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PROJECT:

Consultancy Services for Detailed Feasibility Study and Framing up of Phasewise proposal (DPR) for construction of two tunnels at Z-Morh and at Zojila for all weather connectivity from Srinagar to Leh in Jammu & Kashmir State

ZOJILA TUNNEL

TITLE:

Phase II: Detailed Project Report - Preliminary Tunnel Design

Volume V: Primary Lining Analysis Report

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List of Addendum

1 GENERAL

In this design report the analyses of the primary lining of Zojila Tunnel is presented. The analyses are based on the geological-geotechnical ground conditions as described in DPR Volume IV: Geotechnical Design Report. Basic input parameters are the Ground Types and the Behaviour Types including their descriptions and parameters.

Analytical and numerical analysis are carried out. The influence of the egress tunnel to the main tunnel and vice versa is assessed with numerical analysis. The influence on the primary lining due to seismic actions is assessed with pseudo-static methods, considered in the numerical analysis.

2 REFERENCES

2.1 Project Documents

[1] Study of seismicity and seismotectonics of Zojila Tunnel project area, G S GeoEnVirons Pvt. Ltd., 2011

2.2 Standards, Guidelines & Literature

[2] EN 1997-1 (2004): “Eurocode 7 – Geotechnical Design – Part 1: General Rules”, 2004

[3] IS 1893-1 (2002): “Criteria for Earthquake Resistant Design of Structures, Part I: General Provisions and Buildings”, Bureau of Indian Standard, New Delhi, 2002

[4] Feder, G., Arwanitakis, M.: Zur Gebirgsmechanik ausbruchnaher Bereiche tiefliegender Hohlraumbauten, Berg- und Hüttenmännische Monatshefte, Heft 4, 1976

[5] Eurocode 2 (EN 1992-1-1) Design of concrete structures, Part 1-1 General rules and rules for buildings

2.3 Software

[6] PLAXIS b.v.: PLAXIS 2D 2011, Delft, The Netherlands, 2011.

3 ASSIGNMENT OF PRIMARY SUPPORT CATEGORY

8 Support Categories are developed for the primary support of the Zojila main tunnel tube. The egress tunnel is designed with 6 support categories. A detailed definition of the Support Categories is given in DPR Volume IV: Geotechnical Design Report and the corresponding support category drawings in DPR Volume XI: Drawings (8482B_II-ZOT_EXC-01-12-00 to 8482B_II-ZOT_EXC-16-12-00). A correlation between the Support Categories and the Ground Types are given in the table below.

Tab. 1 Correlation between the Behaviour Types and the Support Categories

Behaviour Type	Support Category
BT 1 Stable	SC A
BT 2 Gravitational block fall	SC B and SC C
BT 3 Shallow stress induced failure	SC D
BT 4 Voluminous stress induced failure	SC E to SC G
BT 7 Crown failure	SC H
BT 8 Ravelling ground	SC H
BT 9 Flowing ground	SC H

4 ANALYTICAL ANALYSES

For the analysis of the support capacity an analytical model [4] is used. With these analyses the correlation between Ground Types, Behaviour Types, ranges of overburden and Support Categories are investigated and determined.

The analysis sheets are given in Addendum 1 of this report. The results are summarized in the table below.

Tab. 2 Support Matrix with indication of analytically calculated cases (light grey…analytical analysis, dark grey…numerical analysis)

<table>
<tr><th></th><th></th><th>GT1</th><th>GT2</th><th>GT3</th><th>GT4</th><th>GT5</th><th>GT6</th><th>GT7</th></tr>
<tr><td rowspan="7">Overburden Range</td><td>0-50 m</td><td>SC H</td><td>SC A,B,C</td><td>SC B,C</td><td>SC B,C</td><td>SC B,C</td><td>SC B,C</td><td>SC H</td></tr>
<tr><td>50-80 m</td><td>-</td><td>SC A,B,C</td><td>SC B,C</td><td>SC B,C</td><td>SC B,C</td><td>SC B,C</td><td>SC D</td></tr>
<tr><td>80-150 m</td><td>-</td><td>SC A,B,C</td><td>SC B,C</td><td>SC B,C</td><td>SC B,C</td><td>SC D</td><td>SC E</td></tr>
<tr><td>150-200 m</td><td>-</td><td>SC A,B,C</td><td>SC B,C</td><td>SC D</td><td>SC B,C</td><td>SC D</td><td>SC F</td></tr>
<tr><td>200-400 m</td><td>-</td><td>SC A,B,C</td><td>SC B,C</td><td>SC D</td><td>SC D</td><td>SC E</td><td>SC F, G</td></tr>
<tr><td>400-500 m</td><td>-</td><td>SC A,B,C</td><td>SC B,C</td><td>SC E</td><td>SC D</td><td>SC E</td><td>SC G</td></tr>
<tr><td>500-650 m</td><td>-</td><td>SC A,B,C</td><td>SC D</td><td>SC E</td><td>SC D</td><td>SC F</td><td>SC G</td></tr>
</table>

Fault zones are not calculated with the simple analytical method because the limited thickness of the fault zone leads to significantly lower displacements and loading of the tunnel lining than in the 2D analysis. More detailed analysis shall be done in the detailed design phase.

The ground parameters are reduced (from elastic to plastic parameters) in the model which covers simplifications within the model (such as circular tunnel geometry) and leads to realistic to conservative results. For the analysis only the shotcrete lining was assumed (neglecting of the bolting) with a general reduction of 20% from the theoretical calculated support pressure for the perfectly circular geometry. The following support pressure was applied.

Tab. 3 Support pressure of the different Support Categories

Support Category	Support pressure [MPa]	Applied reduced support pressure [MPa]
SC A	0.15	0.12
SC B	0.30	0.24
SC C	0.50	0.40
SC D	0.67	0.54
SC E	0.80	0.64
SC F	1.00	0.80
SC G	1.00	0.80
SC H	1.00	0.80

Apart of the analyses given in Addendum 1 various parameter studies, including numerical analyses, are done based on the parameter ranges as described in DPR Volume IV: Geotechnical Design Report. The results of the analysis lead to the following maximum displacements for each support category.

Tab. 4 Maximum displacements of the different Support Categories

	Thickness of inner lining [cm]	Thickness of primary lining [cm]	Expected total displacement of primary lining [cm]
SC A	30	5	1
SC B	30	10	2
SC C	30	15	3
SC D	40	20	5
SC E	40	25	10
SC F	40	30	20
SC G	40	30	35
SC H	40	30	5

5 NUMERICAL ANALYSES

Three numerical analyses are performed to check the results of the analytical model for three different Support Categories with high overburden. The following three cases are analysed and marked dark grey in Tab. 2.

- Analysis 1: Support Category C, Ground Type 3, maximum overburden of approx. 510 m
- Analysis 2: Support Category E, Ground Type 4, maximum overburden of approx. 450 m
- Analysis 3: Support Category G, Ground Type 7, maximum overburden of approx.660 m

5.1 Numerical Model

Numerical analyses are performed by the finite element code PLAXIS on two-dimensional models assuming plane strain conditions.

The geometry of the main tunnel with the main primary support elements are taken from the design drawings 8482B_II-ZOT_EXCA-01-12-00 to 8482B_II-ZOT_EXCA-09-12-00 (support categories A to H) and for the egress tunnel from 8482B_II-ZOT_EXCA-10-12-00 to 8482B_II-ZOT_EXCA-16-12-00 (support categories A to F) respectively.

Homogenous ground conditions are assumed for the numerical analysis. Effects of joints are considered in the rock mass properties as given in Section 5.2.1.

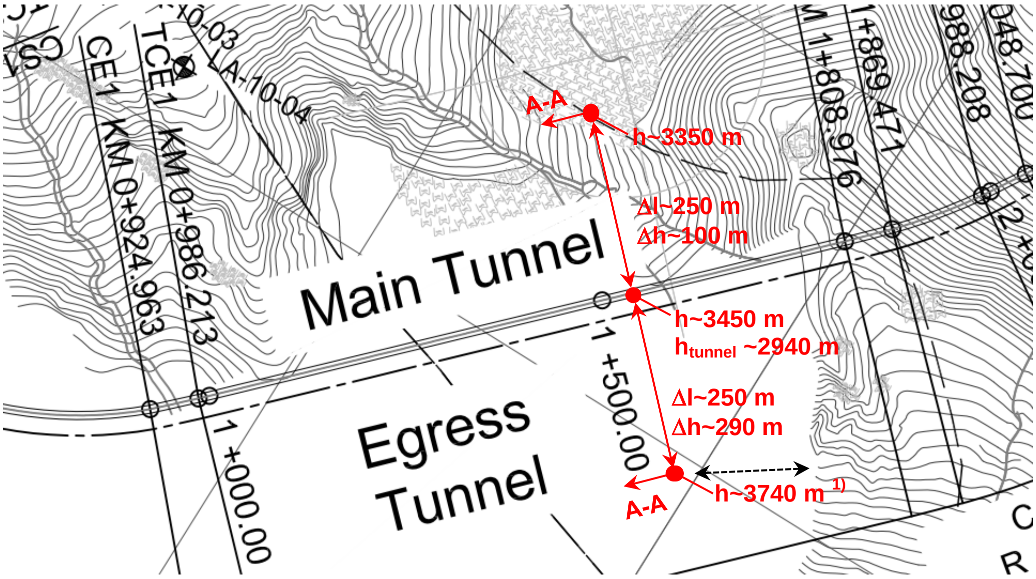
The model boundaries are determined in a distance of approximately 20 times the tunnel diameter to the outer tunnel circumference. The numerical model is therefore about 500 m wide. The height of the three models is different due to the ground surface geometry. The overburden is simulated by a layer with higher unit weight to minimize the height of the model. The model is fixed at the lower boundary in x- and y-direction and at the lateral boundaries in x-direction. The upper boundary is free to move.

The mesh generation is done automatically by the finite element program with 15-noded triangular shaped elements. The mesh is refined in an area of approx. 5 m from the outer geometric point of the bolts.

The calculation cross sections are given in Tab. 5 to Tab. 7 in plain view details of the general layout plan (8482B_II-ZOT_GEN-02-12-00) compared to the numerical model as designed in the numerical code PLAXIS.

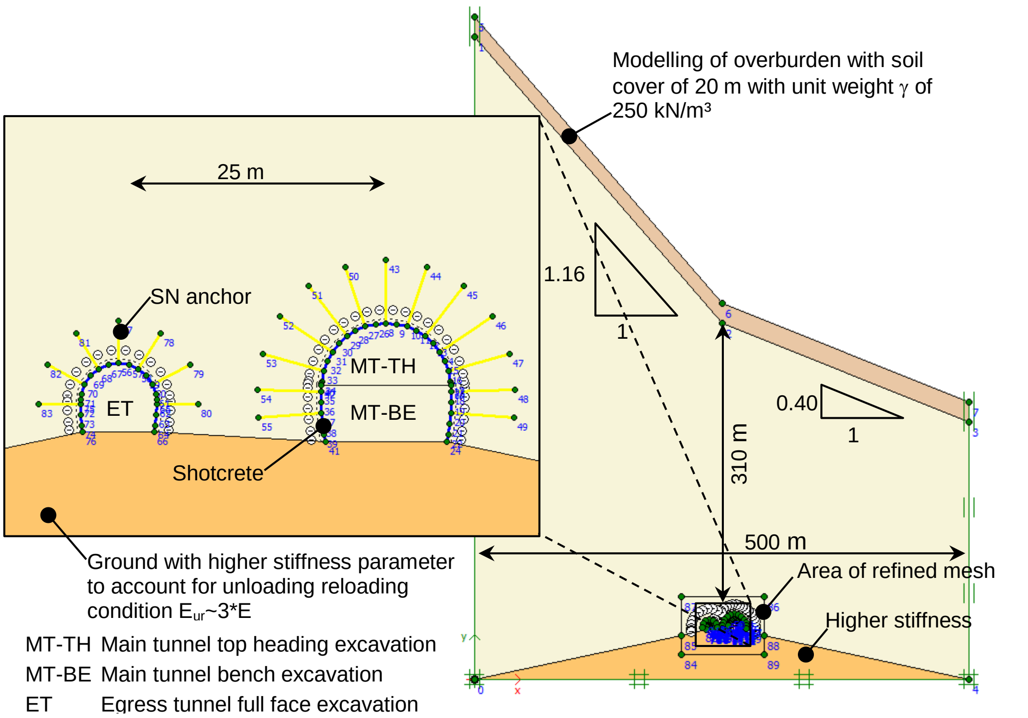
Tab. 5 Calculation cross section analysis 1, Support Category C, A-A

Analysis 1, SC C: Calculation cross section according to general layout of Zojila Tunnel



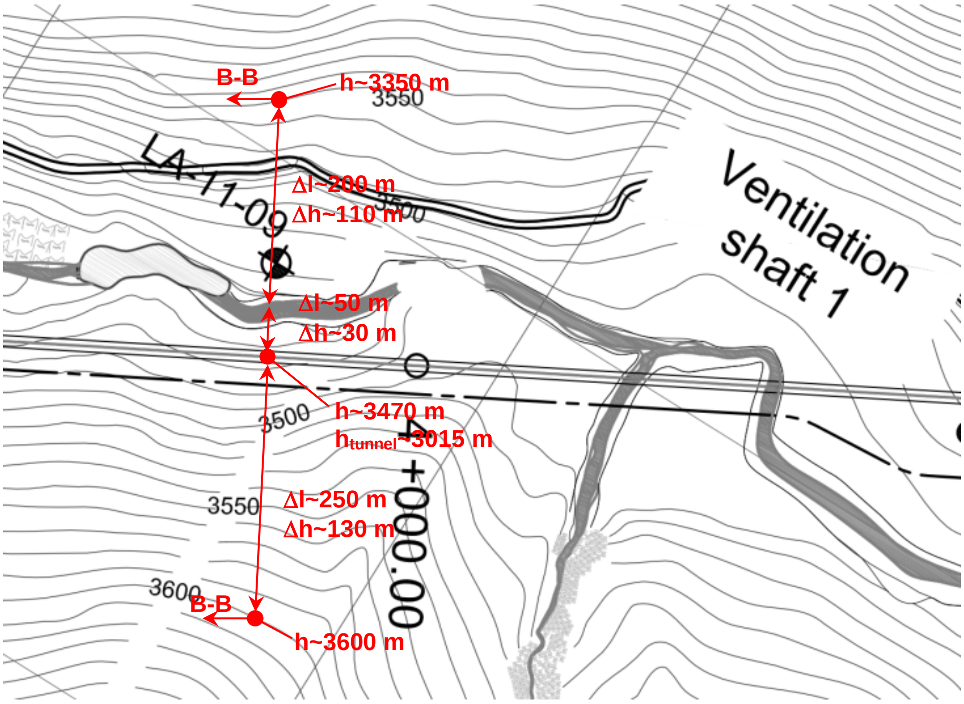
1) height interpolated from next contour line on the right side (see dashed line)

Analysis 1 SC C: Numerical model of calculation cross section A-A

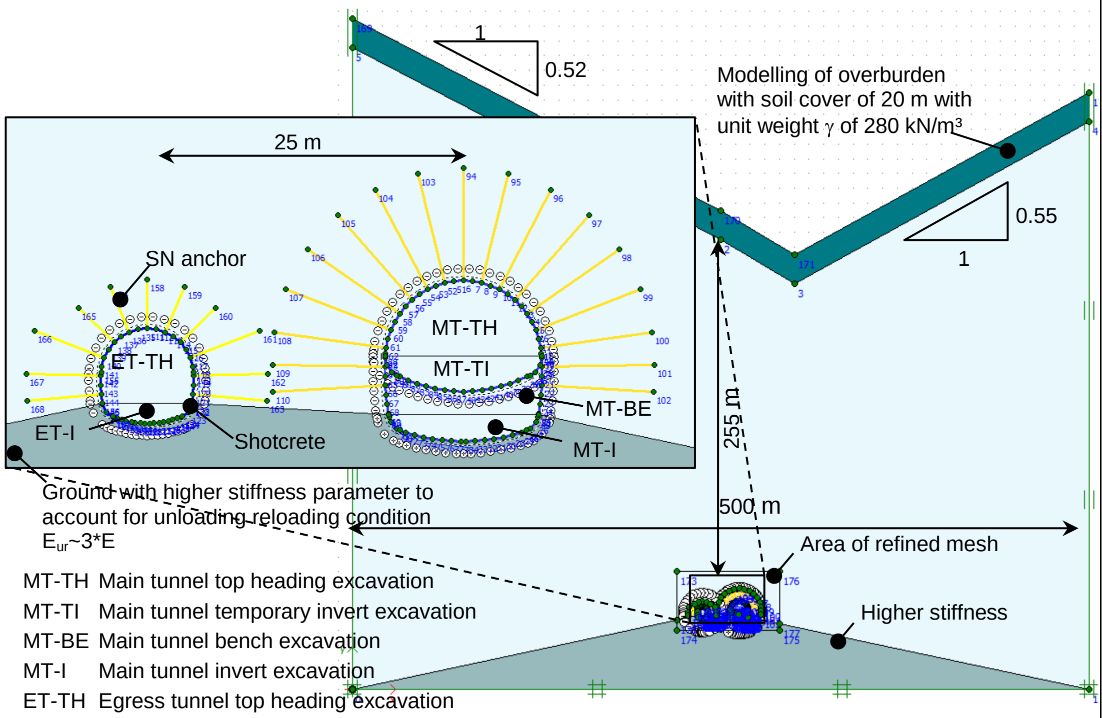


Tab. 6 Calculation cross section analysis 2, Support Category SC E, B-B

Analysis 2, SC E: Calculation cross section according to general layout of Zojila Tunnel

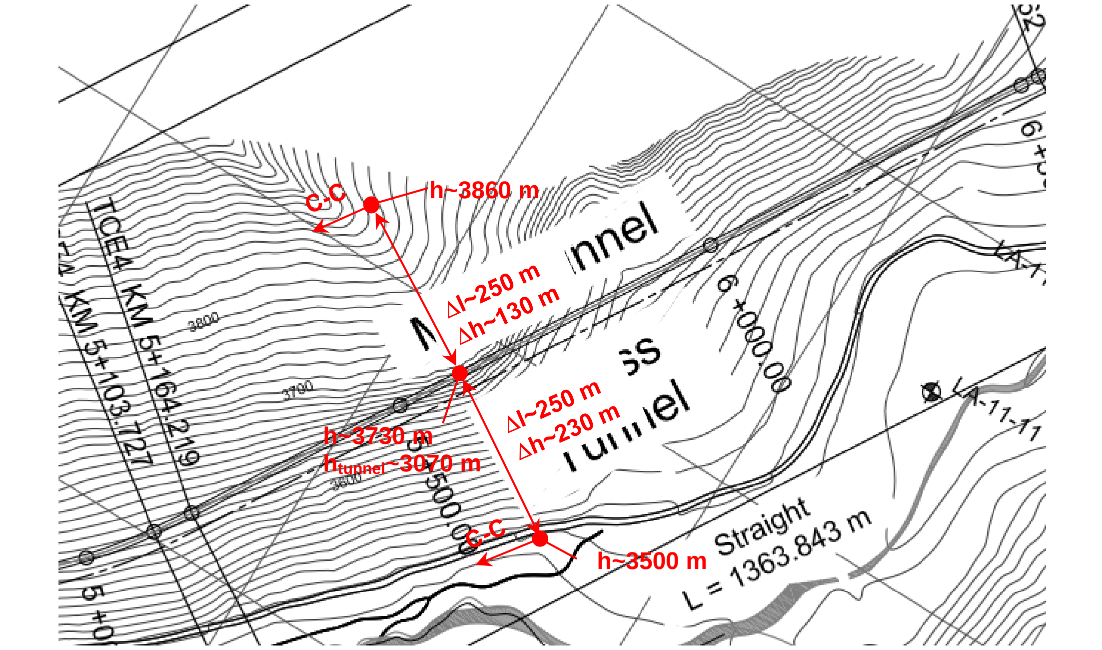


Analysis 2, SC E: Numerical model of calculation cross section B-B

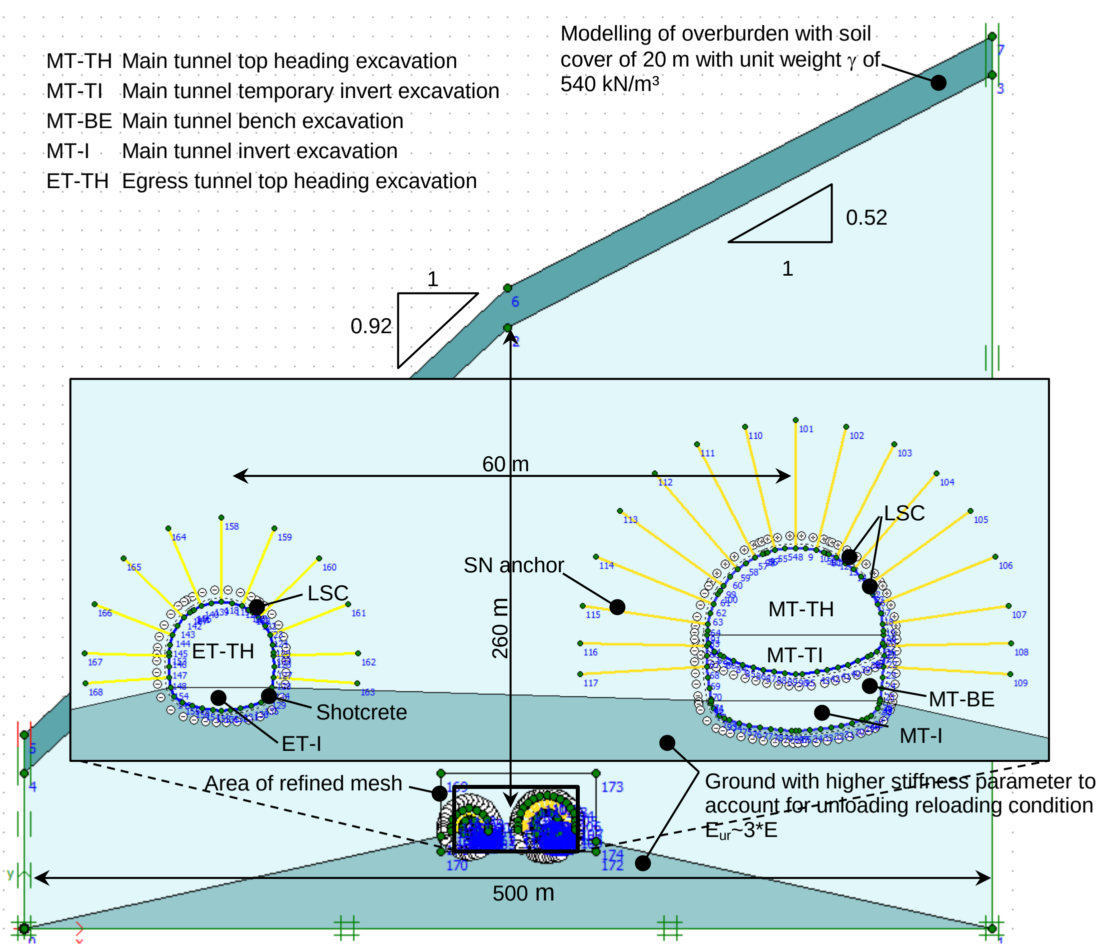


Tab. 7 Calculation cross section analysis 3, Support Category SC G, C-C

Analysis 3, SC G: Calculation cross section according to general layout of Zojila Tunnel



Analysis 3, SC G: Numerical model of calculation cross section C-C



5.2 Material Properties

5.2.1 Rock Mass

Tab. 8 shows the range (poor-good) as well as the expected mean values of the rock mass parameters and the used values in the numerical analyses. These data are taken from the Ground Type tables according to DPR Volume IV: Geotechnical Design Report.

