Solution for Problem Set 4

1. Scope of work

The scope of the present report, is the presentation of the analytical design of a primary support for a tunnel which will be excavated by a conventional method (NATM), below the ground surface for the new Line 4 of Athens Metro.

The tunnel area, is located between "Panepistimio" station and the "Amerikis" trumpet tunnel, as shown in the following ***Figure 1***.

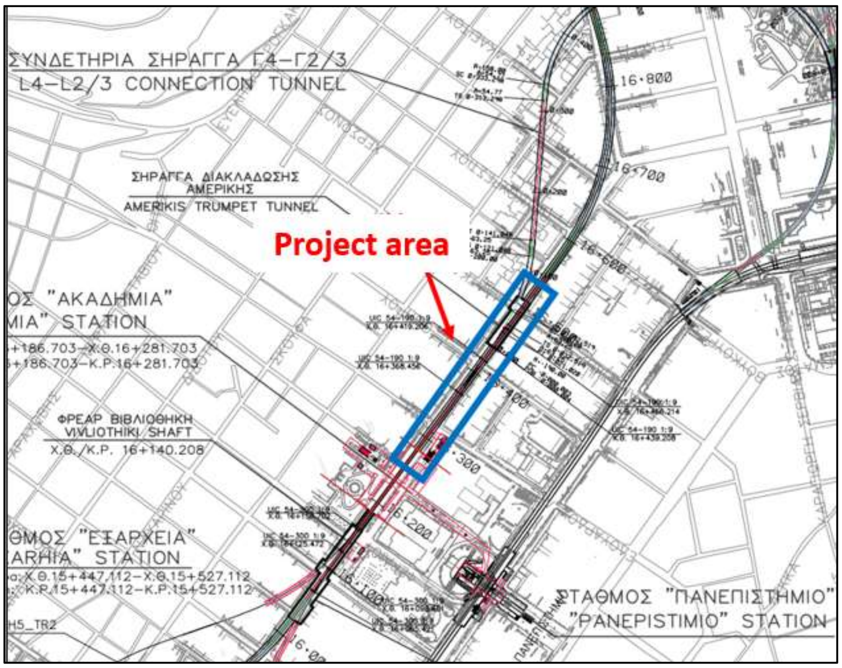

Figure 1. Project area.

The tunnel equivalent diameter D= 10m and will be excavated in the following tree (3) excavation phases:

a) Top Heading
b) Bench
c) Final Invert

Multiple tunnel phase excavation selected, due to the poor ground conditions and the low tunnel overburden height (H), as it is about 20m, measured from the tunnel axis.

The geological layer on the tunnel area, are alluvial deposits (al) for the first 3m below the ground surface and then the weak Athens Schist rockmass formation (sch), where the tunnel section is located.

Tunnel design based on a two – dimensional (2D) numerical analysis, using the RS2 – Rocscience software and the equivalent deconfinement factor (λ) method, 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 and the limitation of the maximal acceptable surficial settlement bellow 30mm.

2. Numerical analysis

2.1. Software

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

2.2. Numerical model

The numerical model limits, are based on the tunnel equivalent diameter (D). The side external limits of the numerical model, placed four (4) tunnel diameters (4 x D) from the tunnel axis, the upper limit 20m above the tunnel axis (equal to the tunnel overburden height) and the bottom limit, placed two (2) tunnel diameters (2 x D) from the tunnel axis. On the following ***Figure 2***, the numerical model is presented. The model separated in two soil layers, where the upper soil layer (al) has 3m thickness (measured from the ground surface) and the lower one is the rockmass layer (sch).

