Solution for Problem Set 6

1. Scope of work

The scope of the present report, is the presentation of the analytical design for a cavern (underground part) of a subway (Athens Metro – Extension of Line 3). Due to the restricted free area for the construction of metro station, an underground part for the station, been designed.

The underground part of the station, as it is presented on the following ***Figure 1***, will be constructed with the conventional tunneling method (NATM), as a tunnel with big spam (cavern) and for the tunnels ventilation, a smaller tunnel near of the main tunnel, is planed to be constructed.

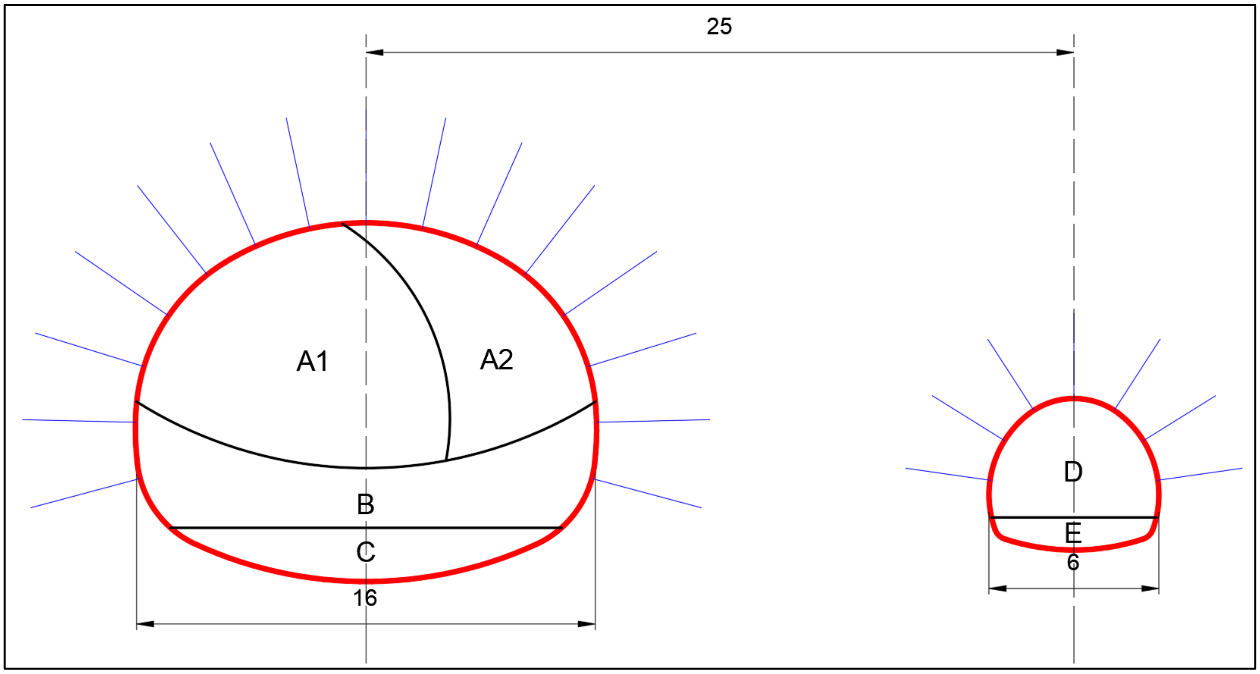

Figure 1. Typical section for the underground part of the station.

The tunnel overburden height (H) for both tunnels, is **25m**. The equivalent tunnel diameter for the main tunnel, is **D= 16m** and for the ventilation tunnel is **D= 6m**. Based on the geological and geotechnical investigation program, both tunnels will be excavated on the Metasandstone – Metasiltstone (AS – STL) geotechnical unit, part of the Athens Schist rockmass formation.

The following stratigraphy is estimated on the station area (depth is measured from the ground surface):

- Depth: 0 to -2m: **Artificial fill (TE)**
- Depth: -2 to -8m: **Metasandstone – Schists (AS – ST)**
- Depth: > -8m: Alteration of **Metasandstone – Metasiltstone (AS – STL)**

Due to the poor ground conditions of the alterations of Metasandstone – Metasiltstone formation, the excavation advance length for both tunnels, set to 1m. Moreover, due the poor ground conditions and the big spam of the main tunnel, the main tunnel (cavern) is planned to be excavated in multiple phase, in order to reduce the surficial settlements and increase the tunnel face stability.

Tunnels design, based on a two – dimensional (2D) numerical analyses, using the RS2 – Rocscience software and the ground deconfinement method by using equivalent tunnel internal peruse (p), in order to simulate the relaxation effect of the third dimension (along the tunnel axis).

For the tunnel lining design, the design standards of EC2 EN 1992-1 taken into account.

2. Numerical analyses

2.1. Software

For the tunnel design, the two – dimensional (2D) numerical software RS2 – Rocscience used.

2.2. Numerical models

The numerical models limits, are based on the tunnel equivalent diameter (D). Both tunnels simulated on the same numerical model, in order to investigate the interaction between the two tunnels. On the numerical model, the side external limits placed four (4) tunnel diameters (4 x D) from the tunnel axis, the upper limit set 25m upper from the tunnel axis (equal to the tunnel overburden height) and the limit, placed two (2) tunnel diameters (2 x D) from the tunnel axis. The tunnel dimeter D, is set to 16m, equal to the dimeter of the cavern. On the following ***Figure 2*** the numerical model is presented, taken into account the stratigraphy, as described in *Chapter 1*.

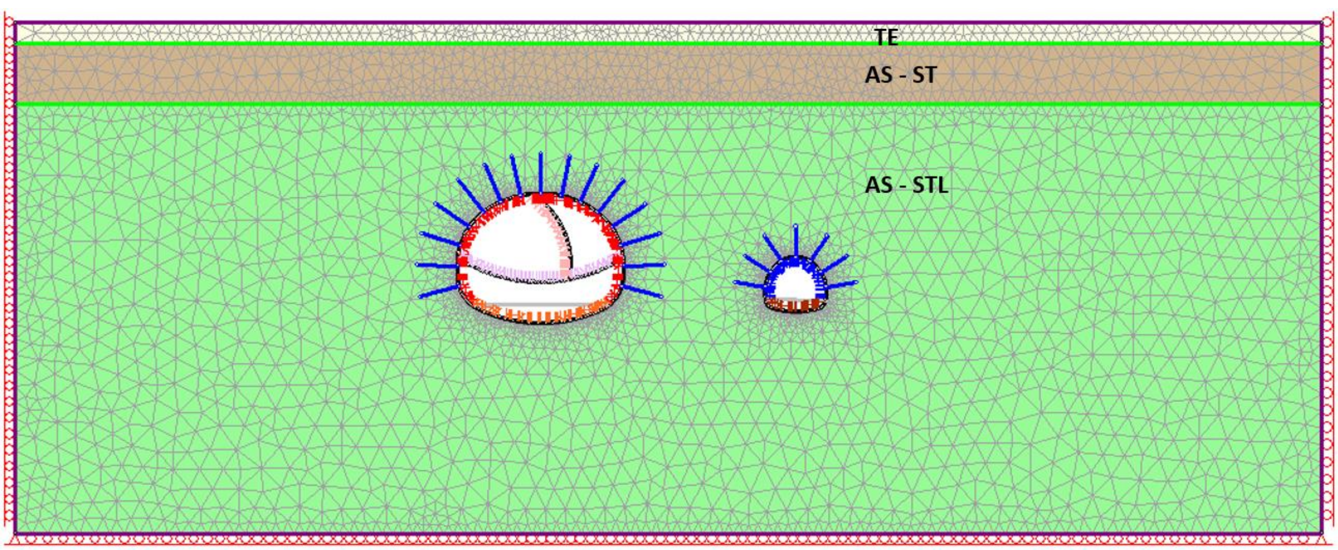

Figure 2. Numerical model.

2.3. Mesh

The numerical model mesh, consist of three nodded triangular continuum elements, where **7886 elements** and **4318 nodes**, used.

2.4. Geotechnical parameters

The following geotechnical parameters, as presented on ***Table 1*** used on the numerical analysis and based on the ground stratigraphy.

Table 1. Geotechnical parameters.

Parameter	TE	AS - ST	AS - STL
Depth	0 – 2m	2 – 8m	>8m
Rockmass unit weight (γ)	21 kN/m^3	23 kN/m^3	23 kN/m^3
Cohesion (c)	10 KPa	60 KPa	-
Friction angle (φ)	28^o	36^o	-
Compressive strength of intact rock (σ_{ci})	-	-	5 MPa
Intact rock modulus (E_i)	-	-	7 GPa
Poisson ratio (v)	0.30	0.30	0.25
mi	-	-	11
GSI	-	-	30
Disturbance factor (D)	-	-	0.4
Dilation angle (δ)	0^o	0^o	0^o
Soil / Rockmass strength (σ_{cm})	33.3 KPa	-	324,3 KPa
Soil / Rockmass modulus (E_m)	40 MPa	200 MPa	344.5 MPa

A plain strain analysis used, taken into account a geostatic field stress loading, with horizontal stress (K_o), 0.5

Due to the usage of hydraulic excavator for both tunnels excavation, the rockmass disturbance factor (D), due to the tunnel excavation, se to 0.4.

For the TE and AS – ST geotechnical units, the Mohr Coulomb failure criterion used and for the rockmass unit AS – STL, the Generalized Hoek & Brown failure criterion (2022), used.

The ground simulated in dry conditions, due to the installation of drainage holes on the tunnels perimeter.

2.5. Support parameters

Both tunnels primary lining, consist of shotcrete lining, lattice girders and rockbolts. Shotcrete lining and lattice girders, simulated as composite beam elements on the excavation perimeter and the rockbolts simulated as truss elements, with radial placement on the tunnel perimeter. All of the supported elements, simulated with elastic behavior. On the following ***Table 2,*** the tunnel support parameters are presented.

Table 2. Tunnel support parameters.

Shotcrete	
Concrete class	C30/37
Reinforcement	T188 wire mesh
Compression strength (f_{ck})	30 MPa
Tensile strength (f_{tk})	6 MPa
Elastic modulus (E_{shot})	17 GPa
Poisson ratio (v)	0.2
Unit weight (γ_{shot})	25 kN/m^3

Rockbolts	
Type	Fully bonded
Diameter (d)	25 mm, 40mm
Tensile capacity (F_{tk})	270 kN, 691 kN
Elastic modulus (E_{steel})	200 GPa
Length (L)	4 m
Pattern	1 x 2 m (longitudinal x radial)
Steel sets	
Type	Lattice girder 4bar-140,Φ26
Elastic modulus (E_{steel})	200 GPa
Longitudinal spacing	1 set every excavation advance length

In order to take into account the effect of the shotcrete time dependent hardening, the installation of shotcrete lining on each excavation step, is separated in three (3) stages, where on each stage the shotcrete thickness and strength is simulated as percentage of the total value, as follow:

- ✓ 1st installation stage: 50% of total thickness, 25% of total strength
- ✓ 2nd installation stage: 100% of total thickness, 50% of total strength
- ✓ 3d installation stage: full values of thickness and strength

2.6. Simulation stages

Tunnel numerical analysis, were based on a two-dimensional (2D) analysis, using the excavation deconfinement method with excavation internal pressure (p), in order to simulate the effect of the third dimension, as the tunnel pre-convergence starts a lot of meters in front of the tunnel face.

Every tunnel excavation step, was separated in four (4) steps, where on the first one the tunnel excavation and ground deconfinement was simulated and on the other excavation steps, the ground deconfinement and the shotcrete hardening as presented on *Chapter 2.5*, were simulated. The sequence of the ground deconfinement and shotcrete hardening simulation, is the following, based on the tunnel face advance length (Y):

- ✓ 1st stage: Excavation deconfinement in the tunnel face (Y=0), No support
- ✓ 2nd stage: Excavation deconfinement in a distance 1 x Y from the tunnel, Shotcrete installation: 50% of total thickness and 25% of total strength, Steel sets installation with full capacity values
- ✓ 3d stage: Excavation deconfinement in a distance 2 x Y from the tunnel face, Shotcrete: 100% of total thickness and 50% of total strength, Bolts installation with full capacity values
- ✓ 4th stage: No internal pressure (deconfinement), Shotcrete with full thickness and strength values

In the numerical analysis, first simulation stage was the geostatic stage, where was the initial condition of the ground without the tunnel construction.

On the following ***Tables 3***, the numerical analysis stages, are presented.

Table 3. Simulation stages.

Stage	Description
1. Geostatic	Simulation of the initial ground conditions, without tunnel construction
Main Tunnel Excavation	
2. Deconfinement Top Heading A (x= 0m)	Main Tunnel: Excavation deconfinement of the Top Heading A area, with internal pressure (p_i) at the tunnel face position (x= 0m).

Stage	Description
3. *Deconfinement Top Heading A (x= -1m)*	Main Tunnel: Excavation deconfinement of the Top Heading A area, with excavation internal pressure (p_i) in a distance x= 1m from the tunnel face. Installation of lattice girders and shotcrete: 50% of total thickness and 25% of total strength.
4. *Deconfinement Top Heading A (x= -2m)*	Main Tunnel: Excavation deconfinement of the Top Heading A area, with excavation internal pressure (p_i) in a distance x= 2m from the tunnel face. Installation of rockbolts. Shotcrete: 100% of total thickness and 50% of total strength.
5. *Support Top Heading A*	Main Tunnel: No internal pressure (p_i). Full parameters of shotcrete lining.
6. *Deconfinement Top Heading B (x= 0m)*	Main Tunnel: Excavation deconfinement of the Top Heading B area, with internal pressure (p_i) at the tunnel face position (x= 0m).
7. *Deconfinement Top Heading B (x= -1m)*	Main Tunnel: Excavation deconfinement of the Top Heading B area, with excavation internal pressure (p_i) in a distance x= 1m from the tunnel face. Installation of lattice girders and shotcrete: 50% of total thickness and 25% of total strength.
8. *Deconfinement Top Heading B (x= -2m)*	Main Tunnel: Excavation deconfinement of the Top Heading B area, with excavation internal pressure (p_i) in a distance x= 2m from the tunnel face. Installation of rockbolts. Shotcrete: 100% of total thickness and 50% of total strength.
9. *Support Top Heading B*	Main Tunnel: No internal pressure (p_i). Full parameters of shotcrete lining.
10. *Deconfinement Bench (x= 0m)*	Main Tunnel: Excavation deconfinement of the Bench area, with internal pressure (p_i) at the tunnel face position (x= 0m).
11. *Deconfinement Bench (x= -1m)*	Main Tunnel: Excavation deconfinement of the Bench area, with excavation internal pressure (p_i) in a distance x= 1m from the tunnel face. Installation of lattice girders and shotcrete: 50% of total thickness and 25% of total strength.
12. *Deconfinement Bench (x= -2m)*	Main Tunnel: Excavation deconfinement of the Bench area, with excavation internal pressure (p_i) in a distance x= 2m from the tunnel face. Installation of rockbolts. Shotcrete: 100% of total thickness and 50% of total strength.
13. *Support Bench*	Main Tunnel: No internal pressure (p_i). Full parameters of shotcrete lining.
14. *Deconfinement Final Invert (x= 0m)*	Main Tunnel: Excavation deconfinement of the Final Invert area, with internal pressure (p_i) at the tunnel face position (x= 0m).
15. *Deconfinement Final Invert (x= -1m)*	Main Tunnel: Excavation deconfinement of the Final Invert area, with excavation internal pressure (p_i) in a distance x= 1m from the tunnel face. Installation of shotcrete: 50% of total thickness and 25% of total strength
16. *Deconfinement Final Invert (x= -2m)*	Main Tunnel: Excavation deconfinement of the Final Invert area, with excavation internal pressure (p_i) in a distance x= 2m from the tunnel face. Shotcrete: 100% of total thickness and 50% of total strength.
17. *Support Final Invert*	Main Tunnel: No internal pressure (p_i). Full parameters of shotcrete lining.
Ventilation Tunnel Excavation	
18. *Deconfinement Top Heading (x= 0m)*	Ventilation Tunnel: Excavation deconfinement of the Top Heading area, with internal pressure (p_i) at the tunnel face position (x= 0m).
19. *Deconfinement Top Heading (x= -1m)*	Ventilation Tunnel: Excavation deconfinement of the Top Heading area, with excavation internal pressure (p_i) in a distance x= 1m from the tunnel face. Installation of lattice girders and shotcrete: 50% of total thickness and 25% of total strength
20. *Deconfinement Top Heading (x= -2m)*	Ventilation Tunnel: Excavation deconfinement of the Top Heading area, with excavation internal pressure (p_i) in a distance x= 2m from the tunnel face.