Tab. 8 Rock mass properties for Ground Types

Ground Type		γ [kN/m³]	φ [°]	**c** [MPa]	**E** [GPa]	ν [-]	**GSI** [-]	**UCS** [GPa]
GT2	Massive Metabasite (Green Schist)	27-30 29	40-50 45	7-30 15	70-180 110	0.1-0.2 0.15	70-95 80	30-170 80
GT3	Jointed/Foliated Metabasite (Green Schist)	27-30 28	35-45 40	5-20 10	20-160 70	0.1-0.2 0.15	50-75 60	20-100 50
GT4	Faulted Metabasite (Green Schist)	27-29 28	23-35 27	1.5-5 2.5	2-20 6	0.15-0.25 0.20	25-55 35	5-20 8
GT5	Jointed/Foliated Phyllites and Slates (Zojila Formation)	26-29 28	27-40 33	2-8 4	10-80 35	0.1-0.2 0.15	50-75 60	6-35 15
GT6	Intensively Foliated Phyllites and Slate (Zojila Formation)	26-29 28	23-35 27	1-4 2	2-20 6	0.1-0.2 0.15	35-55 40	3-15 6
GT7	Faulted Phyllites and Slates (Zojila Formation)	26-28 27	20-27 23	0.5-2 1	0.5-3 1.5	0.15-0.25 0.20	20-40 30	1.5-7 3.5

γ Specific weight
φ Friction angle
c Cohesion
E Young's modulus (primary loading); E-module unload/reload = 3xE
ν Poisson's ratio

It is pointed out, that the parameter ranges are overlapping, so that the combination of minimum parameters from one Ground Type is already within the range of the next weaker Ground Type.

The rock mass is simulated with the linear elastic, perfectly plastic Mohr-Coulomb constitutive model. Tensile stresses are generally not admitted in the analysis. The parameters are valid for drained conditions. It is assumed, that drainage measures dissipate possible pore pressures and excess pore pressures.

The Young's moduli presented in the tables above are used to describe the deformation behaviour of the rock mass for primary loading conditions. Below excavation level the moduli are put 3 times higher to consider more realistic heave effects due to excavation.

5.2.2 Shotcrete

Shotcrete of C25/30 with fibre reinforcement or wire mesh reinforcement is designed for primary lining of support categories A and B. The shotcrete lining was modelled with linear elastic plate elements. The thickness varies in the support categories.

The modulus of elasticity is estimated with an average value of 2500 MPa from experience of other projects considering the gaining of strength over time as well as consideration of shrinking and creeping effects.

Tab. 9 Material properties for shotcrete without additional support elements

Shotcrete				
Support Category	SC	[-]	A	B
thickness	d	[m]	0,05	0,10
length	l	[m]	1,00	1,00
area	A	[m²]	0,05	0,10
moment of inertia	I	[m⁴]	1,04E-05	8,33E-05
specific weight	γ	[kN/m³]	25	25
E-Modulus	E	[kN/m²]	2,5E+06	2,5E+06
Poisson's ratio	ν	[-]	0,20	0,20
PLAXIS Plate-Element	**EA**	[kN/m]	1,25E+05	2,50E+05
	EI	[kNm²/m]	26	208
	w	[kN/m/m]	1,25	2,50

EA / EI Axial stiffness / bending stiffness of the element per m tunnel length
w Element weight per m tunnel length

5.2.3 Shotcrete with Steel Sets

In support categories C to H steel sets (lattice girders) are installed in the shotcrete primary lining. The shotcrete thicknesses as well as the type of the lattice girders (and the spacing) vary in the different support categories. The lining is modelled with linear elastic plate elements.

The modulus of elasticity for shotcrete is estimated with an average value of 2500 MPa from experience of other projects considering the gaining of strength over time as well as consideration of shrinking and creeping effects.

Tab. 10 Material properties for shotcrete with steel sets

Shotcrete with Steel Sets						
Support Category	SC	[-]	C	D	E	F,G,H
Shotcrete						
thickness	d	[m]	0,15	0,20	0,25	0,30
length	l	[m]	1,00	1,00	1,00	1,00
area	A_1	[m²]	0,15	0,20	0,25	0,30
moment of inertia	I_1	[m^4]	2,81E-04	6,67E-04	1,30E-03	2,25E-03
E-Modulus	E_1	[kN/m²]	2,5E+06	2,5E+06	2,5E+06	2,5E+06
specific weight	γ	[kN/m³]	25	25	25	25
axial stiffness	E_1A_1	[kN/m]	3,75E+05	5,00E+05	6,25E+05	7,50E+05
bending stiffness	E_1I_1	[kNm²/m]	703	1667	3255	5625
element weight	w_1	[kN/m/m]	3,75	5,00	6,25	7,50
Lattice Girders						
type	LG	[-]	50/20/30	70/20/30	95/20/30	95/20/30
area	A_2	[m²]	1,34E-03	1,34E-03	1,34E-03	1,34E-03
moment of inertia	I_2	[m^4]	1,93E-06	3,06E-06	4,85E-06	4,85E-06
E-Modulus	E_2	[kN/m²]	2,1E+08	2,1E+08	2,1E+08	2,1E+08
spacing	l	[m]	1,75	1,50	1,25	1,25
sets per m tunnel	n	[pcs]	0,57	0,67	0,80	0,80
axial stiffness	nE_2A_2	[kN/m]	1,61E+05	1,88E+05	2,25E+05	2,25E+05
bending stiffness	nE_2I_2	[kNm²/m]	232	643	1019	1019
Combined Section						
Poisson's ratio	ν	[-]	0,20	0,20	0,20	0,20
PLAXIS Plate-Element	**EA**	[kN/m]	5,36E+05	6,88E+05	8,5E+05	9,75E+05
	EI	[kNm²/m]	935	2310	4274	6644
	w	[kN/m/m]	3,75	5,00	6,25	7,50

EA Axial stiffness of the element per m tunnel length: $EA = E_1A_1 + nE_2A_2$

EJ Bending stiffness of the element per m tunnel length: $EJ = E_1J_1 + nE_2J_2$

w Element weight per m tunnel length (weight of lattice girders is disregarded)

5.2.4 Rock Bolts

Rock bolts are modelled as "geogrid"-elements of elastoplastic material type which can sustain only tensional but not compressive forces. Main input parameters are the extensional stiffness and the maximum axial force per meter tunnel. The part of the extensional stiffness carried by the grouted body is not considered.

Tab. 11 gives an overview of the material properties. The upper values are given for one single rock bolt. In PLAXIS the "spacing" has to be introduced, which corresponds to the spacing of the rock bolts in longitudinal tunnel direction. The material properties have to be adapted with the spacing length as the input for "geogrids" is per tunnel meter.

Tab. 11 Material properties for rock bolts

Rock Bolts							
Support Category	SC	[-]	A	B	C	D	E, F,G
type			Frictional bolts		Grouted bolts		
min. breaking load	N_{Ult}	[kN]	200	200	300/350	300/350	300/350
yielding load	$N_{p(1)}$	[kN]	180	180	220/260	220/260	220/260
E-Modulus	E	[kN/m²]	2,1E+08	2,1E+08	2,1E+08	2,1E+08	2,1E+08
tube diameter	Ø	[mm]	54	54	28/36	28/36	28/36
material thickness	t	[mm]	3	3	-	-	-
area	A	[mm²]	509	509	616/1017	616/1017	616/1017
axial stiffness	$EA_{(1)}$	[kN]	1,07E+5	1,07E+5	1,29E+5 2,14E+5	1,29E+5 2,14E+5	1,29E+5 2,14E+5
spacing	L_{spac}	[m]	3,50	2,50	1,75	1,50	1,25
PLAXIS Geogrid-Element	**EA**	[kN/m]	30540	42750	73920 122285	86240 142380	103488 170856
	$\mathbf{N_p}$	[kN/m]	51,4	72,0	125/149	147/173	176/208

L_{spac} Spacing of rock bolts in direction of tunnel length; varies in the different support categories

$EA_{(1)}$ Axial stiffness of one rock bolt

EA Axial stiffness of rock bolts per meter tunnel

N_p Maximum axial tensional force per meter tunnel

5.2.5 Yielding Steel Elements

In support category F and G the top heading of the primary support lining is divided by two and four longitudinal gaps respectively, with a height of 50 cm each. In the gaps yielding steel cylinders are provided to control the deformation behaviour (LSC-Lining Stress Controllers).

The yielding steel elements are modelled as elastoplastic plate elements. It is assumed, that each element yields at a normal force of 700 kN and has a negligible resistance on moments.

Tab. 12 Material properties for yielding steel elements

Yielding Steel Elements				
Support Category	SC	[-]	ET-E	MT-G
sprayed concrete thickness	d	[m]	0,25	0,30
yielding force per element	F_p	[kN]	700	700
number of elements per m tunnel		# / m	4	4
total yielding force	$F_{p,TOT}$	[kN]	2800	2800
height of elements	h	[m]	0,43	0,43
activation length for F_p	$l_{p,min}$	[m]	0,06	0,06
strain at F_p	ε_p	[-]	0,14	0,14
equivalent area per m tunnel	A	[m²]	0,25	0,30
equivalent stress at F_p per m tunnel	σ_p	[kN/m²/m]	11200	9333
E-Modulus (back-analysis)	E	[kN/m²]	80287	66903
PLAXIS Plate-Element	**EA**	[kN/m]	20071	20071
	EI	[kNm²/m]	175	175
	w	[kN/m/m]	7,5	7,50
	M_p	[kNm/m]	1	1
	N_p	[kN/m]	2800	2800

E-modulus from back-analysis: $E = \sigma_p / \varepsilon_p$

EA Axial stiffness of the element per m tunnel length: $EA = F_{p,TOT} / \varepsilon_p$

EJ Bending stiffness of the element per m tunnel length

w Element weight per m tunnel length (disregarded)

N_p Yielding axial force

M_p Yielding moment

5.2.6 Interface-Elements

Interface-elements are arranged around the outer surface of the tunnel primary lining to the surrounding rock to take account of the rock-structure interaction. In the interfaces the friction angle of the surrounding rock is reduced to 1/2 φ.

5.3 Initial Stresses and Construction Sequence

The initial stress condition is generated with the so-called gravity loading condition which is applied for non-horizontal ground surface. For a linear elastic perfectly plastic constitutive model (Mohr Coulomb model) as applied in the numerical analysis, the earth pressure at rest is governed by the Poisson's ratio ν ($k_0=\nu/[1-\nu]$). An additional material set is introduced for generation of initial stresses only, with same material properties of initial material except a Poisson's ratio ν of 0.495. The Poisson's ration is chosen in such way to receive an earth pressure at rest k_0 of approx. 1.0. A Poisson's ration ν of 0.5 would lead to numerical problems and is therefore not permitted in the software. After initial stresses are generated the

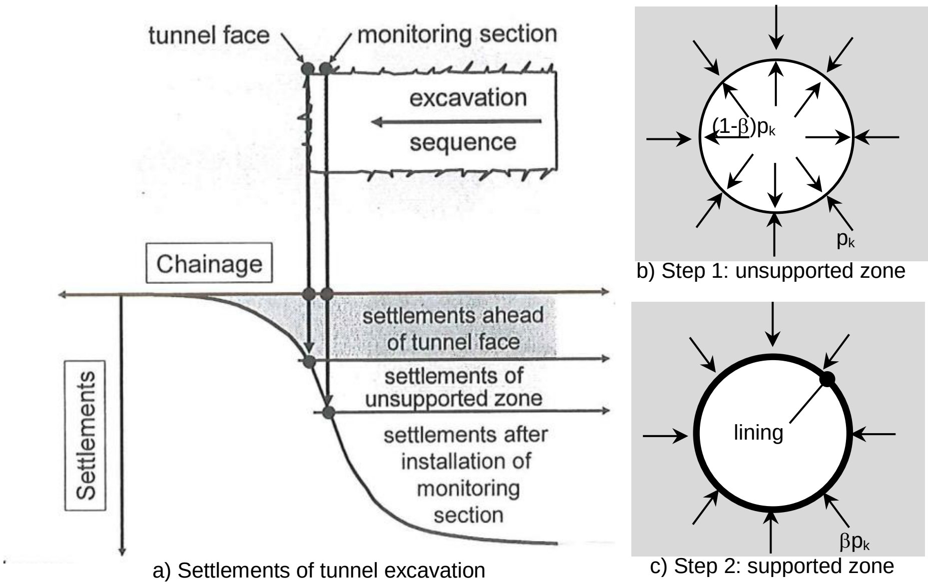
Poisson's ratio of the initial material sets according to Tab. 8 are used for subsequent calculation steps.

The construction sequence for the support categories A to H (main tunnel) and A to F (egress tunnel) are shown in the drawings 8482B_II-ZOT_EXCA-01-12-00 to 8482B_II-ZOT_EXCA-16-12-00.

The egress tunnel is excavated in advance of the main tunnel with full face excavation.

Each excavation step (full face, top heading, bench or invert) is excavated in two stages in the analysis to account for the settlement ahead of the tunnel and the settlement of the unsupported zone (see Fig. 1 a). In PLAXIS this is considered by the so-called β-method, where β is the pre-relaxation factor. The initial stress on the tunnel is p_k (see Fig. 1 b). In the first excavation step the internal pressure is reduced from p_k to $(1-\beta)p_k$, hence displacements are accumulated which represents the settlement ahead of tunnel plus settlement of unsupported zone. In the next excavation step the primary support is installed and the internal pressure is reduced to zero hence the tunnel is fully excavated. In PLAXIS this can be done by using a reduced ultimate level of ΣMstage where the β-value corresponds to 1- ΣMstage.

In this way the three dimensional arching effects in longitudinal direction of the tunnel drift are considered in the two dimensional analysis.



a) Settlements of tunnel excavation
b) Step 1: unsupported zone
c) Step 2: supported zone

Fig. 1 Settlement of tunnel crown with respect to chainage of excavation face

The pre-relaxation factor is governed by the behaviour of the ground material, the time of installation of the support, excavation cross section, advance rate etc. The pre-relaxation factors β are determined in such way that approx. 1/3 of the overall displacements are accumulated before installation of the primary support. Thus the pre-relaxation factors vary with numerical analysis, advance rate and face.

The construction sequences and the corresponding values for ΣMstage (=1-β) for all additional analyses are shown in Tab. 13.

Tab. 13 Construction sequence of additional numerical analyses

Analysis 1				
Stage	**Description**	**ΣMstage**		**Comment**
0	Initial stress state	1.00		balance
1	Full face egress tunnel excavation	0.30		unbalance
2	Full face egress tunnel support installation	1.00		balance
3	Top heading main tunnel excavation	0.30		unbalance
4	Top heading main tunnel support installation	1.00		balance
5	Bench main tunnel excavation	0.30		unbalanced
6	Bench main tunnel support installation	1.00		balanced
Analysis 1: Apllication of pseudo-static EQ				
7	Earthquake loading	1.00		balance
Analysis 2 & 3				
Stage	**Description**	**ΣMstage**		**Comment**
		Analysis 2	Analysis 3	
0	Initial stress state	1.00	1.00	balance
1	Top heading egress tunnel excavation	0.40	0.95	unbalance
2	Top heading egress tunnel support installation	1.00	1.00	balance
3	Invert egress tunnel excavation	0.30	0.80	unbalance
4	Invert egress tunnel support installation	1.00	1.00	balanced
5	Top heading main tunnel excavation	0.50	0.92	unbalance
6	Top heading main tunnel support installation	0.40	0.45	unbalance
7	Temporary invert main tunnel excavation	0.50	0.45	unbalance
8	Temporary invert main tunnel support installation	1.00	1.00	balanced
9	Bench main tunnel excavation	0.50	0.50	unbalanced
10	Bench main tunnel support installation	0.40	0.40	unbalance
11	Invert main tunnel excavation	0.30	0.80	unbalance
12	Invert main tunnel support installation	1.00	1.00	balanced
Analysis 2 & 3: Apllication of pseudo-static EQ				
12	Earthquake loading	1.0	1.0	balance

5.4 Seismic Load Case

The seismic loading due to earthquake is considered in the numerical analysis by applying pseudo-static horizontal and vertical accelerations onto the model. The accelerations considered in the design are based on the relevant Indian Standards and on the peak ground accelerations determined by a seismic study [1], where different seismic scenarios were investigated in the project region. The used horizontal and vertical coefficient k_h and k_v of acceleration due to earthquake at ground surface are 0.24 and 0.12. These values coincide with peak ground accelerations given in IS-1893-1 [3] for seismic zone IV and correspond to the maximum credible earthquake. However in general the design basis earthquake is used in seismic analyses according to IS-1893-1 [3], thus the design is on the conservative side. The seismic accelerations in structures below 30 m of the ground surface can be taken as half the design value (see [3]). Consequently seismic accelerations of 0.12g horizontal and 0.06g vertical are applied in the numerical analysis. In analysis 3 (Support Category G) the design basis earthquake is used for the structural analysis.

Liquefaction phenomena are unlikely, based on the predicted ground types, presented in DPR Volume IV: Geotechnical Tunnel Design Report, and therefore not investigated during the primary lining design.

The structural seismic design is carried out in Section 6 with respect to the internal reactions due to seismic loading, determined in the numerical model and according to EN 1992-1-1 [2].

5.5 Simulations and Results

5.5.1 Simulations

Tab. 14 shows again the numerically investigated Primary Support Categories.

Tab. 14 Support Category cases investigated by numerical analyses

Analysis	Support Category	Ground Type	Over burden	Calculation (PLAXIS File)
1	SC C	GT3_mean	500 m	8482B_II-ZOT_SCC-GT3-mean-500 m
2	SC E	GT6_mean	455 m	8482B_II-ZOT_SCE-GT6-mean-600 m
3	SC G	GT7_mean	660 m	8482B_II-ZOT_SCG-GT7-mean-660m

5.5.2 Results for Analysis 1 – SC C, GT3_mean, overburden 500 m

Main Input Parameters

Rock Type	Ground Type	γ [kN/m³]	φ [°]	c [kPa]	E [MPa]	ν [-]
Jointed/Foliated Metabasite (Green Schist)	GT3	28	40,0	10000	70000	0,15

γ Specific weight
φ Friction angle
c Cohesion
E Young's modulus (primary loading); E-module unload/reload = 3xE
ν Poisson's ratio

Shotcrete			
shotcrete thickness	d	[m]	0,15
spacing steel sets	l	[m]	1,75
Poisson's ratio	ν	[-]	0,20
PLAXIS Plate-Element	**EA**	[kN/m]	5,36E+05
	EI	[kNm²/m]	935
	w	[kN/m/m]	3,75

EA / EI Axial stiffness / bending stiffness of the element per m tunnel length
w Element weight per m tunnel length

Rock Bolts				
type			ET Grouted Ø 28	MT Grouted Ø 36
spacing	L_{spac}	[m]	1,75	1,75
PLAXIS	**EA**	[kN/m]	73920	122285
Geogrid-Element	**N_p**	[kN/m]	125	149

L_{spac} Spacing of rock bolts in direction of tunnel length
EA Axial stiffness of rock bolts per meter tunnel
N_p Maximum axial tensional force per meter tunnel
ET Egress Tunnel
MT Main Tunnel

