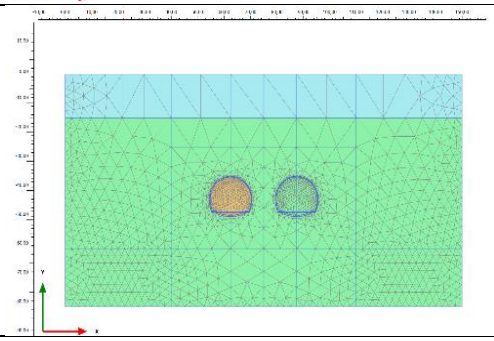
The statistics of the discretization elements of the models are given in table 4.

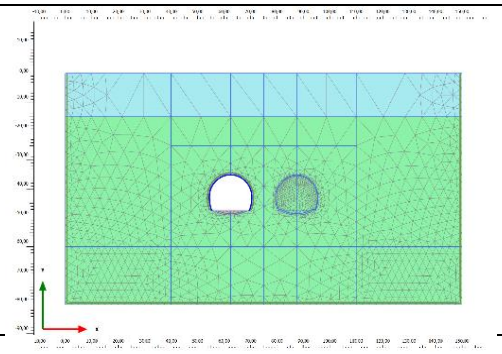
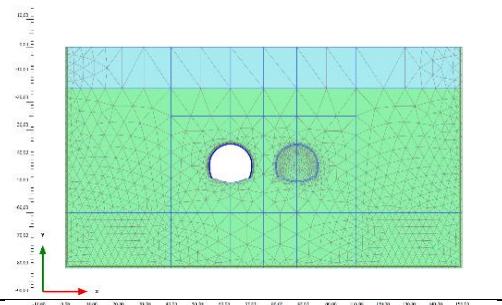
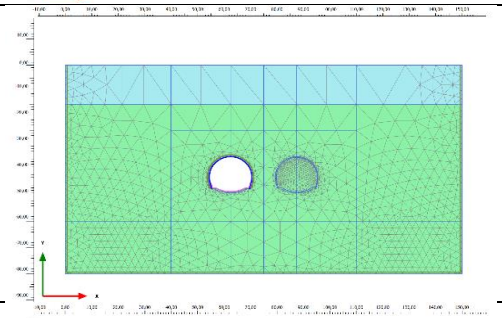
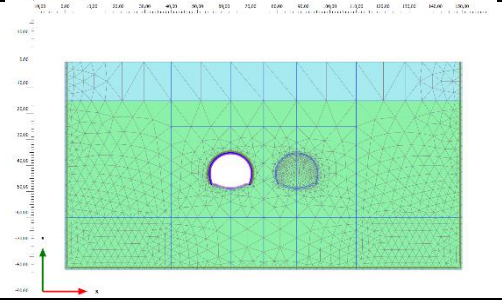
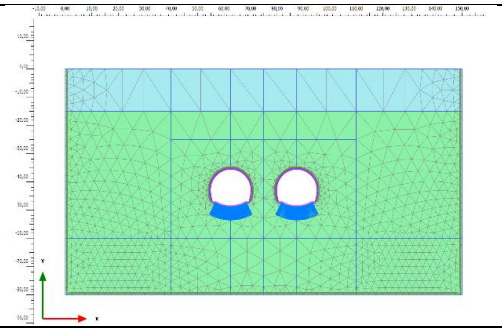
Table 4. Statistics of discretization elements

Total number of:	**Section1**	**Section 2**
Nodes	29240	30520
Triangular elements	3430	3590

The calculations were performed in steps in 15 level of program resulting in the calculation schemes presented in table 5 for the section where the thickness of the rocks covering the tunnel is equal to the maximum value encountered along its route (35 m) (section 2).

Table 5. Phases and calculation schemes corresponding to section 2 [2, 3].

The level of program	**Calculation scheme corresponding to section 2**
0 Generation of the original state of tension in the massif, before the execution of the gallery, with the application of the own weight of the terrain	
1 Cancellation of displacements resulting from the application of its own weight. Carrying out the preventive consolidation on the front of the gallery on the left	

2 Excavation simulation left gallery, throughout the section with activation of temporary support / lining and further excavation	
3 Excavation for raft/inverted vault	
4 Execution of raft / inverted vault and walls; subsequently advancing front	
5 Complete final support / final lining and complete left gallery excavation	
6-12 Sequences analogous to levels of program 1-5, less cancellation of already verified displacements, for the gallery on the right	
13 Activation of force systems that simulate the action exerted by the process of re-heave the terrain on the support / lining of the galleries	
14 Deactivation of temporary supports/linings in the first Level of program, considering that in the long run they are no longer effective; the actions exercised by the massive are totally transferred to the final support / lining	
15 Application of seismic forces, by introduction of horizontal acceleration of ±0.10*g*	

4.2.2. Analysis results

The calculations made allowed to obtain the entire evolution of the state of stresses and deformations of the tunnel (gallery) -ground complex. It was possible to evaluate the extent of the plastic areas around the excavation and the order of magnitude of the movements on the perimeter of the gallery. At the same time, the states of solicitation in the pre-lining and the final lining in different phases of execution were obtained.

Thus, it was possible to evaluate by numerical modeling with finite elements, for the two representative sections of the tunnel route, the following geomechanical characteristics that affect their long-term resistance structure:

- total displacements/deformations (SLE analyzes);
- relative shear stresses τ/τ_{max} (SLE and SLU analyzes - group 2);
- bending moments in the final support / lining (SLE, SLU-group 2 and earthquake SLU analyzes).

For example, in figures 6-11 [2, 3] are presented the distributions of the analyzed geomechanical characteristics (SLE and SLU) for section 1, phases 15 and 16, and in figures 12-17 [2, 3] the distributions of the same phenomena for section 2, phases 15 and 16.