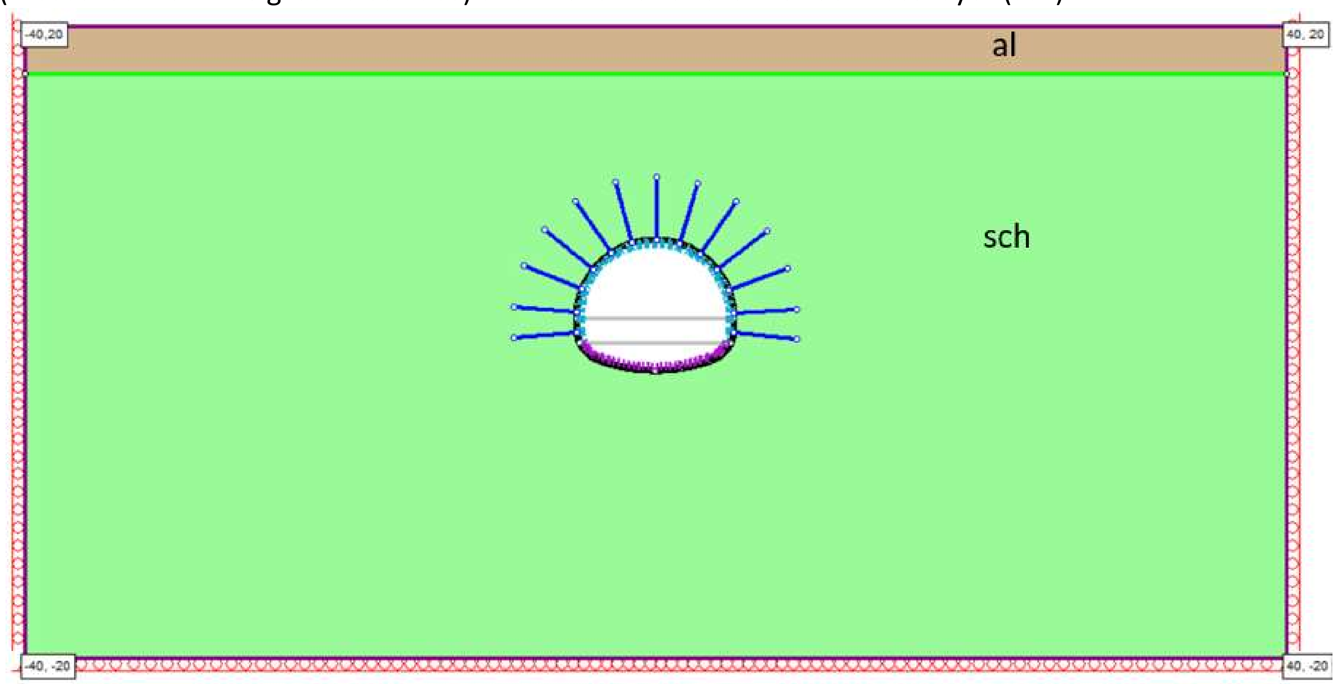

Figure 2. Numerical model.

2.3. Mesh

The numerical model mesh, consist of three nodded triangular elements. The total numerical model elements, are 8840 and the total numerical model nodes, are 4874.

2.4. Geotechnical parameters

The two ground layers (al and sch), simulated with elastoplastic behavior, using the Mohr Coulomb failure criterion. On the following table the geotechnical input parameters for the two ground layers, are presented.

Table 1. Geotechnical parameters of ground layers.

Parameter	al	sch
Ground unit weight (γ)	23 kN/m^3	26 kN/m^3
Cohesion (c)	10 KPa	50 KPa
Friction angle (φ)	25°	35°
Soil / Rockmass modulus (E_m)	30 MPa	350 MPa
Poisson ratio (v)	0.3	0.25
Dilation angle (δ)	6.25°	8.75°

A plain strain analysis used, taken into account a geostatic field stress loading, with horizontal stress (K_o), 0.8.

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

2.5. Support parameters

Tunnel primary lining, consist of shotcrete lining, steel sets and rockbolts. Shotcrete lining and steels sets, 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, the tunnel support parameters are presented, for each tunnel support type.

Note that the tunnel lining, was simulated with full values of thickness and strength, after each installation stage.

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
Tensile capacity (F_{tk})	270 kN
Elastic modulus (E_{steel})	200 GPa
Length (L)	4 m
Pattern	1 x 1.5 m (longitudinal x radial)
Steel sets	
Type	HEB 120
Elastic modulus (E_{steel})	200 GPa
Longitudinal spacing	1 set every excavation advance length

2.6. Simulation stages

Tunnel numerical analysis, was based on a two-dimensional (2D) analysis, using the deconfinement method 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 two steps, where on the first one the excavation step deconfinement was simulated and on the second one, the installation of the primary support was simulated. Moreover, the first simulation stage of the numerical analysis, was the geostatic stage, where was the initial condition of the ground without the tunnel construction.

On the following table, the numerical analysis stages are presented.

Table 3. Numerical analysis, simulation stages.

Stage	Description
1. Geostatic	Simulation of the initial ground conditions, without tunnel construction
2. Deconfinement Top Heading	Relaxation – deconfinement of the Top Heading area with ground excavation and simultaneously adding of internal load, based on the deconfinement factor (λ).
3. Support Top Heading	Installation of the Top Heading area support, without internal load.
4. Deconfinement Bench	Relaxation – deconfinement of the Bench area with ground excavation and simultaneously adding of internal load, based on the deconfinement factor (λ).
5. Support Bench	Installation of the Bench area support, without internal load.
6. Deconfinement Invert	Relaxation – deconfinement of the Invert area with ground excavation and simultaneously adding of internal load, based on the deconfinement factor (λ).
7. Support Invert	Installation of the Inver area support, without internal load.

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:

$\mathbf{p = (1-\lambda) \times p_o}$, 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, as it is presented on the following Figure.