Stresses, Plastic Zone

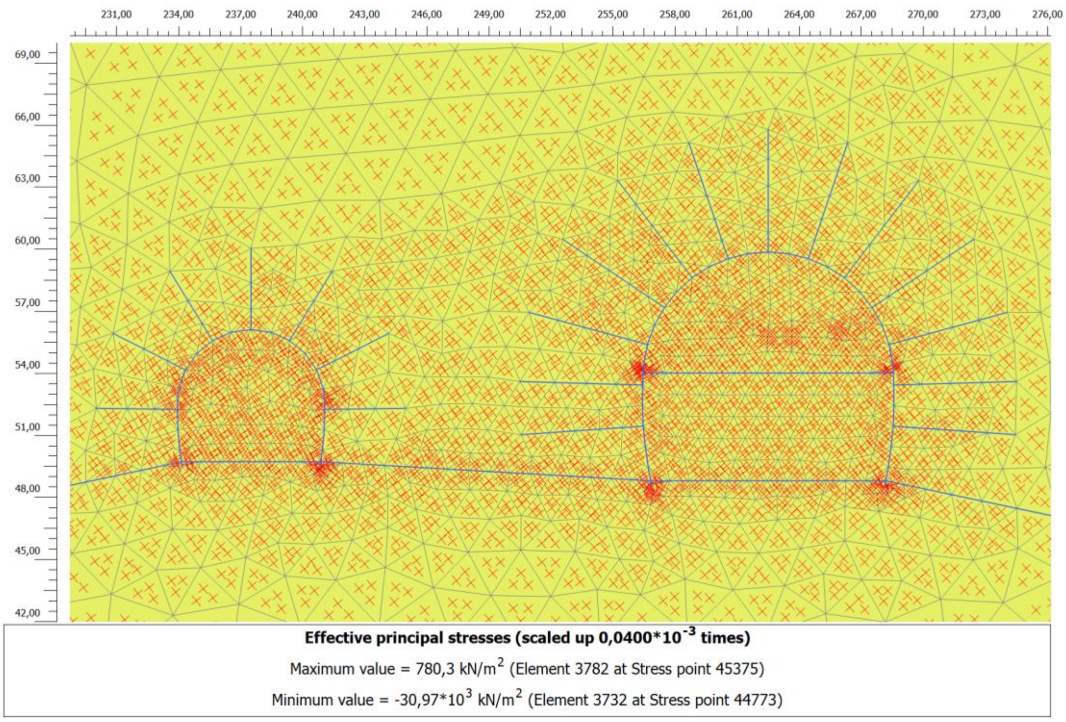


Fig. 2 Initial mean principal effective stresses direction

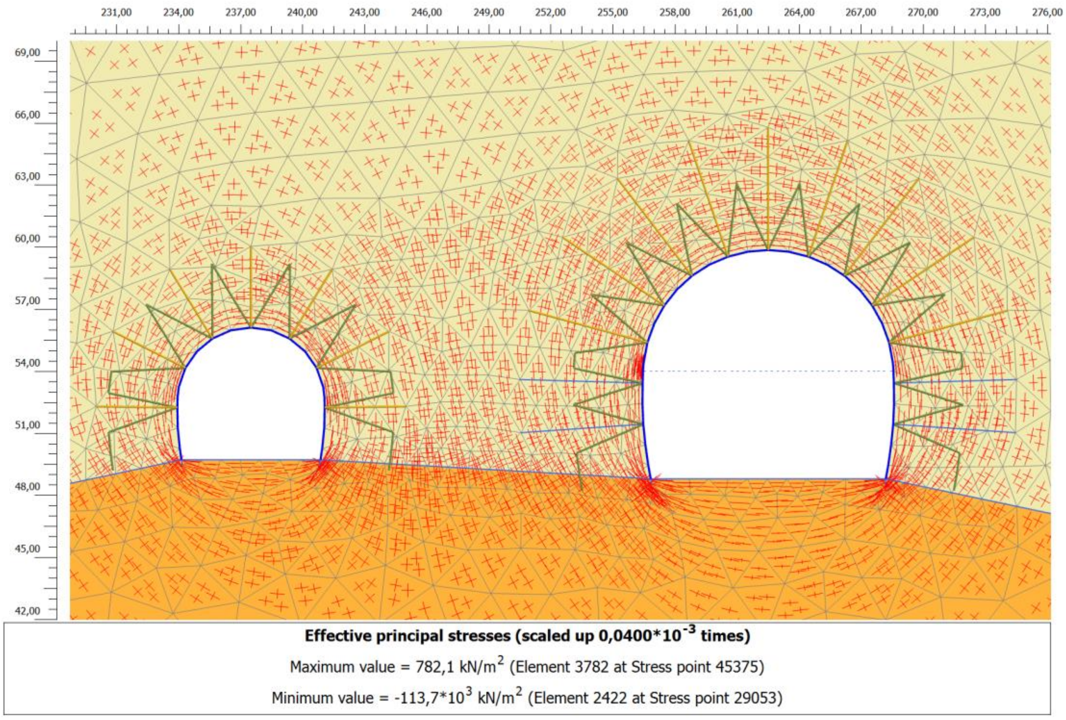


Fig. 3 Mean principal effective stresses direction

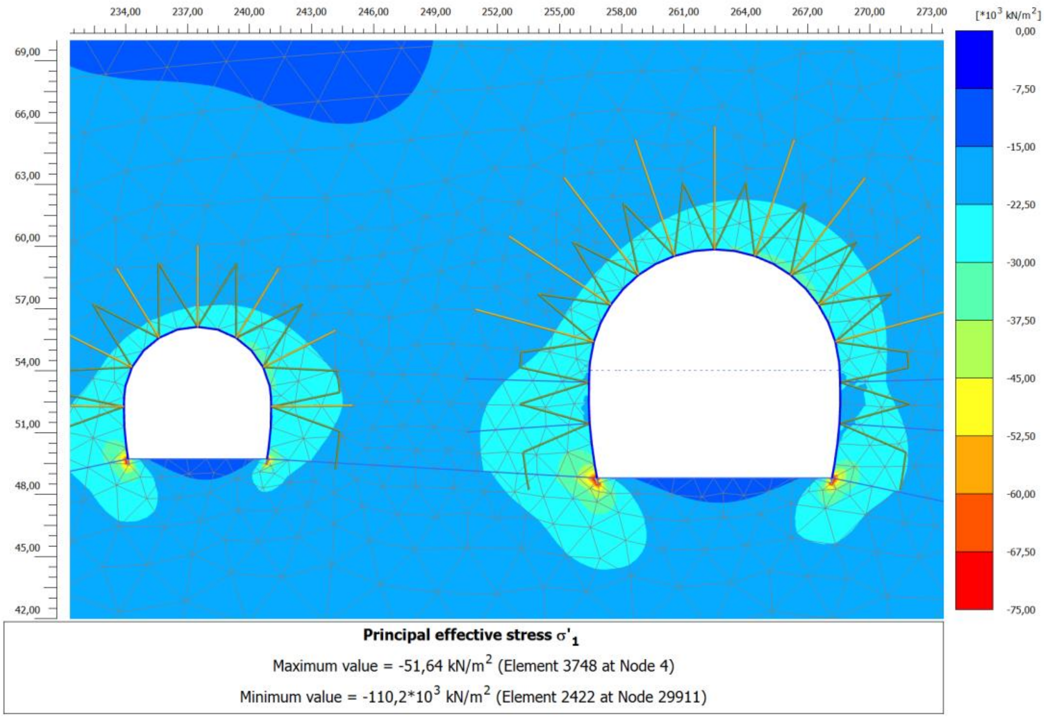


Fig. 4 Mean principal effective stresses (shadings)

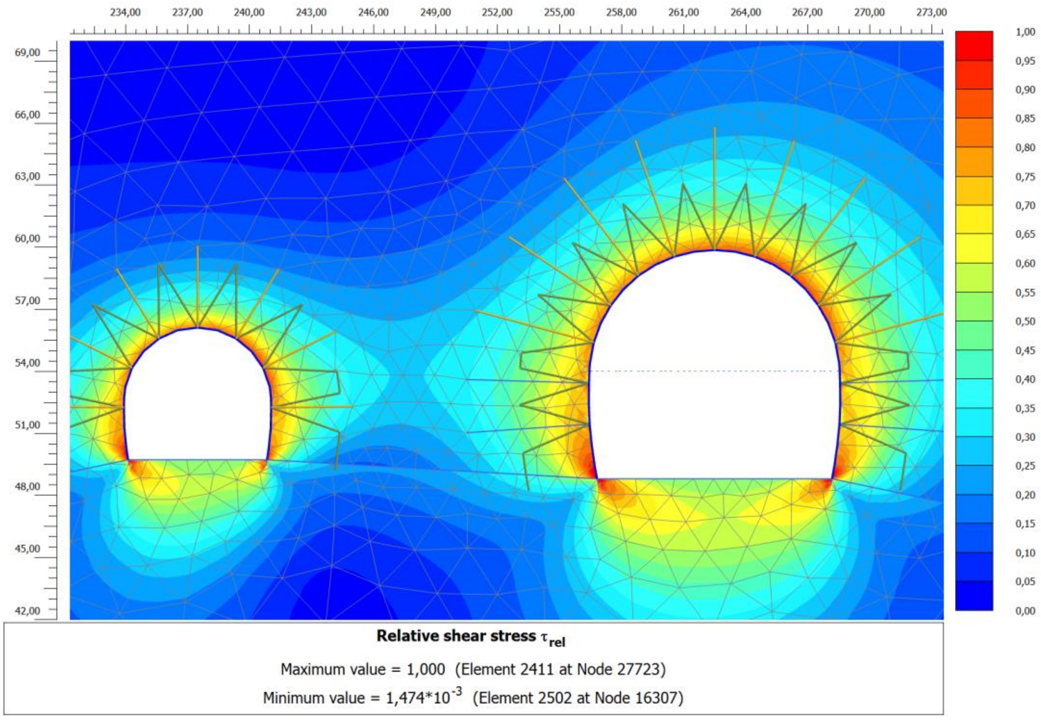


Fig. 5 Relative shear stresses (shadings) = strength factor contours

Fig. 2 shows the initial mean principal stress directions before tunnel excavation. Due to the non-horizontal ground surface the principal stress directions are not orientated vertical/horizontal. The principal stress directions are influenced by the tunnel excavation as shown in Fig. 3. In Fig. 5 the plastic zone around the excavation (red coloured zone) are given. There are only small areas of plastic zones around the tunnel excavation, especially in areas with high stresses due to numerical reasons such as the contact area of the primary lining and the bench floor.

Deformations

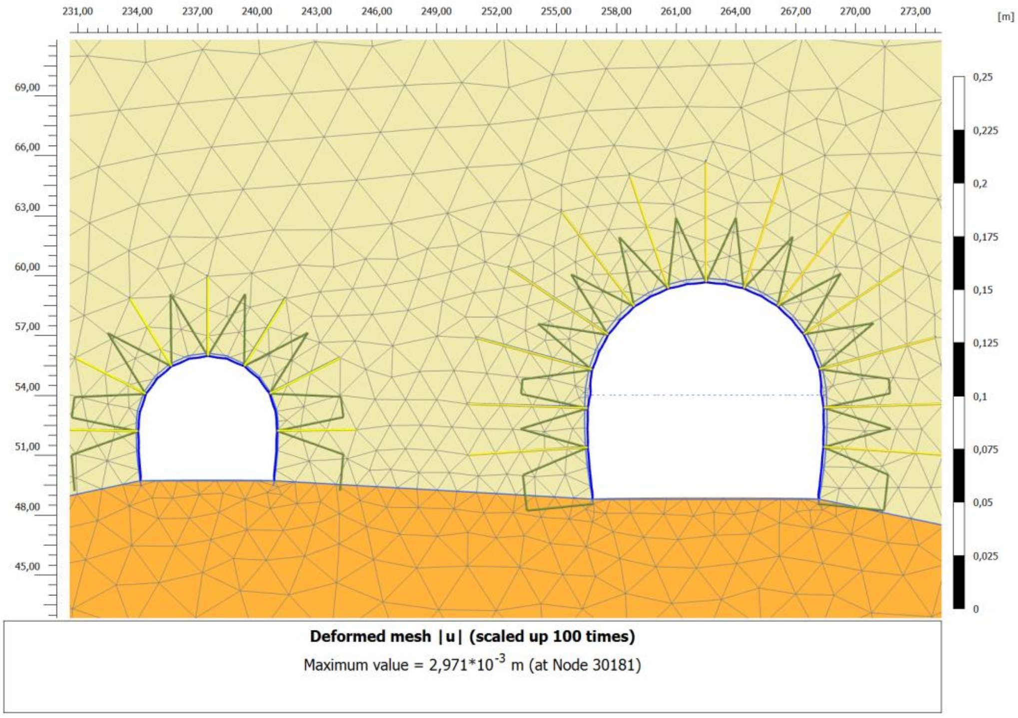


Fig. 6 Deformed mesh

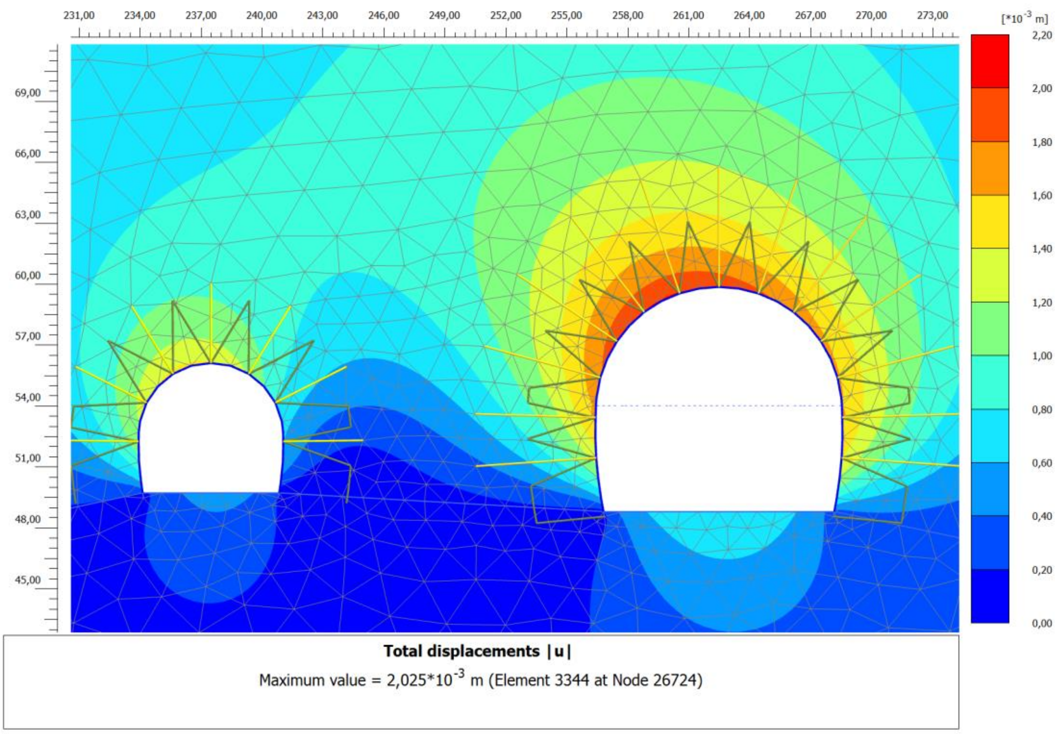


Fig. 7 Total displacements (shadings)

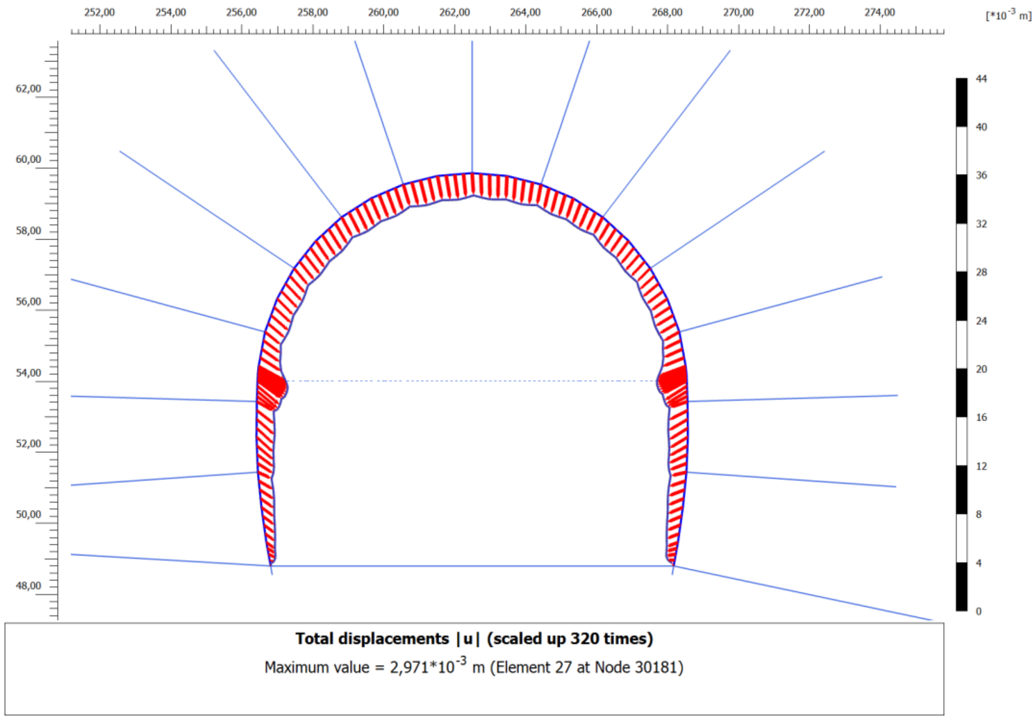


Fig. 8 Total displacements of primary lining, main tunnel

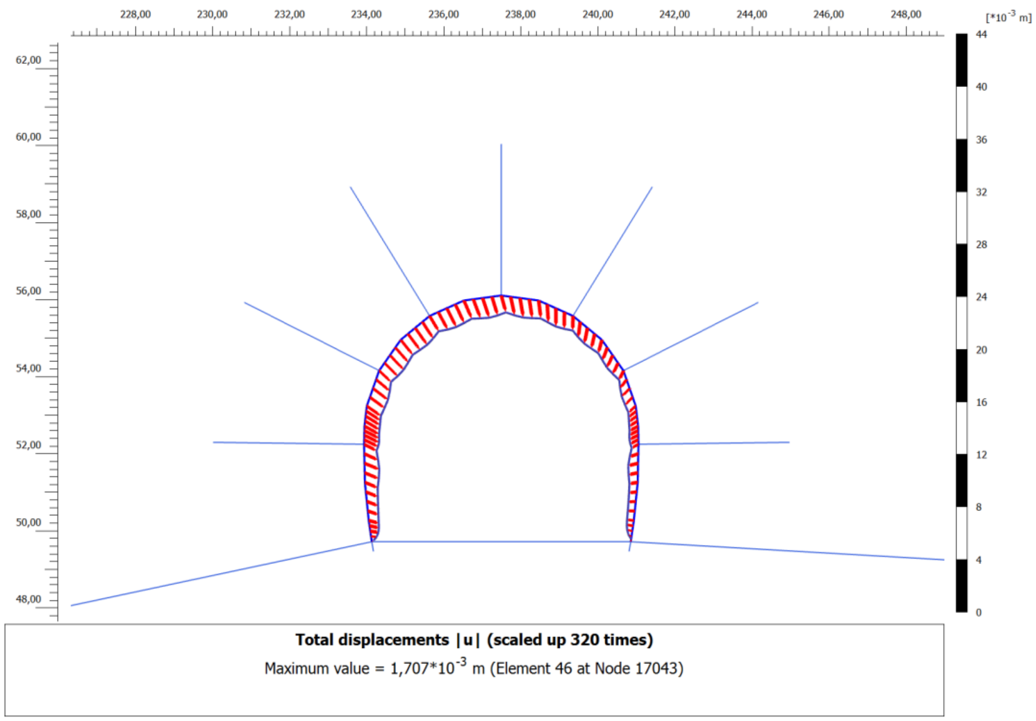


Fig. 9 Total displacements of primary lining, egress tunnel

The maximum vertical deformation occurs at the top heading of the main tunnel with app. 3 mm. The maximum horizontal deformation at the lower part of the bench is also app. 2.5 mm. In the egress tunnel approx. half of the deformations of the main tunnel are generated. The given displacements are the overall displacements (displacements ahead of the face and displacements in the unsupported zone plus displacements of the lining, for detailed description refer to Section 5.3). The displacements ahead of the tunnel and in the unsupported zone are determined to approx. 0.6 mm in main and 0.3 mm in egress tunnel.

Internal Forces

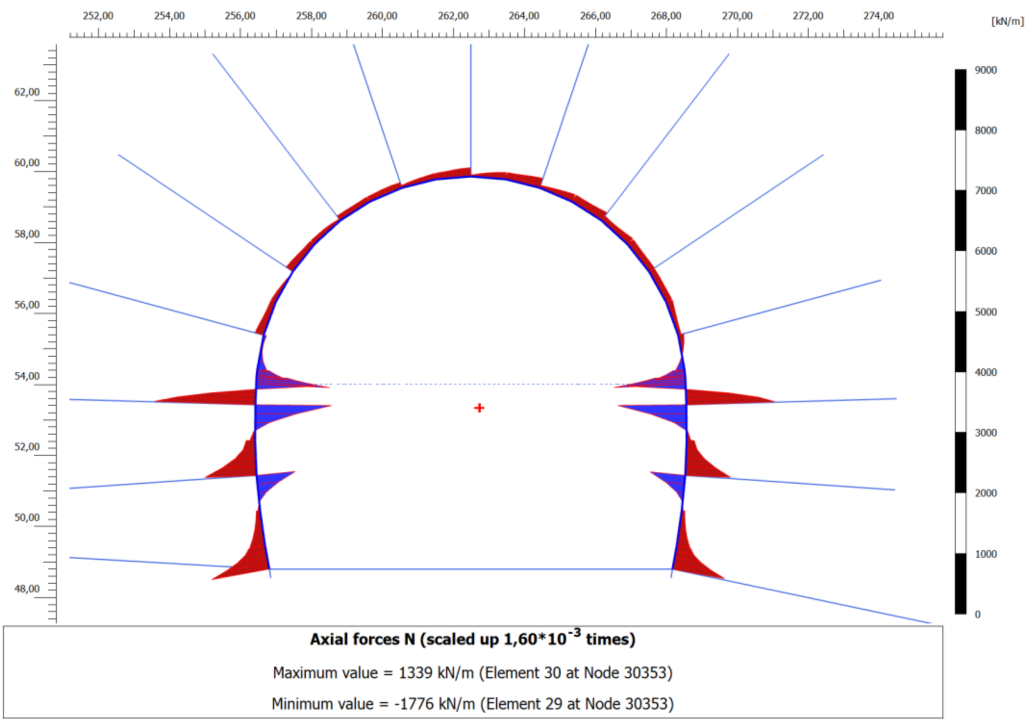


Fig. 10 Axial forces of primary lining, main tunnel

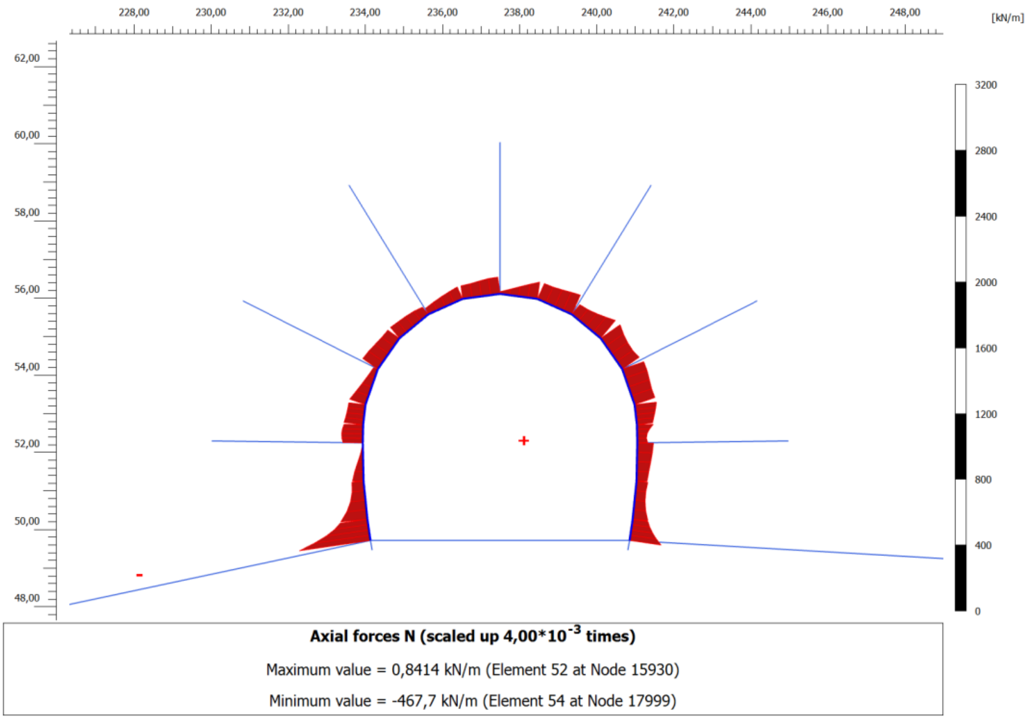


Fig. 11 Axial forces of primary lining, egress tunnel

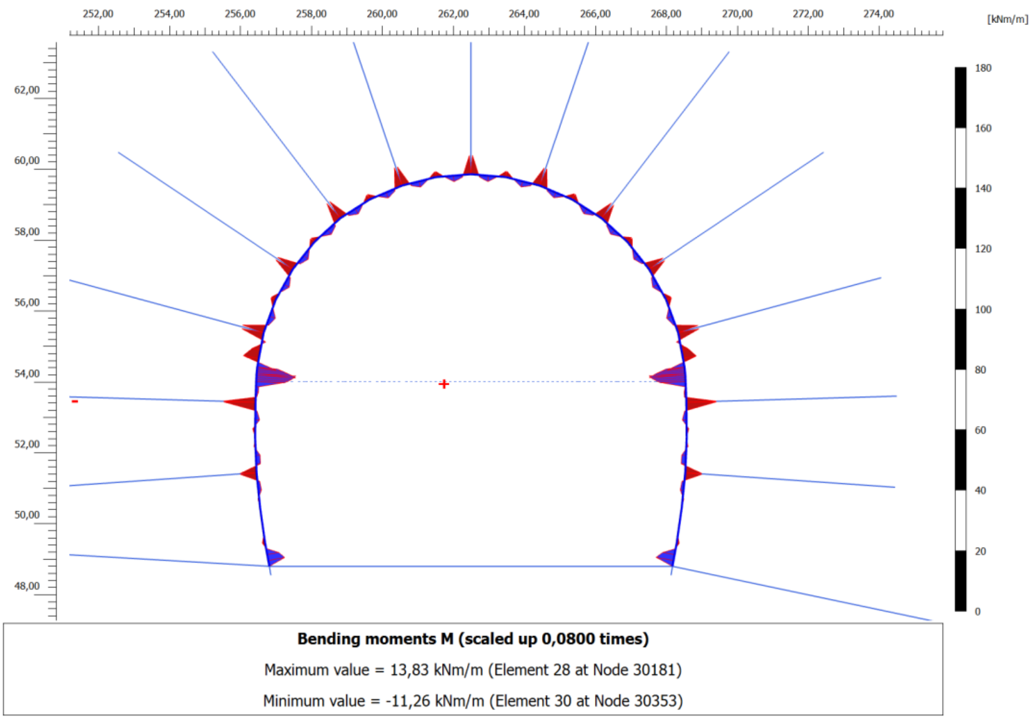
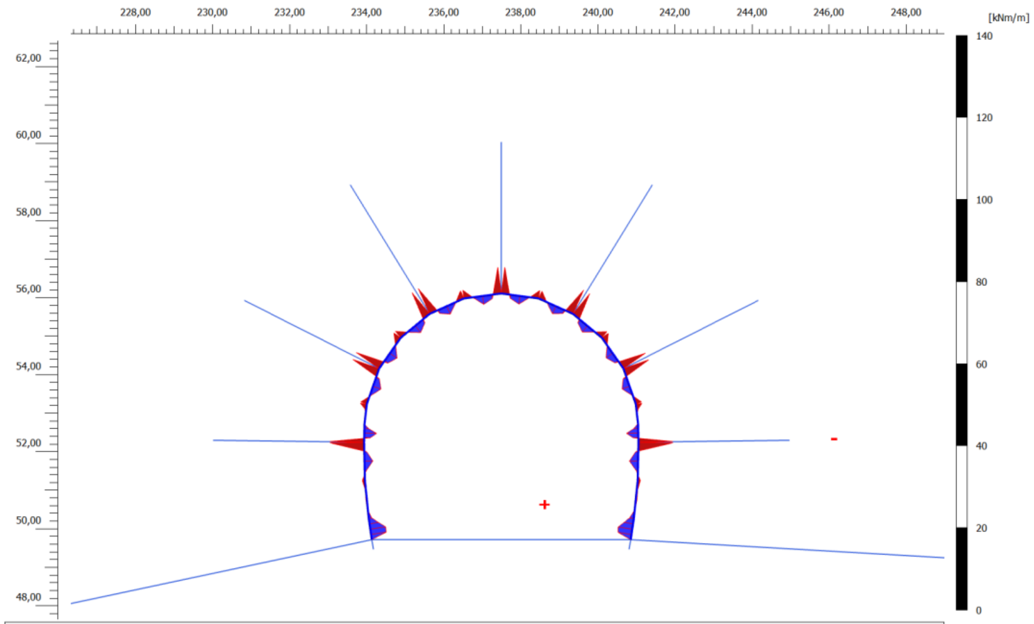