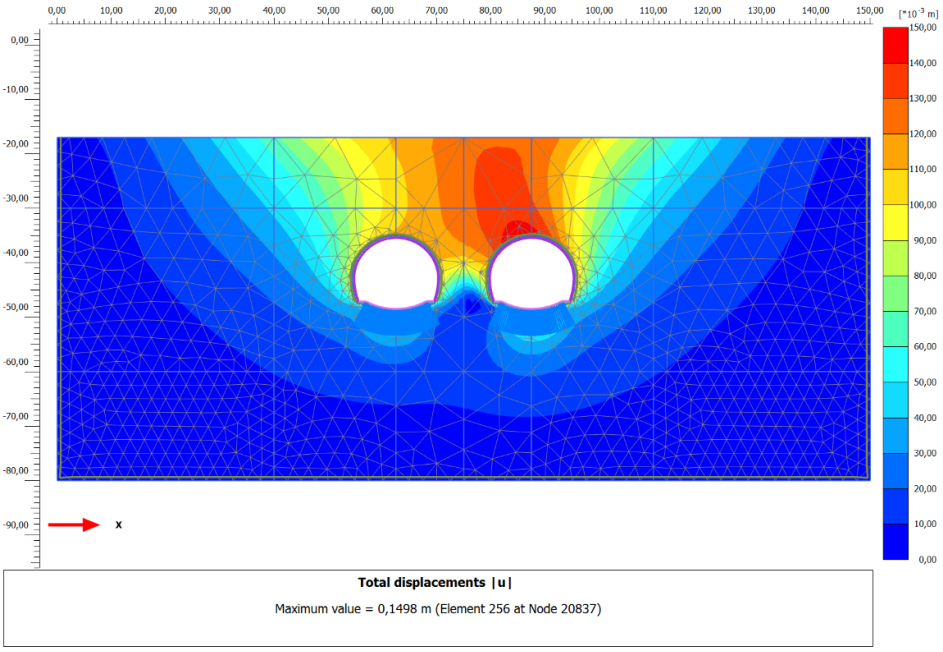


Figure 6. Distribution of displacements in section 1, Phase 14. SLE analysis

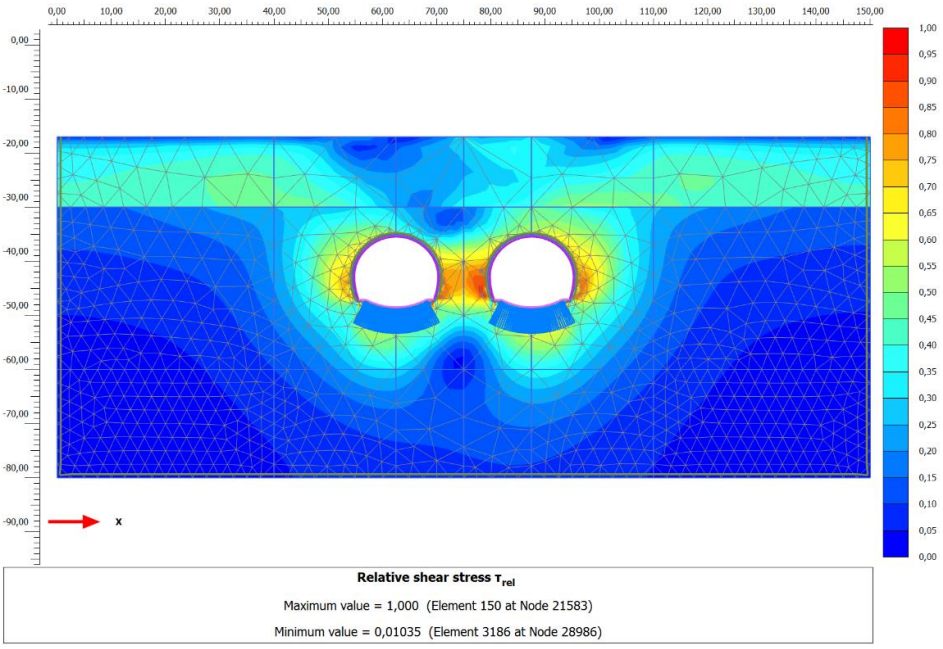


Figure 7. Distribution of relative shear stresses in section 1, Phase 14. SLE analysis

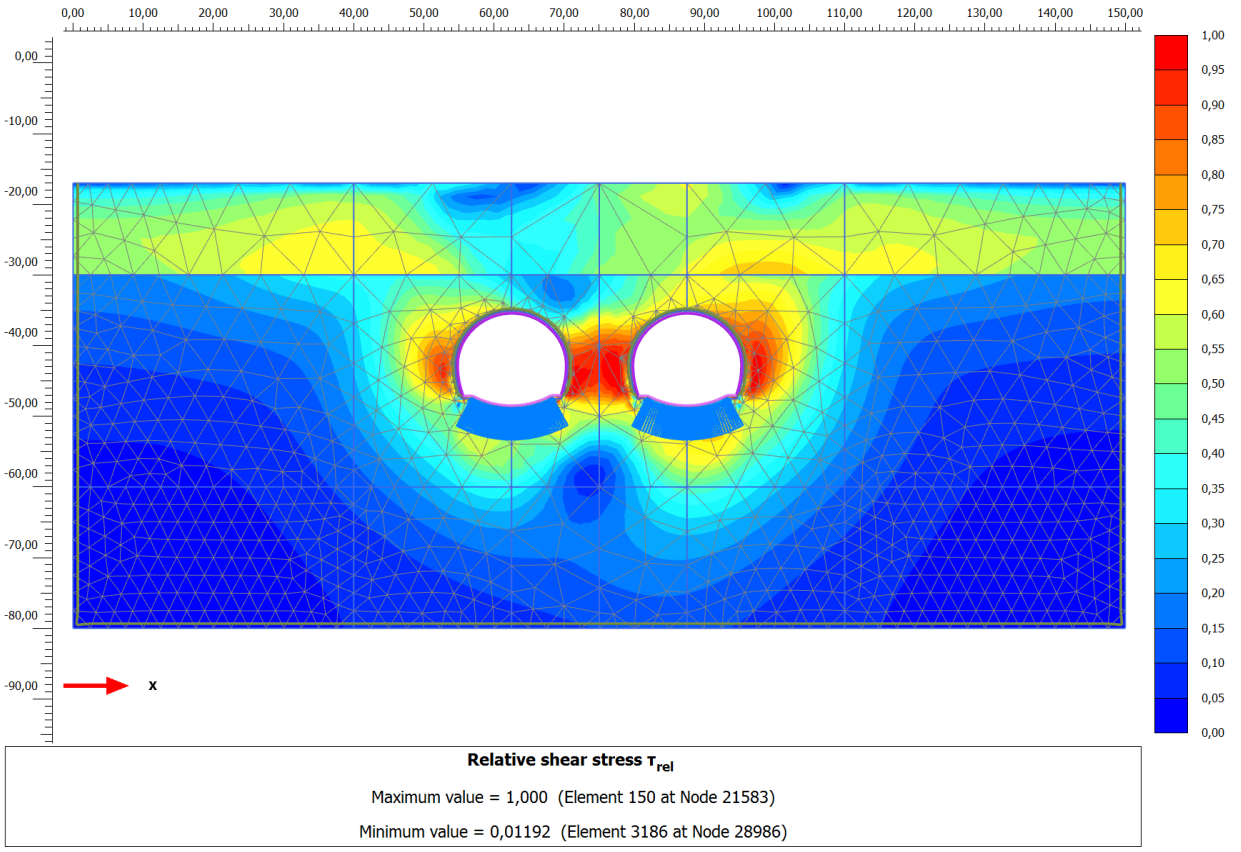


Figure 8. Distribution of relative shear stresses in section 1, Phase 14. SLU analysis- group 2