Stage	Description
	Installation of rockbolts. Shotcrete: 100% of total thickness and 50% of total strength.
21. Support Top Heading	Ventilation Tunnel: No internal pressure (p_i). Full parameters of shotcrete lining.
22. Deconfinement Final Invert (x= 0m)	Ventilation Tunnel: Excavation deconfinement of the Invert area, with internal pressure (p_i) at the tunnel face position (x= 0m).
23. Deconfinement Invert (x= -1m)	Ventilation Tunnel: Excavation deconfinement of the Invert area, with excavation internal pressure (p_i) in a distance x= 1m from the tunnel face. Installation of shotcrete: 50% of total thickness and 25% of total strength.
24. Deconfinement Invert (x= -2m)	Ventilation Tunnel: Excavation deconfinement of the Invert area, with excavation internal pressure (p_i) in a distance x= 2m from the tunnel face. Shotcrete: 100% of total thickness and 50% of total strength.
25. Support Invert	Ventilation Tunnel: No internal pressure (p_i). Full parameters of shotcrete lining.

2.7. Deconfinement factor (λ)

The effect of the third -dimension, as the tunnel pre-convergence starts a lot of meters in front of the tunnel face, simulated by the equivalent deconfinement factor (λ), based on the ground convergence – confinement curve.

The internal pressure (p), is calculated as follow:

$p= (1-\lambda) \times p_o$, where p_o is the geostatic field stress.

The ground deconfinement due to the tunnel excavation, was simulated by an internal pressure (p) which added in the excavation perimeter.

In order to calculate the deconfinement factor (λ) for using in the numerical analysis, the longitudinal displacement profile (LDP) method by *Chern et al. 1998* and the convergence – confinement method by *Kavvadas M. 2004*, used.

On the numerical analysis, the input parameter for the internal pressure (p), is the stage factor (β), which is based on the deconfinement factor (λ), and can be calculated as follow: **$\beta= (1-\lambda)$**.

On the following Figures, the convergence confinement curve and the tunnel longitudinal displacement profile (LDP) for the geotechnical unit (AS - STL), for both tunnels with D= 6m and 16m, are presented. Also, on the following Tables characteristics values from the convergence – confinement curve and the deconfinement ratio (λ) on the position of the tunnel support installation, are presented.

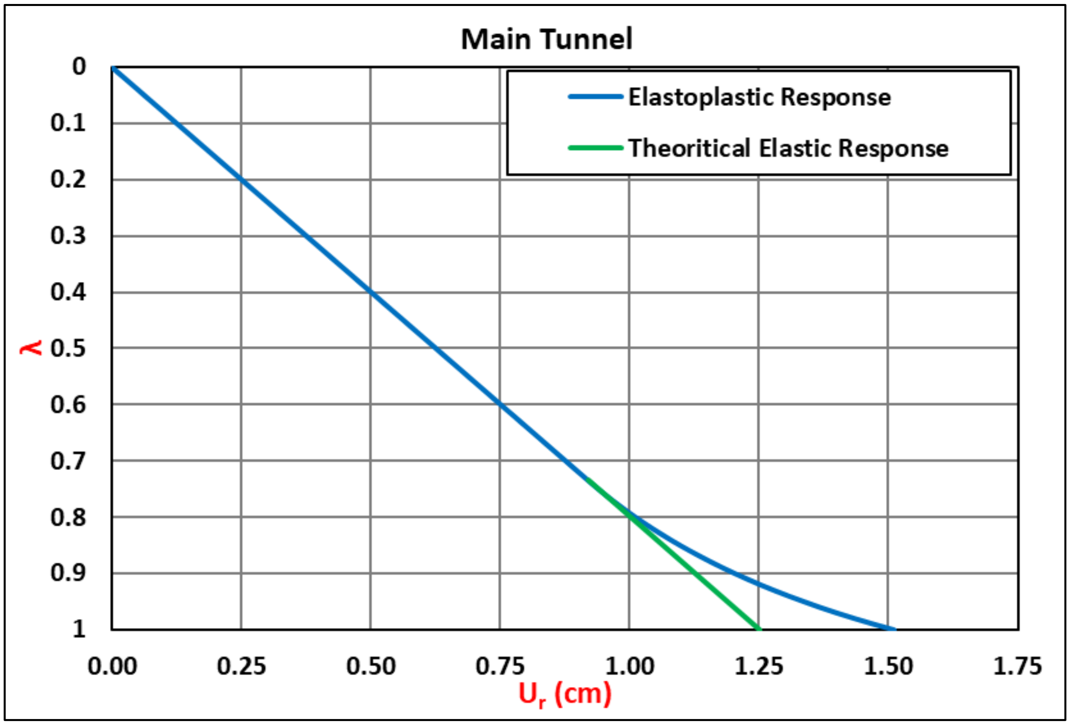

Figure 3. Ground convergence – confinement curve, for the main tunnel.

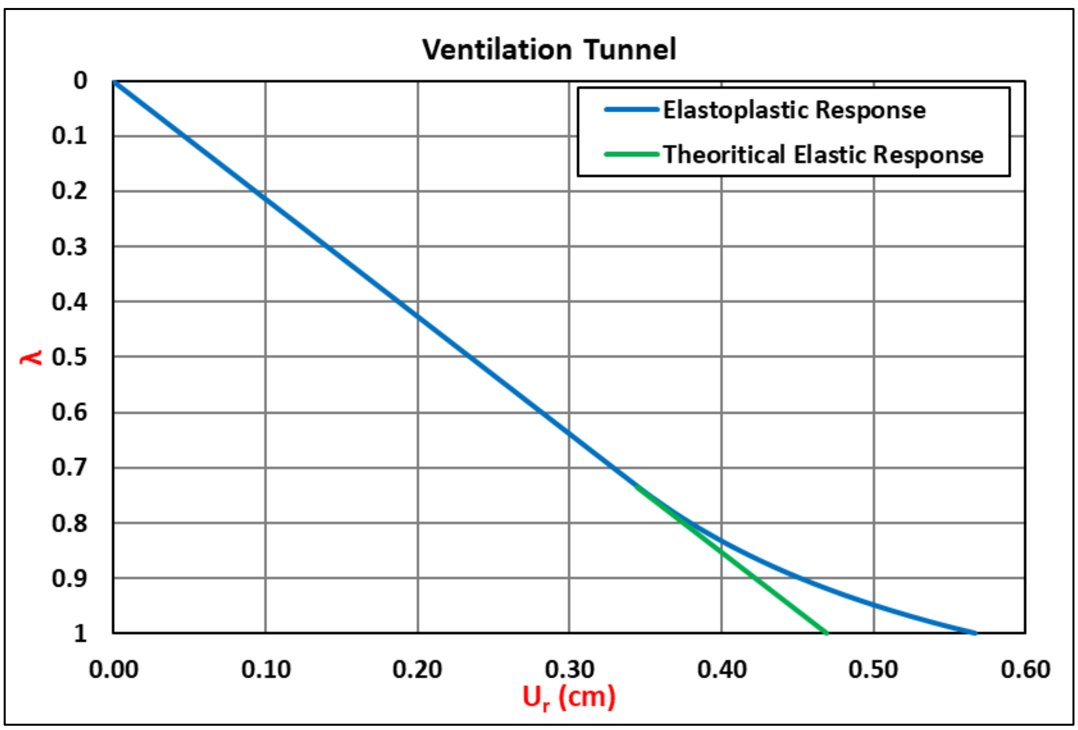

Figure 4. Ground convergence – confinement curve, for engineering geological unit B.

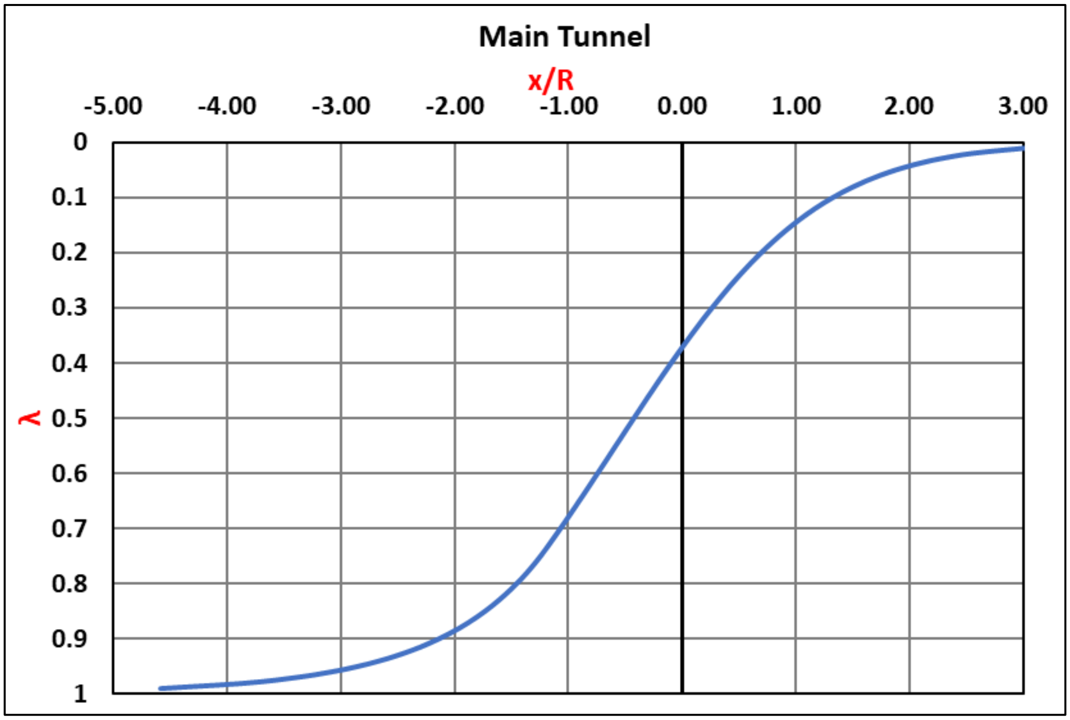

Figure 5. Longitudinal displacement profile (LDP), for the main tunnel.

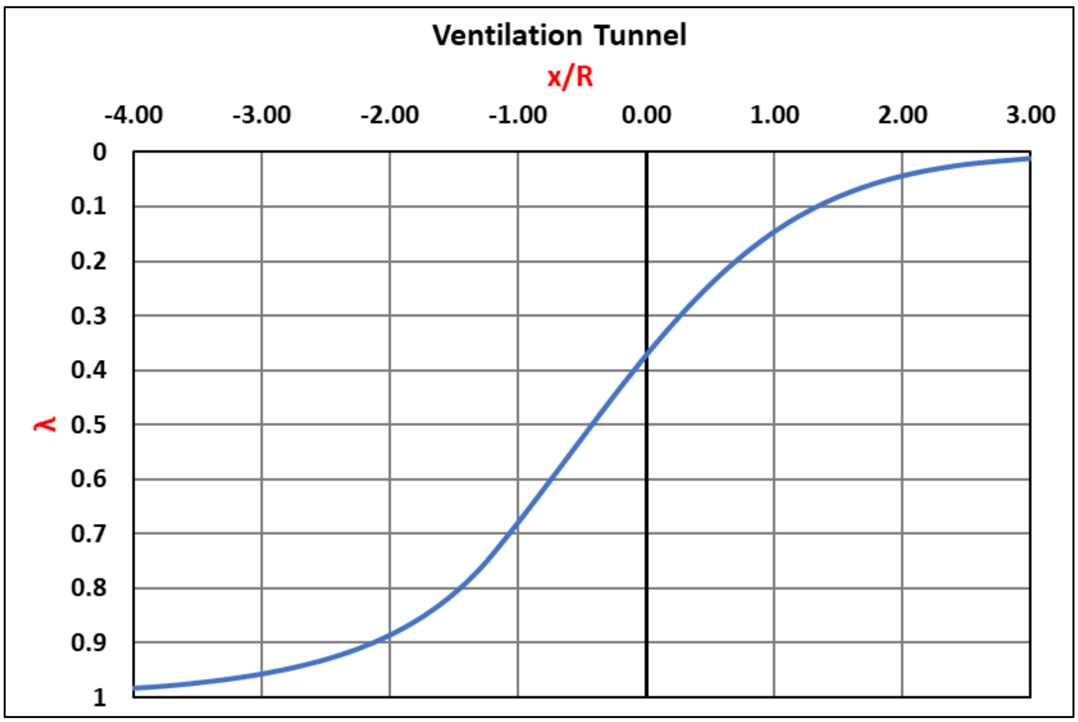

Figure 6. Longitudinal displacement profile (LDP), for engineering geological unit B.

Table 4.Deconfinement characteristics values, for main tunnel.

Parameter	Value
Geostatic field stress (p_o)	431,3 KPa
Ground strength (σ_{cm})	324,3 KPa
Overload factor (N_s)	2,66
Critical deconfinement factor (λ_{cr})	0,73
Deconfinement factor at the tunnel face (λ, x=0m)	0,37
Deconfinement factor at excavation 1 x advance length (λ, x=-1m)	0,41
Deconfinement factor at excavation 2 x advance length (λ, x=-2m)	0,45
Stage factor at excavation 1 x advance length (β, x=0m)	0,63
Stage factor at excavation 2 x advance length (β, x=-1m)	0,59
Stage factor at excavation 3 x advance length (β, x=-2m)	0,55

Table 5.Deconfinement characteristics values, for ventilation tunnel.