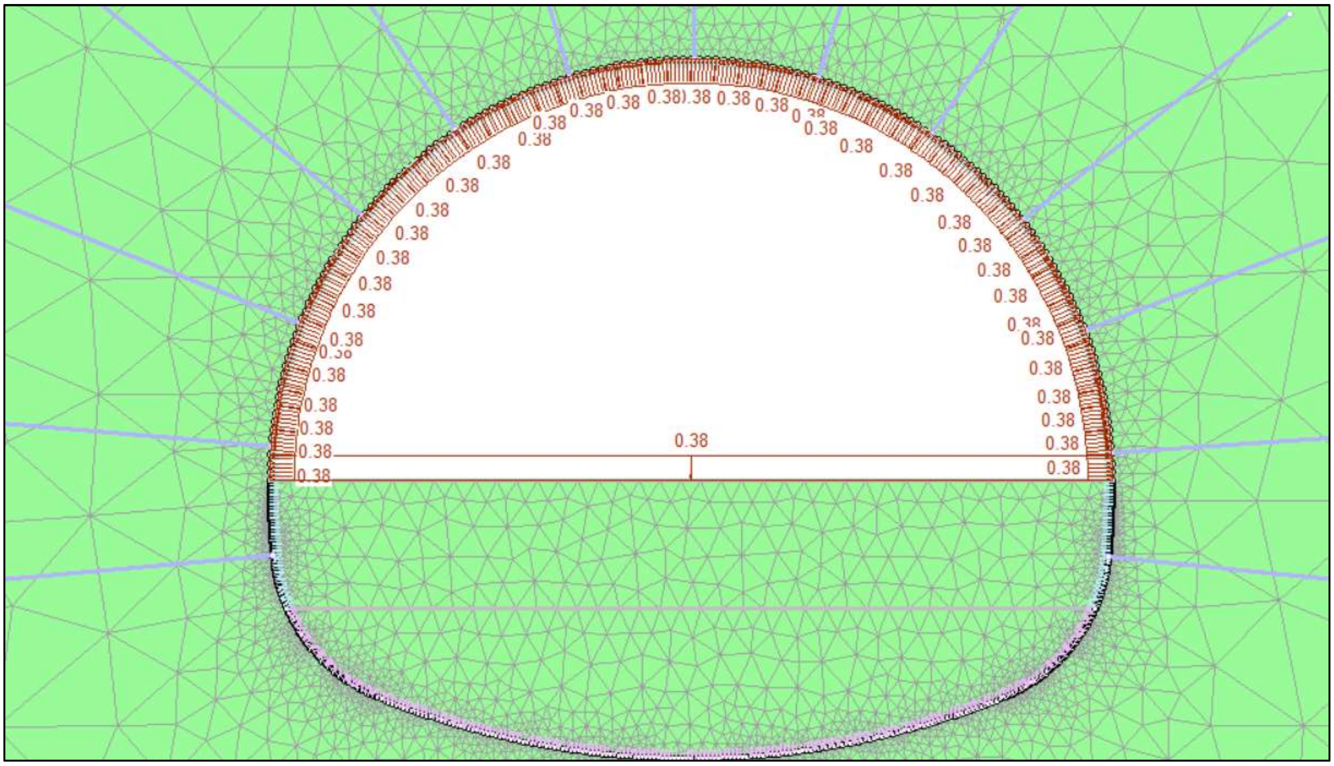

Figure 3. Simulation of the ground deconfinement.

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: **β= (1-λ)**.

On the following Figures, the convergence confinement curve and the tunnel longitudinal displacement profile (LDP) for the rockmass unit (sch), are presented. Also, on the following Table characteristics values from the convergence – confinement curve and the deconfinement ratio (λ) on the position of the tunnel support installation (x= -1m, equal to the tunnel face advance length), are presented.

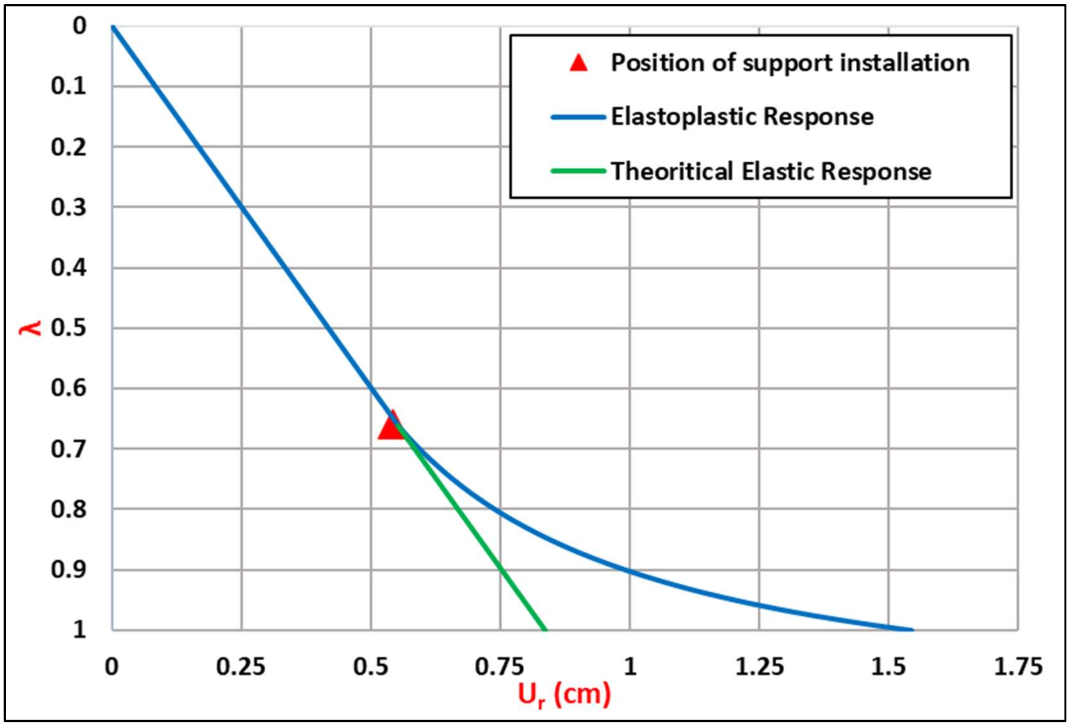

Figure 4. Ground convergence – confinement curve, for the ground unit (sch).