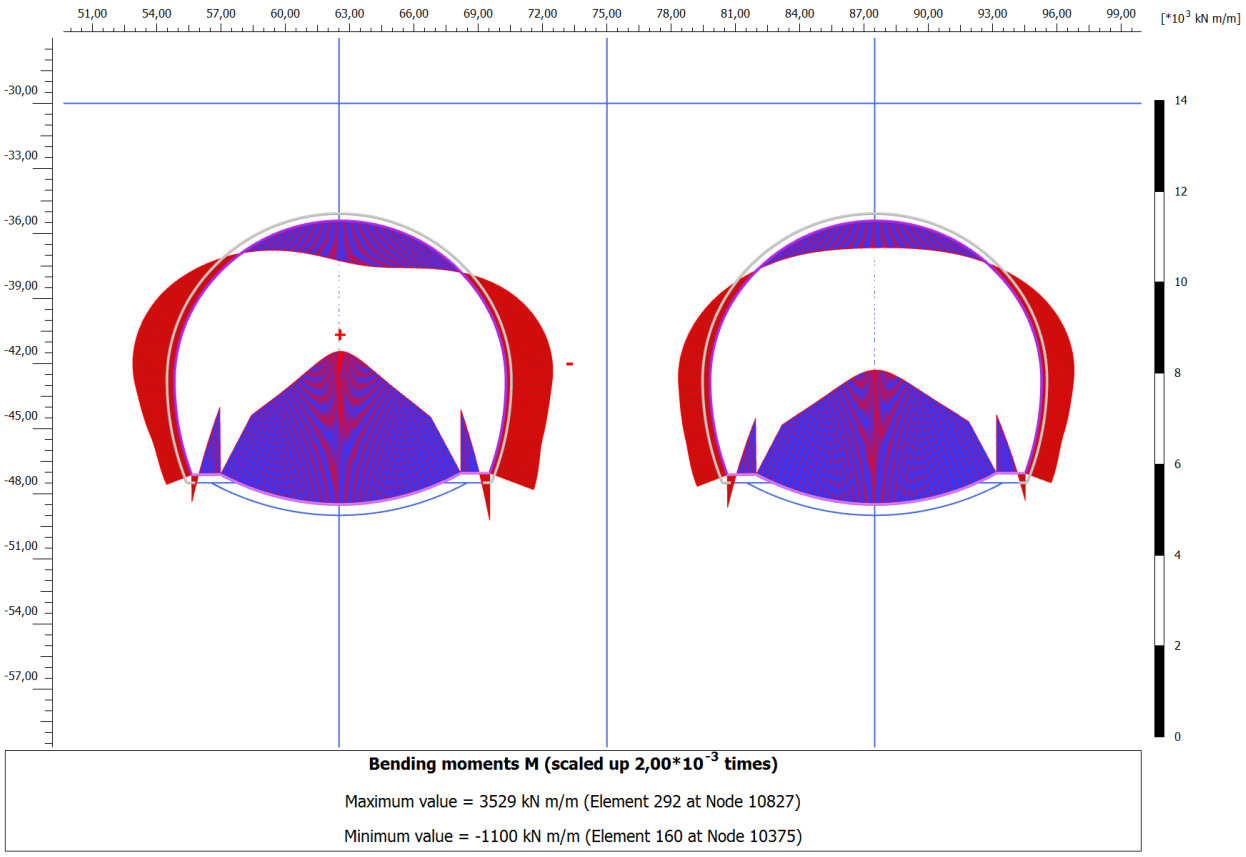


Figure 9. Diagram of bending moments in the final support/lining in section 1, Phase 14. SLE analysis

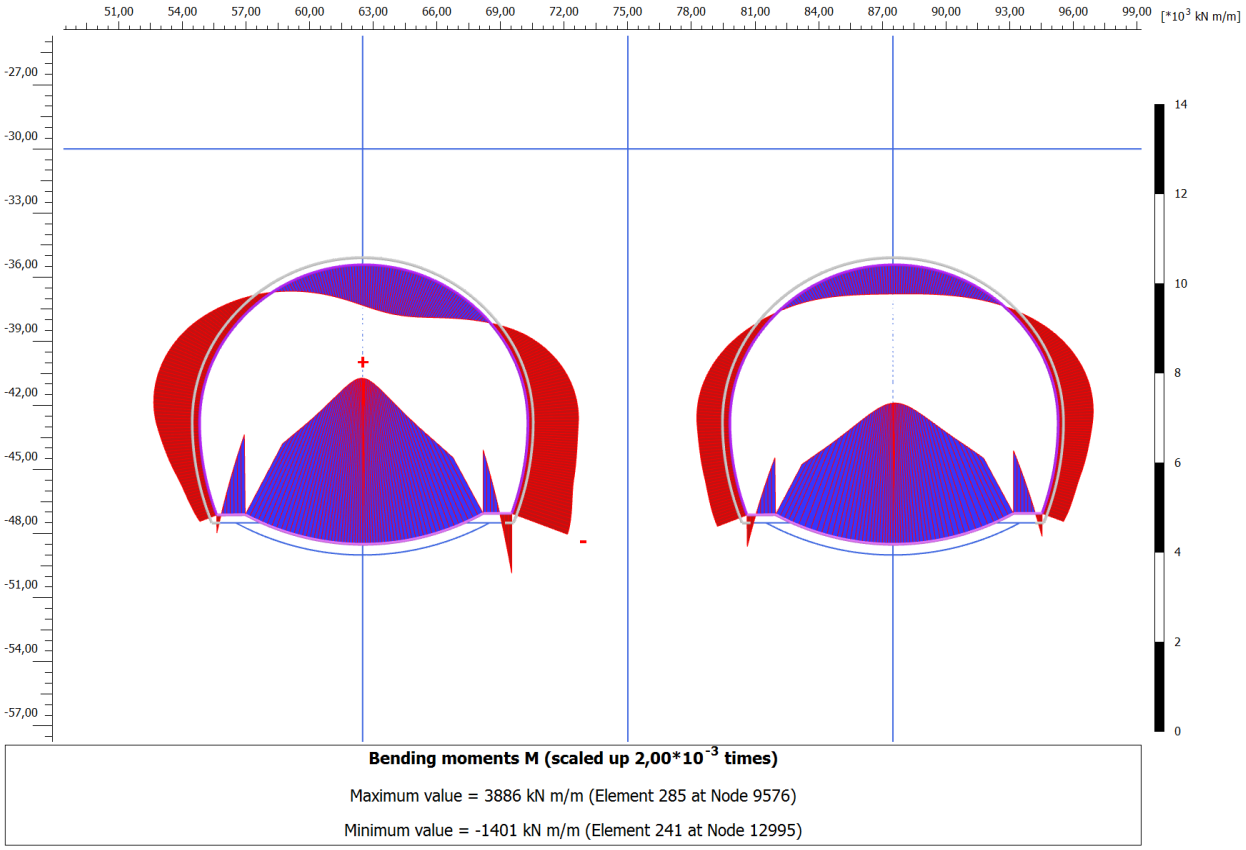


Figure 10. Diagram of bending moments in the final support/lining in section 1, Phase 14. SLU analysis-group 2