Parameter	Value
Geostatic field stress (p_o)	431,3 KPa
Ground strength (σ_{cm})	324,3 KPa
Overload factor (N_s)	2,66
Critical deconfinement factor (λ_{cr})	0,73
Deconfinement factor at the tunnel face (λ, x=0m)	0,37
Deconfinement factor at excavation 1 x advance length (λ, x=-1m)	0,48
Deconfinement factor at excavation 2 x advance length (λ, x=-2m)	0,58
Stage factor at excavation 1 x advance length (β, x=0m)	0,63
Stage factor at excavation 1 x advance length (β, x=-1m)	0,52
Stage factor at excavation 3 x advance length (β, x=-2m)	0,42

Note, that in all excavation phases, the same deconfinement ratio (λ), is used.

2.8. Restrains

On the numerical models, the following restrains were used:

- Rollers on the sides and bottom limits
- Pins on the two corners of the bottom limit
- Free the upper surface

3. Results – Main Tunnel

3.1. Tunnel primary lining design – Support bearing capacity

Based on the limitation of the minimal shotcrete thickness t= 15cm, numerical analysis with the previous shotcrete thickness done, in order to check the lining bearing capacity, according to EC2 EN 1992-1. It is observed that the minimal shotcretes thickness t=15cm, it is acceptable according to EC2 EN 1992-1, as the combination points between lining bending moment (M) and thrust (N), are inside the lining capacity envelope in all simulation stages, as presented on the following Figures.

✓ Main Tunnel

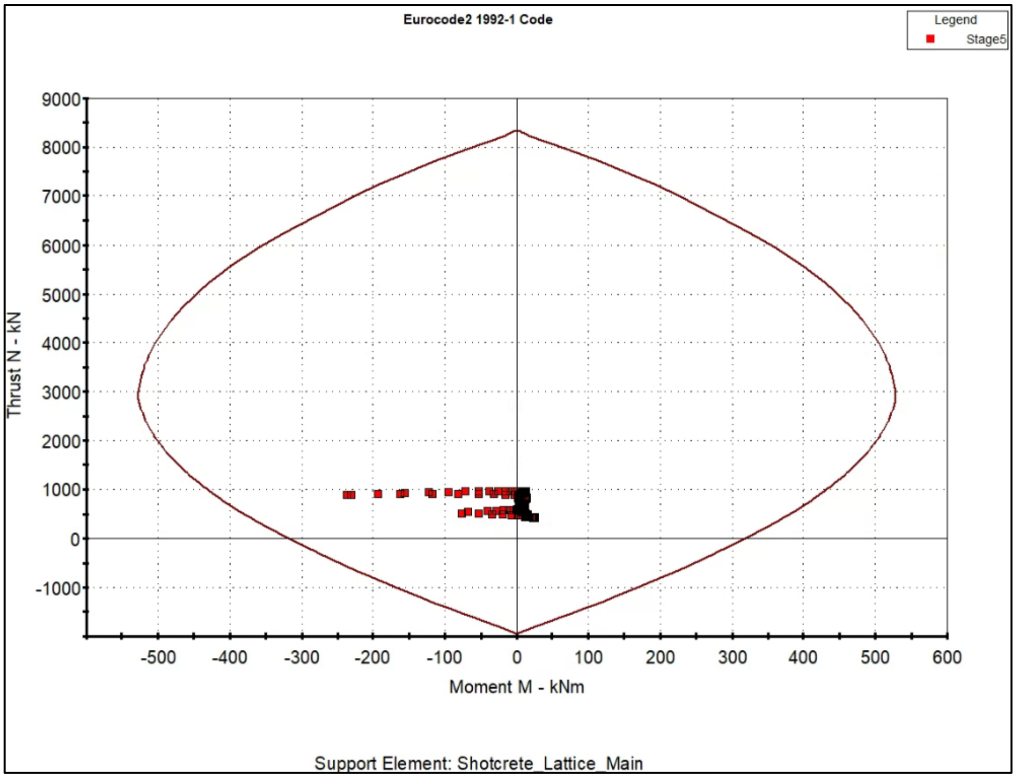

Figure 7. Support capacity plot for the shotcrete lining on the perimeter of the Top Heading A area at the Top Heading A excavation phase, of main tunnel.

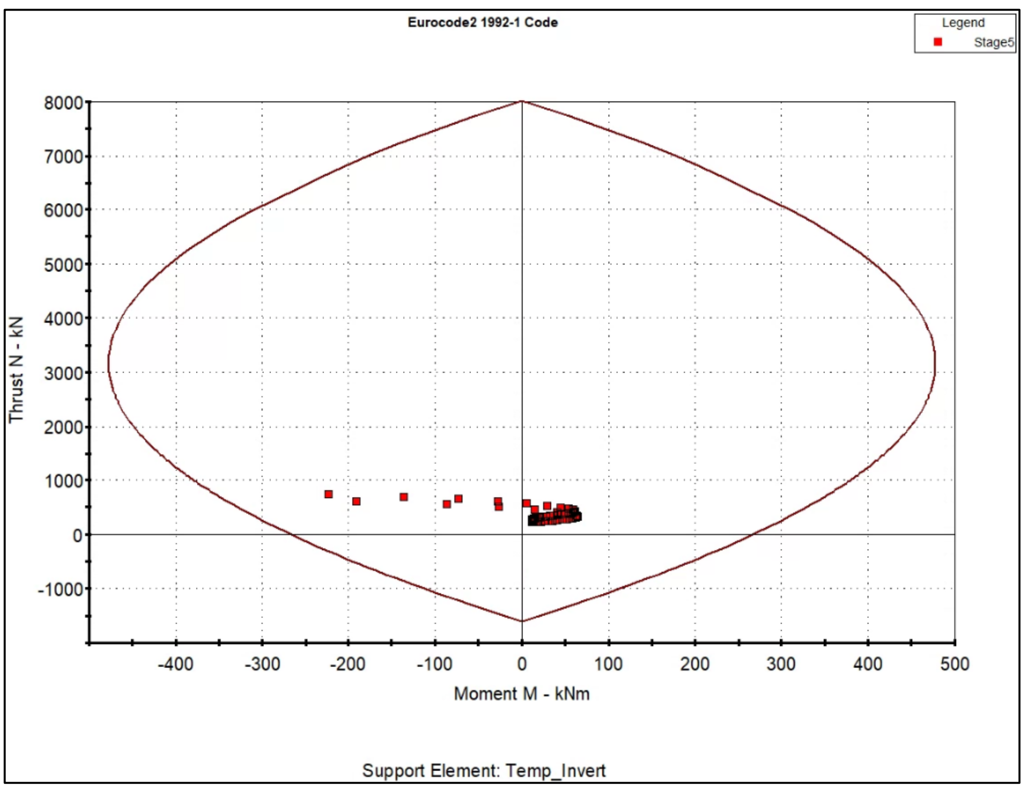

Figure 8. Support capacity plot for the shotcrete lining on the Temporary Invert area at the Top Heading A excavation phase, of main tunnel.

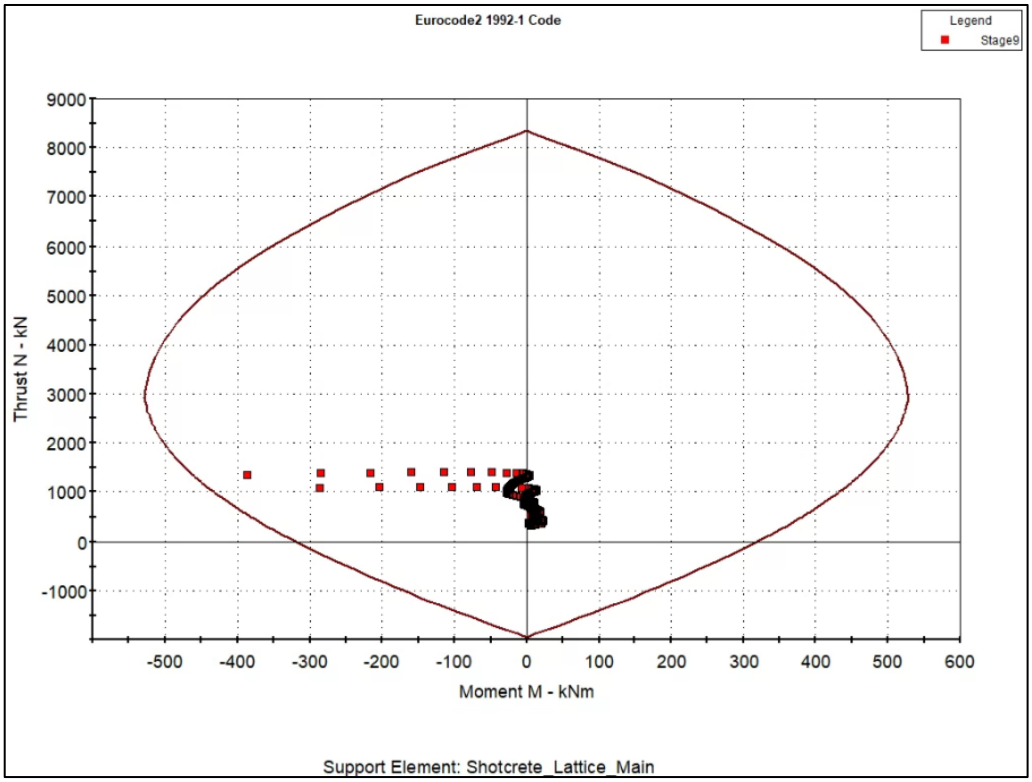

Figure 9. Support capacity plot for the shotcrete lining on the perimeter of the Top Heading B area at the Top Heading B excavation phase, of main tunnel.

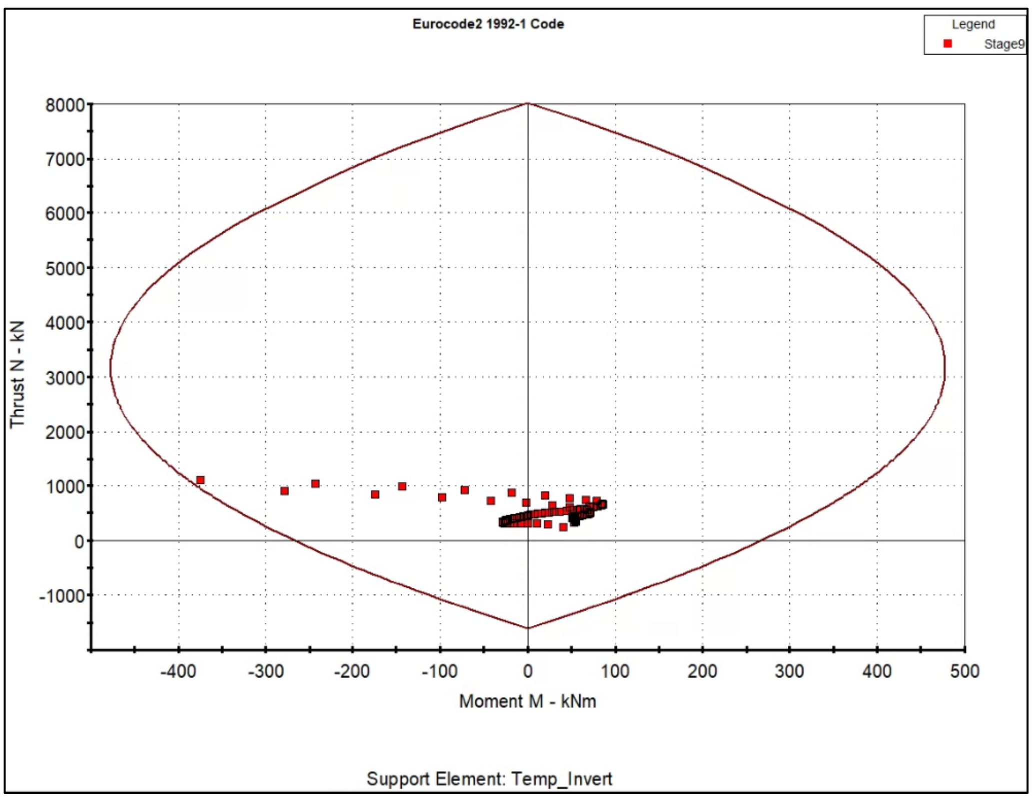

Figure 10. Support capacity plot for the shotcrete lining on the Temporary Invert area at the Top Heading B excavation phase, of main tunnel.

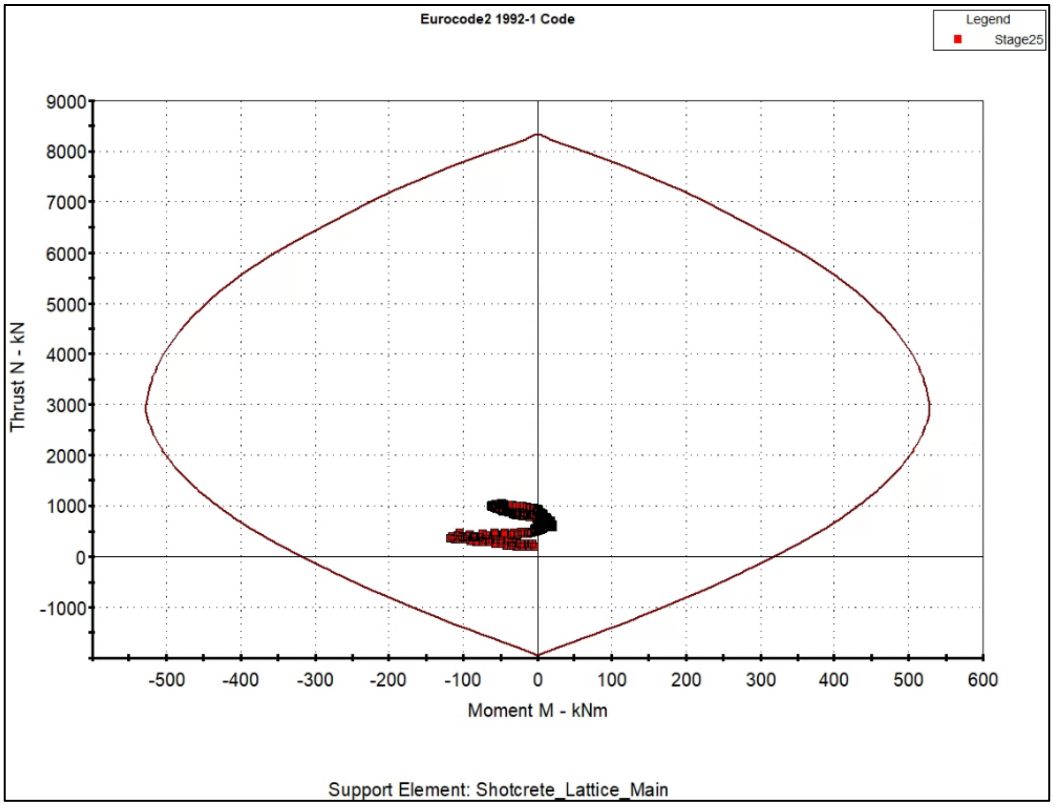

Figure 11. Support capacity plot for the shotcrete lining on the perimeter of the Bench area at the Bench excavation phase, of main tunnel.