Fig. 12 Bending moments of primary lining, main tunnel



Bending moments M (scaled up 0,100 times)
Maximum value = 4,035 kNm/m (Element 54 at Node 17095)
Minimum value = -8,930 kNm/m (Element 38 at Node 23021)

Fig. 13 Bending moments of primary lining, main tunnel

Fig. 10 to Fig. 13 show the internal forces of the final calculation step.

The maximum axial forces are determined as per analysis to approx. 1.3 MN compression and 1.7 MN tension for the main tunnel and approx. 0.5 MN compression for the egress tunnel. The value of axial tension is resulting from numerical singularities in the connection between bench and top heading lining and is therefore neglected in further design of the primary lining (see Section 6). The average axial force in the primary lining is determined to approx. 0,25 MN compression in the main tunnel crown and 0,15 MN in the egress tunnel.

The bending moments are determined below approx. ±15 kN/m in both tunnels.

The structural design of the primary lining, based on the above given internal forces is given Section 6.

<u>Radial Bolting</u>

Fig. 14 and Fig. 15 show the axial forces in the rock bolts of the final calculation step. It can be shown that no rock bolt reach the yield strength.

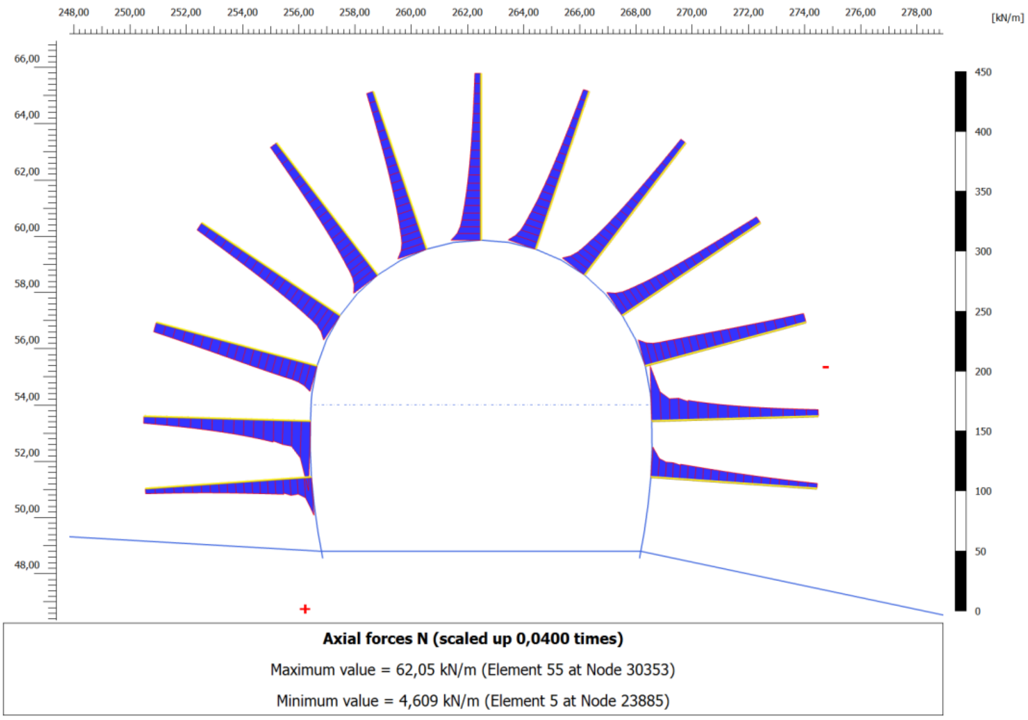


Fig. 14 Axial forces of rock bolts, main tunnel

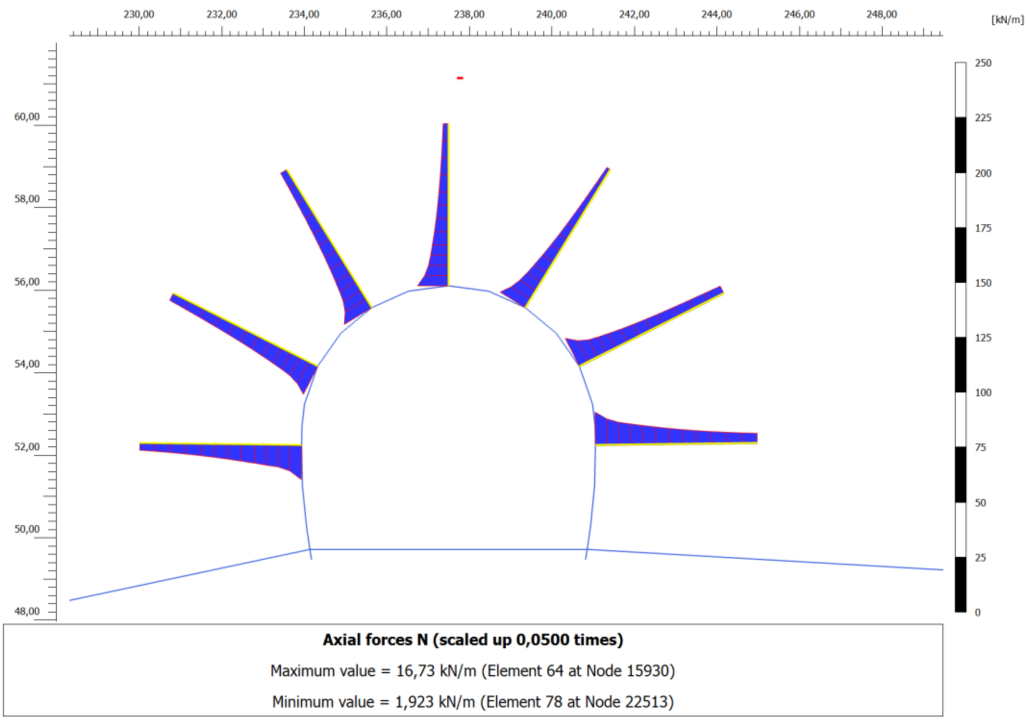


Fig. 15 Axial forces of rock bolts, egress tunnel

5.5.3 Results for Analysis 2 – SC E, GT6_mean, overburden 455 m

Main Input Parameters

Rock Type	**Ground Type**	γ [kN/m³]	φ [°]	**c** [kPa]	**E** [MPa]	ν [-]
Intensively Foliated Phyllites and Slate (Zojila Formation)	GT6	28,0	27,0	2000	6000	0,15

γ Specific weight
φ Friction angle
c Cohesion
E Young's modulus (primary loading); E-module unload/reload = 3xE
ν Poisson's ratio

Shotcrete					
shotcrete thickness	d	[m]	0,20 *)	0,25 **)	0,25
spacing steel sets	l	[m]			1,25
Poisson's ratio	ν	[-]	0,20	0,20	0,20
PLAXIS Plate-Element	**EA**	[kN/m]	5,00E+05	6,25E+05	8,50E+05
	EI	[kNm²/m]	1667	3255	4274
	w	[kN/m/m]	5,00	6,25	6,25

*) temporary invert and **) invert

Rock Bolts				
type			ET Grouted Ø 28	MT Grouted Ø 36
spacing	L_{spac}	[m]	1,25	1,25
PLAXIS Geogrid-Element	**EA**	[kN/m]	86240	170856
	N_p	[kN/m]	147	208

L_{spac} Spacing of rock bolts in direction of tunnel length
EA Axial stiffness of rock bolts per meter tunnel
N_p Maximum axial tensional force per meter tunnel
ET Egress Tunnel
MT Main Tunnel

<u>Stresses, Plastic Zone</u>

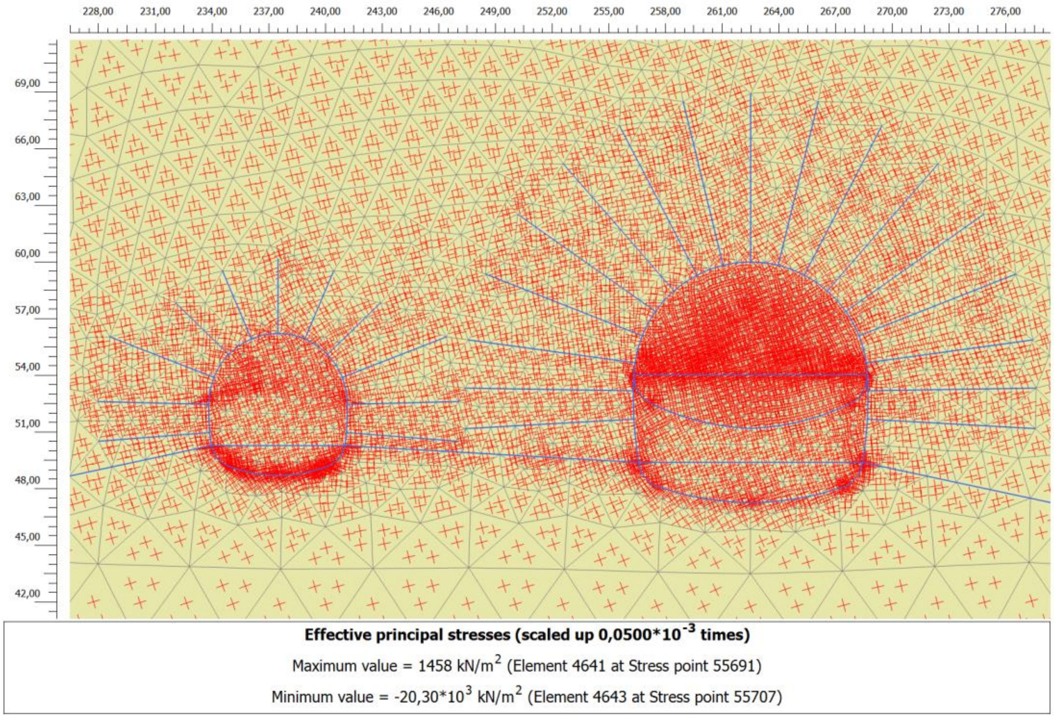


Fig. 16 Initial mean principal effective stresses direction

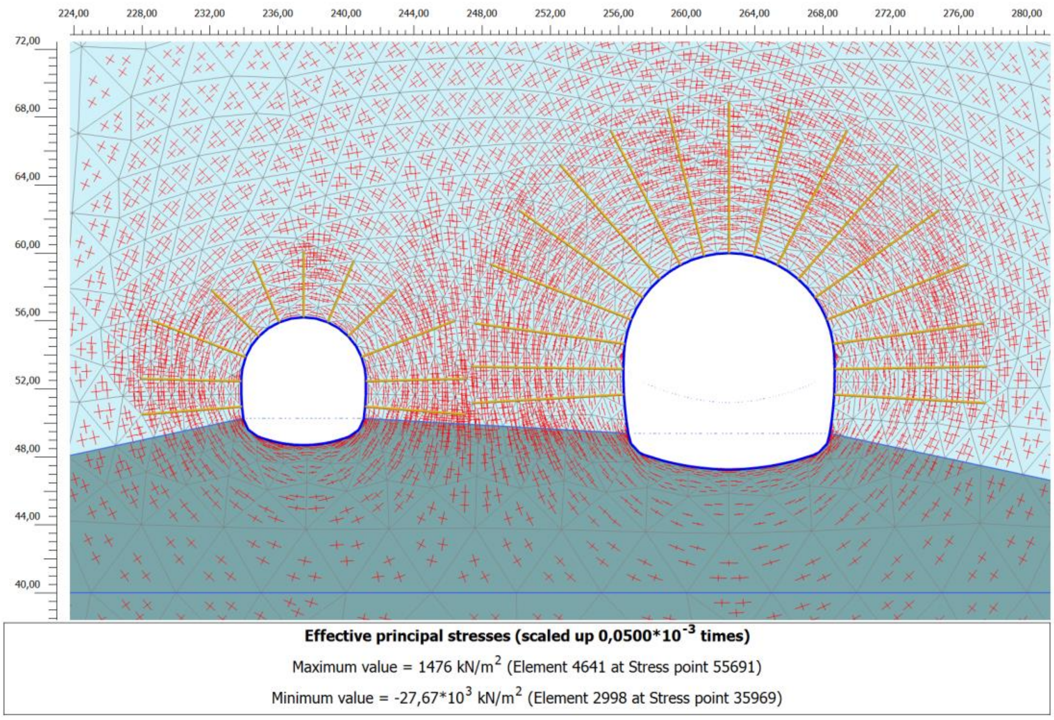


Fig. 17 Mean principal effective stresses direction

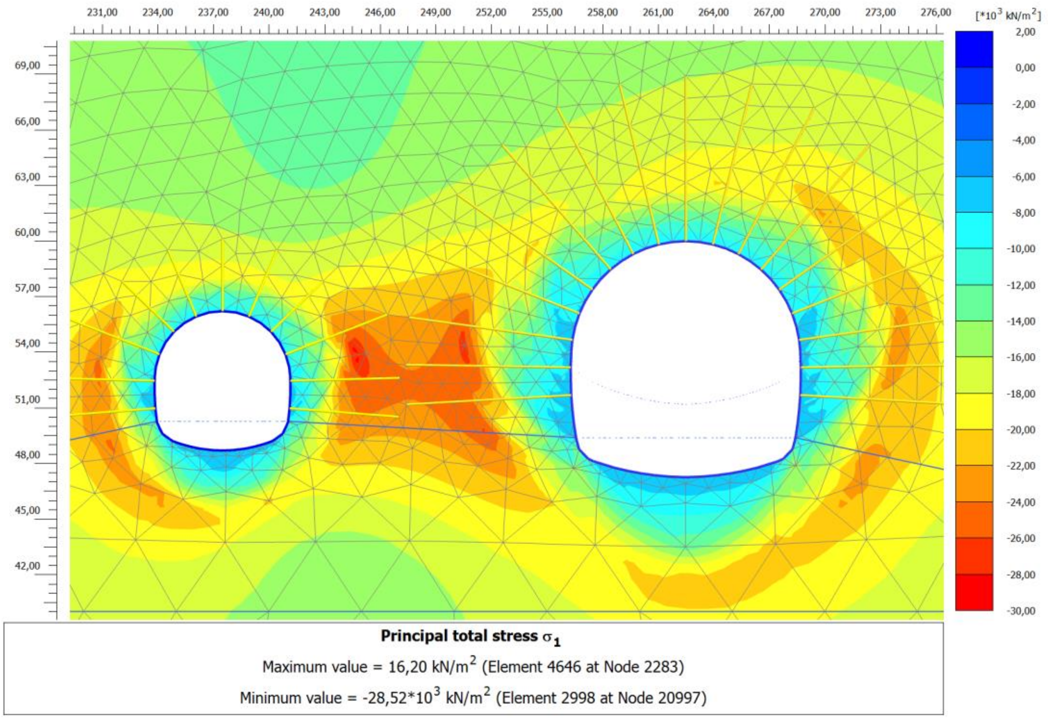


Fig. 18 Mean principal effective stresses (shadings)

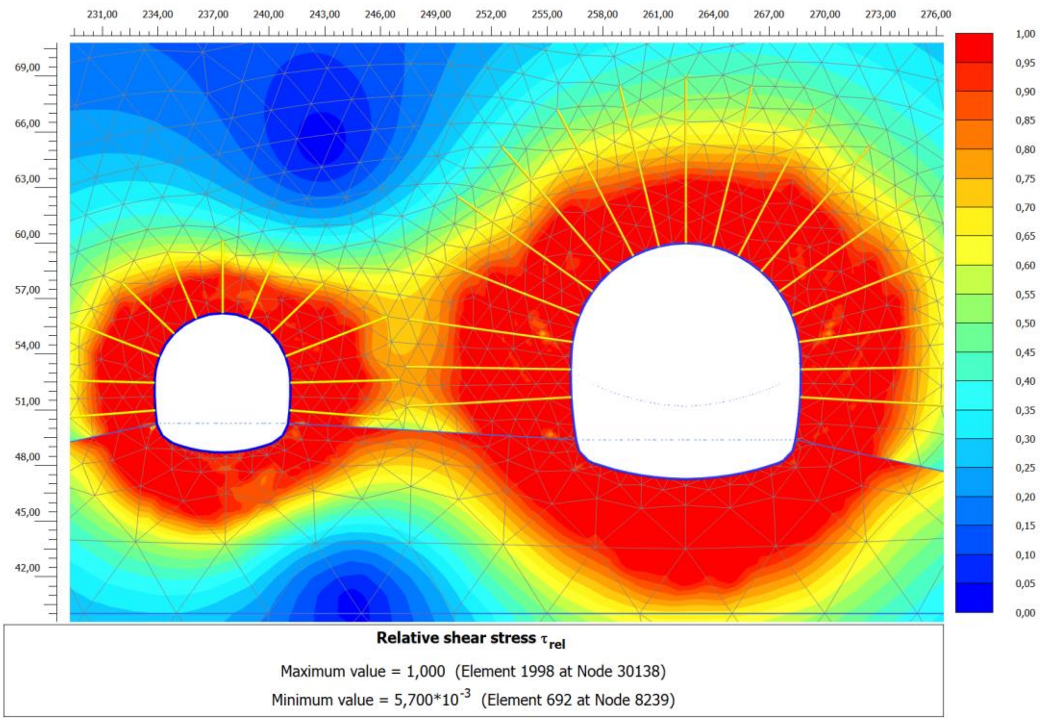


Fig. 19 Relative shear stresses (shadings) = strength factor contours

The initial mean principal stress directions are not orientated in vertical/horizontal direction due to the non-horizontal ground surface above the tunnel (see Fig. 16).

The mean principle stress directions are influenced by the tunnel excavation and after excavation orientated according to the tunnel geometry (see Fig. 17). Fig. 19 shows the plastic zone around the excavation (red coloured zone) and the strength factor contours. It is shown that the plastic radius around the main tunnel is approx. 6 m and around the egress tunnel approx. 3 m. The plastic zones of the main and egress tunnel do not interfere (see Fig. 19).