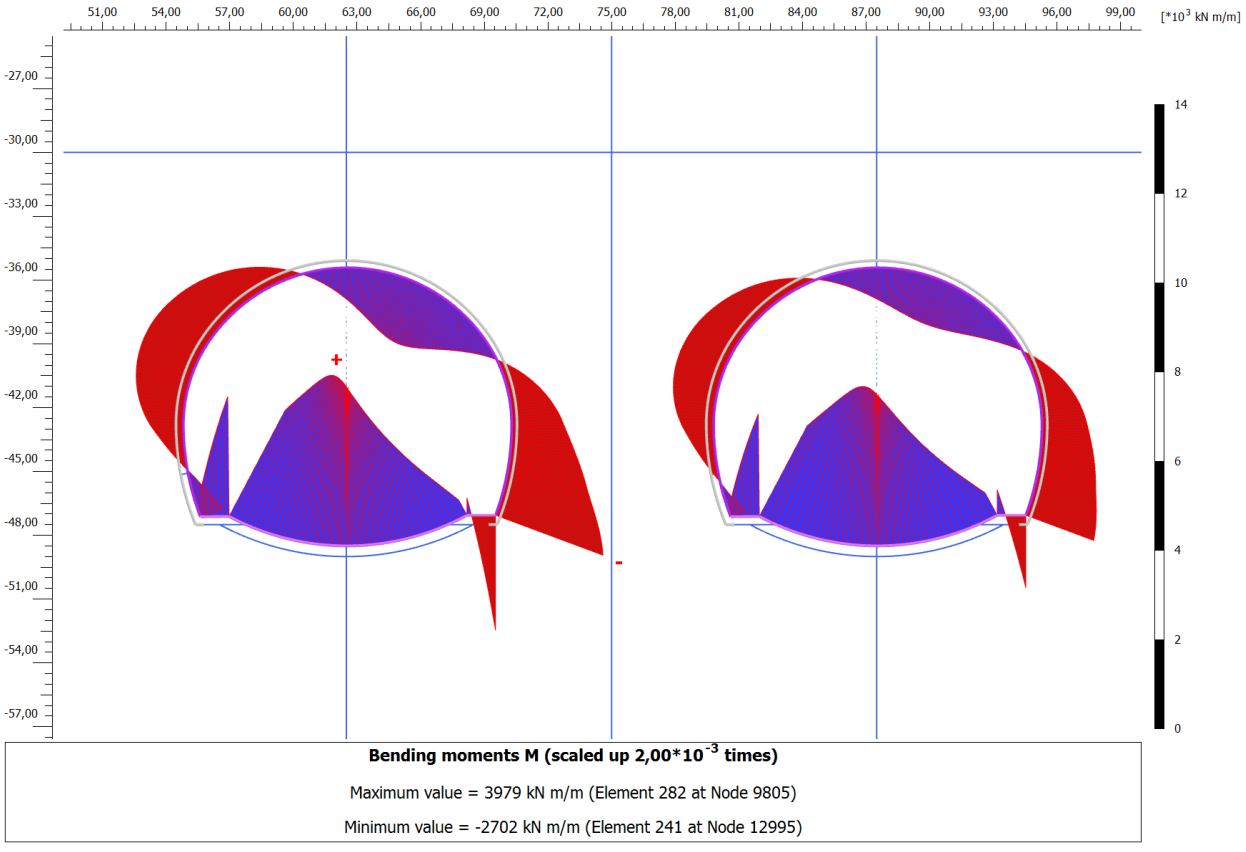


Figure 11. Diagram of bending moments in the final support/lining in section 1, Phase 15. earthquake SLU analyzes