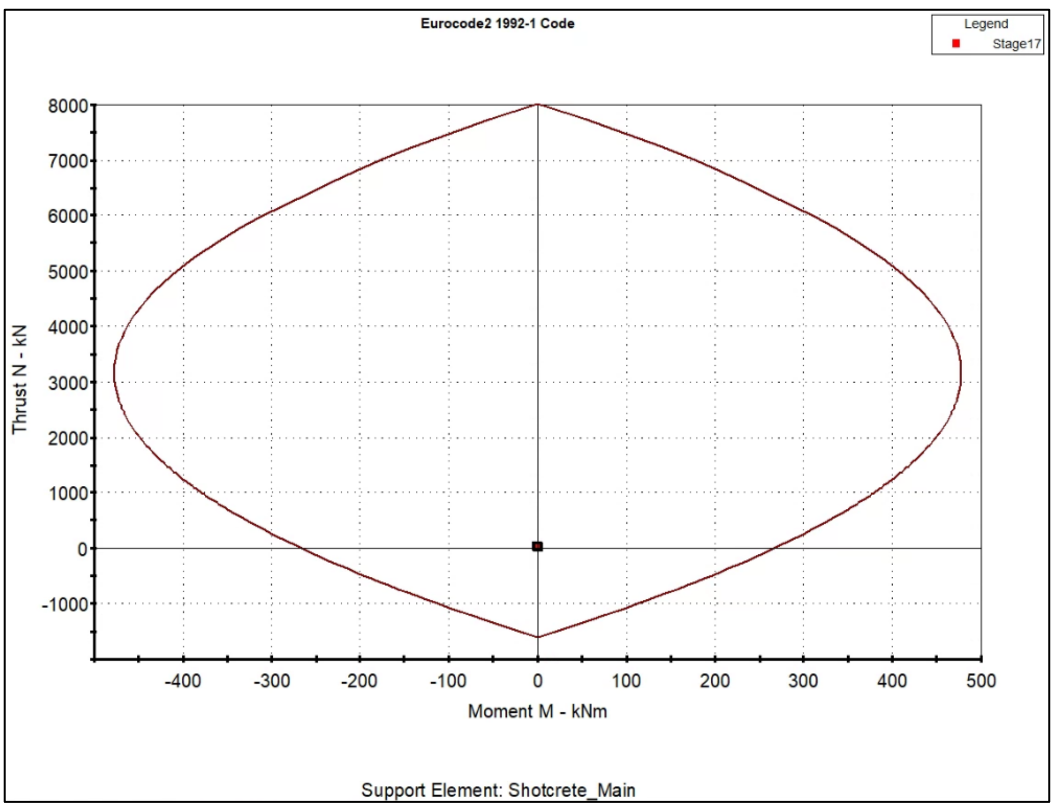

Figure 12. Support capacity plot for the shotcrete lining on the perimeter of the Final Invert area at the Final Invert excavation phase, of main tunnel.

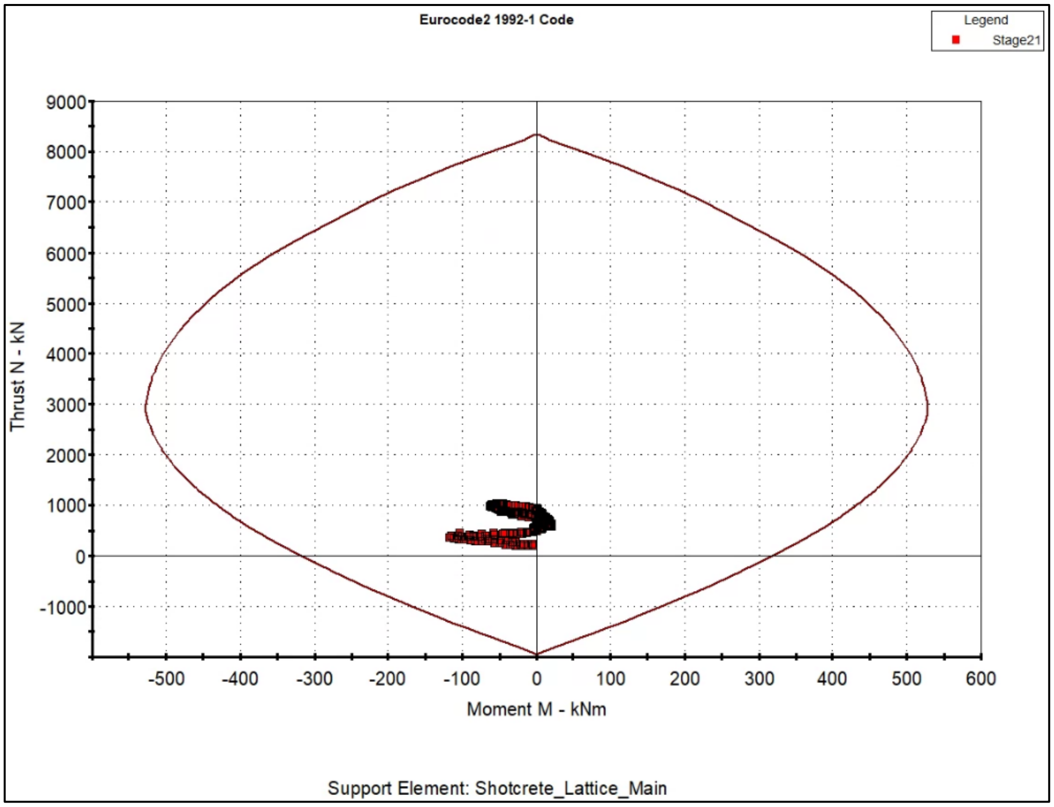

Figure 13. Support capacity plot for the shotcrete lining on the perimeter of the main tunnel area at the Top Heading excavation phase, of ventilation tunnel.

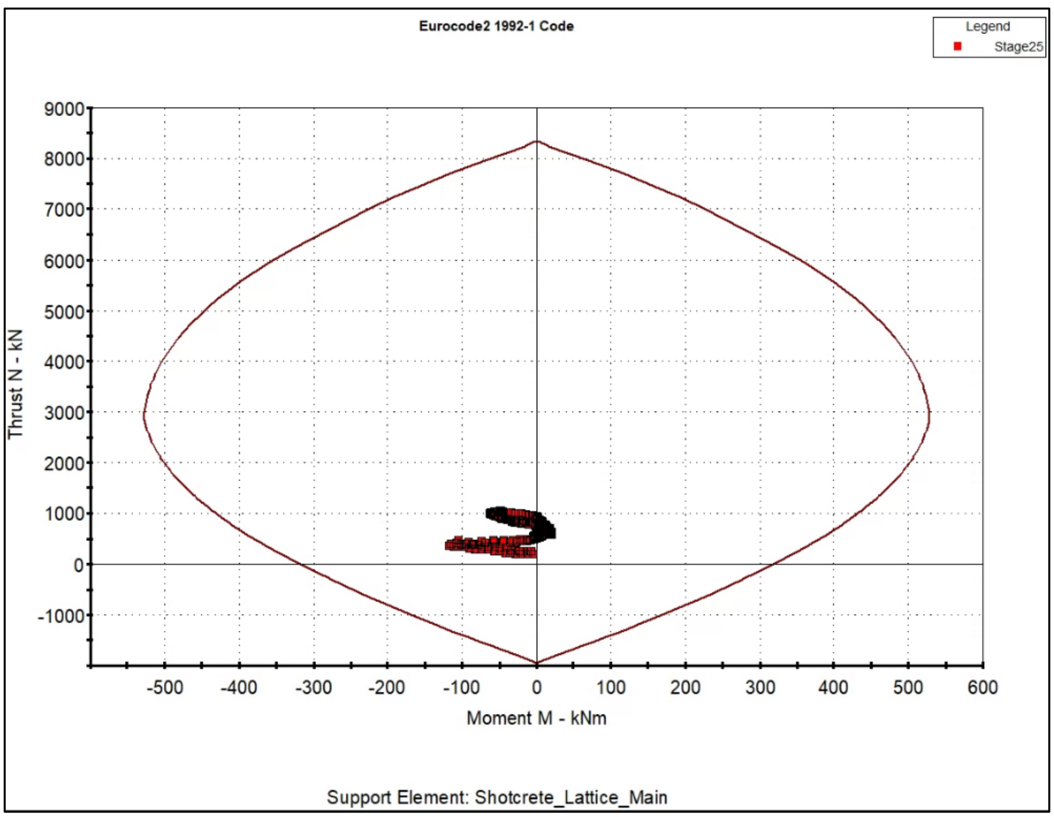

Figure 14. Support capacity plot for the shotcrete lining on the perimeter of the main tunnel area at the Final Invert excavation phase, of ventilation tunnel.

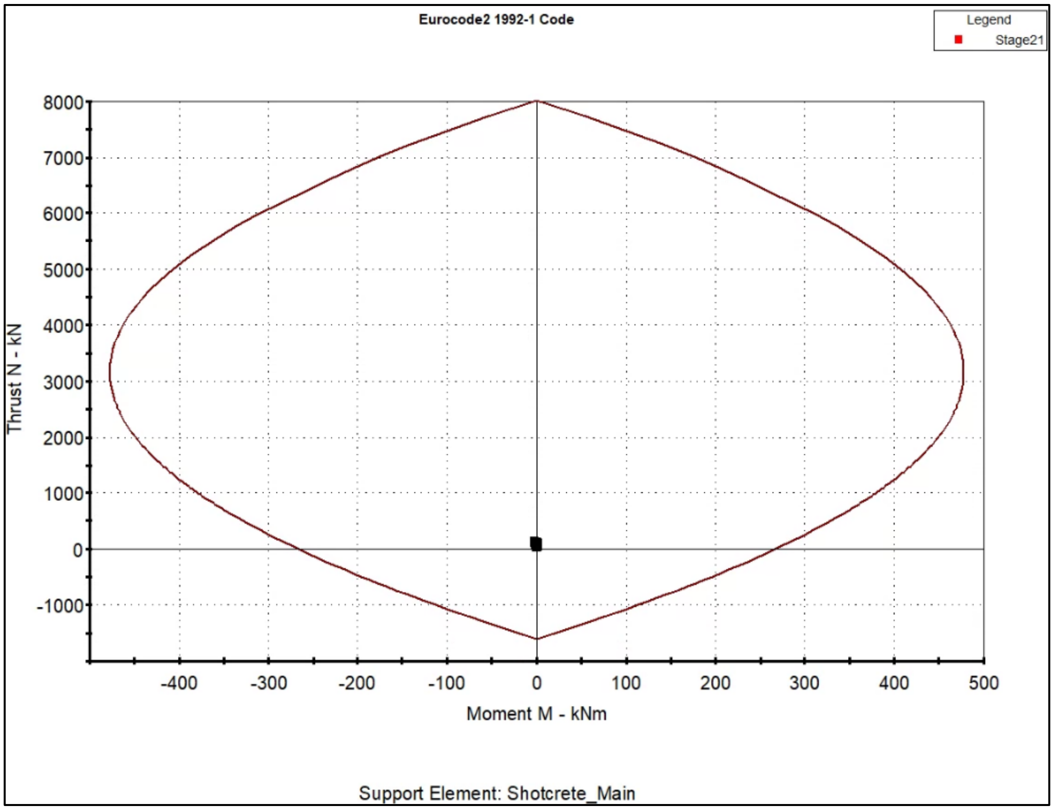

Figure 15. Support capacity plot for the shotcrete lining on the Final Invert of the main tunnel area at the Top Heading excavation phase, of ventilation tunnel.

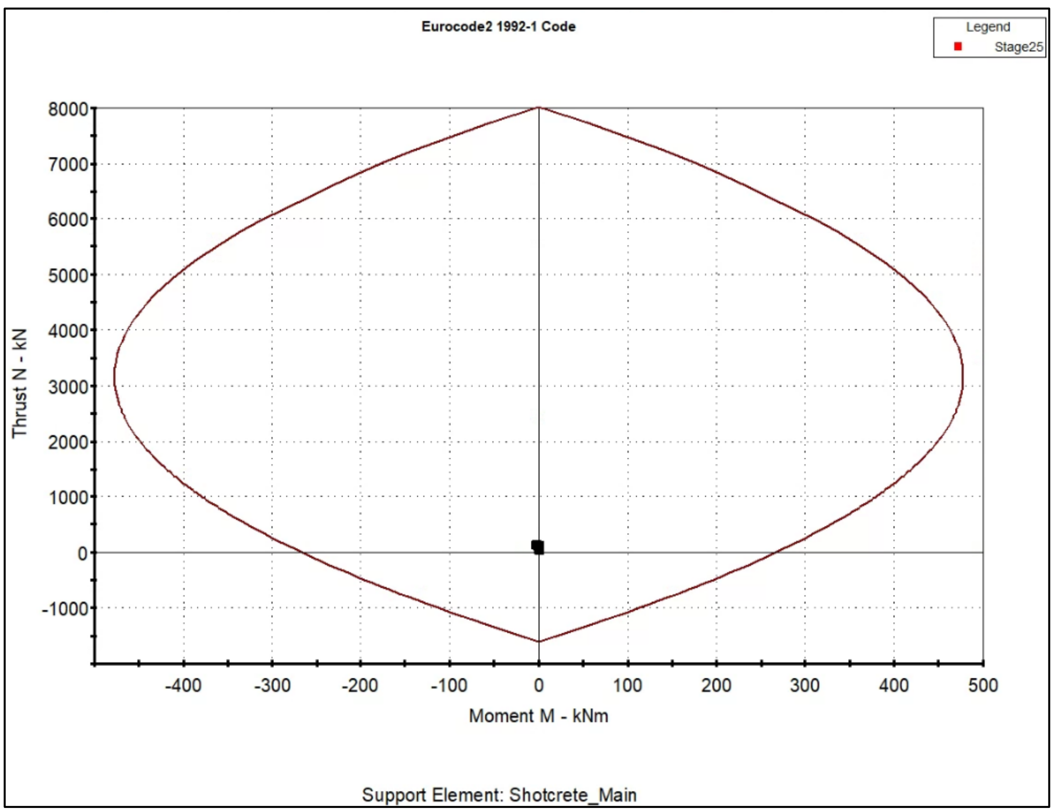

Figure 16. Support capacity plot for the shotcrete lining on the Final Invert of the main tunnel area at the Final Invert excavation phase, of ventilation tunnel.