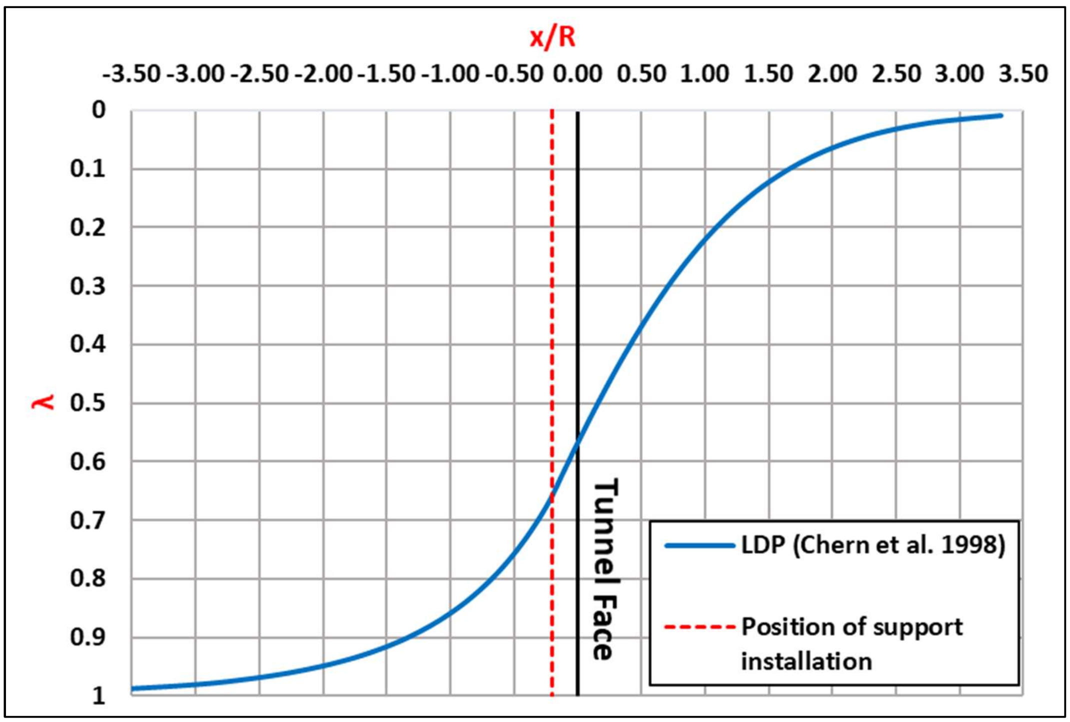

Figure 5. Longitudinal displacement profile (LDP), , for the ground unit (sch).

Table 4.Deconfinement characteristics values.

Parameter	Value
Geostatic field stress (p_o)	468 KPa
Ground strength (σ_{cm})	192.1 KPa
Overload factor (N_s)	4.87
Critical deconfinement factor (λ_{cr})	0.66

Parameter	Value
Deconfinement factor at the tunnel face (λ, x=0m)	0.57
Deconfinement factor at excavation advance length (λ, x=-1m)	0.66
Stage factor at excavation advance length (β)	0.34

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

2.8. Restrains

On the numerical model, the following restrains used:

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

3. Results

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. From the numerical analysis, it is observed that the minimal shotcretes thickness t=15cm, it is acceptable according to EC2 EN 1992-1, as the combination between lining bending moment (M) and thrust (N), are inside the lining capacity envelope in all simulation stages. On the following ***Figure 6*** is presented the support capacity envelope for the tunnel combined lining with steel sets and shotcrete and on ***Figure 7***, is presented the support capacity envelope for tunnel invert shotcrete.