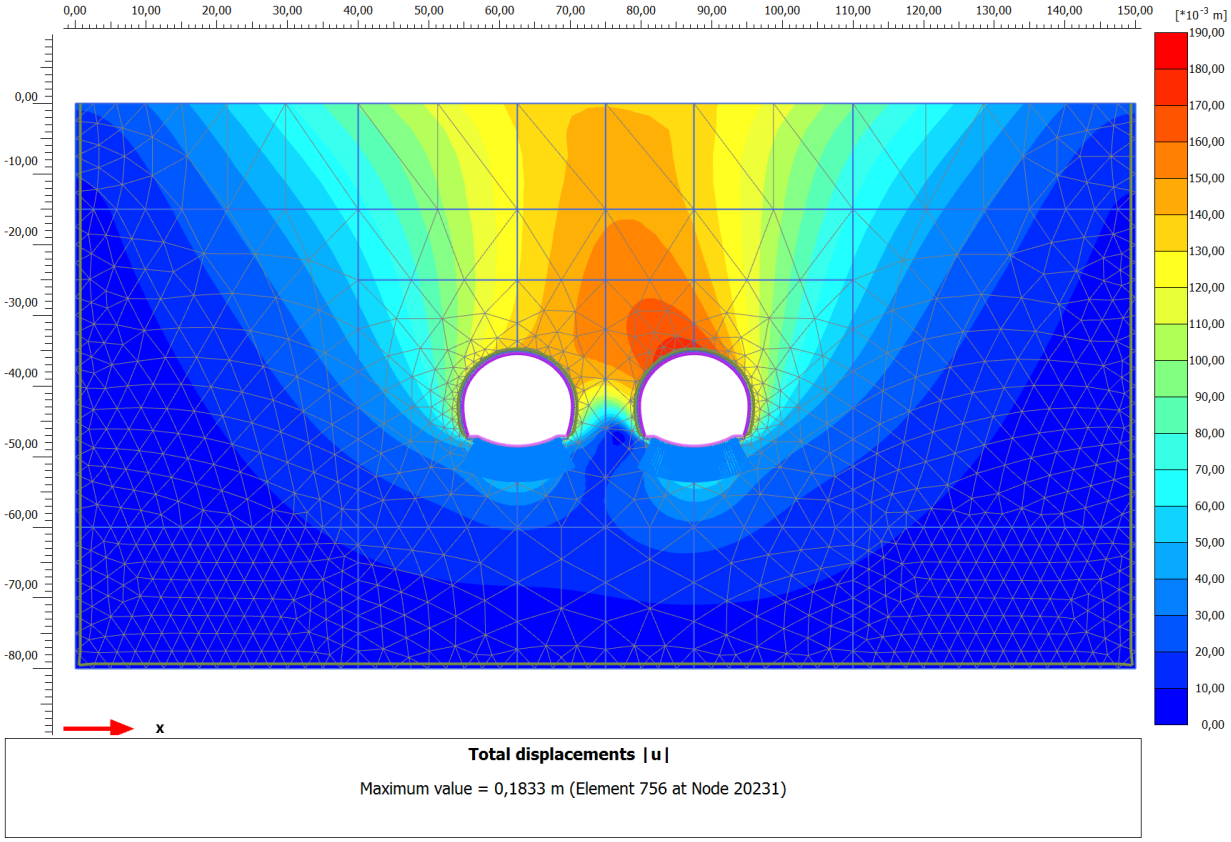


Figure 12. Distribution of displacements in section 2, Phase 14. SLE analysis

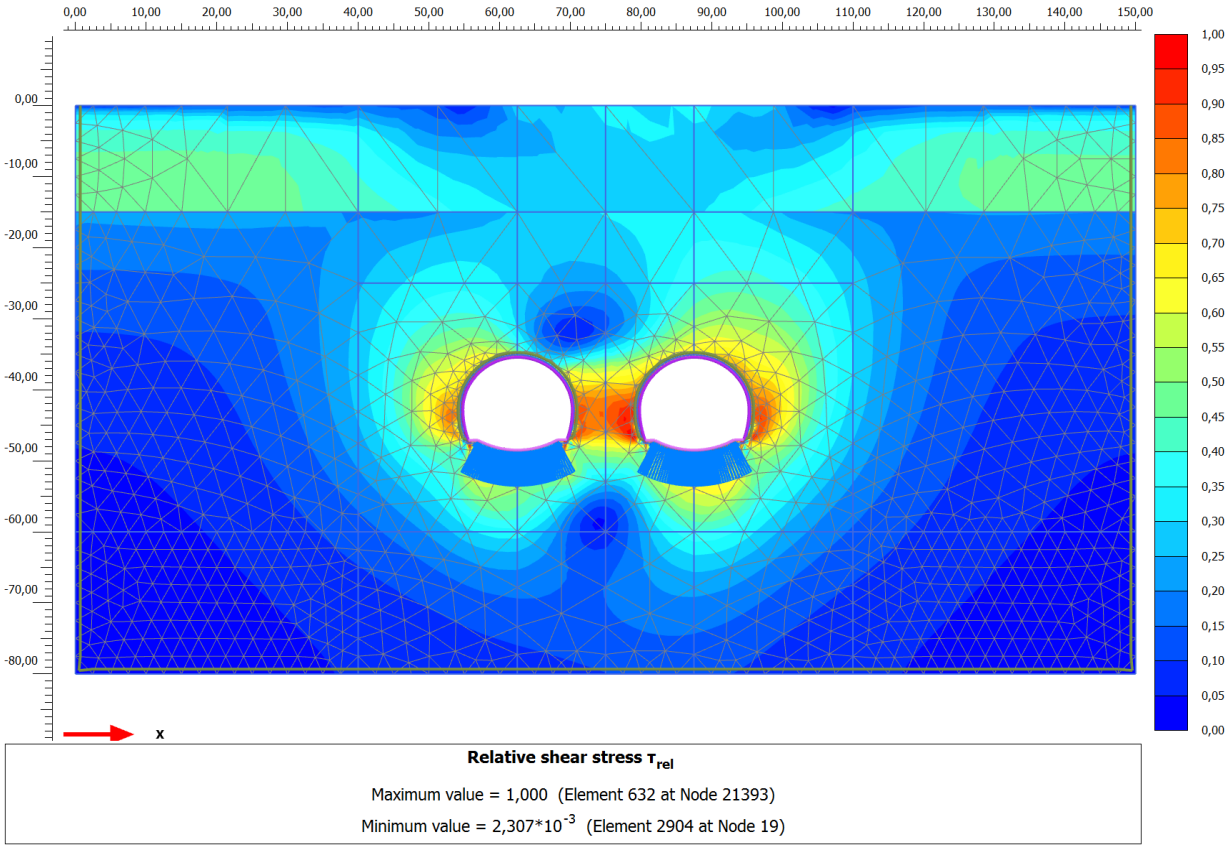


Figure 13. Distribution of relative shear stresses in section 2, Phase 14. SLE analysis

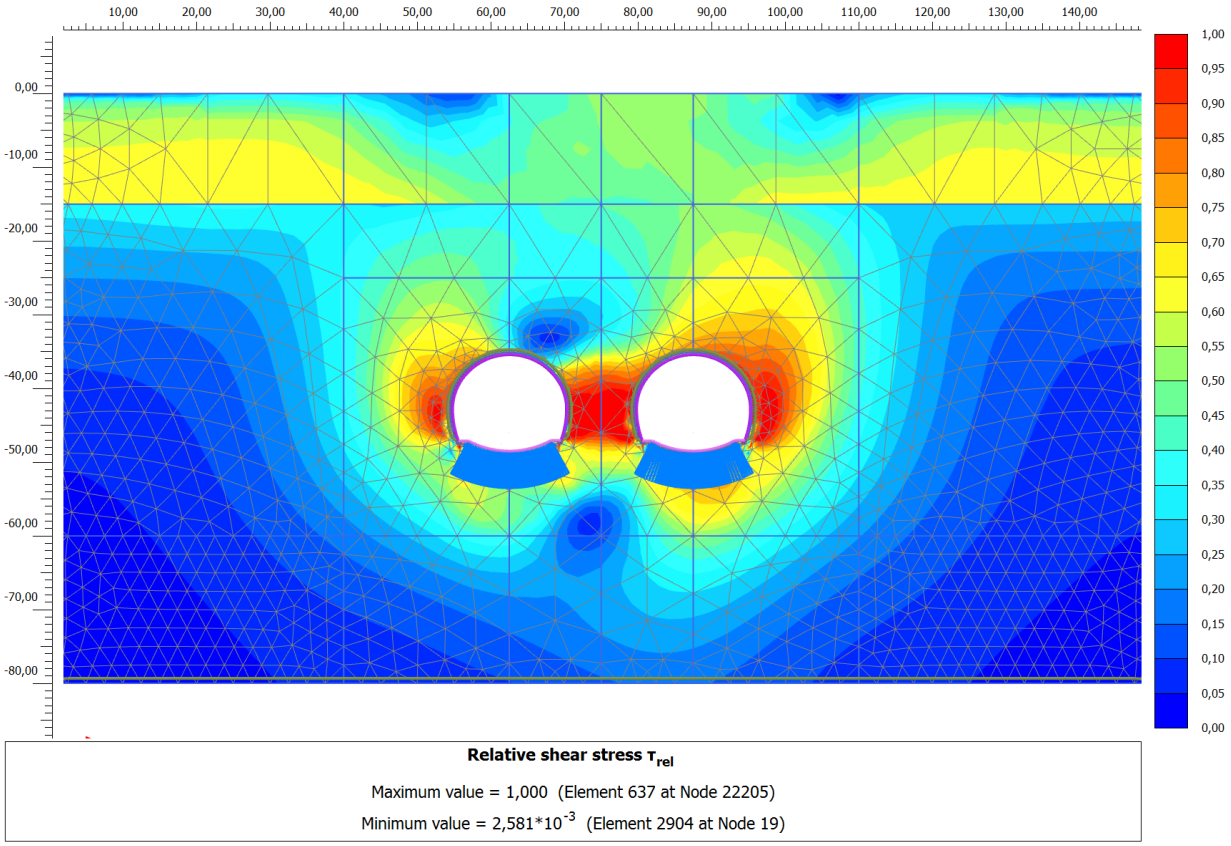


Figure 14. Distribution of relative shear stresses in section 2, Phase 14. SLU analysis-group 2