✓ **Ventilation Tunnel**

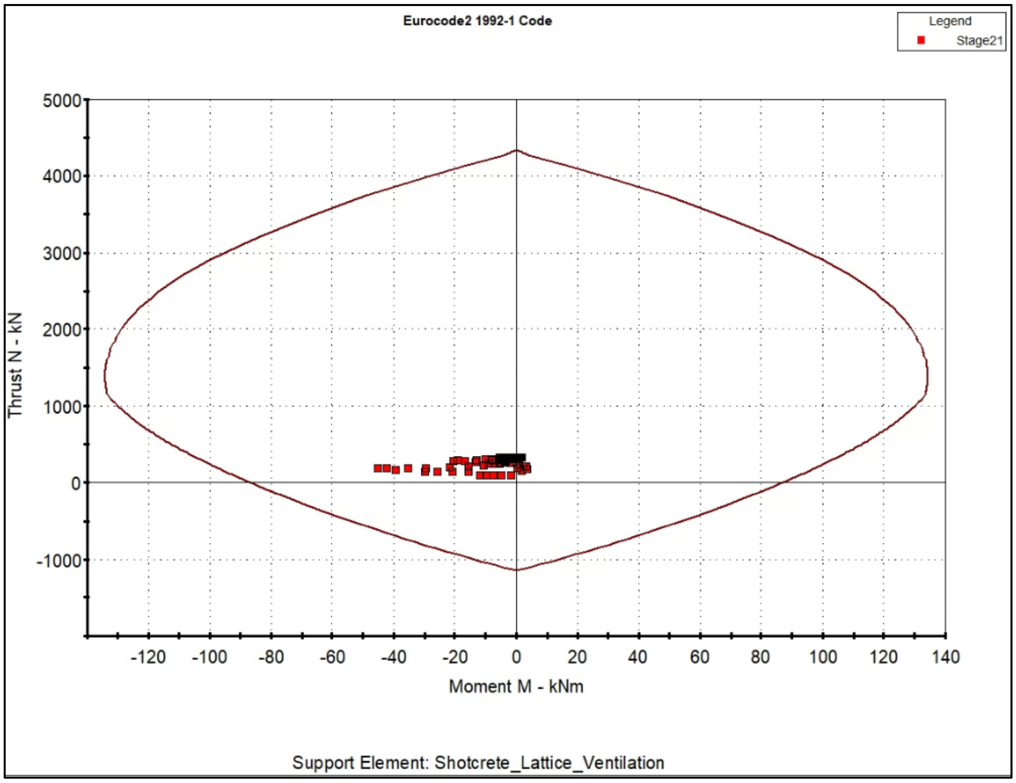

Figure 17. Support capacity plot for the shotcrete lining on the Top Heading area at the Top Heading excavation phase, of ventilation tunnel.

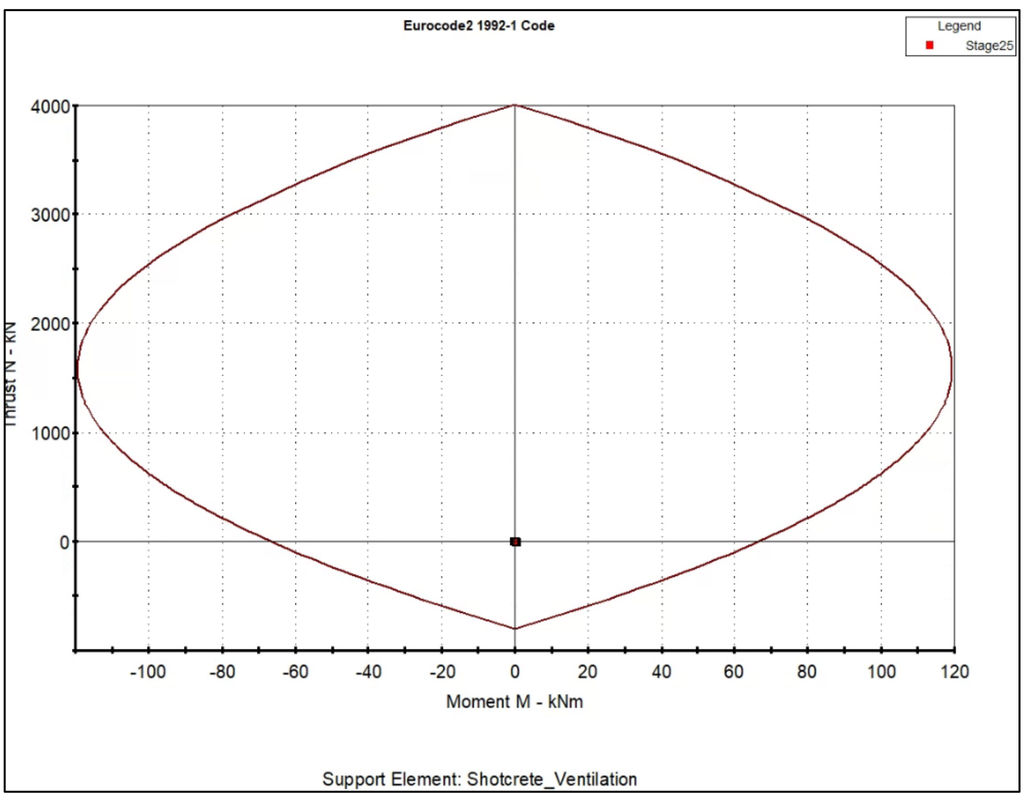

Figure 18. Support capacity plot for the shotcrete lining on the Final Invert area at the Final Invert excavation phase, of ventilation tunnel.

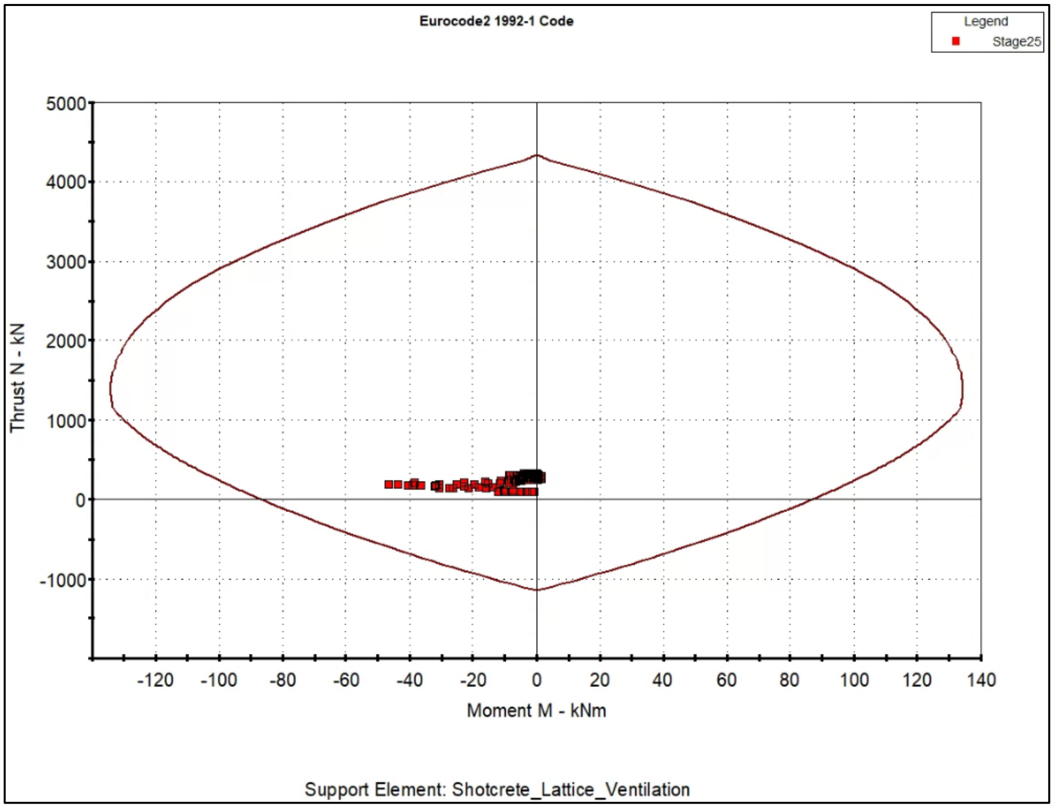

Figure 19. Support capacity plot for the shotcrete lining on the Top Heading area at the Final Invert excavation phase, of ventilation tunnel.

Moreover, as it is presented on the following ***Figure 20,*** the maximal axial force on tunnel rockbolts, is F= 119.6 kN, which means that bolts are not yielded and the safety factor (SF) of bolts under tension loading, taken into account the baring capacity of them P_y= 270 kN, is SF= (270 kN/ 119.6 kN)= **2.26.**

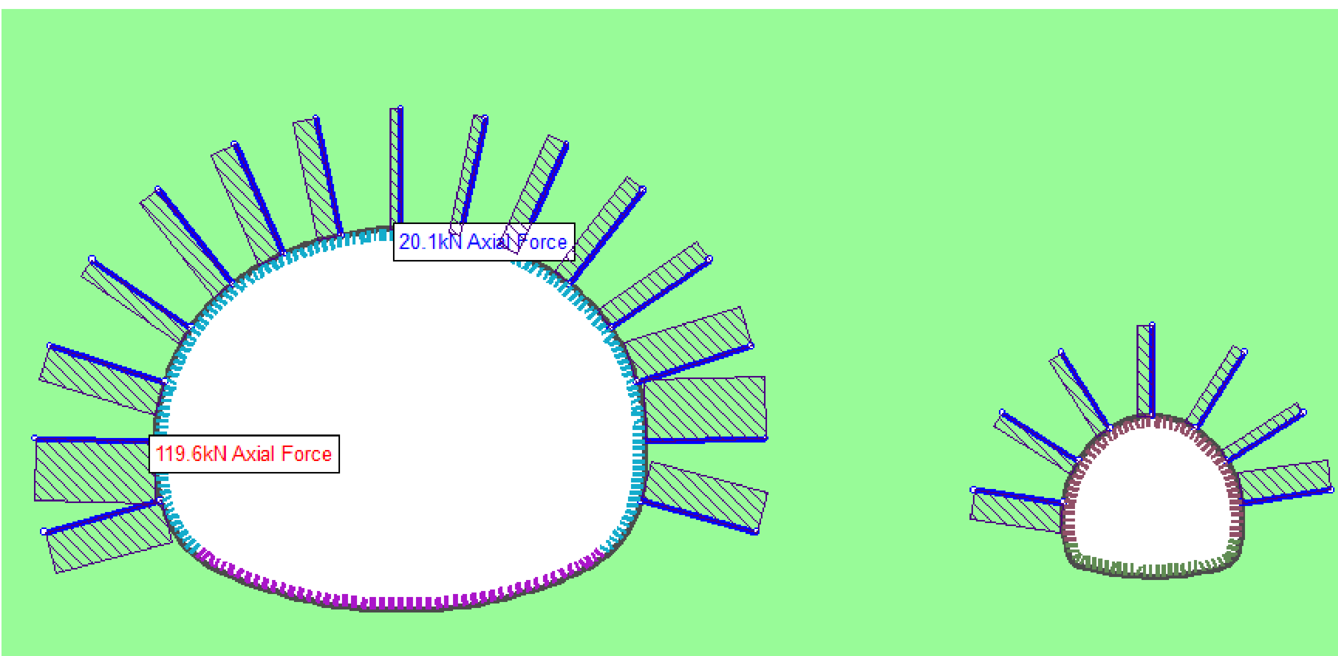

Figure 20. Axial force on tunnel perimeter rockbolts.

3.2. Tunnel displacements

Based on the proposed tunnel primary lining, as described on *Chapter 3.1*, tunnel displacements were recorded on three (3) positions on the main tunnel and on two (2) positions on the ventilation tunnel, as presented on the following ***Figure 21***.

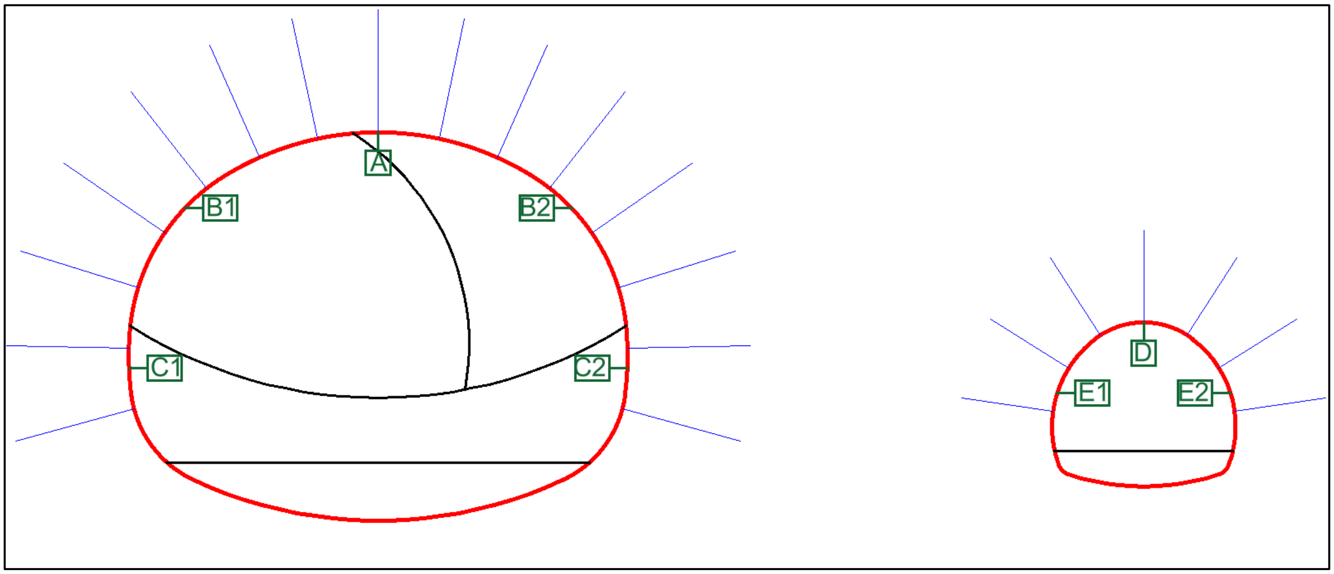

Figure 21. Positions on the tunnel perimeter for recording tunnel displacements.

On the following ***Table 6***, total displacements for the main tunnel and on the following ***Table 7***, total displacements for the ventilation tunnel, are presented for every excavation phase. Moreover, on the following ***Figures 22 & 23***, charts with the increment of the total displacements for both tunnels in every excavation step, are presented.

Table 6. Total displacement measurements on the main tunnel

	Phase	Point A	Point B1	Point B2	Point C1	Point C2
Main Tunnel	Top Heading A	-	7,7 mm	-	-	-
	Top Heading B	21,8 mm	12,6 mm	13,0 mm	-	-
	Bench	26,1 mm	18,3 mm	18,6 mm	8,1 mm	7,9 mm
	Final Invert	26,5 mm	18,7 mm	18,9 mm	8,7 mm	8,3 mm
Ventilation Tunnel	Top Heading	27,1 mm	19,0 mm	19,7 mm	8,9 mm	8,6 mm
	Final Invert	27,2 mm	19,1 mm	19,8 mm	9,0 mm	8,6 mm

Table 7. Total displacement measurements on the ventilation tunnel.