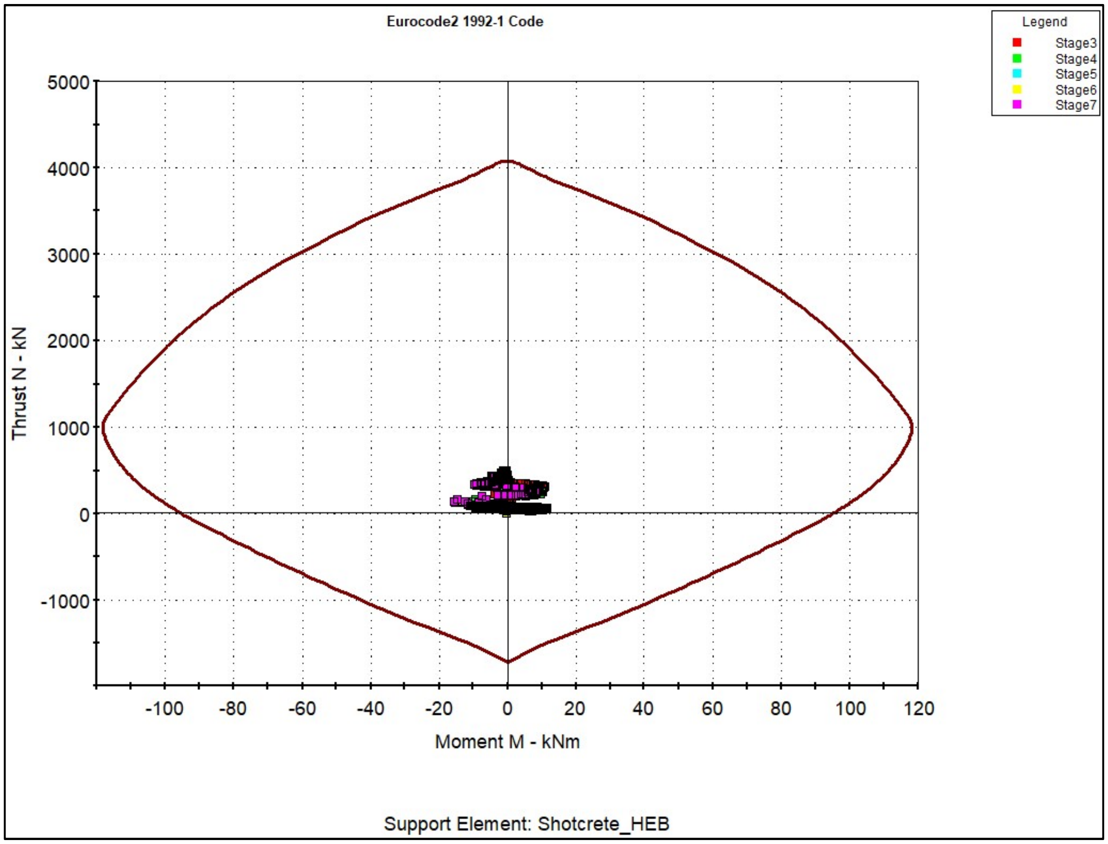

Figure 6. Support capacity plot for the composite lining of shotcrete and steel sets.

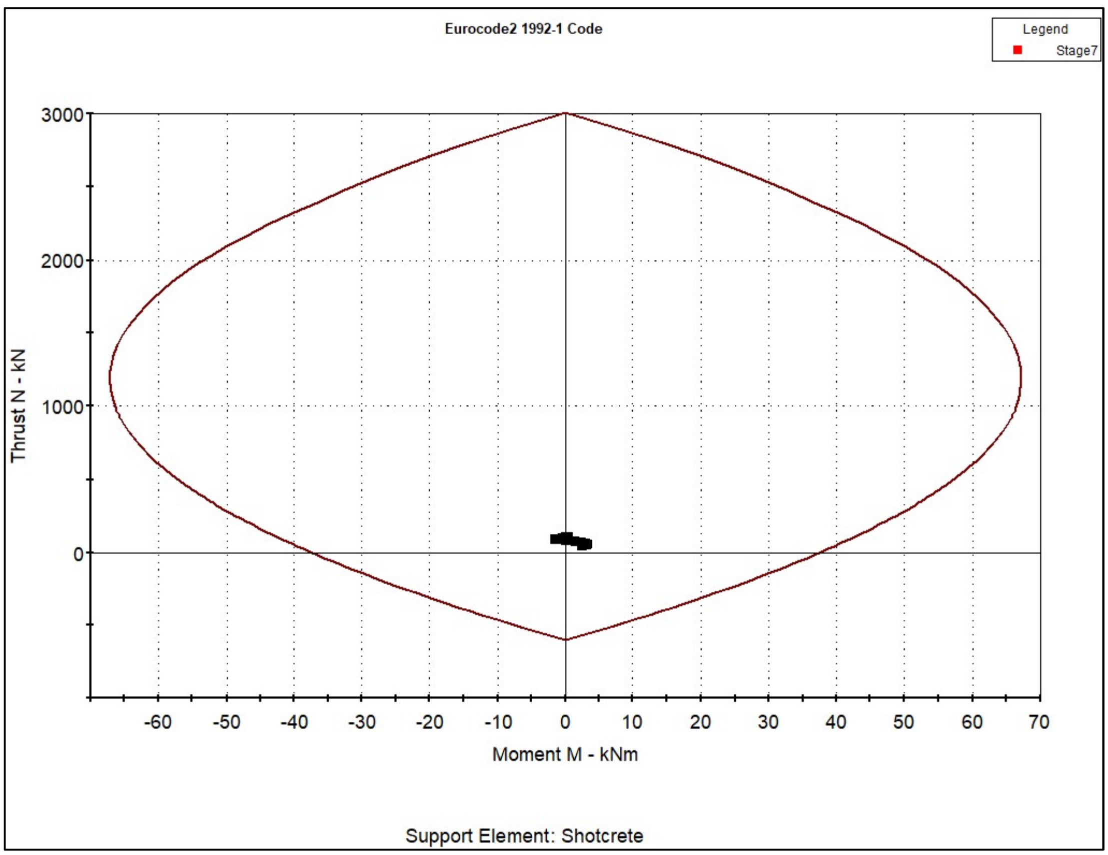

Figure 7. Support capacity plot for the tunnel invert lining.

Moreover, as it is presented on the following ***Figure 8,*** the maximal axial force on tunnel rockbolts, is F= 187.83 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/ 187.83 kN)= **1.44**.