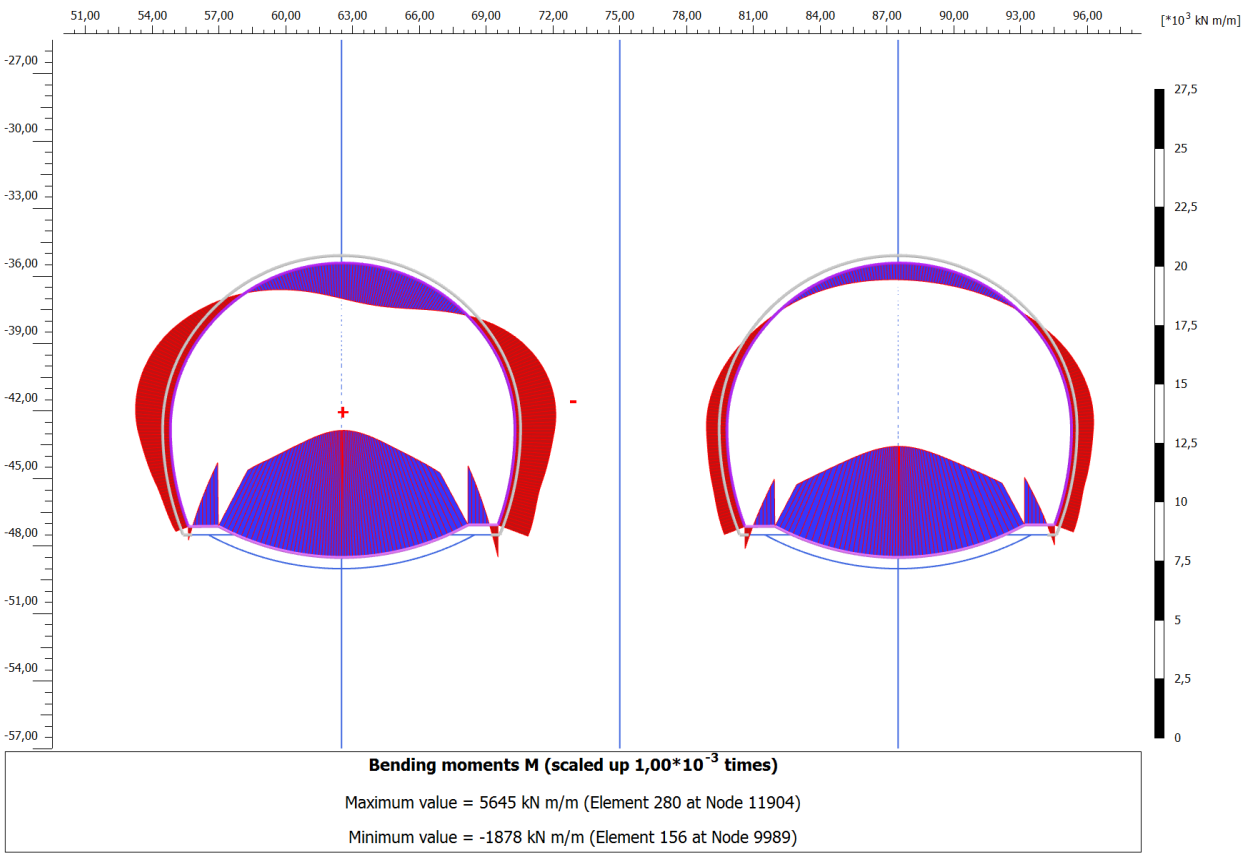


Figure 15. Diagram of bending moments in the final support/lining in section 2, Phase 14. SLE analysis

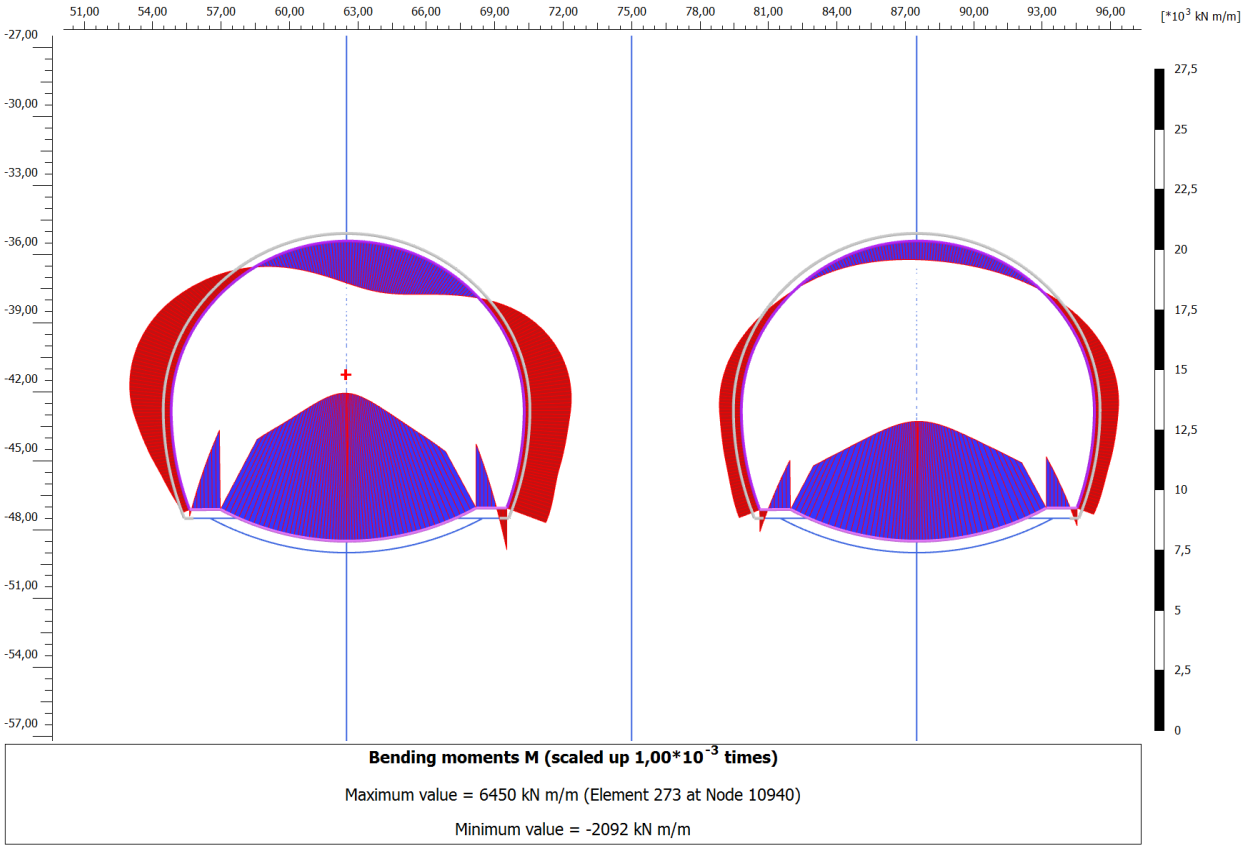


Figure 16. Diagram of bending moments in the final support/lining in section 2, Phase 14. SLU analysis-group 2