Phase	Point D	Point E1	Point E2
Top Heading	6,4 mm	4,8 mm	3,5 mm
Final Invert	6,9 mm	5,5 mm	4,1 mm

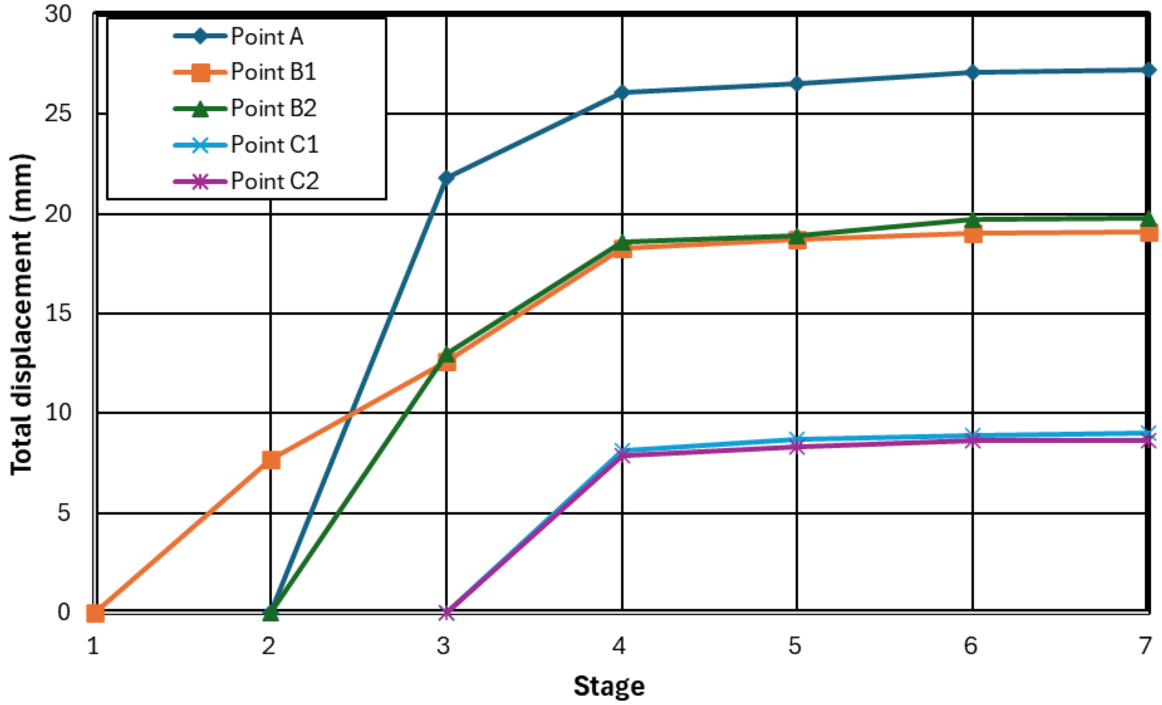

Figure 22. Total displacement chart of main tunnel (Stage 2: Top Heading A – Main Tunnel, Sage 3: Top Heading B – Main Tunnel, Sage 4: Bench – Main Tunnel, Sage 5: Final Invert – Main Tunnel, Sage 6: Top Heading –Ventilation Tunnel, Sage 7: Final Invert –Ventilation Tunnel).

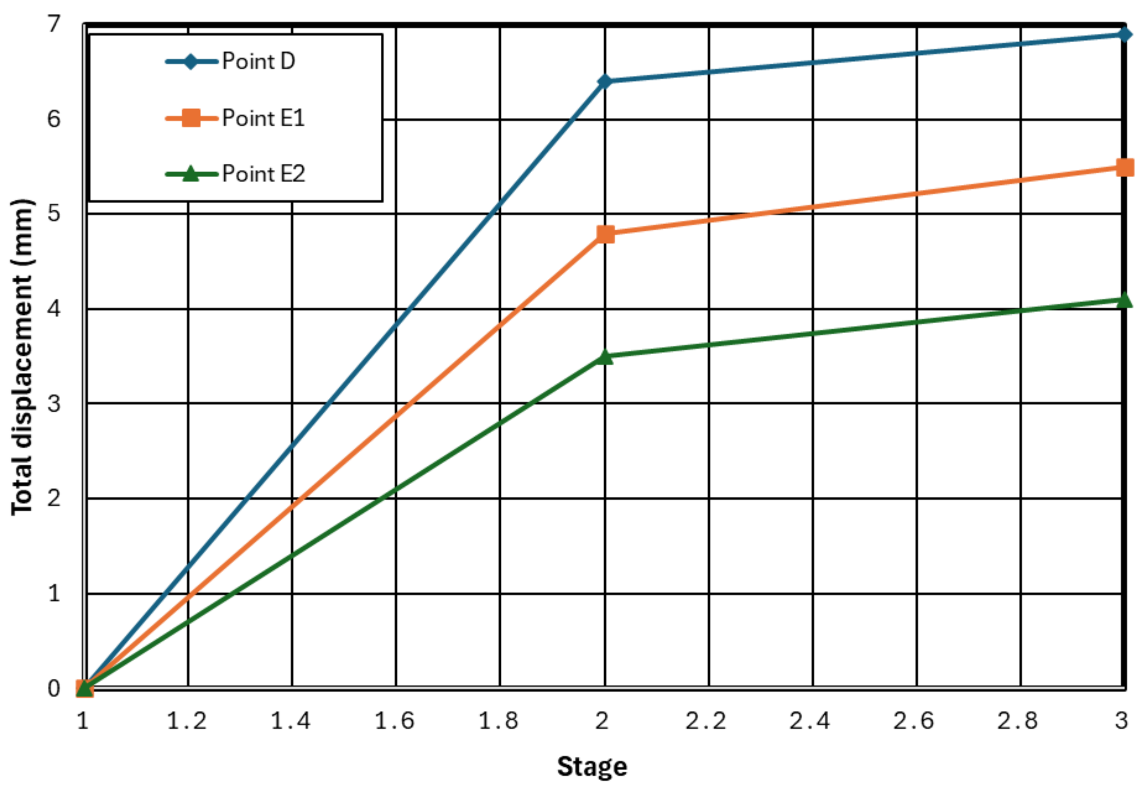

Figure 23. Total displacement chart of ventilation tunnel (Sage 1: Top Heading –Ventilation Tunnel, Sage 2: Final Invert – Ventilation Tunnel).

From the previous ***Table 6*** and ***Figure 23***, it is obvious that it is an interaction between the two tunnels (main and ventilation), as the construction of the ventilation tunnel, causes an increment on the total displacements of the main tunnel. Specifically, the additional displacements on the main tunnel due to the interaction between the two tunnels, is about 1 mm.

On the following ***Figures 24 & 25***, the formation of the total displacements around the excavations, are presented.

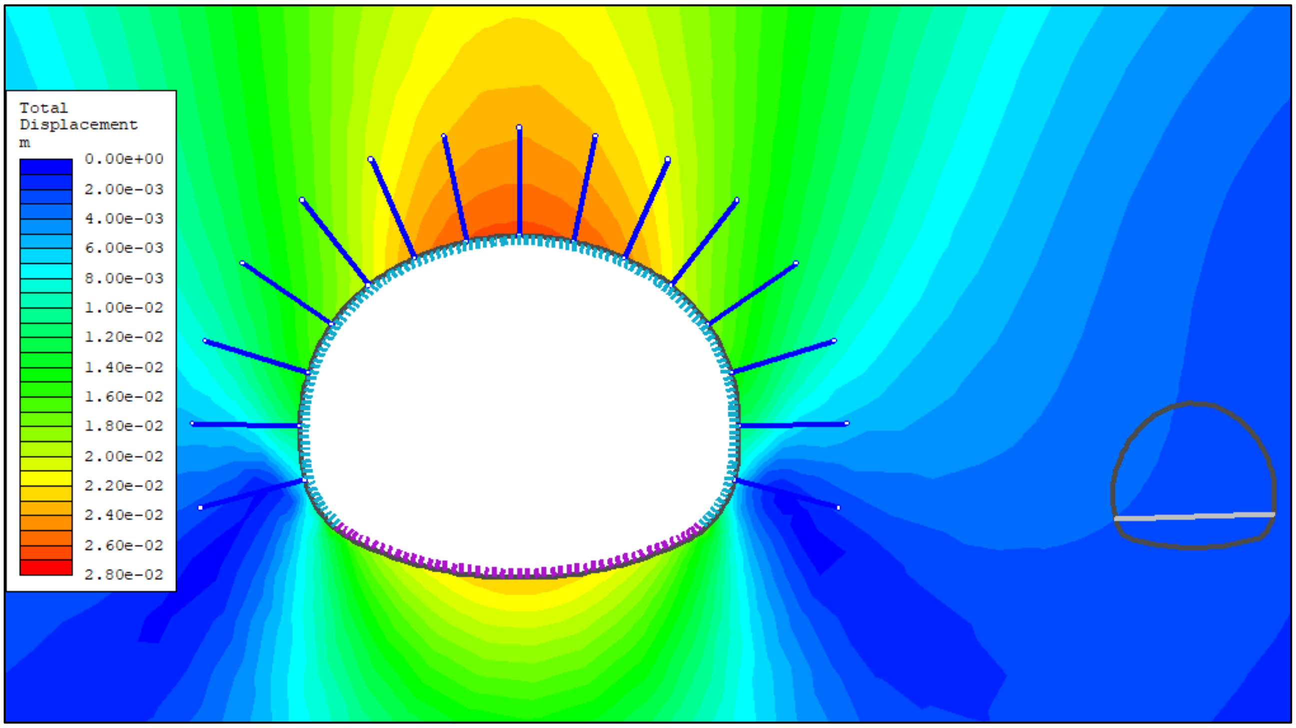

Figure 24. Total displacements after the construction of the main tunnel.

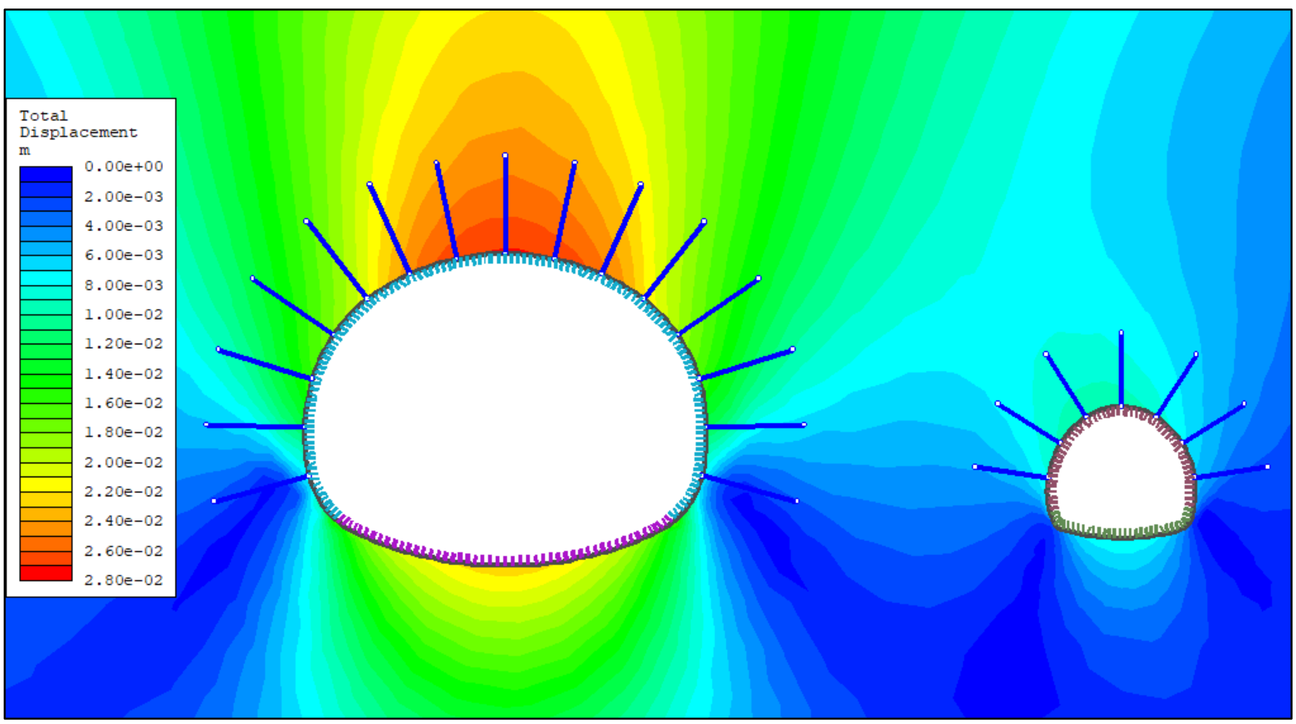

Figure 25. Total displacements after the construction of the ventilation tunnel.

3.3. Surficial settlements

Based on the proposed tunnel primary lining, as described on *Chapter 3.1*, the maximal surficial settlement (ground vertical displacement) due to the tunnels excavation, is **s= 19,6mm** and characterized as acceptable surficial settlement according to the limitation of maximal acceptable settlement of 30mm. On the following ***Figure 26,*** is presented the surficial settlement profile for every excavation step.

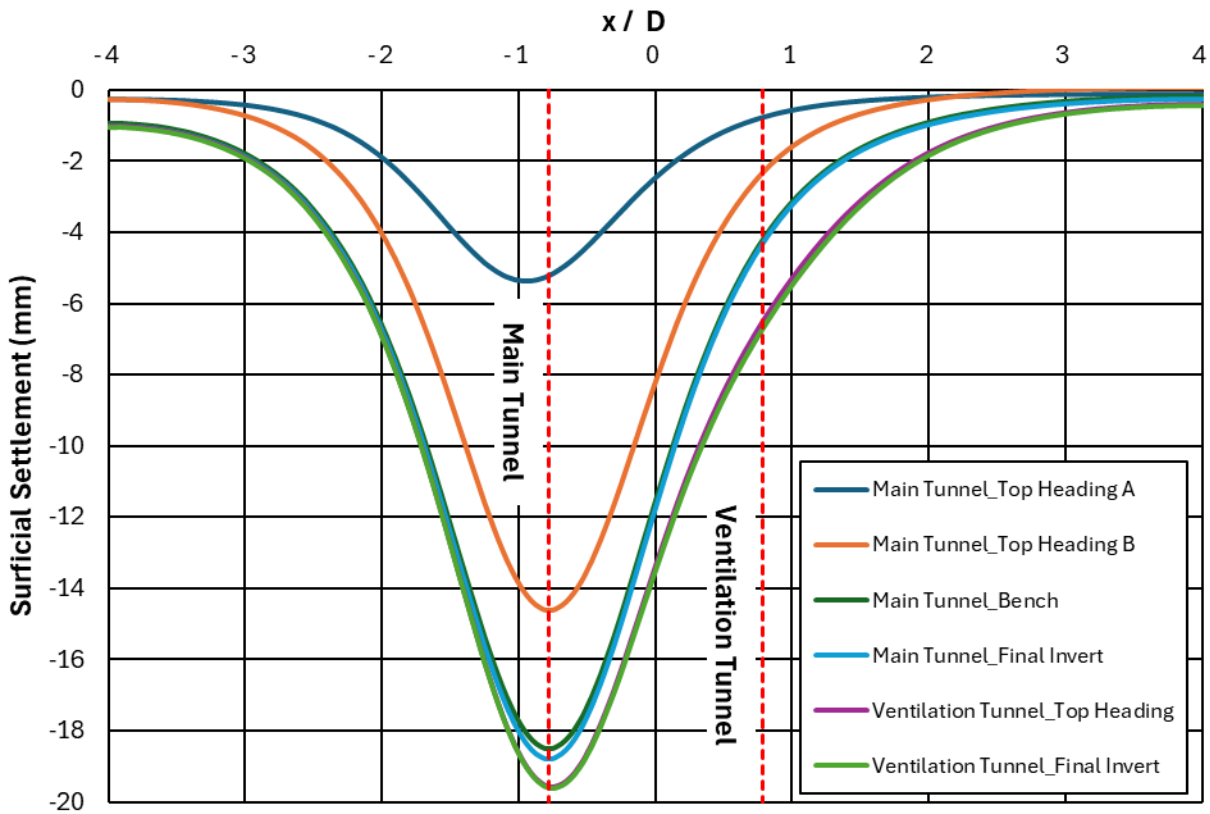

Figure 26. Surficial settlement profile.