Deformations

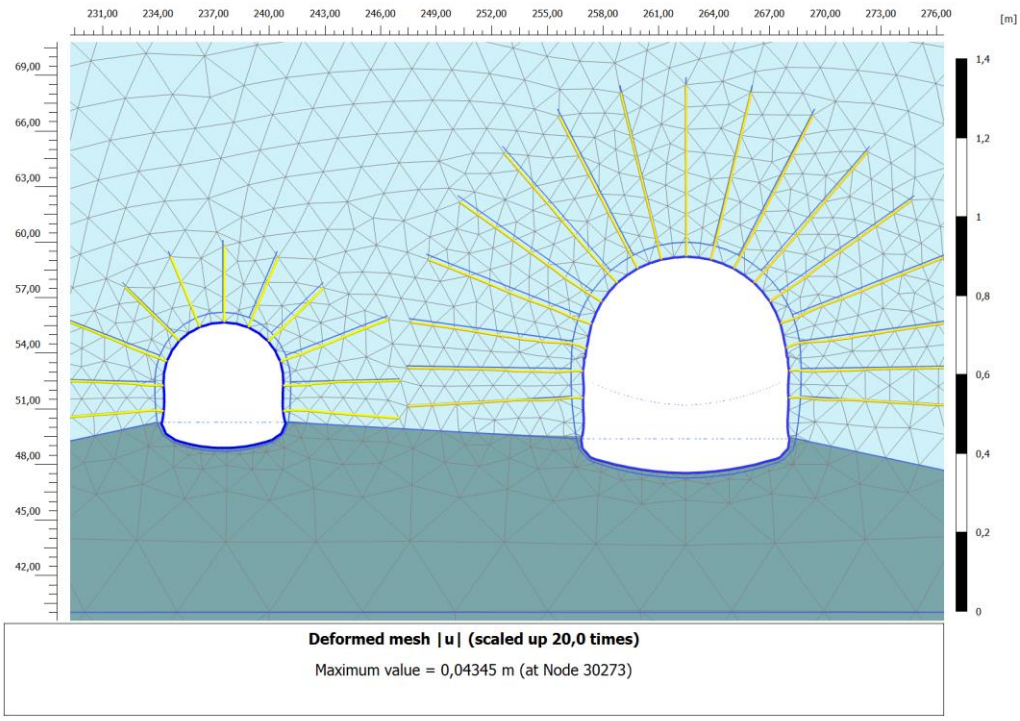


Fig. 20 Deformed mesh

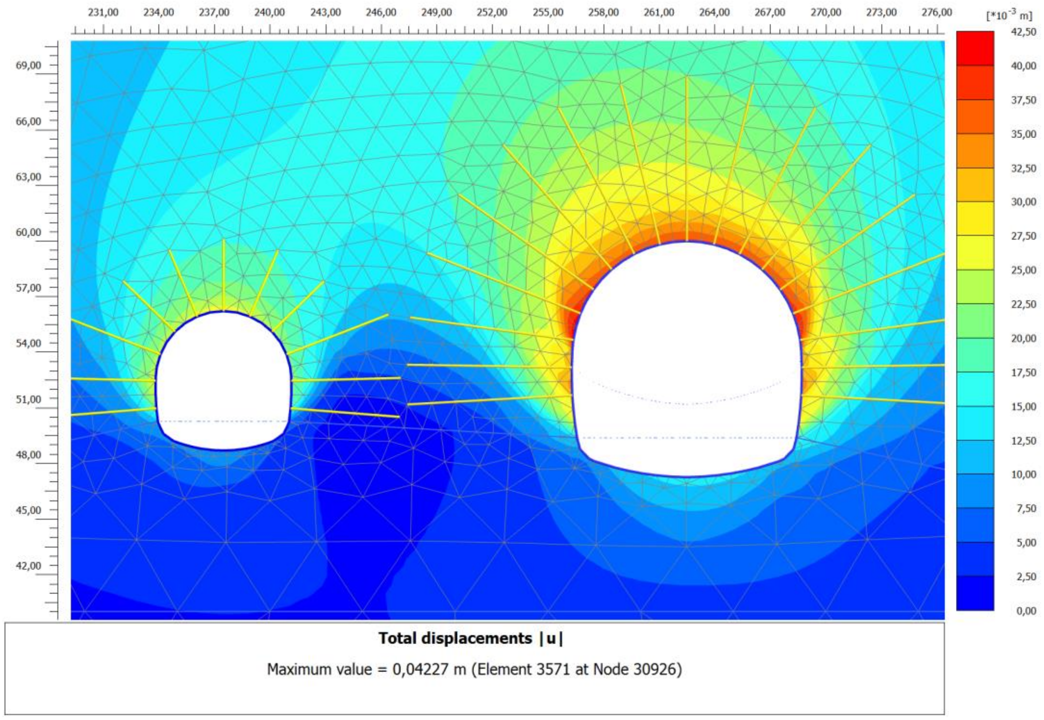


Fig. 21 Total displacements (shadings)

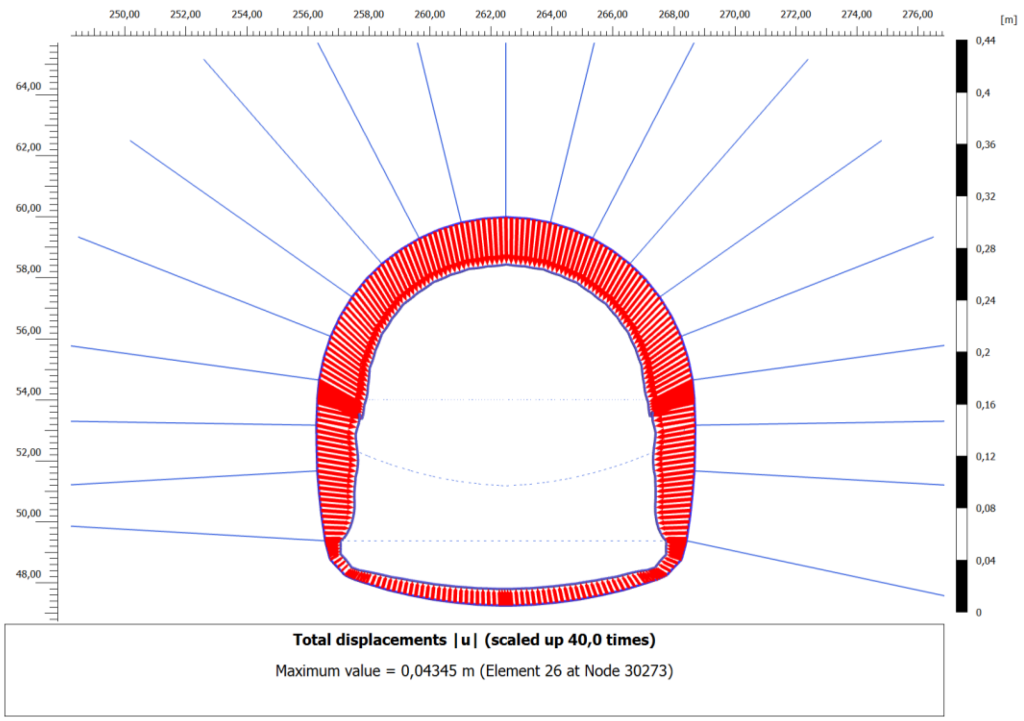


Fig. 22 Total displacements of primary lining, main tunnel

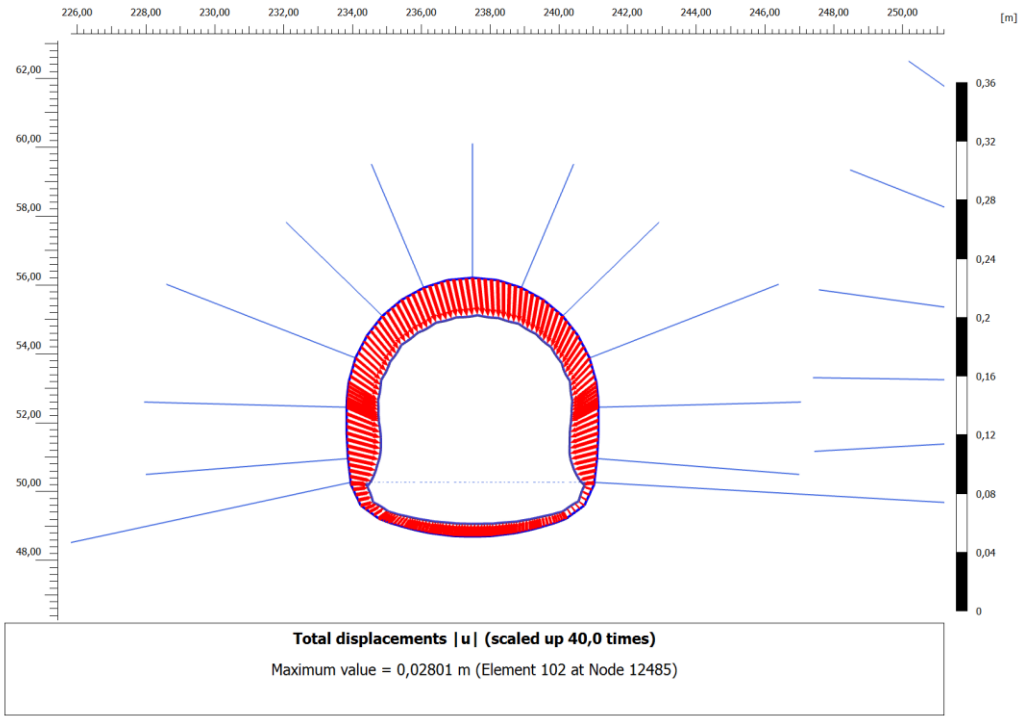


Fig. 23 Total displacements of primary lining, egress tunnel

The maximum vertical deformation occurs at the top heading in both tunnels with approx. 43 mm in main tunnel and approx. 28 mm in egress tunnel. The maximum vertical heave of the invert is approx. 13 mm (main tunnel) and 9 mm egress tunnel respectively. The maximum horizontal deformation at the lower part of the bench is app. 36 mm for main and approx. 26 mm for egress tunnel excavation. The given displacements are the overall displacements (displacements ahead of the face and displacements in the unsupported zone plus displacements of the lining, for detailed description refer to Section 5.3). The displacements ahead of the tunnel and in the unsupported zone are determined to approx. 12 mm in main and 6 mm in egress tunnel.

Internal Forces

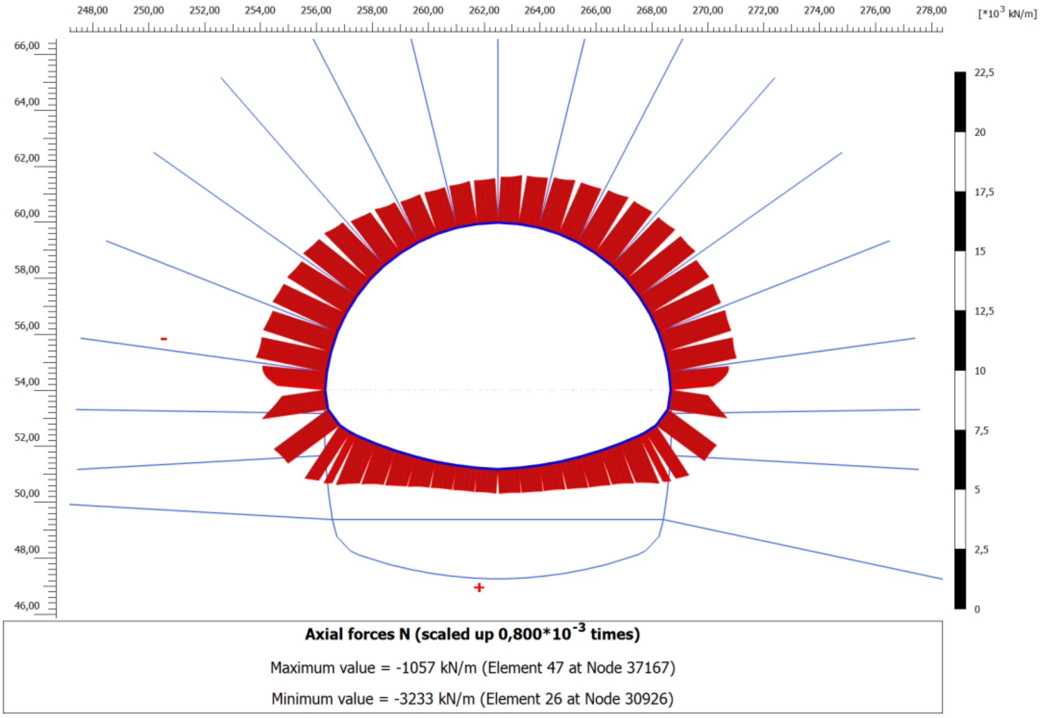


Fig. 24 Axial forces of temporary invert, main tunnel

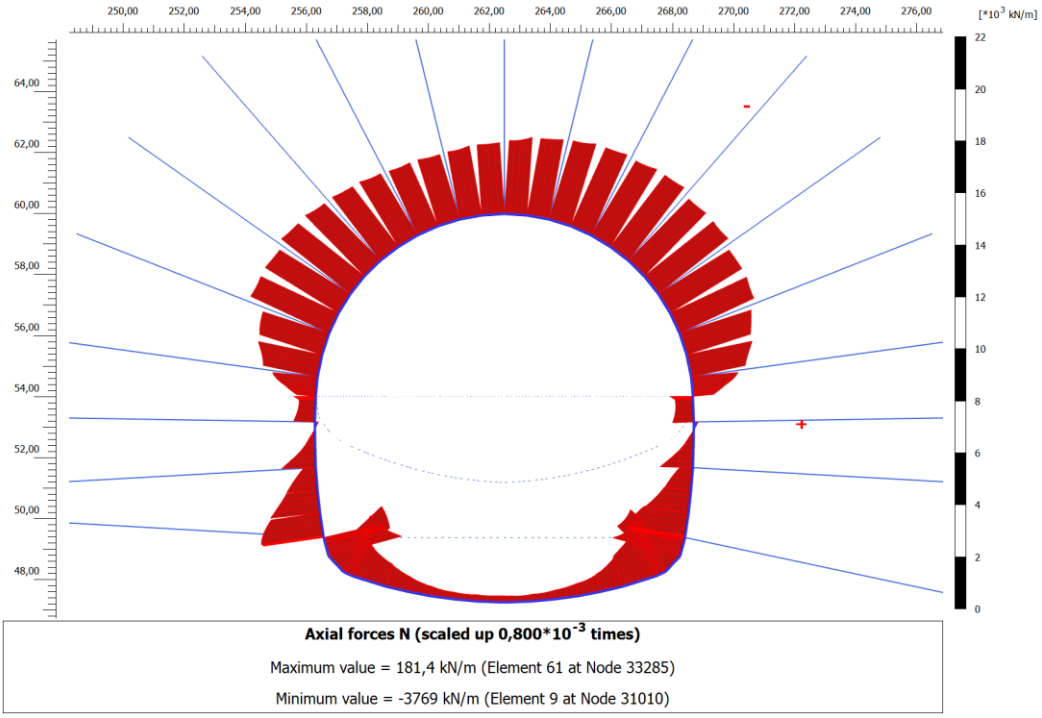


Fig. 25 Axial forces of primary lining (final step), main tunnel

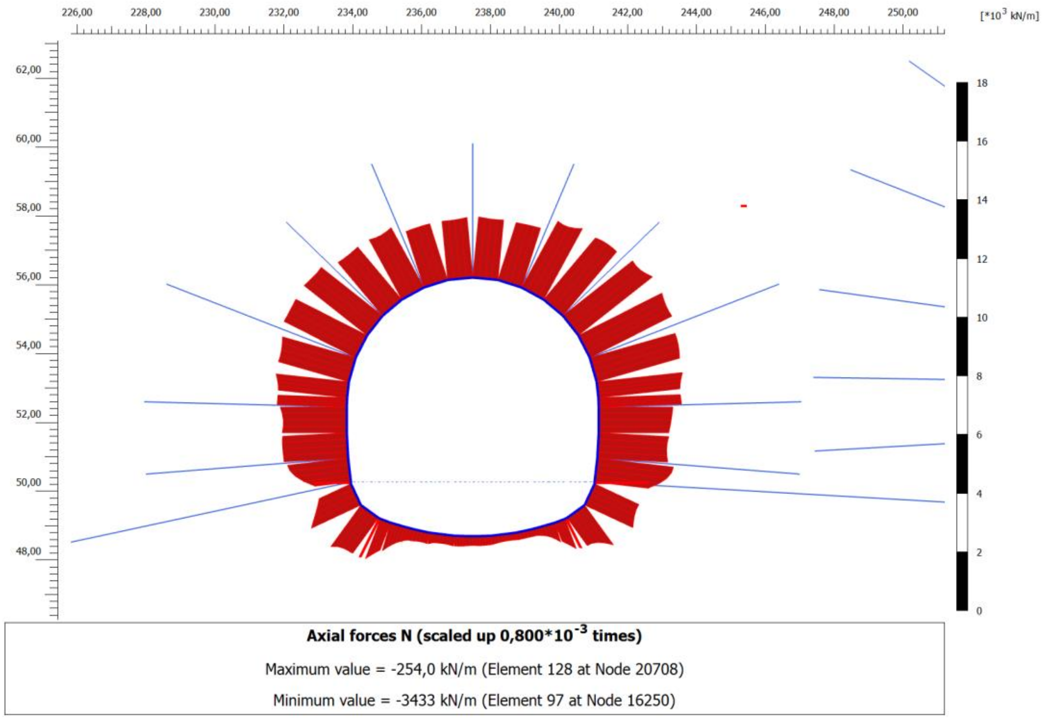


Fig. 26 Axial forces of primary lining (final step), egress tunnel

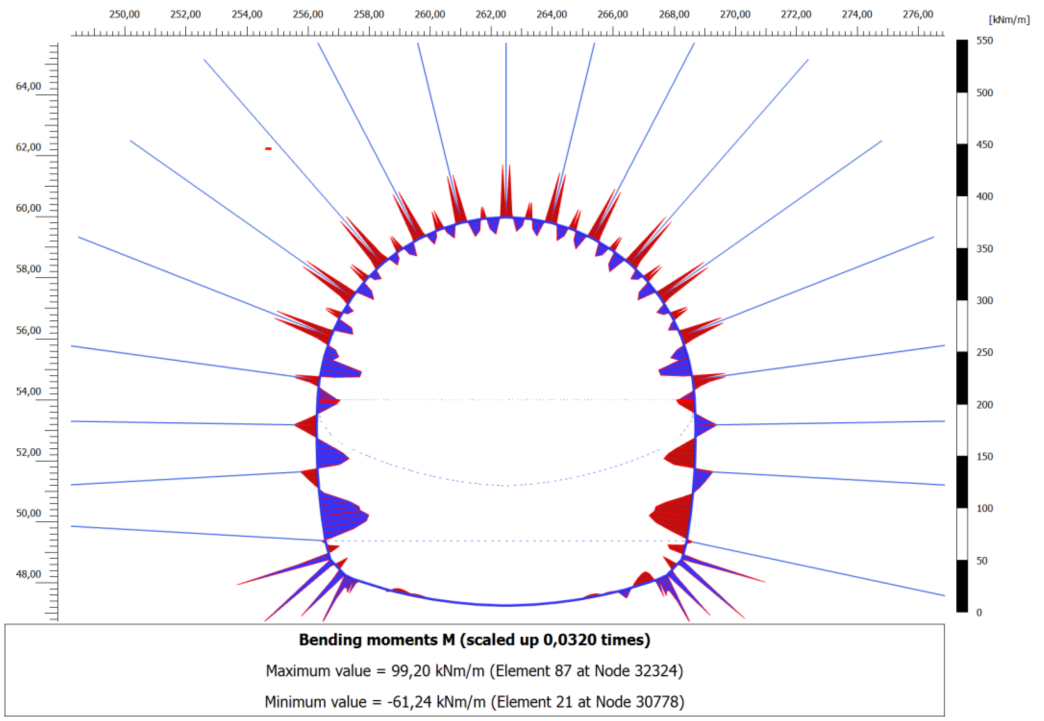


Fig. 27 Bending moments of primary lining, main tunnel (final step)

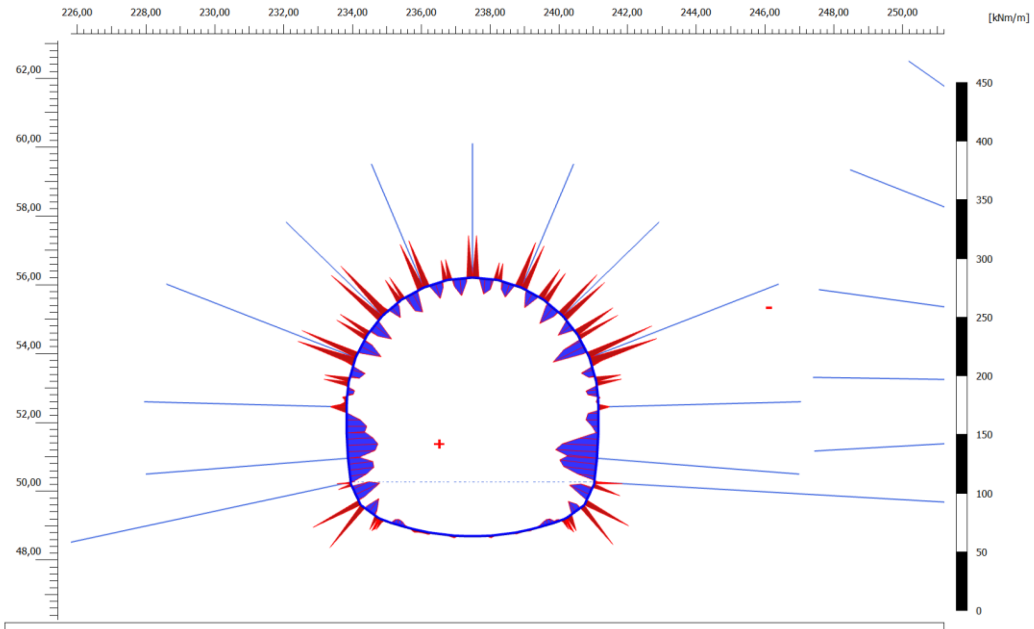


Fig. 28 Bending moments of primary lining, egress tunnel (final step)

Fig. 25 to Fig. 28 show the internal forces of the final calculation step.

The maximum normal force in the primary lining is determined to approx. 3.8 MN in main and 3.4 MN in egress tunnel primary lining. Due to the closure of the primary lining no singularities occur in this numerical analysis (compare analysis 1 no closure of primary lining).

The maximum bending moments are determined to approx. 100 kNm in main and approx. -65 kNm in egress tunnel respectively.

The structural design of the primary lining, based on the above internal forces is given in Section 6.

<u>Radial Bolting</u>

Fig. 29 and Fig. 30 show the axial forces in the rock bolts of the final calculation step. It can be seen that the rock bolts reach the ultimate level in most parts.