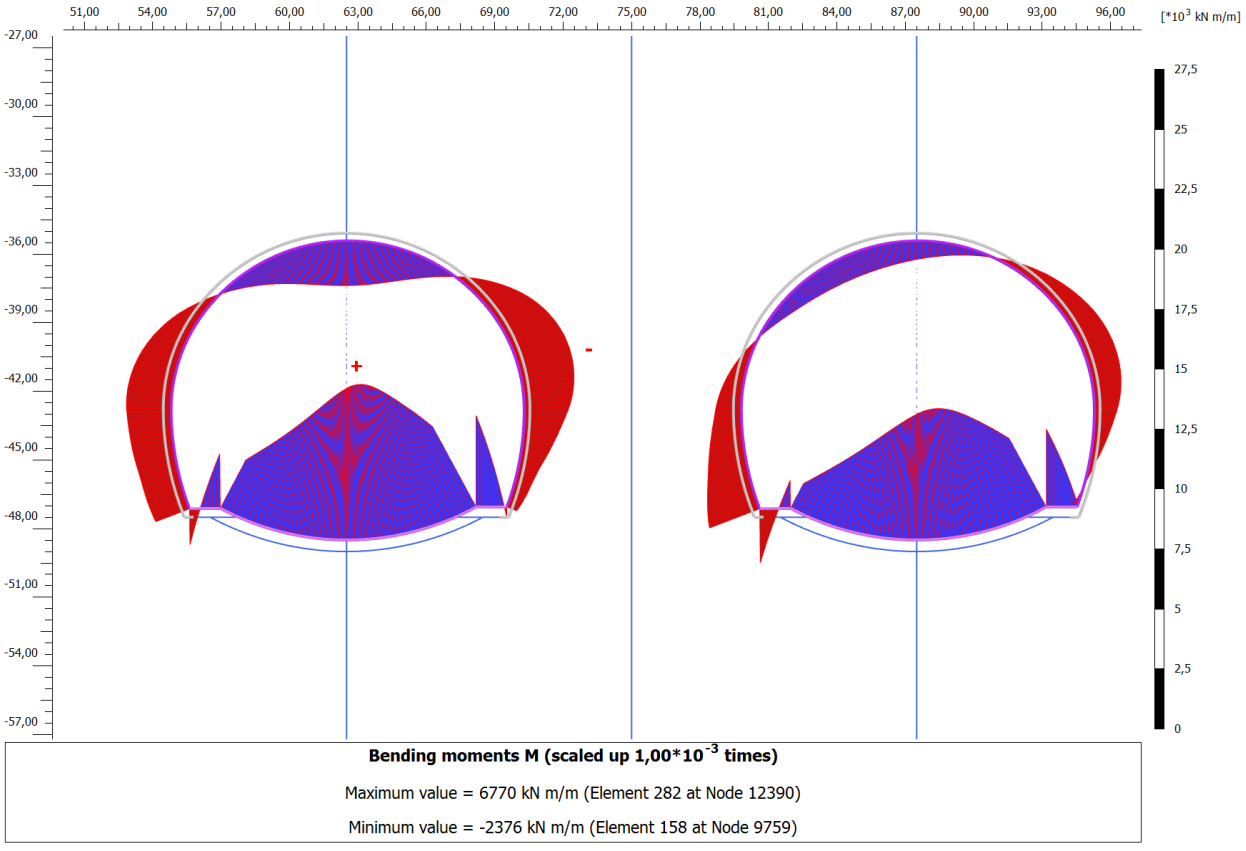


Figure 17. Diagram of bending moments in the final support/lining in section 2, Phase 15. earthquake SLU analyzes

4.3. Static checks of final support / lining

The checks will be made in the two sections characteristic of the route of a gallery/tunnel (1 and 2) and depending on the reference limit state (SLU and SLE). The choice of the size of the support / lining was based on the Norm with the code NP112-2014.

Section 1 (H = 16 m)

The support / lining is made of two parts:
- Raft/inverted Vault with a thickness of 150cm;
- Cap with a thickness of 100 cm.

Section 2 (H = 35 m)

The support / lining is made of two parts:
- Raft/inverted Vault with a thickness of 150cm;
- Cap with a thickness of 100 cm.

Tables 6 and 7 summarize the results of the calculations to verify the stability of the final support / lining of the tunnels taking into account the values of the geomechanical characteristics established using the MEF program (Plaxis® 2D v.2015), results that were partially presented in previous paragraphs.

Table 6. Verification of the stability of the final support/ lining of tunnels in **section 1**

Calculation element	**Geomechanical characteristic**	**Checks for bending moments**			**Checks at shear force**
		SLU		**SLE**	**SLU**
		Group 1	**Group 2**		
Raft/inverted Vault	Max shear force, kN/m, N_{ed}	-1937	-	-1433,36	3352
	Shear force at break, kN/m, $V_{rd,s}$	-	-	-	4369
	Safety coefficient min, $c.s._{min}=N_{ed}/V_{rd,s}$	-	-	-	**1,30**
	Max bending moment, kN·m/m, M_{ed}	4764,5 (stretched fiber on the side)	-	3528,2	-
	Bending moment at break, kN·m/m, M_{Rd}	6331,82	-	-	-
	Safety coefficient min, $c.s._{min}=M_{Rd}/M_{ed}$	**1,329**	-	-	-
Cap	Max shear force, kN/m, N_{ed}	-2588	-3050	-1917	468 (gr.2)
	Shear force at break, kN/m, $V_{rd,s}$	-	-	-	537
	Safety coefficient min, $c.s._{min}=N_{ed}/V_{rd,s}$	-	-	-	**1,15**
	Max bending moment, kN·m/m, M_{ed}	1315 (stretched fiber on the side)	-2047 (stretched fiber on the side)	974,2	-
	Bending moment at break, kN·m/m, M_{Rd}	2016,7	-2188,5	-	-
	Safety coefficient min, $c.s._{min}=M_{Rd}/M_{ed}$	**1,533**	**1,069**	-	-

Table 7. Verification of the stability of the final support/ lining of tunnels in **section 2**

Calculation element	**Geomechanical characteristic**	**Checks for bending moments**			**Checks at shear force**
		SLU		**SLE**	**SLU**
		Group 1	**Group 2**		
Raft/inverted Vault	Max shear force, kN/m, N_{ed}	-3021	-	-2235	3963
	Shear force at break, kN/m, $V_{rd,s}$	-	-	-	4369
	Safety coefficient min, $c.s._{min}=N_{ed}/V_{rd,s}$	-	-	-	**1,10**
	Max bending moment, kN·m/m, M_{ed}	7620,3 (stretched fiber on the side)	-	5643,6	-
	Bending moment at break, kN·m/m, M_{Rd}	9187	-	-	-
	Safety coefficient min, $c.s._{min}=M_{Rd}/M_{ed}$	**1,206**	-	-	-
Cap	Max shear force, kN/m, N_{ed}	-4529,7	-3680	-3346	815 (gr.1)
	Shear force at break, kN/m, $V_{rd,s}$	-	-	-	839
	Safety coefficient min, $c.s._{min}=N_{ed}/V_{rd,s}$	-	-	-	**1,03**
	Max bending moment, kN·m/m, M_{ed}	2312,8 (stretched fiber on the side)	-2441 (stretched fiber on the side)	1720	-
	Bending moment at break, kN·m/m, M_{Rd}	2878	-2638	-	-
	Safety coefficient min, $c.s._{min}=M_{Rd}/M_{ed}$	**1,244**	**1,081**	-	-

Conclusion: Considering the values of the safety coefficients determined for the two calculation elements (raft and cap) it can be concluded that the dimensions chosen for the final support / lining of the tunnels are satisfactory for the stability of the construction as a whole.