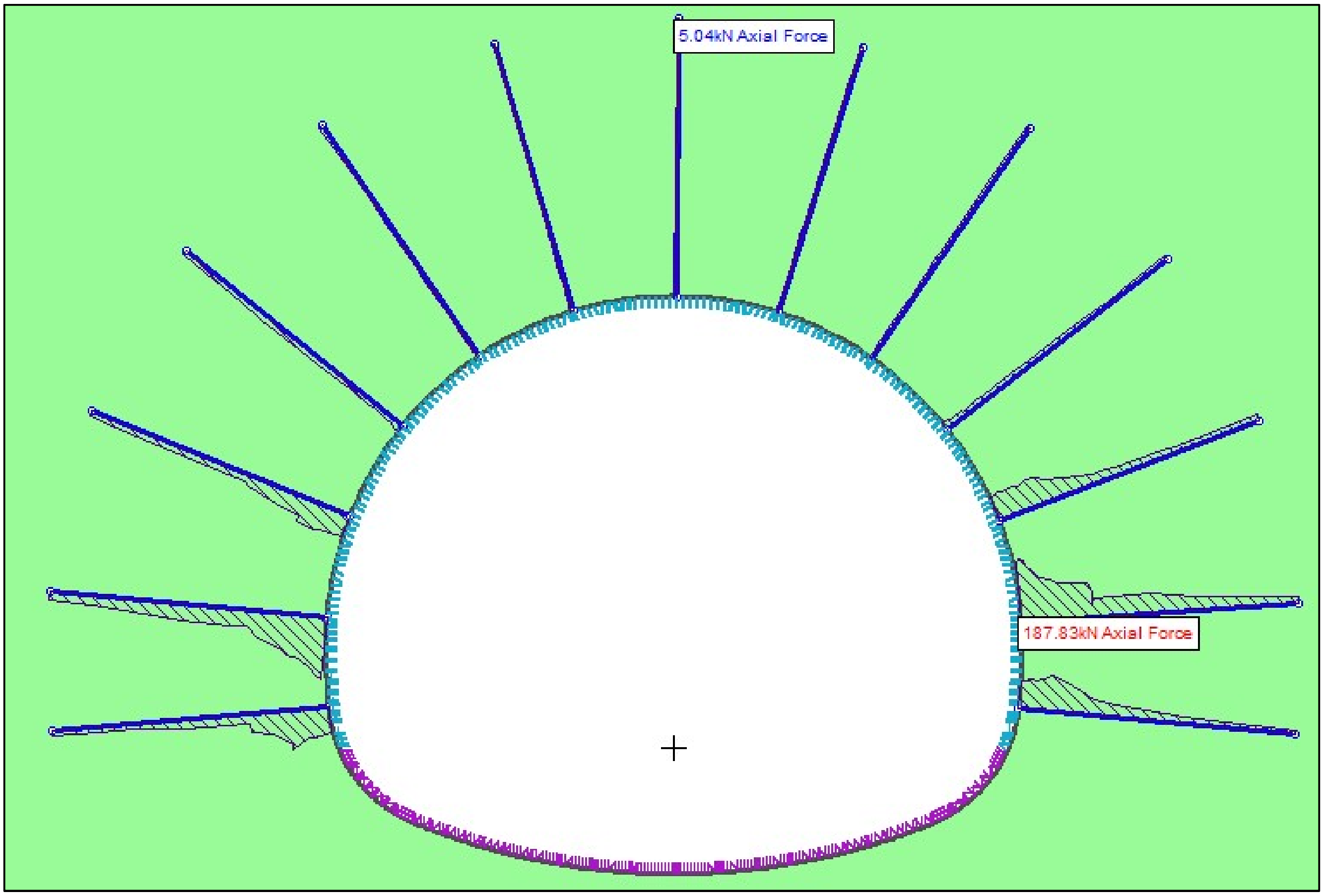

Figure 8. Axial force on tunnel perimeter rockbolts.

<u>Thus, the proposed tunnel primary lining, consist of shotcrete with total thickness t=15cm and HEB 140 steel sets for the Top Heading and Bench excavation phases and shotcrete with total thickness t=15cm, for the tunnel final invert.</u>

3.2. <u>Surficial settlements</u>

Based on the proposed tunnel primary lining, as described on *Chapter 3.1*, the maximal surficial settlement (ground vertical displacement) due to the tunnel excavation, is **s= 5.7mm** and characterized as acceptable surficial settlement according to the limitation of maximal acceptable settlement of 30mm. On the following ***Figure 8,*** is presented the surficial settlement profile for every excavation step.