3.4. Plastic zone

Based on *Tables 4 & 5*, as the tunnel overload factor Ns= 2,66 > 1, a plastic zone around the excavation is expected to be formed. On the following ***Figure 27***, the formation of yield – plastic zone around the excavations due to the tunnel construction, is presented. The maximal length of plastic zone, measured from the excavation perimeter, for the main tunnel is estimated **L= 9,2m**, and for the ventilation tunnel is estimated **L= 3,7m.**

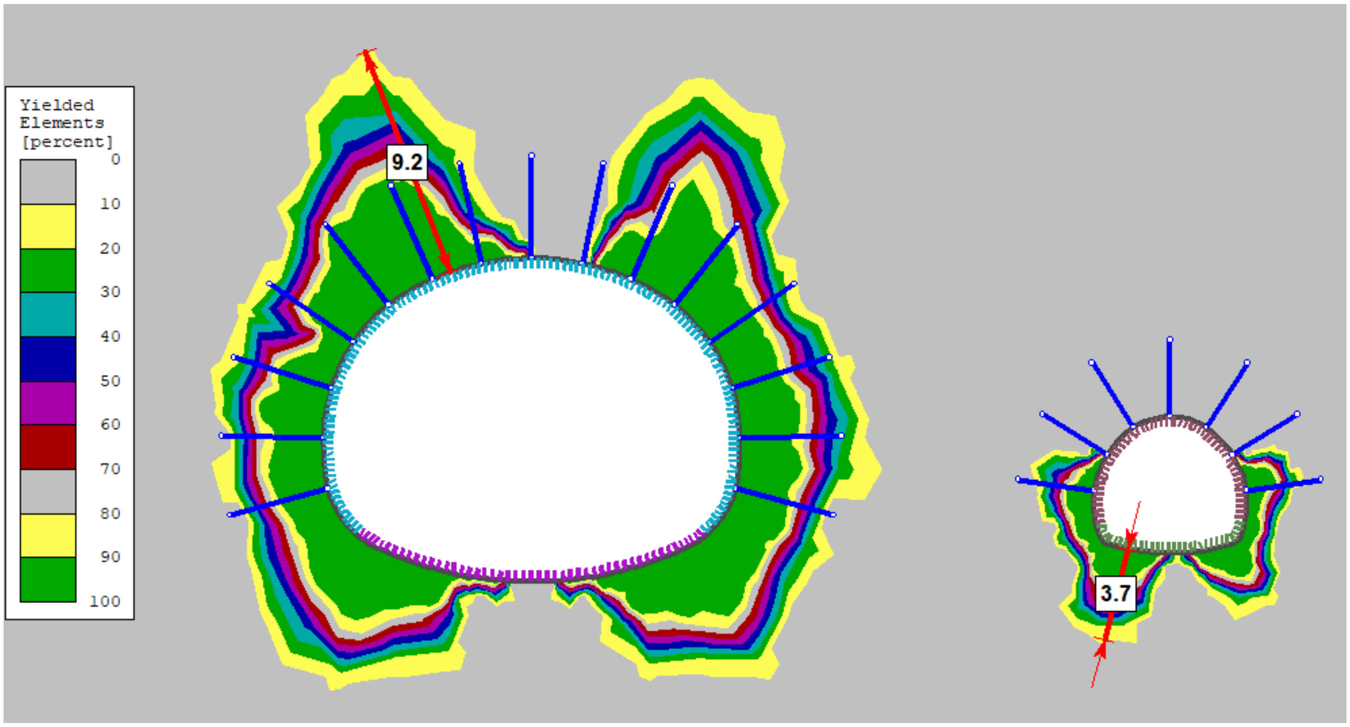

Figure 27. Plastic zone formation on the final excavation stage for both tunnels.

3.5. Lining Data

Based on the proposed primary lining design, on the following ***Table 8***, calculated liner values, are presented for both tunnels.

Table 8. Liner data values.

Parameter	Main Tunnel	Ventilation Tunnel
Axial Force (N)	940,1 kN	53 kN
Bending Moment (M)	28,6 kNm	14,6 kNm
Shear Force (Q)	50,6 kN	27,1 kN

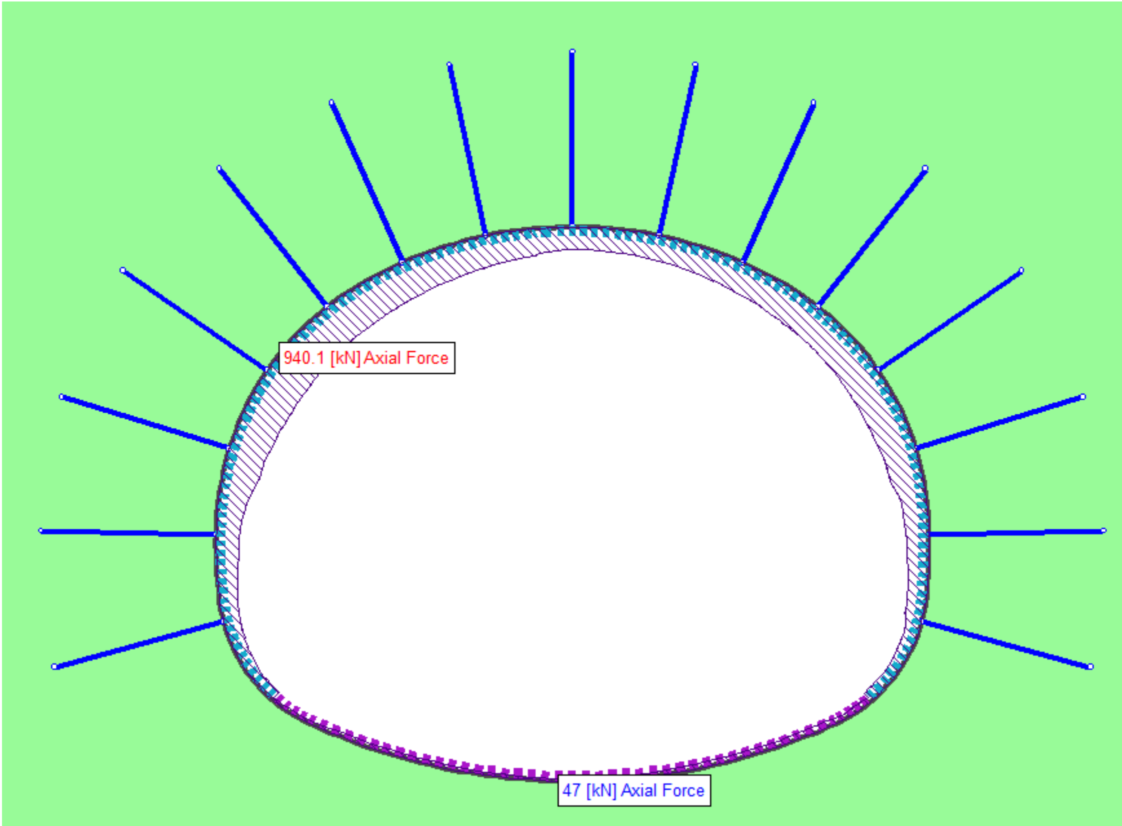

Figure 28. Liner axial force (N) of the main tunnel.

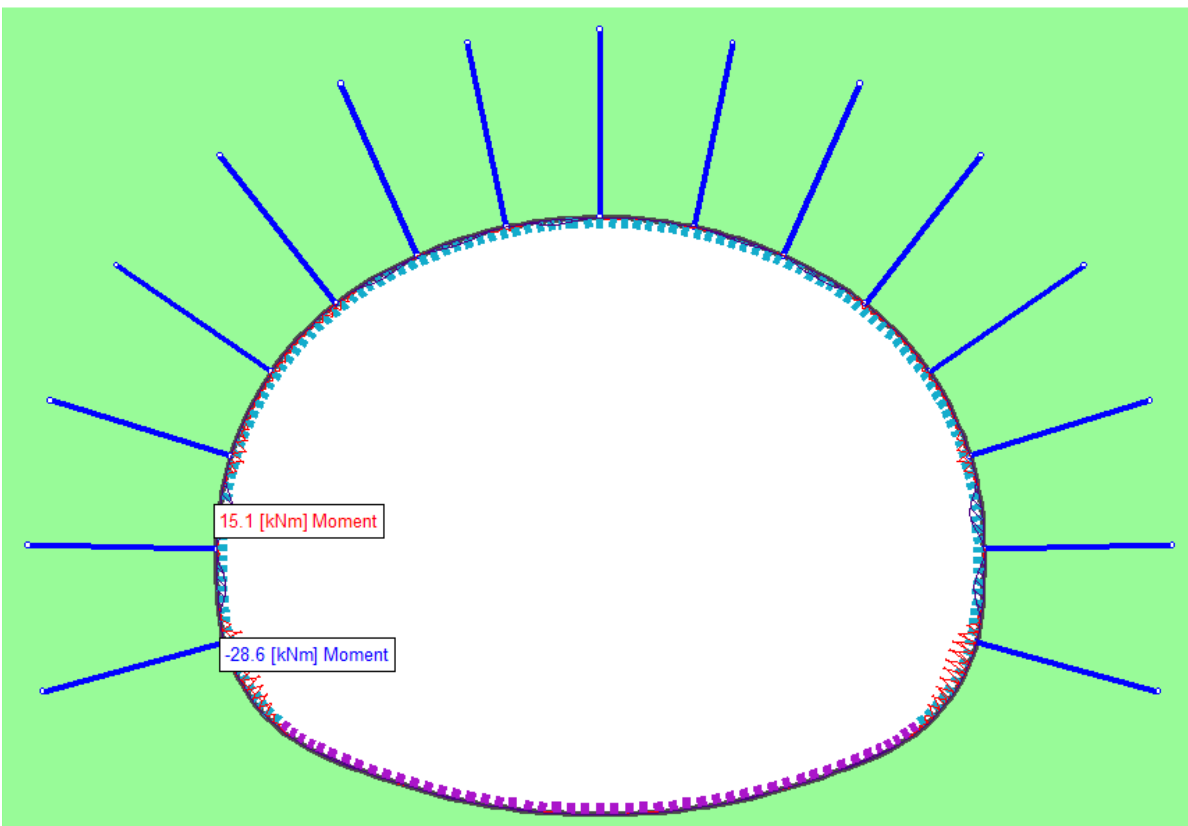

Figure 29. Liner bending moment (M) of the main tunnel.

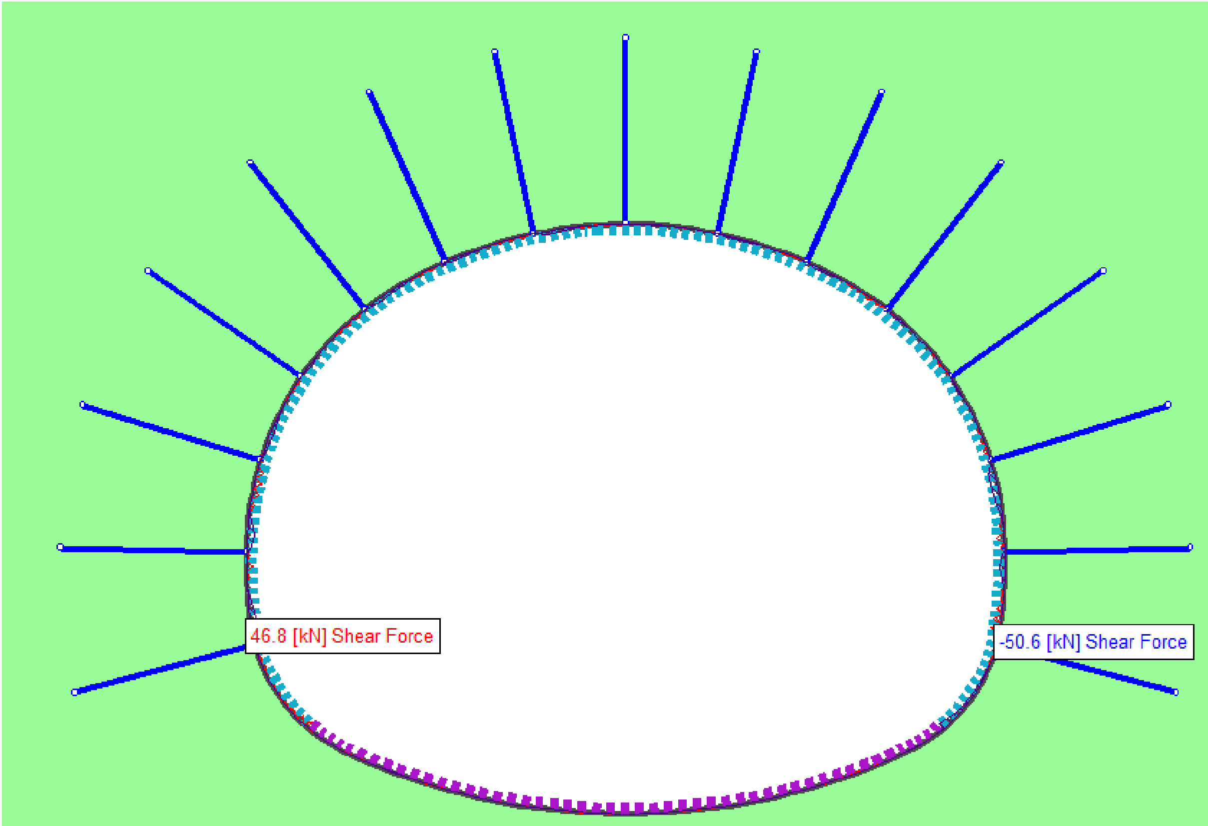

Figure 30. Liner shear force (Q) of the main tunnel.