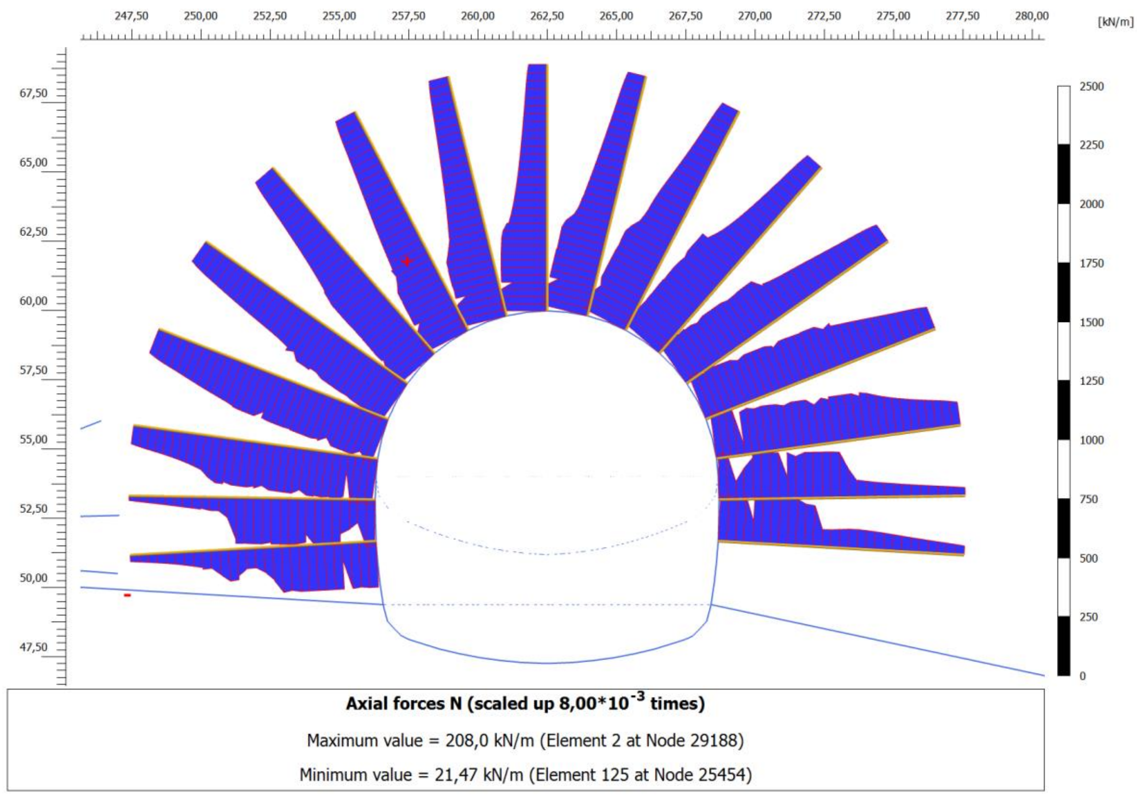


Fig. 29 Axial forces of rock bolts, main tunnel

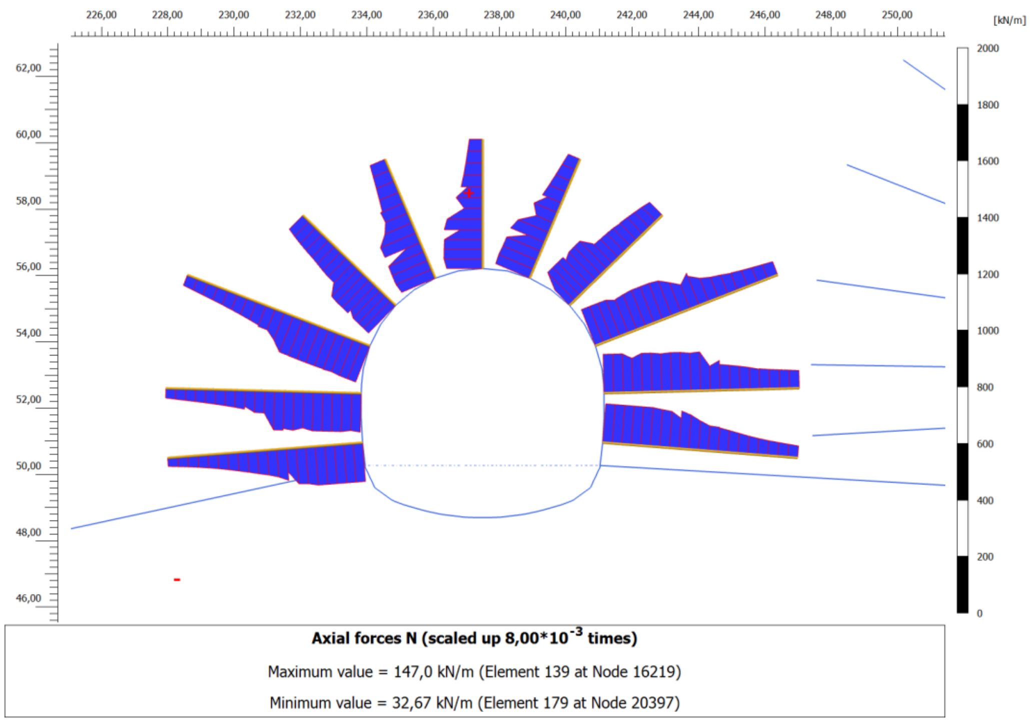


Fig. 30 Axial forces of rock bolts, egress tunnel

5.5.4 Results for Analysis 3 – SC E, GT7_mean, overburden 660 m

Main Input Parameters

Rock Type	Ground Type	γ [kN/m³]	φ [°]	c [kPa]	E [MPa]	ν [-]
Faulted Phyllites and Slates (Zojila Formation)	GT7	27,0	23,0	1000	1500	0,20

γ Specific weight
φ Friction angle
c Cohesion
E Young's modulus (primary loading); E-module unload/reload = 3xE
ν Poisson's ratio

Shotcrete					
shotcrete thickness	d	[m]	0,25 *)	0,30 **)	0,30
spacing steel sets	l	[m]			1,25
Poisson's ratio	ν	[-]	0,20	0,20	0,20
PLAXIS Plate-Element	**EA**	[kN/m]	6,25E+05	7,00E+05	9,75E+05
	EI	[kNm²/m]	3255	5625	6644
	w	[kN/m/m]	6,25	7,50	7,50

*) temporary invert and **) invert

Rock Bolts				
type			ET Grouted Ø 28	MT Grouted Ø 36
spacing	L_{spac}	[m]	1,25	1,25
PLAXIS Geogrid-Element	**EA**	[kN/m]	86240	170856
	$\mathbf{N_p}$	[kN/m]	147	208

L_{spac} Spacing of rock bolts in direction of tunnel length
EA Axial stiffness of rock bolts per meter tunnel
N_p Maximum axial tensional force per meter tunnel
ET Egress Tunnel
MT Main Tunnel

<u>Stresses, Plastic Zone</u>

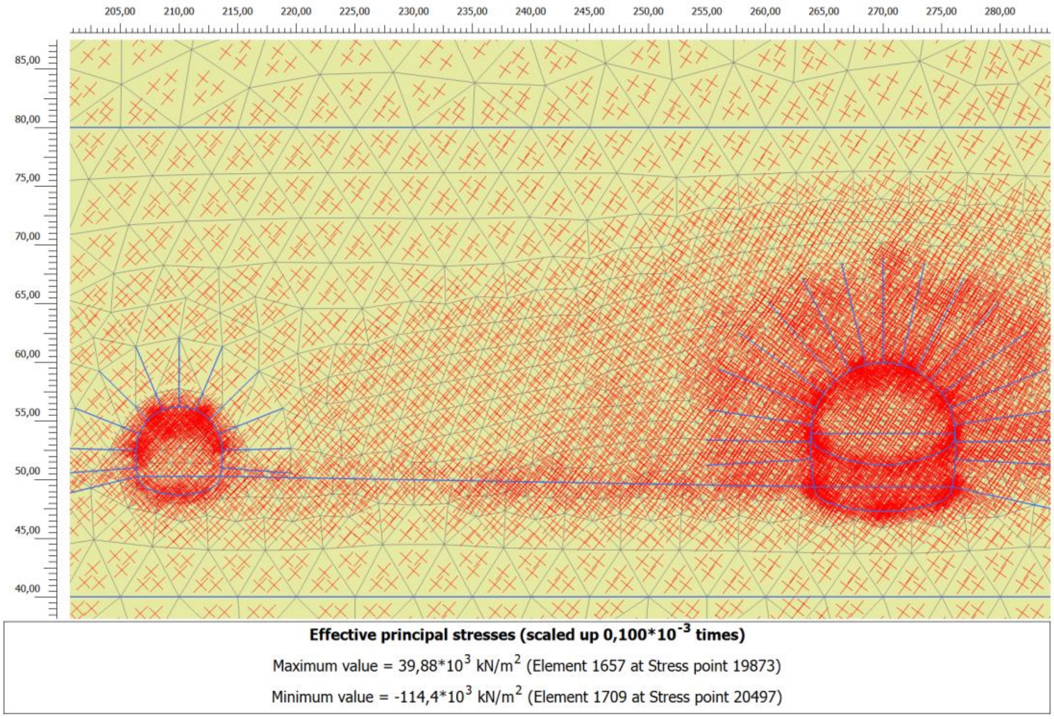


Fig. 31 Initial mean principal effective stresses direction

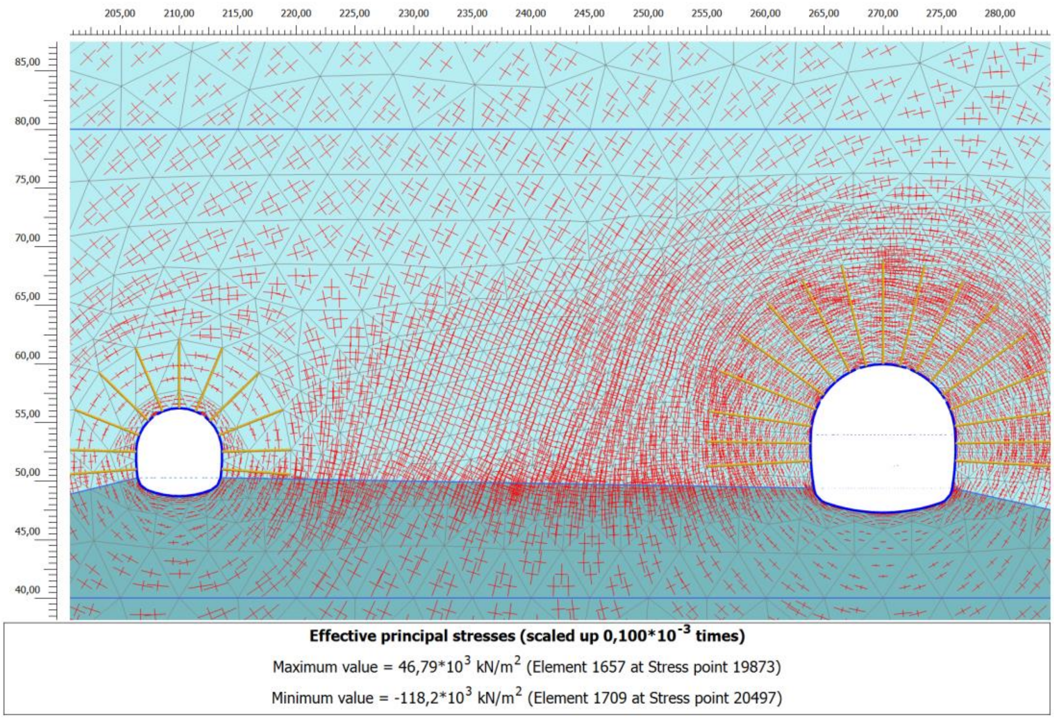


Fig. 32 Mean principal effective stresses direction

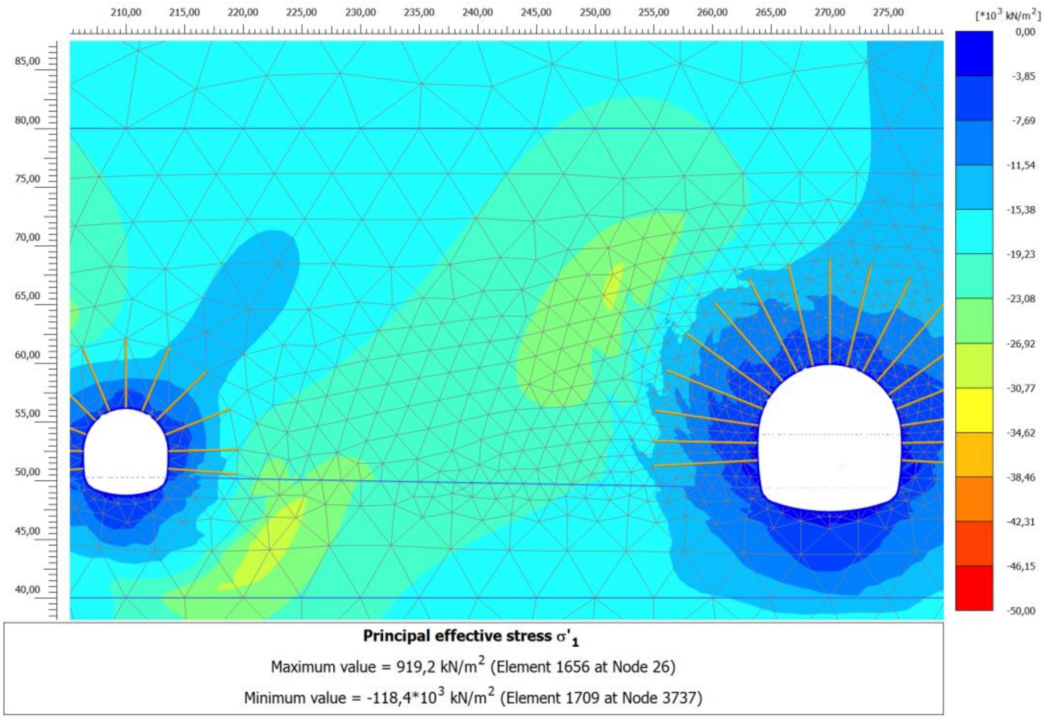


Fig. 33 Mean principal effective stresses (shadings)

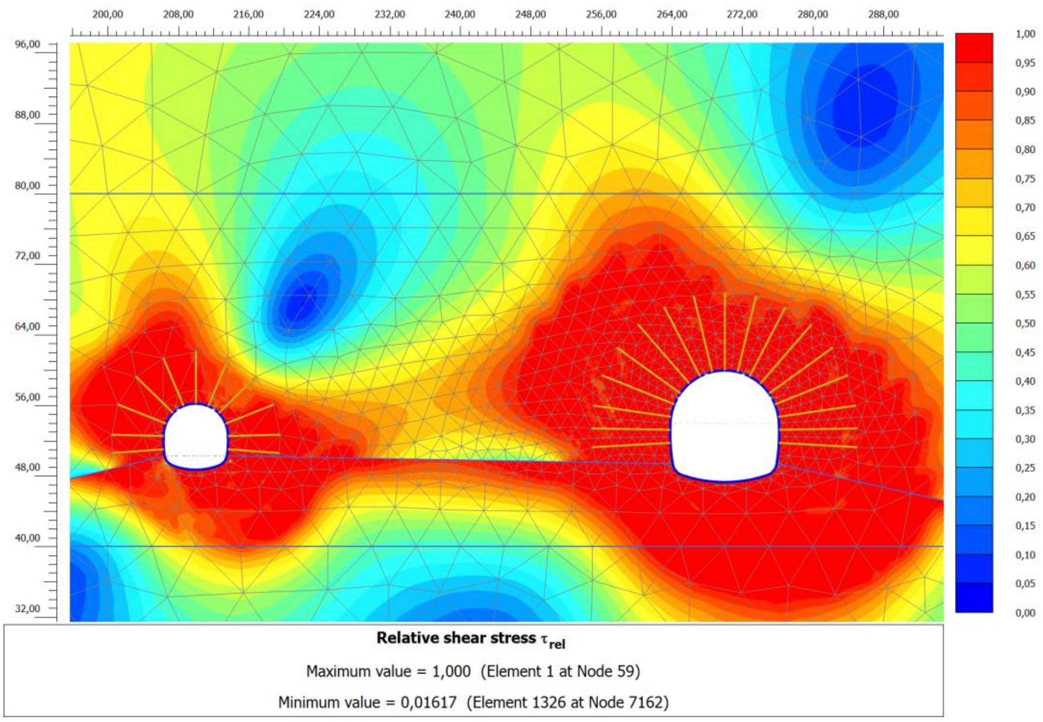


Fig. 34 Relative shear stresses (shadings) = strength factor contours

The initial mean principal stress directions are not orientated in vertical/horizontal direction due to the non-horizontal ground surface above the tunnel (see Fig. 31).

The mean principle stress directions are influenced by the tunnel excavation and after excavation orientated according to the tunnel geometry (see Fig. 32). Fig. 34 shows the plastic zone around the excavation (red coloured zone) and the strength factor contours. It is shown that the plastic radius around the main tunnel is approx. 15 m and around the egress tunnel approx. 10 m (see Fig. 19).

<u>Deformations</u>

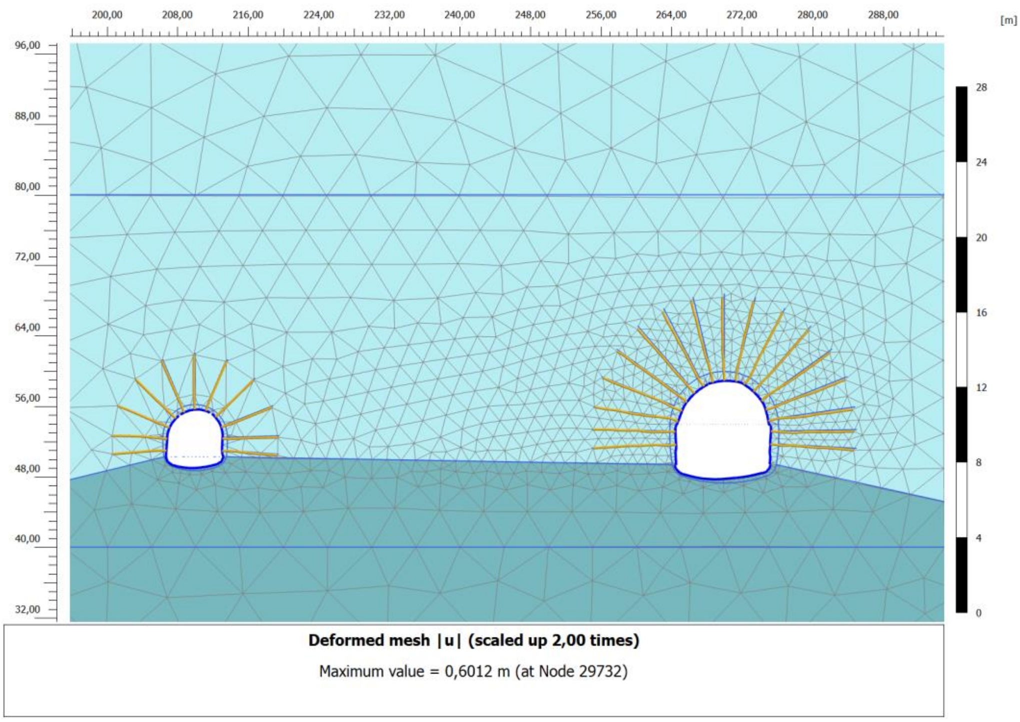


Fig. 35 Deformed mesh

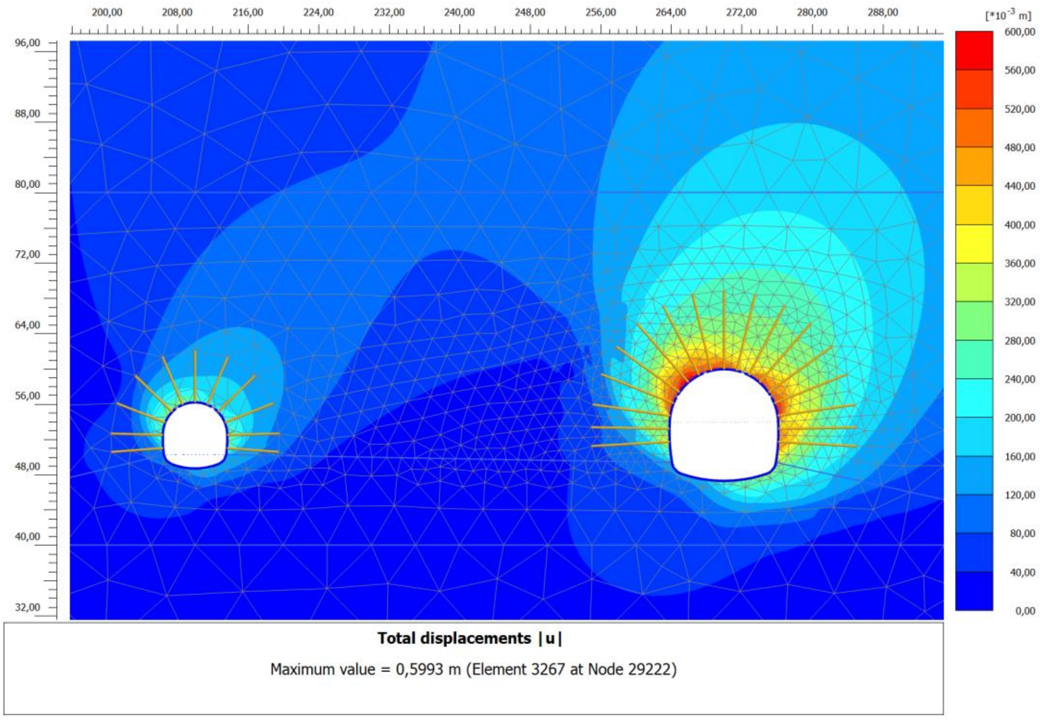


Fig. 36 Total displacements (shadings)

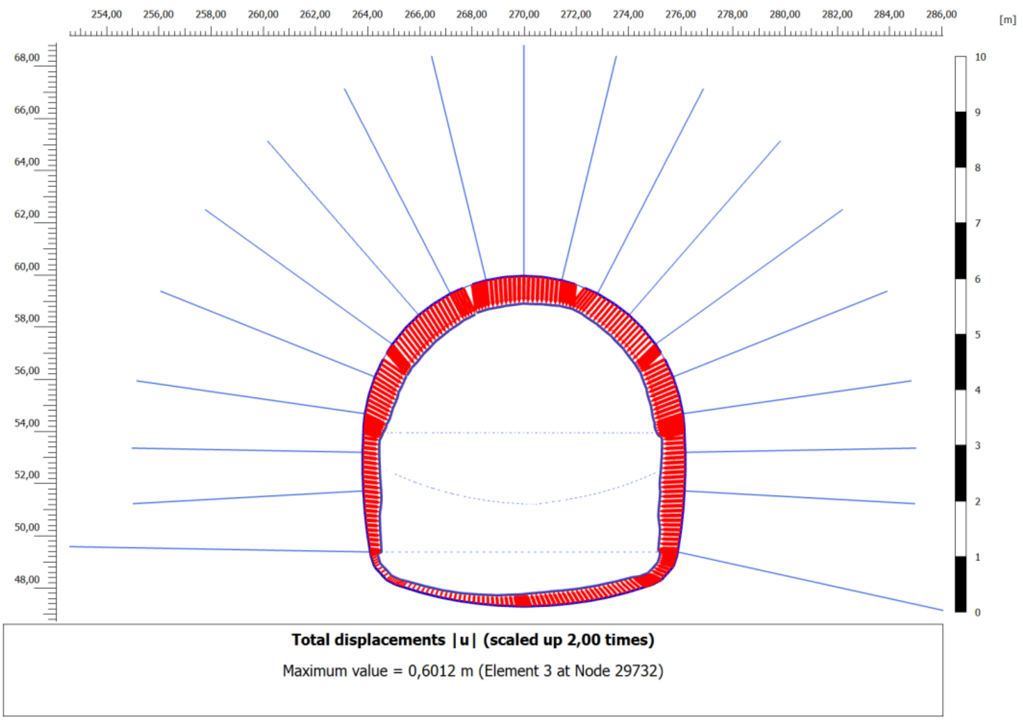


Fig. 37 Total displacements of primary lining, main tunnel

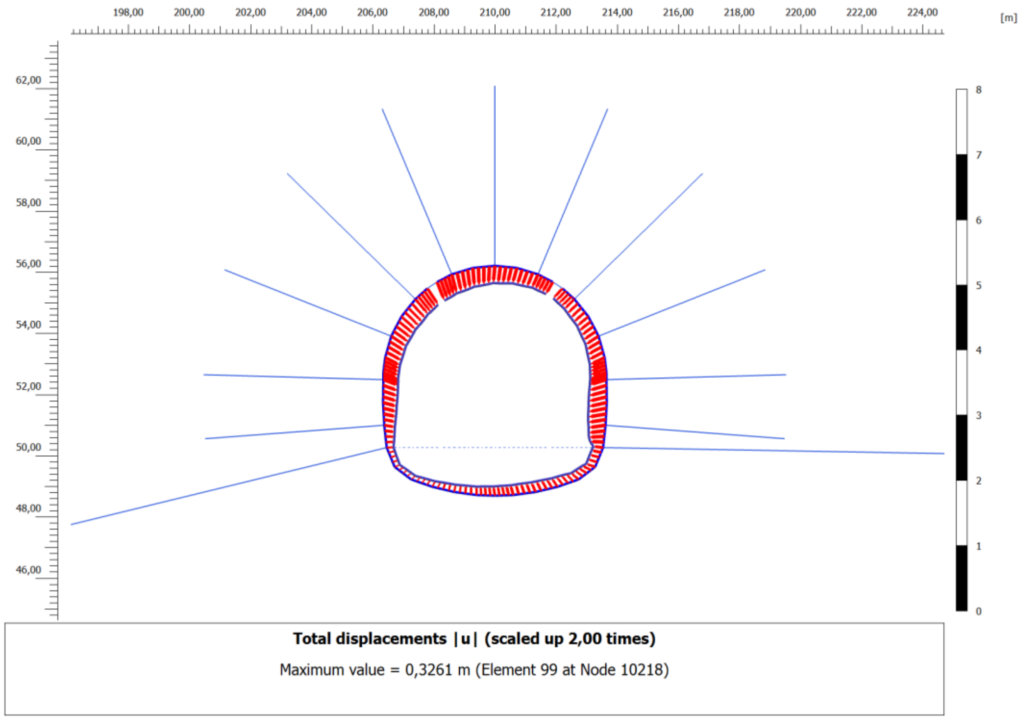


Fig. 38 Total displacements of primary lining, egress tunnel

The maximum vertical deformation occurs at the top heading in both tunnels with approx. 60 mm in main tunnel and approx. 33 mm in egress tunnel. The maximum vertical heave of the invert is approx. 13 mm (main tunnel) and 9 mm egress tunnel respectively. The maximum horizontal deformation at the lower part of the bench is app. 36 mm for main and approx. 26 mm for egress tunnel excavation. The given displacements are the overall displacements (displacements ahead of the face and displacements in the unsupported zone plus displacements of the lining, for detailed description refer to Section 5.3). The displacements ahead of the tunnel and in the unsupported zone are determined to approx. 25 mm in main and 20 mm in egress tunnel.