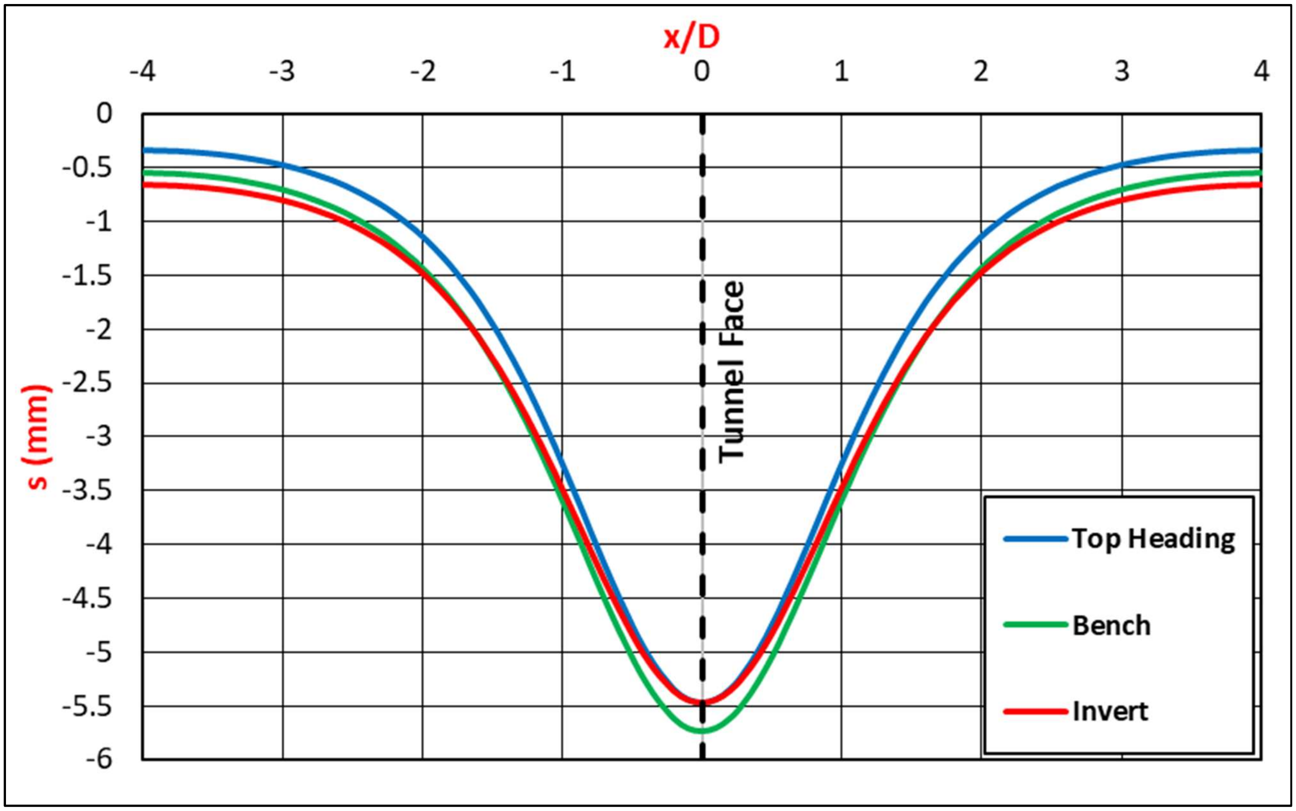

Figure 9. Surficial settlement profile.

3.3. Tunnel displacements

Based on the proposed tunnel primary lining, as described on *Chapter 3.1*, on the following ***Table 3***, total displacements on the tunnel crown every excavation step, are presented. Moreover, on the following ***Figure 10***, total displacements around excavation perimeter on the final excavation step, are presented.

Table 5. Total displacement on the tunnel crown.

Phase	Total displacement
Top Heading	9,2 mm
Bench	9,4 mm
Invert	9,1 mm

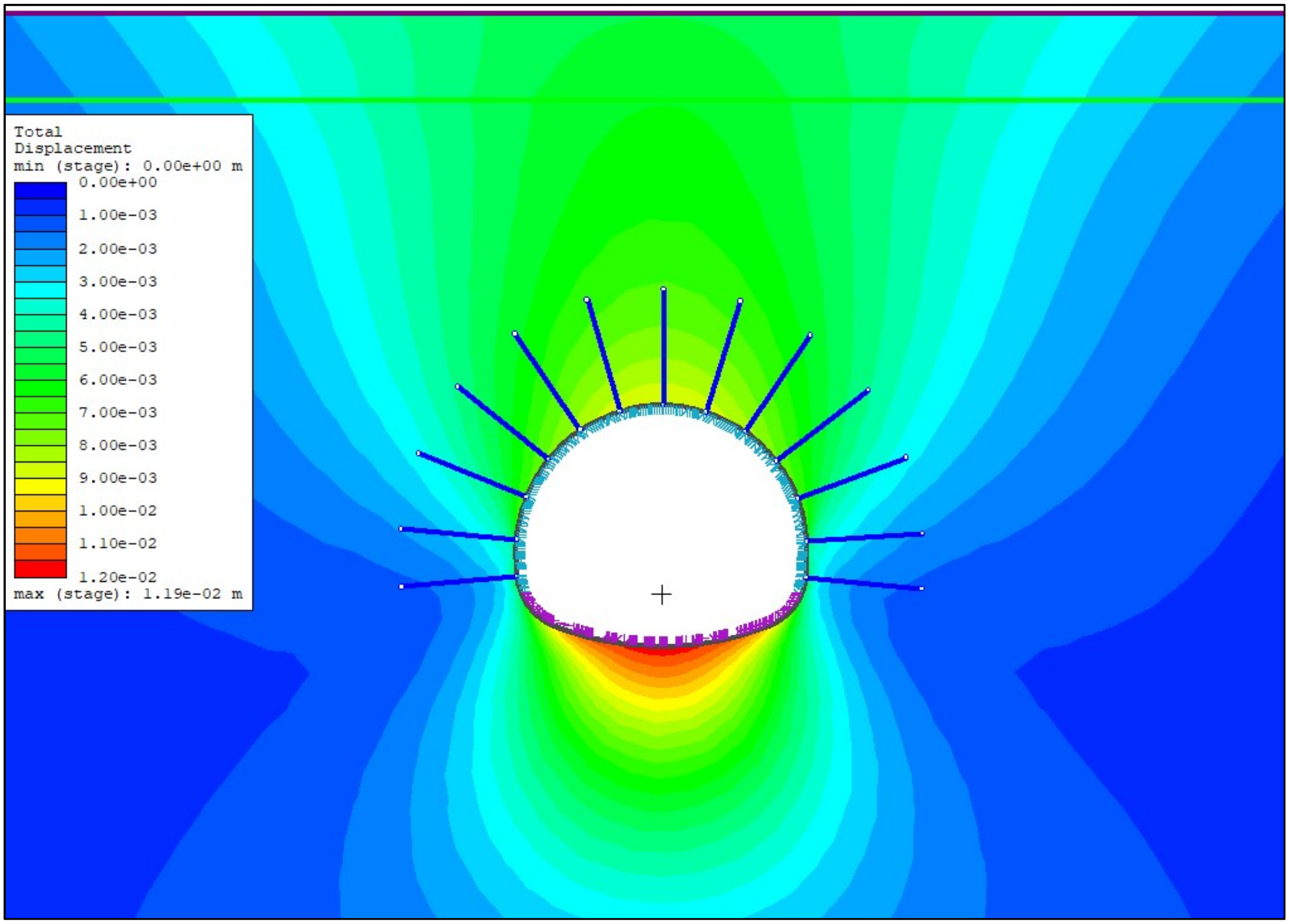

Figure 10. Total displacements on the final excavation stage.