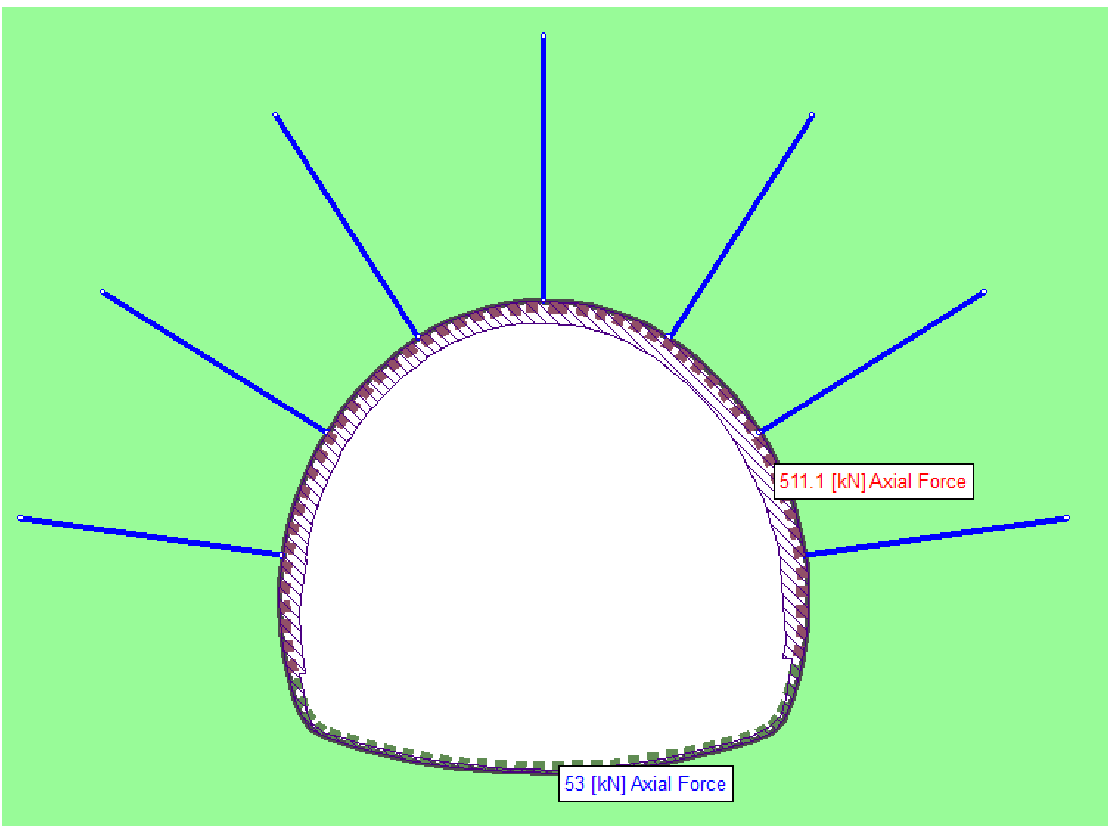

Figure 31. Liner axial force (N) of the ventilation tunnel.

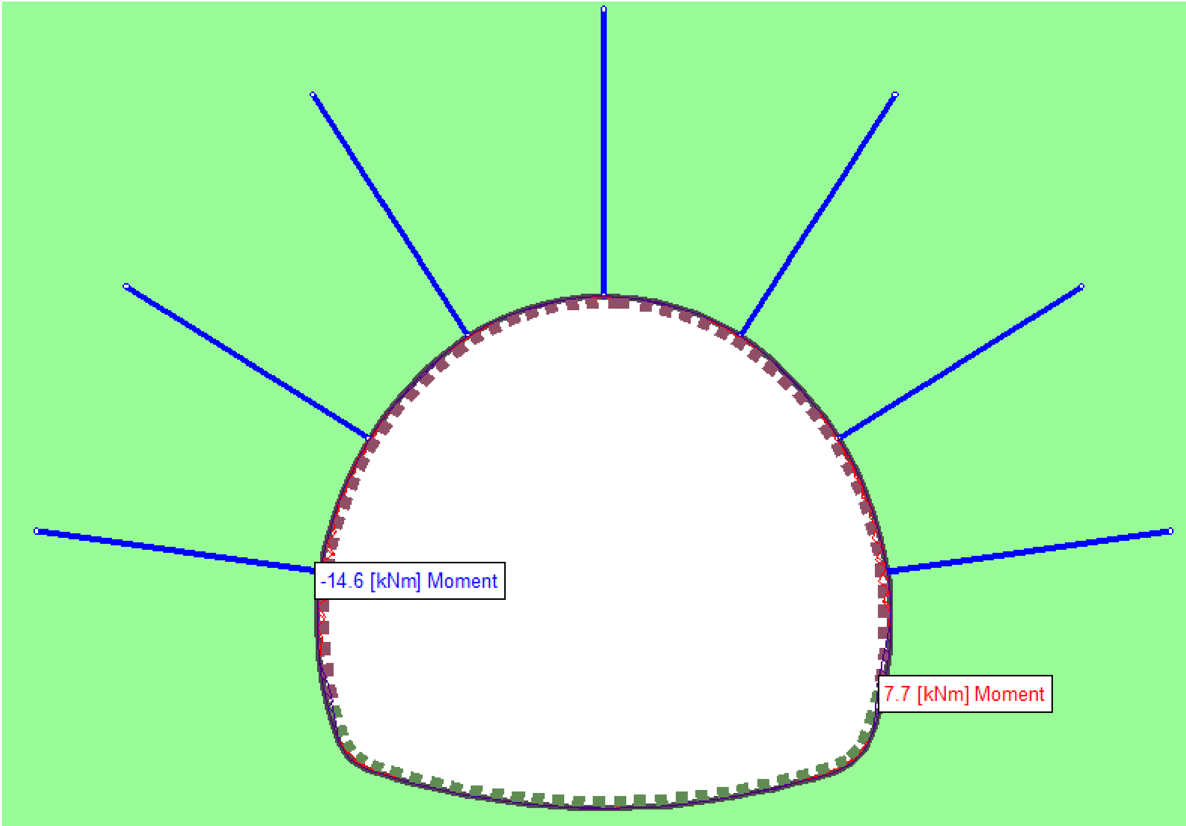

Figure 32. Liner bending moment (M) of the ventilation tunnel.

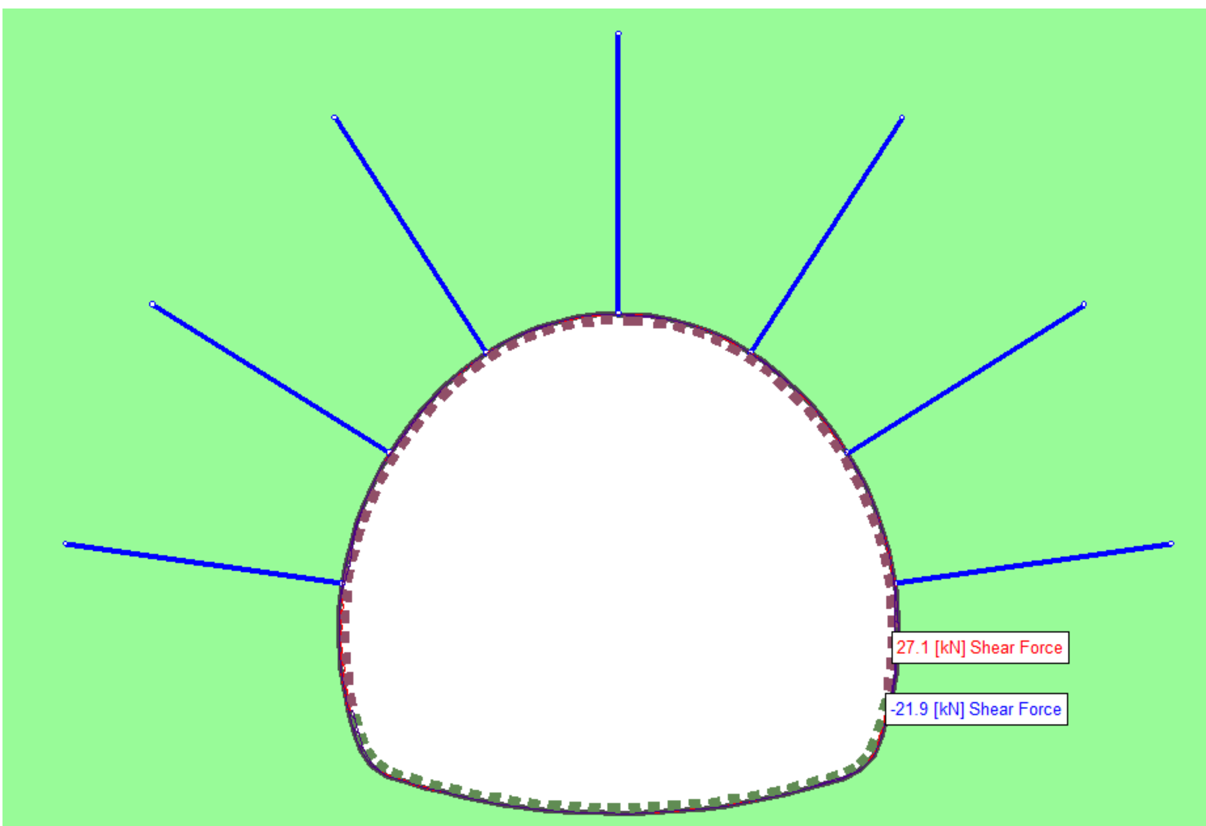

Figure 33. Liner shear force (Q) of the ventilation tunnel.

3.6. Face Stability

For the tunnel primary lining design of the main tunnel, the tunnel face stability factor of the unsupported tunnel face (FS_o), is calculated as follow:

$$\boldsymbol{FS_o = \frac{2}{(1-\lambda)\times N_s}}$$

, where λ: is the deconfinement factor on the tunnel face position and N_s: is the overload factor.

According to ***Table 4***, λ= 0,37 and N_s= 2,66

Thus, **FS_o= 1,19**, which means that the tunnel face stability it is not acceptable, as the minimal tunnel face stability factor could be > 1,5.

In order to ensure the minimal acceptable tunnel face stability factor (FS) ≥ 1,5, tunnel face reinforcement by fiberglass nails, is selected.

The tunnel face stability factor after the tunnel face reinforcement by fiberglass nails, can be calculated as follow:

$$\boldsymbol{FS_{FG} = FS_o + \frac{1}{(1-\lambda)}\times\left(\frac{\sigma_3}{p_o}\right)\times tan^2\left(45^\circ+\frac{\varphi}{2}\right)}$$

, where λ: is the deconfinement factor on the tunnel face position, σ_3: is the horizontal stress on the tunnel face due to the tunnel face reinforcement, p_o: is the geostatic field stress and ϕ: is the rockmass friction angle.

Taken into account the minimal acceptable tunnel face stability factor FS_{FG} ≥ 1,5, the appropriate horizontal stress on the tunnel face due to the tunnel face reinforcement by fiberglass nails, is **σ_3= 22,9KPa**.

The horizontal stress (σ_3), is correlated with the number of tunnel face fiberglass nails (n), with the following formula:

$$\boldsymbol{\sigma_3 = \frac{n\times F_y}{FS_F\times A}}$$

, where F_y: is the tensile capacity of each nail, FS_F: is the material safety factor of nails and A: is the tunnel face reinforcement area.

Using fiberglass nails, with F_y= 280 kN, FS_F= 1,2 in a tunnel face area A= 104,9 m^2, the appropriate number of fiberglass nails, is **n= 11**, which means tunnel face reinforcement density **ρ= 0,1 nail /m^2**.

According to Rankine's theory, the tunnel face failure prism (wedge) angle from the horizontal surface, can be estimated as follow: β= 45° + ϕ/2= 45° + 35°/2 → **β= 62,5°**. Thus the maximum distance (X_{max}) of the failure prism in front of the tunnel face, can be calculated as follow, taken into account the tunnel face height (h) of the top heading area, where h= 8m (half of the tunnel diameter):

X_{max}= h/tanβ= 8m/tan(62,5°)= **4,2m**.

Thus, the minimal overlap distance ($L_{overlap}$), of the fiberglass nails could be 4,2m and selected **$L_{overlap}$=4,5m**.

On the following ***Figure 34***, is presented the proposed tunnel face reinforcement by fiberglass nails.

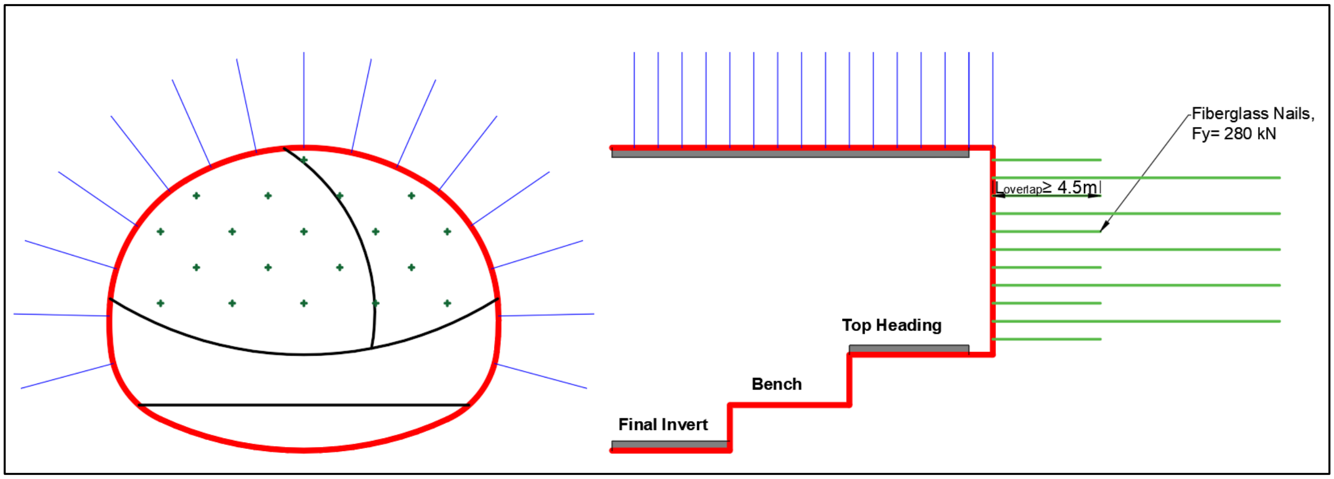

Figure 34. Proposed tunnel face stability reinforcement by fiberglass nails.

3.7. <u>Proposed tunnel primary lining</u>

The proposed tunnel primary lining and the excavation phases for both tunnels, based on the numerical analyses, are presented on the following tables.

Table 9. Tunnel primary lining, for main tunnel.

Excavation Phase	Excavation advance length	Shotcrete	Wire mesh	Bolts	Steel sets	Tunnel face pre - support
Top Heading A	1 m	C30/37, t=15 cm	3 layers T188	5 fully bonded rockbolts Φ25, L= 2m, s= 1m, every 2excavation's length	Lattice girder 4bar-140,Φ26, every excavation length	11 Fiberglass Nails, F_y=280kN, $L_{overlap}$≥ 4,5m
Top Heading B	1 m	C30/37, t=15 cm	3 layers T188	6 fully bonded rockbolts Φ25, L= 2m, s= 1m, every 2excavation's length	Lattice girder 4bar-140,Φ26, every excavation length	
Temporary Invert	1 m	C30/37, t=15 cm	3 layers T188	-	-	-
Bench	1 m	C30/37, t=15 cm	3 layers T188	4 fully bonded rockbolts Φ25, L= 2m, s= 1m, every 2excavation's length	Lattice girder 4bar-140,Φ26, every excavation length	-

Excavation Phase	Excavation advance length	Shotcrete	Wire mesh	Bolts	Steel sets	Tunnel face pre - support
Final Invert	1 m	C30/37, t=15 cm	3 layers T188	-	-	-

Table 10. Tunnel primary lining, for ventilation tunnel.

Excavation Phase	Excavation advance length	Shotcrete	Wire mesh	Bolts	Steel sets
Top Heading	1 m	C30/37, t=15 cm	2 layers T188	7 fully bonded rockbolts Φ25, L= 2m, s= 1m, every 2excavation’s length	Lattice girder 4bar-140,Φ26, every excavation length
Final Invert	1 m	C30/37, t=15 cm	2 layers T188	-	-