5. Tunnel construction technology

Analysis to determine the behavior of the rock / earth mass during the excavation of the gallery / tunnel and to identify possible interventions to stabilize the excavation and stop the deformations within acceptable limits showed that the fronts are unstable in the absence of reinforcements, which requires support / linings immediately behind the advancing front.

For the construction of each tunnel, it is planned to divide its length into two different sections, each characterized by small differences related to the execution technology [2].

For each of them, the works that will be executed, at the stage of the current knowledge of the land stability, are the following:

A. Entrance section and the next 20 m.

• *External micro-piles*

Execute a single row of micro-piles starting from the forward front (figure 18)

• *Consolidation of the front with VTR elements (classic tubes)*

Fiberglass piles are used on the front as the reinforcing element must be able to be destroyed during the subsequent excavation phase of the front. The injections will be extended along the entire length of the tubes. These can be made by the valve *packe*r process, which allows the passage of the cement mortar from one drilling (fiberglass pilot) to another through a plastic tube.

• *Excavation of rocks in the tunnel section*

The excavation of the galleries will be done on the entire section, according to the principles of the ADECO-RS method, in 1.0 m strips. According to the results of the preliminary calculations, each step of advancing the excavation will be preceded by the execution of the temporary support. For this purpose, the use of Ø60″/40 VTR valves with a minimum nominal tensile strength of 600 kN is foreseen.

Figure 18. Micro-piles for front consolidation [2, 3].

In order to highlight the possible hydraulic accumulations, in the crossed land, exploration drillings will be performed in advance, equipped with micro-cracked PVC drainage tubes protected with geotextile. The diameter of the pipes will be at least 60 mm and the length about 30 m. These drains will be made every 20 m of excavation.

Considering the nature of the terrains/rocks/soils in the tunnel area, their excavation will be done with hydraulic excavators to be equipped with buckets with teeth. In the sections with layered rocks can also be used hammers or hydraulic chisels, quickly applicable to the dipper arm, and very easy to replace. In order to obtain an excavation section as close as possible to the calculation profile, the bucket and/or the teeth of the excavators must be hydraulically maneuverable on both sides in relation to the longitudinal axis.

• *Temporary support/lining consisting from gunite with welded mesh and metal belts (double T profiles).*

Temporary support/lining is a phase immediately after excavation. It is necessary for the temporary support of the excavation until the execution of the final support/final lining to avoid problems of relaxation of the cap material.

Temporary support / temporary lining will be made using 2 IPE 240 belts coupled with 1 m wheelbase, and C28/35 concrete spritz ($\sigma_{R,\,med} \geq 18$ N/mm^2) fiber-reinforced with a thickness of 30 cm and welded mesh ø6″/10x10.

Each belt will consist of three elements made of articulated pre-assembled steel profiles that will be transported to the gallery with trolleys, belt mounting devices or cranes.

Before initiating the operations of mounting the belt, it is necessary to complete the operations of regularization and pre-concreting of the front. Only after ensuring perfect cleanliness and the integrity of the excavation surface will it be possible to switch to belt positioning with the help of specialized equipment.

After performing these operations, the belt is lifted, positioned, the belt is supported with the machine arm and the belts are fastened with bolts. Each positioning belt is connected with metal chains to the previous one, already mounted. Depending on the characteristics of the land crossed, it may be necessary to use welded nets positioned above the chains and fixed to them with wire.

In order to consolidate and secure the area where the new pre-lining (belt, chains and nets) was laid, the guniting operation is performed.

• *Making the raft on the already excavated section of the tunnel* (figure 19).

Figure 19. Making the raft [2, 3].

• *Construction (depending on needs) of radial anchors*
• *Realization of radial drainage pipes and collector*
• *Waterproofing with PVC membrane protected above and below by geotextile of minimum weight 400 g/m^2 coupled with PVC membranes of thickness > 2mm* (figure 20)

Figure 20. Waterproofing of the inner surface of the tunnel [2, 3].

• *Final support /final reinforced concrete lining on the entire section*

The final support /final lining, with a constant thickness of 100 cm in the cap and 150 cm in the inverted vault /raft, will be made of reinforced concrete throughout the section.

It is made with mobile formwork/mobile platform that advances along the direction of advance of the front (figure 21). After mounting the reinforcement (BST 500 S), positioning and fixing the mobile formwork, proceed to the operation of pumping the concrete C35 / 45 (R_{ck} > 45 N/mm^2) in the formwork. After the concrete has hardened (1-2 days), when it reaches the strength provided for the stripping phase, the next section is prepared for pouring.

Figure 21. The final support [2, 3].

B. Current section

The same steps as above are valid, with the following modifications:

- there are no micro-piles;
- the final support can be reinforced only at the raft and not in the cap.

In addition, an additional procedure can be considered, namely the radial arrangement of anchors on the cross section of the tunnel.

Note: The tunnel execution technology presented in point 5 is also proposed in the Technical Project [10] but now the E2 tunnel section is being tendered again, so it is possible that the new designer will propose another tunnel execution technology such as drilling the entire section of the tunnel.

6. Conclusions

Based on the general characteristics of the area, geology, geomorphology, climatic conditions and geotechnical research conducted and presented in the paper, it was established that the area crossed by tunnels falls into *geotechnical category 3 with major geotechnical risk.*

The stability analyzes were performed with the MEF Plaxis® 2D v.2015 program, with which the calculation models with finite elements in flat deformation (2D) were built, in the hypothesis of the elasto-plastic behavior of the ground in which the tunnels are dug, taking into account consider the stress-strain characteristics.

The construction of the tunnels will always be accompanied by the change of the state of effort-deformation of the land around them, which leads to active stresses on their support / lining, and on the sealing structures, stresses that depend on many factors whose influence cannot be predicted. accuracy in the design phase.

That is why it is indispensable to set up an instrumentation network, during the execution of the work, through which to permanently control the deforming behavior of the lands and their evolution. Based on the measurements, it is possible to highlight, in advance, any anomalies or situations of potential danger that require additional interventions, ie the adoption of execution methodologies different from those normally used.

Regardless of which tunnel execution technology will be chosen, the geotechnical research carried out [3] and presented in the paper can be used while remaining basic.

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