Internal Forces

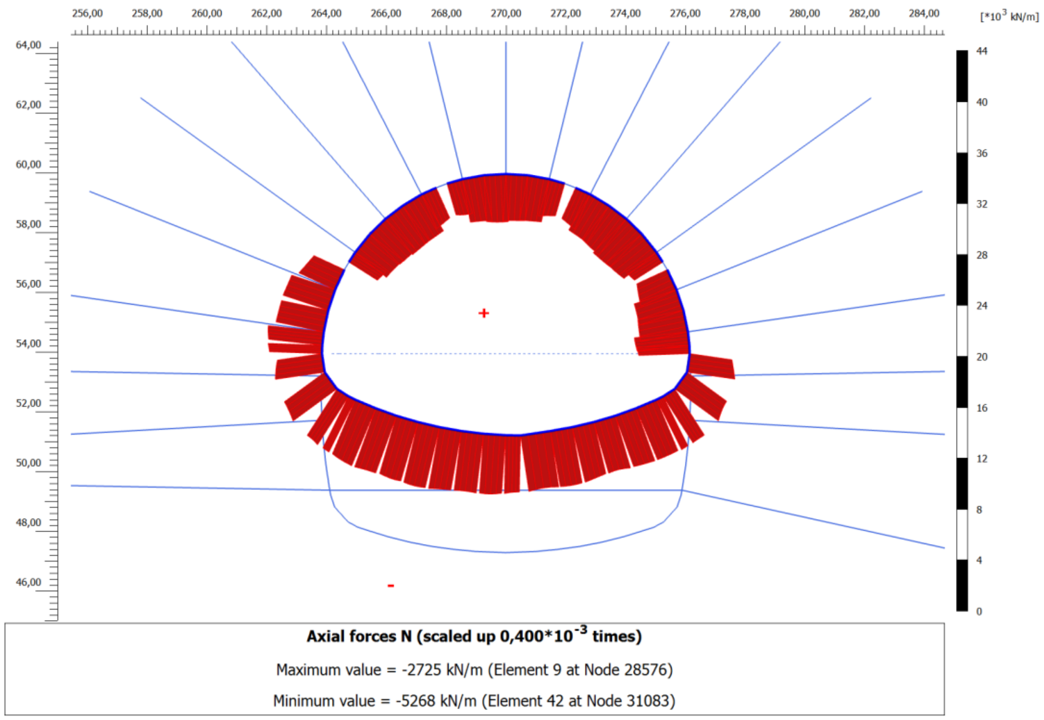


Fig. 39 Axial forces of temporary invert, main tunnel

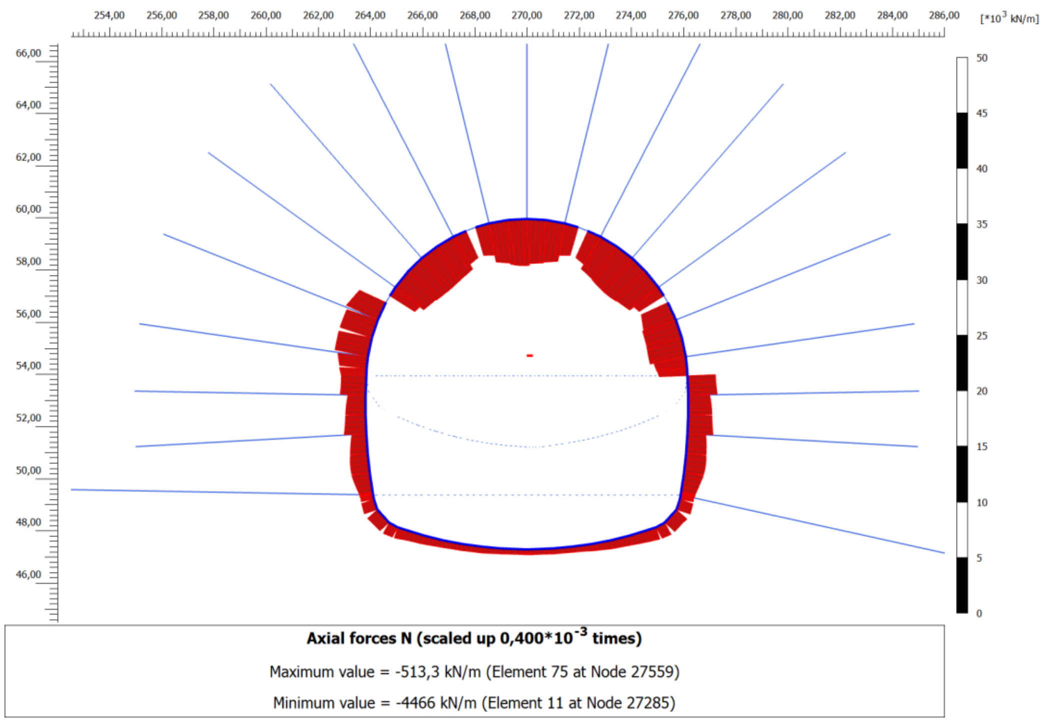


Fig. 40 Axial forces of primary lining (final step), main tunnel

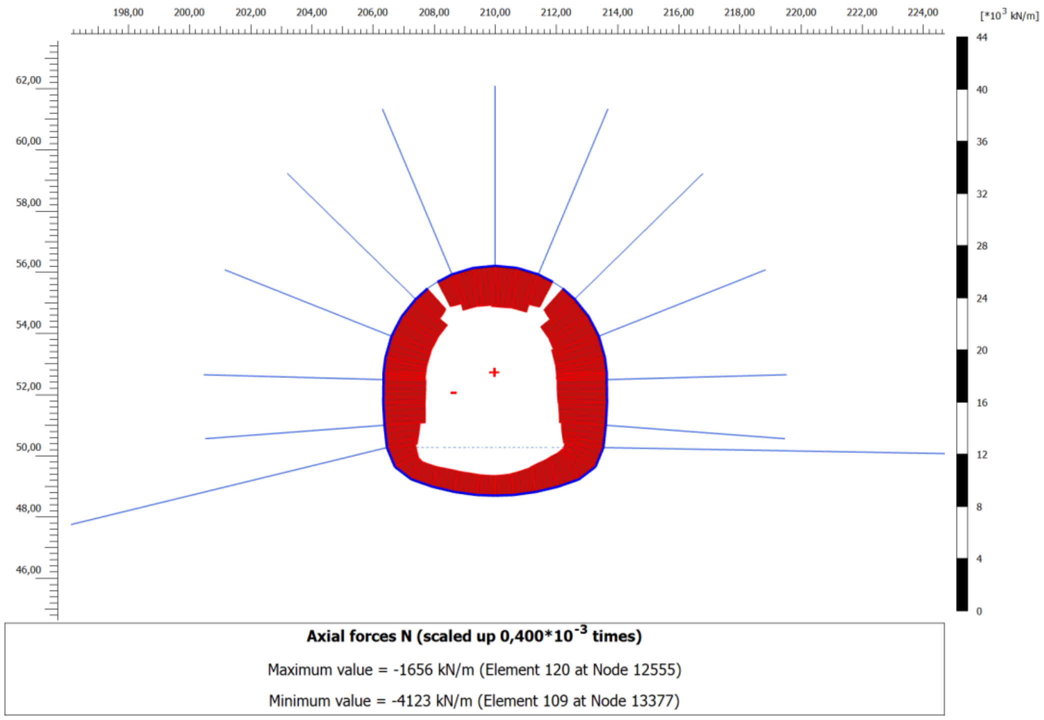


Fig. 41 Axial forces of primary lining (final step), egress tunnel

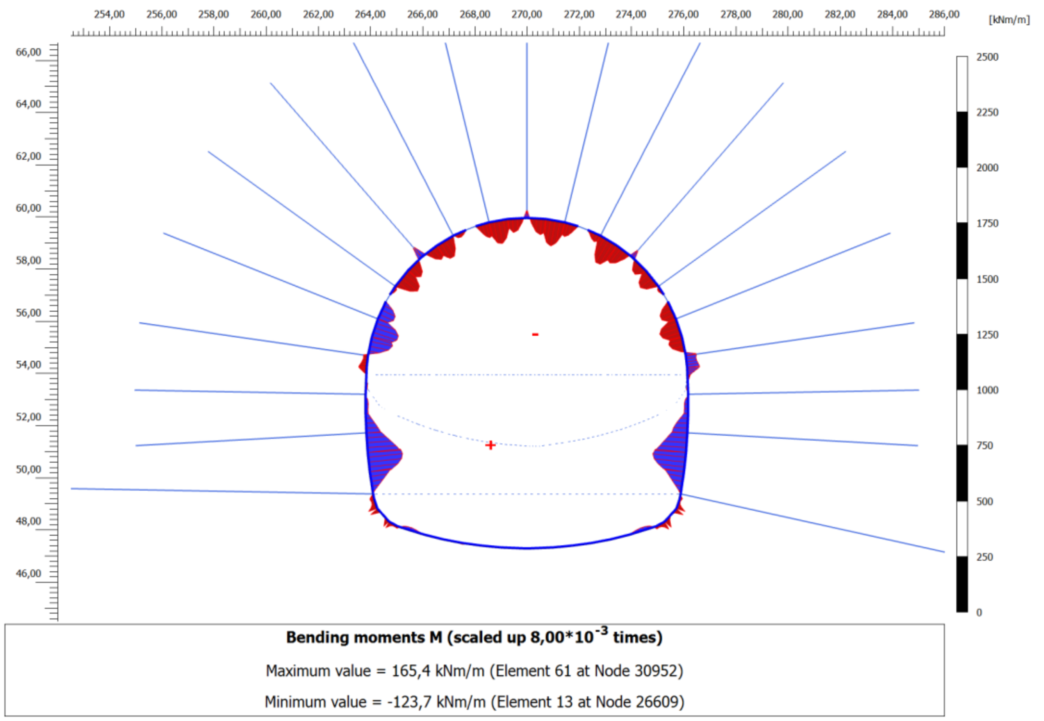


Fig. 42 Bending moments of primary lining, main tunnel (final step)

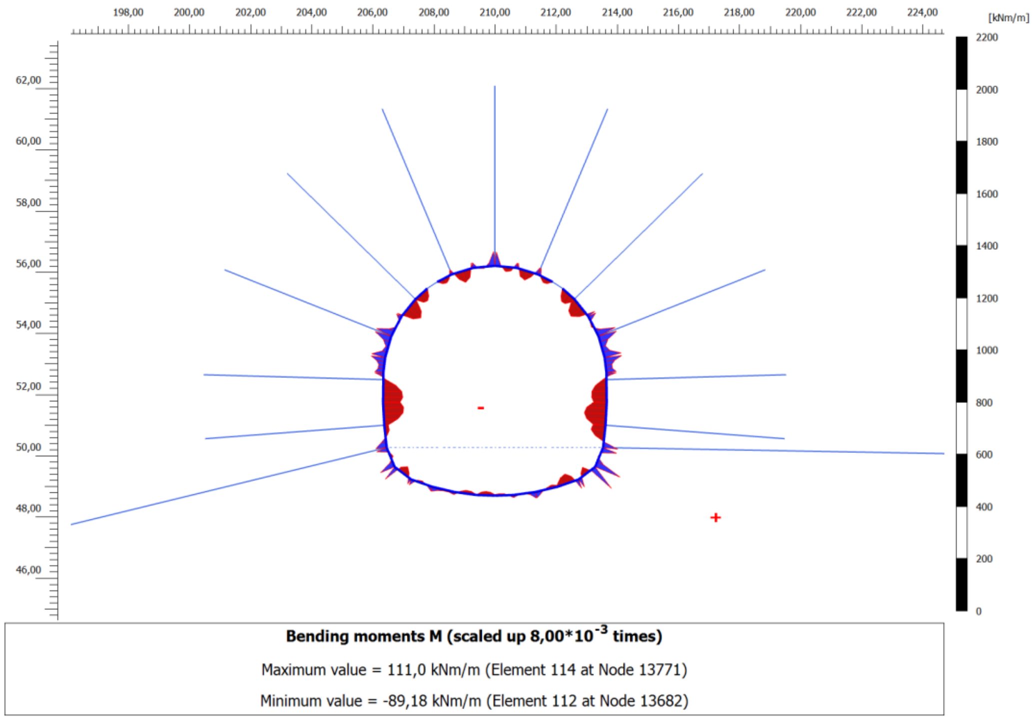


Fig. 43 Bending moments of primary lining, egress tunnel (final step)

Fig. 40 to Fig. 43 show the internal forces of the final calculation step.

The maximum normal force in the primary lining is determined to approx. 4.5 MN in main and 4.1 in egress tunnel primary lining. Due to the closure of the primary lining no singularities occur in this numerical analysis (compare analysis 1 no closure of primary lining).

The maximum bending moments are determined to approx. 165 kNm in main and approx. 110 kNm in egress tunnel respectively. The primary lining is designed in Section 6.

Radial Bolting

Fig. 44 and Fig. 45 show the axial forces in the rock bolts of the final calculation step. It can be seen that the rock bolts reach the ultimate level in most parts.

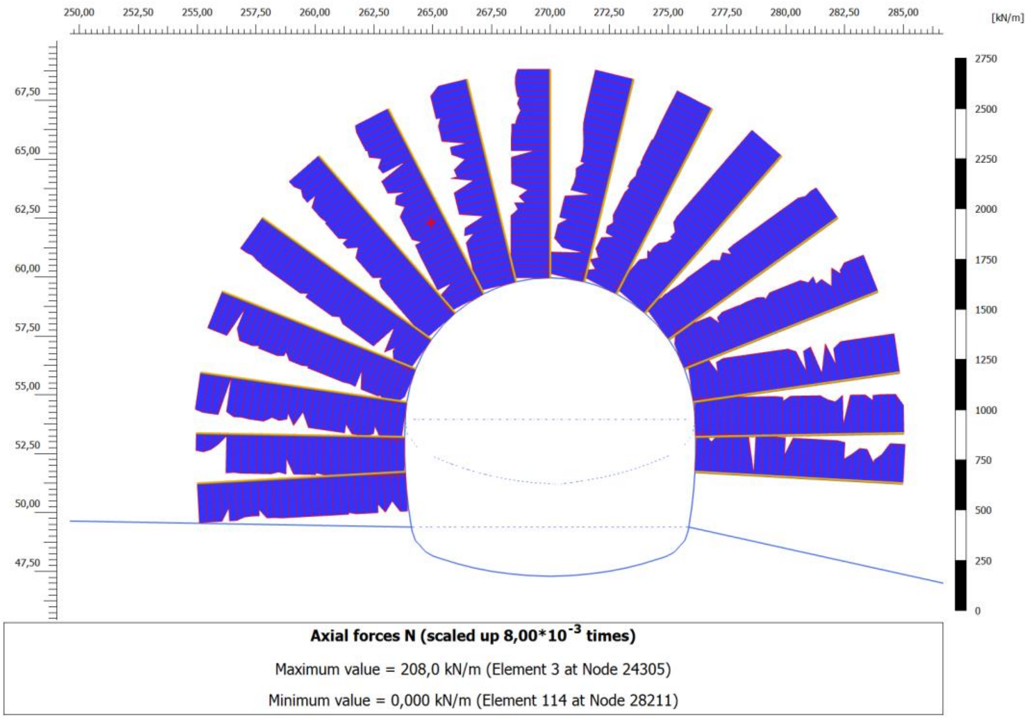


Fig. 44 Axial forces of rock bolts, main tunnel

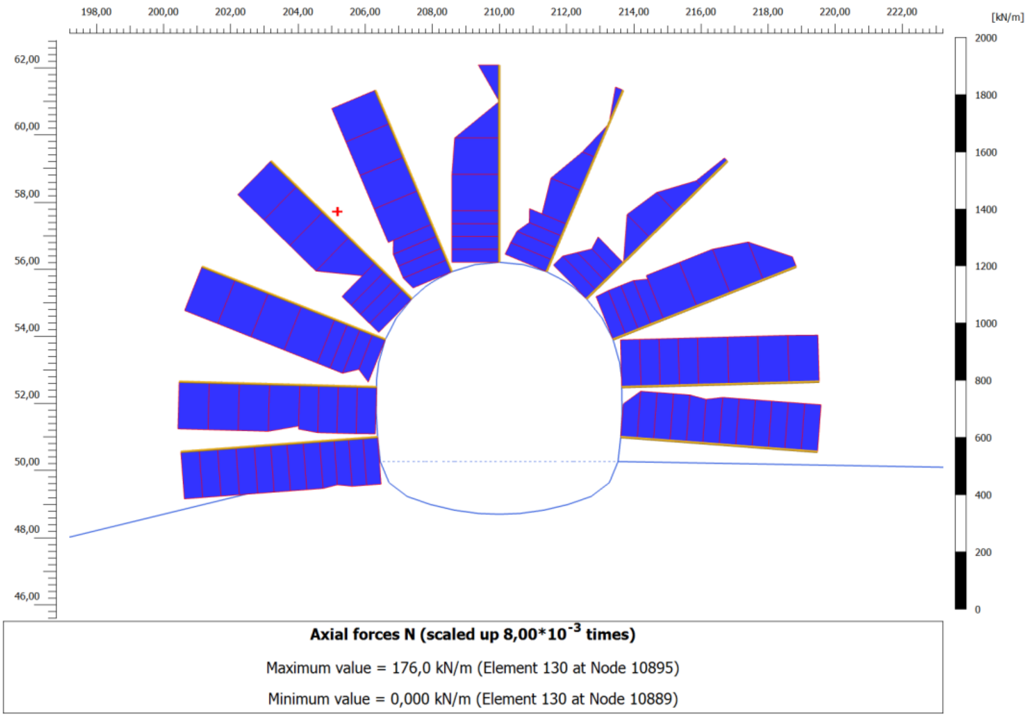


Fig. 45 Axial forces of rock bolts, egress tunnel

6 STRUCTURAL ANALYSES OF THE PRIMARY LINING

The design of the primary lining is carried out according to EN 1992-1-1 [5] with partial safety facotrs applied on internal reactions (bending moment and normal force) and material strength (concrete and reinforcement). A safety factor of 1.2 is applied on internal reactions of the primary lining due to the fact that the primary lining is only a temporary support meassure. The partial safety factor during exceptional load case is 1.0 (primary lining design with consideration of seismic loading). The safety factors onto the characeteristic strength is 1.5 (concrete) and 1.15 (steel) in normal load cases and 1.2 (concrete) and 1.0 (steel) in exceptional load cases. The partial safety factors are summarized in Tab. 15.

Tab. 15 Partial safety factors considered in the primary lining design

	Partial safety factor internal reactions γ	Partial safety factor concrete γ_c	Partial safety factor steel γ_s
Normal load case	1.20	1.50	1.15
Exceptional load case	1.00	1.20	1.00

The structural analysis is based on the internal reactions determined by the above given numerical analysis (Analysis 1 to 3, see Section 5.5.2 to 5.5.4).

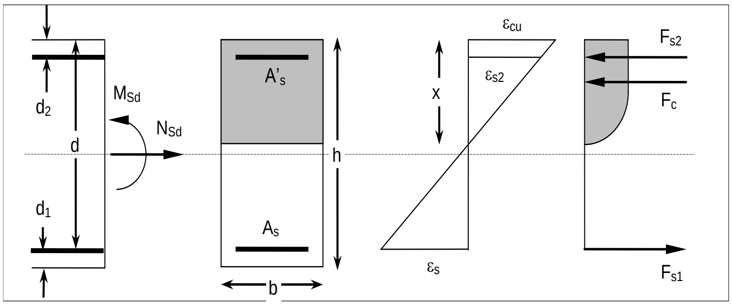
The shotcrete lining is reinforced with wire mesh with Ø6 mm each 10 cm (2.8 cm²/m) in SC B to SC C and with Ø8 mm each 10 cm (5.03 cm²/m) in SC D to H. The lattice girder of each support category (see Tab. 10) is added to the reinforcement. The lattice girder consists of three bars, on the one side one bar with Ø30 mm and on the other side two bars with Ø20 mm each are situated. Thus an additional reinforcement area of 7.1 cm² per lattice girder is considered in the structural design of the primary lining.

The structural design sheets of the three analysis (Analysis 1 to 3) for the main and egress tunnel for normal and exceptional load cases are given in Fig. 46 to Fig. 57. The blue lines in the corresponding diagrams represent the design capacity of the primary lining with respect to the combinations of normal forces and bending moments. The red dots are the actual design normal forces and bending moments, based on the numerical analysis.

It is shown that the primary lining of the main and egress tunnel is capable to withstand the loading (all load combinations = red dots lie within the capacity of the lining = blue lines).