3.4. Proposed tunnel primary lining

The proposed tunnel primary lining and the excavation phases, based on the numerical analysis, is presented on the following table and Figure.

Table 6. Tunnel primary lining.

Excavation Phase	Excavation advance length	Shotcrete	Wire mesh	Rockbolts	Steel sets
Top Heading	1 m	C30/37, t=15 cm	1 layer T188	11 bolts fully bonded Φ25, L= 4m, s= 1.5m, every excavation length	HEB 120, every excavation length
Bench	1 m	C30/37, t=15 cm	1 layer T188	2 bolts fully bonded Φ25, L= 4m, s= 1.5m, every excavation length	HEB 120, every excavation length
Invert	1 m	C30/37, t=15 cm	1 layer T188	-	-

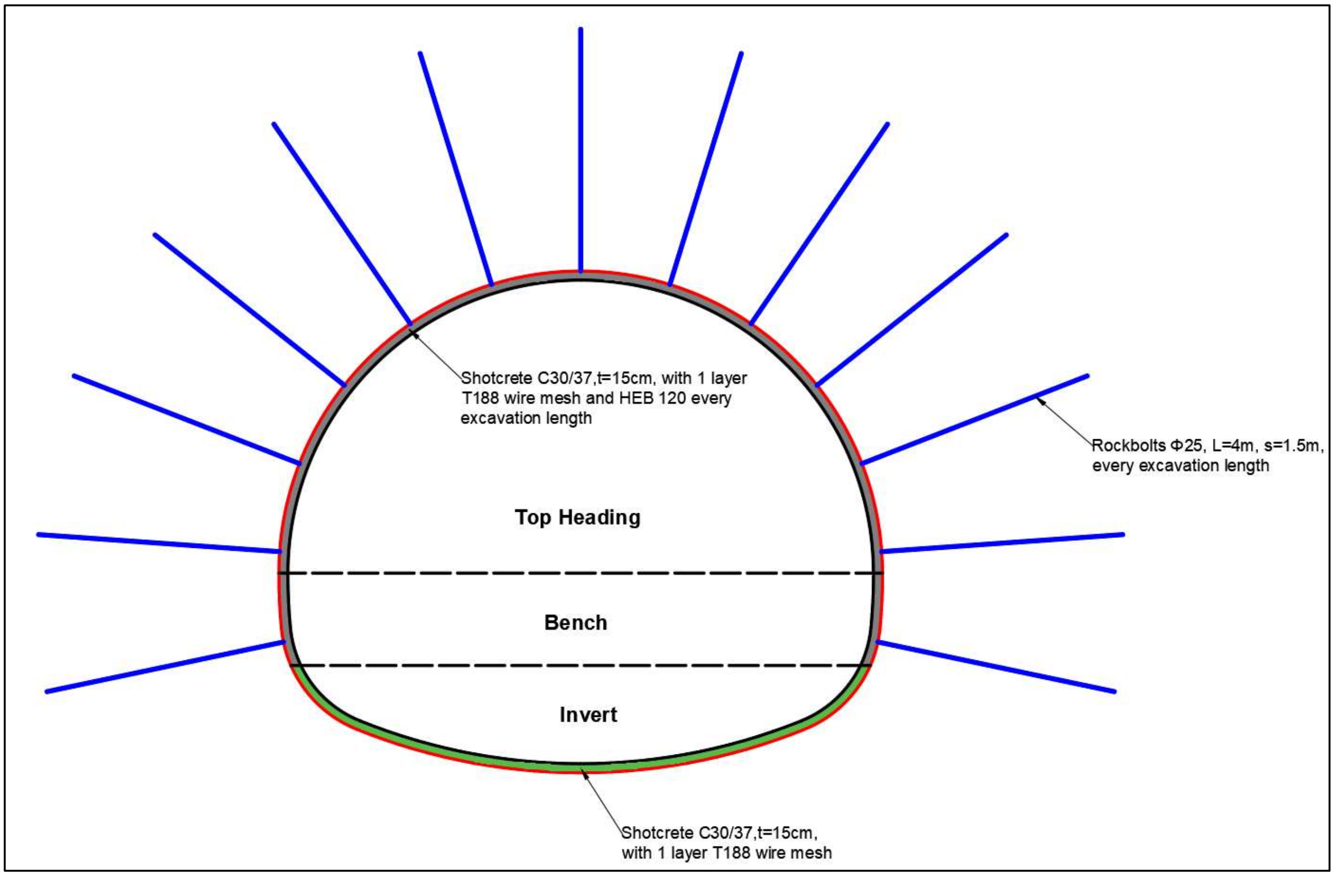

Figure 11. Tunnel proposed primary lining.