Project no..:	8482B	Editor:	MBu
Project:	Zojila Tunnel	Date:	26.04.2013
Position:	Main Tunnel SC-C	Notes:	acc. num. analysis 1

Design for bending and axial force according EC 2 (2004) rectangular section with symmetrical reinforcement



Materials

Reinforcing steel			S 500	Strength class for concrete			C 25/30
characteristic yield strength of reinforcement	f_{yk}	[MPa]	500	characteristic compressive cylinder strength of concrete	f_{ck}	[MPa]	25
partial factor	γ_s	[-]	1,15	partial factor	γ_c	[-]	1,5
design yield strength of reinf.	f_{yd}	[MPa]	435	coeff. for long term effects	α_{cc}	[-]	1
				design value of concr. comp.	f_{cd}	[MPa]	16,7

Geometry

height	h	[cm]	15	Cross sec. area of reinf.	A_s=A'$_s$	[cm²]	6,89
width	b	[cm]	100	edge distance	d_1, d_2	[cm]	2,5
area	A	[m²]	0,15	reinforcement ratio	ρ_s	[-]	0,9%

Calculation values

ω_{tot}	[-]	0,240	$N_{c,Rd}$	[kN]	1216,3
d_1/h	[-]	0,13	$N_{s,Rd}$	[kN]	598,9
κ	[-]	0,416	$M_{c,Rd}$	[kNm]	45,6
β	[-]	0,810	$M_{s,Rd}$	[kNm]	32,9
x	[cm]	9,02			

Diagram

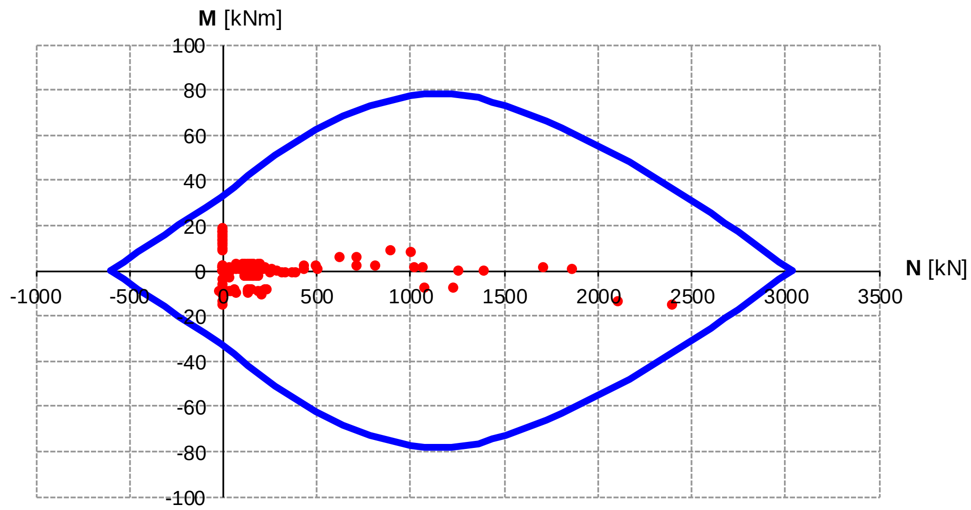
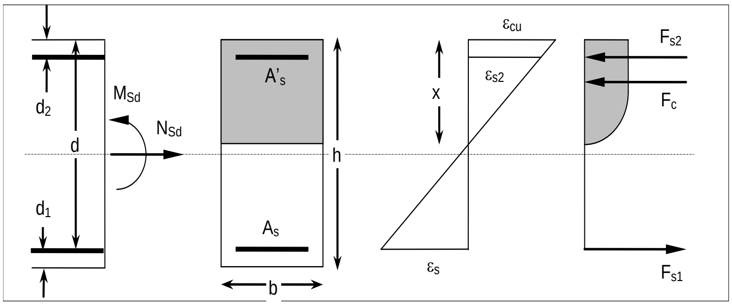


Fig. 46 Structural design, primary lining, main tunnel, SC C, normal load case

Project no..:	8482B	Editor:	MBu
Project:	Zojila Tunnel	Date:	26.04.2013
Position:	Egress Tunnel SC-C	Notes:	acc. num. analysis 1

Design for bending and axial force according EC 2 (2004)
rectangular section with symmetrical reinforcement



Materials

Reinforcing steel			S 500	Strength class for concrete			C 25/30
characteristic yield strength of reinforcement	f_{yk}	[MPa]	500	characteristic compressive cylinder strength of concrete	f_{ck}	[MPa]	25
partial factor	γ_s	[-]	1,15	partial factor	γ_c	[-]	1,5
design yield strength of reinf.	f_{yd}	[MPa]	435	coeff. for long term effects	α_{cc}	[-]	1
				design value of concr. comp.	f_{cd}	[MPa]	16,7

Geometry

height	h	[cm]	15	Cross sec. area of reinf.	A_s=A'_s	[cm²]	6,89
width	b	[cm]	100	edge distance	d_1, d_2	[cm]	2,5
area	A	[m²]	0,15	reinforcement ratio	ρ_s	[-]	0,9%

Calculation values

ω_{tot}	[-]	0,240		$N_{c,Rd}$	[kN]	1216,3
d_1/h	[-]	0,13		$N_{s,Rd}$	[kN]	598,9
κ	[-]	0,416		$M_{c,Rd}$	[kNm]	45,6
β	[-]	0,810		$M_{s,Rd}$	[kNm]	32,9
x	[cm]	9,02				

Diagram

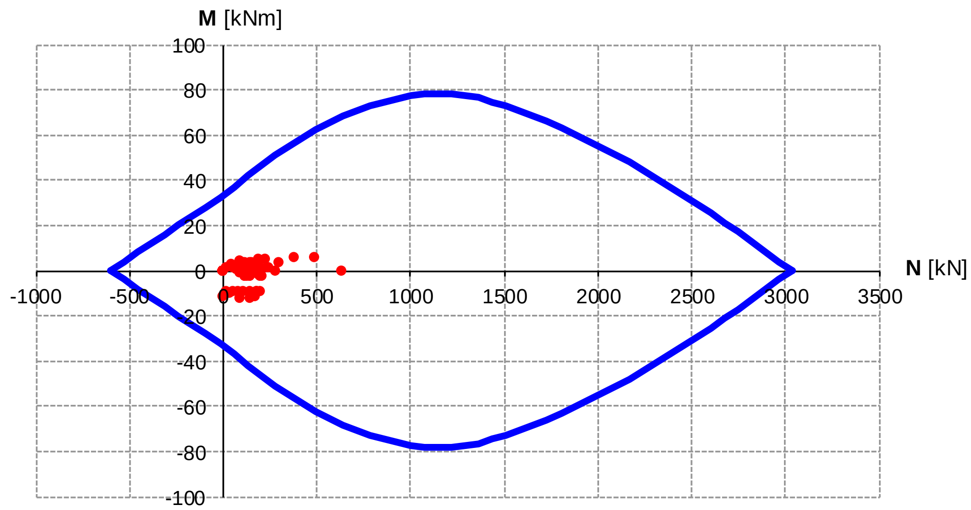
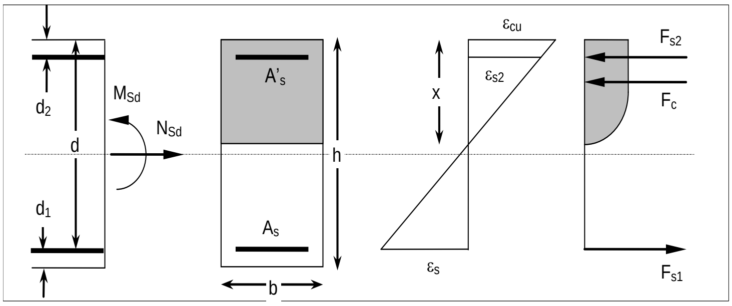


Fig. 47 Structural design, primary lining, egress tunnel, SC C, normal load case

Project no..:	8482B	Editor:	MBu
Project:	Zojila Tunnel	Date:	26.04.2013
Position:	Main Tunnel SC-C/EQ	Notes:	acc. num. analysis 1

Design for bending and axial force according EC 2 (2004) rectangular section with symmetrical reinforcement



Materials

Reinforcing steel			S 500	Strength class for concrete			C 25/30
characteristic yield strength of reinforcement	f_{yk}	[MPa]	500	characteristic compressive cylinder strength of concrete	f_{ck}	[MPa]	25
partial factor	γ_s	[-]	1	partial factor	γ_c	[-]	1,2
design yield strength of reinf.	f_{yd}	[MPa]	500	coeff. for long term effects	α_{cc}	[-]	1
				design value of concr. comp.	f_{cd}	[MPa]	20,8

Geometry

height	h	[cm]	15	Cross sec. area of reinf.	A_s=A'_s	[cm²]	6,89
width	b	[cm]	100	edge distance	d_1, d_2	[cm]	2,5
area	A	[m²]	0,15	reinforcement ratio	ρ_s	[-]	0,9%

Calculation values

ω_{tot}	[-]	0,220		$N_{c,Rd}$	[kN]	1520,4
d_1/h	[-]	0,13		$N_{s,Rd}$	[kN]	688,7
κ	[-]	0,416		$M_{c,Rd}$	[kNm]	57,0
β	[-]	0,810		$M_{s,Rd}$	[kNm]	37,9
x	[cm]	9,02				

Diagram

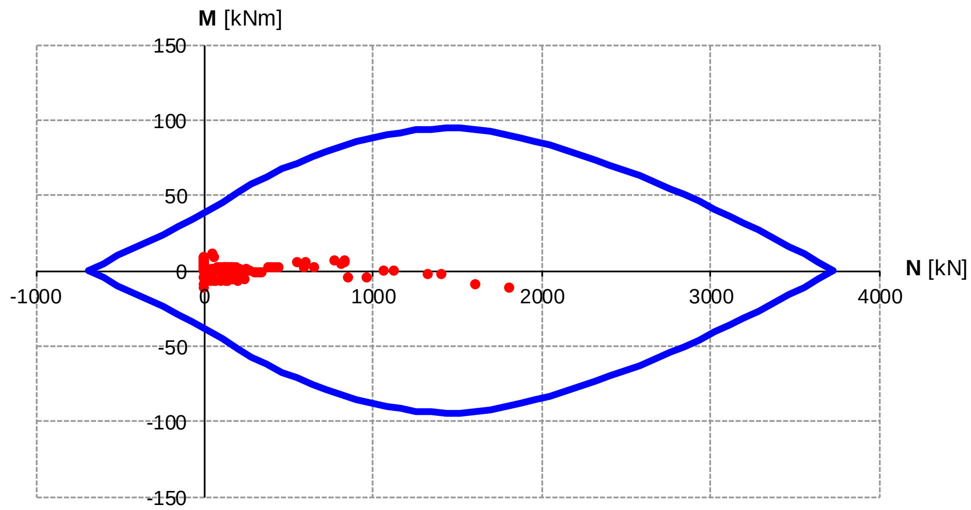
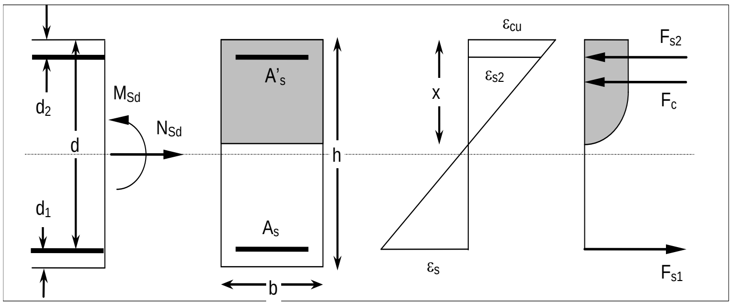


Fig. 48 Structural design, primary lining, main tunnel, SC C, exceptional load case

Project no..:	**8482B**	Editor:	MBu
Project:	Zojila Tunnel	Date:	26.04.2013
Position:	**Egress Tunnel SC-C/E**	Notes:	acc. num. analysis 1

Design for bending and axial force according EC 2 (2004)
rectangular section with symmetrical reinforcement



Materials

Reinforcing steel			**S 500**	Strength class for concrete			**C 25/30**
characteristic yield strength of reinforcement	f_{yk}	[MPa]	500	characteristic compressive cylinder strength of concrete	f_{ck}	[MPa]	25
partial factor	γ_s	[-]	1	partial factor	γ_c	[-]	1,2
design yield strength of reinf.	f_{yd}	[MPa]	500	coeff. for long term effects	α_{cc}	[-]	1
				design value of concr. comp.	f_{cd}	[MPa]	20,8

Geometry

height	h	[cm]	**15**	Cross sec. area of reinf.	A_s=A'$_s$	[cm²]	**6,89**
width	b	[cm]	**100**	edge distance	d_1, d_2	[cm]	**2,5**
area	A	[m²]	0,15	reinforcement ratio	ρ_s	[-]	0,9%

Calculation values

ω_{tot}	[-]	0,220	$N_{c,Rd}$	[kN]	1520,4
d_1/h	[-]	0,13	$N_{s,Rd}$	[kN]	688,7
κ	[-]	0,416	$M_{c,Rd}$	[kNm]	57,0
β	[-]	0,810	$M_{s,Rd}$	[kNm]	37,9
x	[cm]	9,02			

Diagram

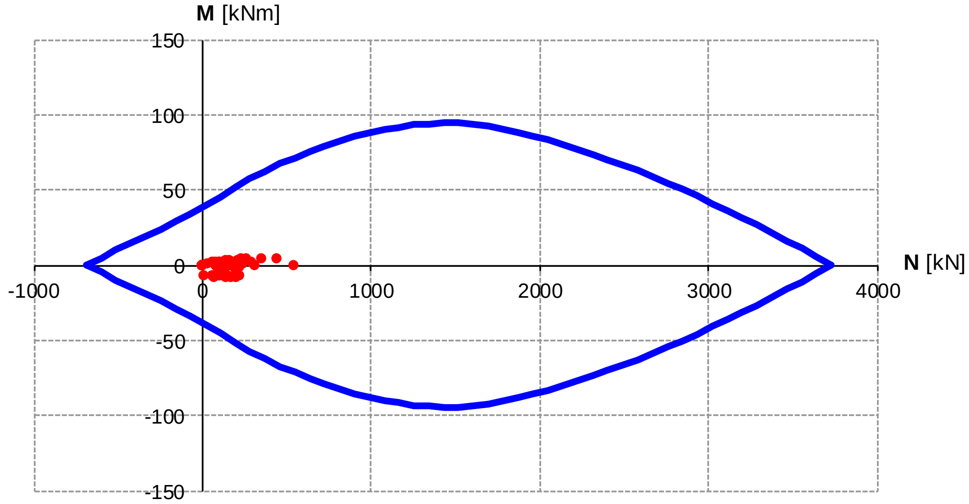
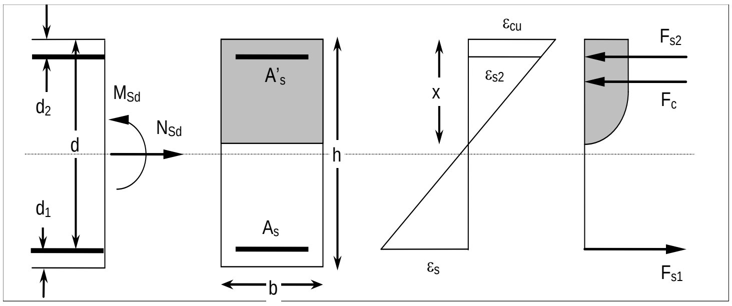


Fig. 49 Structural design, primary lining, egress tunnel, SC C, exceptional load case

Project no..:	**8482B**	Editor:	MBu
Project:	Zojila Tunnel	Date:	26.04.2013
Position:	**Main Tunnel SC-E**	Notes:	acc. num. analysis 2

Design for bending and axial force according EC 2 (2004) rectangular section with symmetrical reinforcement



Materials

Reinforcing steel			**S 500**	Strength class for concrete			**C 25/30**
characteristic yield strength of reinforcement	f_{yk}	[MPa]	500	characteristic compressive cylinder strength of concrete	f_{ck}	[MPa]	25
partial factor	γ_s	[-]	1,15	partial factor	γ_c	[-]	1,5
design yield strength of reinf.	f_{yd}	[MPa]	435	coeff. for long term effects	α_{cc}	[-]	1
				design value of concr. comp.	f_{cd}	[MPa]	16,7

Geometry

height	h	[cm]	**25**	Cross sec. area of reinf.	A_s=A'_s	[cm²]	**9,76**
width	b	[cm]	**100**	edge distance	d_1, d_2	[cm]	**2,5**
area	A	[m²]	0,25	reinforcement ratio	ρ_s	[-]	0,8%

Calculation values

ω_{tot}	[-]	0,204		$N_{c,Rd}$	[kN]	2027,2
d_1/h	[-]	0,08		$N_{s,Rd}$	[kN]	849,0
κ	[-]	0,416		$M_{c,Rd}$	[kNm]	126,7
β	[-]	0,810		$M_{s,Rd}$	[kNm]	89,1
x	[cm]	15,03				

Diagram

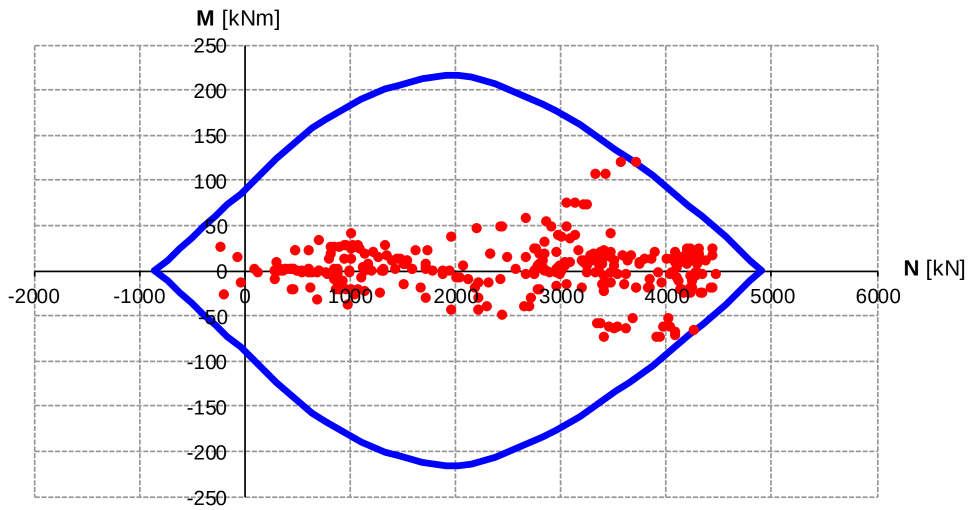
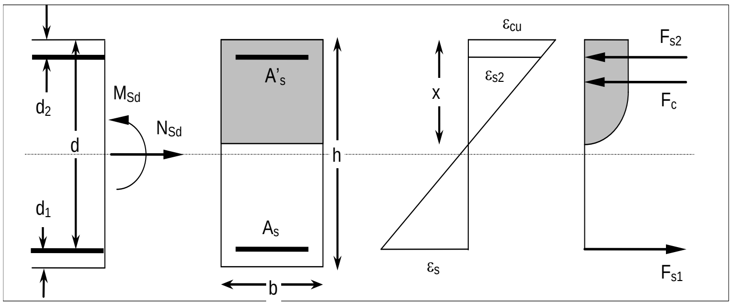


Fig. 50 Structural design, primary lining, main tunnel, SC E, normal load case

Project no..:	8482B	Editor:	MBu
Project:	Zojila Tunnel	Date:	26.04.2013
Position:	Egress Tunnel SC-E	Notes:	acc. num. analysis 2

Design for bending and axial force according EC 2 (2004)
rectangular section with symmetrical reinforcement



Materials

Reinforcing steel			S 500	Strength class for concrete			C 25/30
characteristic yield strength of reinforcement	f_{yk}	[MPa]	500	characteristic compressive cylinder strength of concrete	f_{ck}	[MPa]	25
partial factor	γ_s	[-]	1,15	partial factor	γ_c	[-]	1,5
design yield strength of reinf.	f_{yd}	[MPa]	435	coeff. for long term effects	α_{cc}	[-]	1
				design value of concr. comp.	f_{cd}	[MPa]	16,7

Geometry

height	h	[cm]	25	Cross sec. area of reinf.	A_s=A'_s	[cm²]	9,76
width	b	[cm]	100	edge distance	d_1, d_2	[cm]	2,5
area	A	[m²]	0,25	reinforcement ratio	ρ_s	[-]	0,8%

Calculation values

ω_{tot}	[-]	0,204	$N_{c,Rd}$	[kN]	2027,2
d_1/h	[-]	0,08	$N_{s,Rd}$	[kN]	849,0
κ	[-]	0,416	$M_{c,Rd}$	[kNm]	126,7
β	[-]	0,810	$M_{s,Rd}$	[kNm]	89,1
x	[cm]	15,03			

Diagram

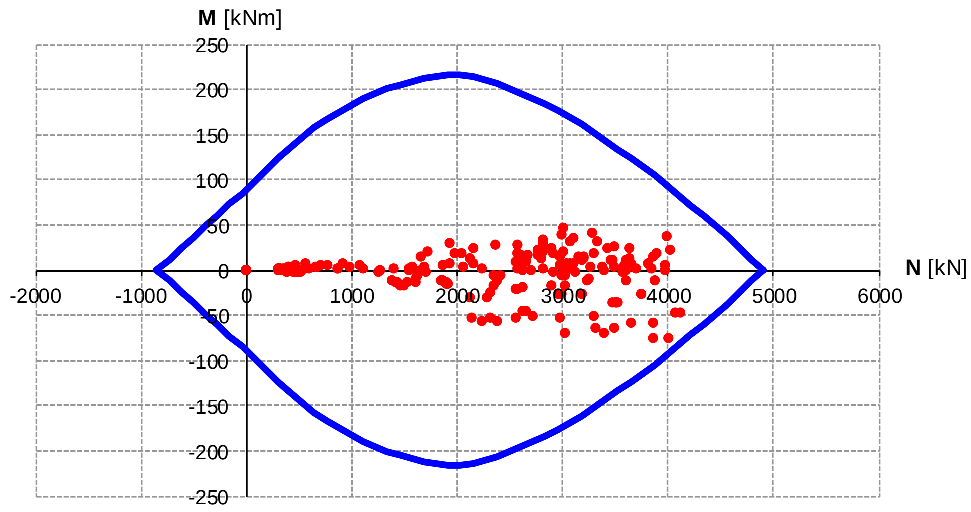
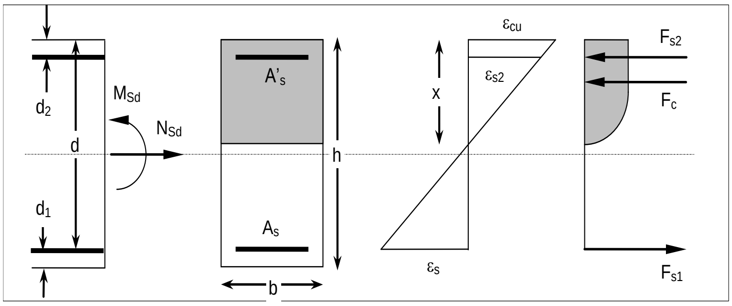


Fig. 51 Structural design, primary lining, egress tunnel, SC E, normal load case

Project no..:	**8482B**	Editor:	MBu
Project:	Zojila Tunnel	Date:	26.04.2013
Position:	**Main Tunnel SC-E/EQ**	Notes:	acc. num. analysis 2

Design for bending and axial force according EC 2 (2004)
rectangular section with symmetrical reinforcement



Materials

Reinforcing steel			**S 500**	Strength class for concrete			**C 25/30**
characteristic yield strength of reinforcement	f_{yk}	[MPa]	500	characteristic compressive cylinder strength of concrete	f_{ck}	[MPa]	25
partial factor	γ_s	[-]	1	partial factor	γ_c	[-]	1,2
design yield strength of reinf.	f_{yd}	[MPa]	500	coeff. for long term effects	α_{cc}	[-]	1
				design value of concr. comp.	f_{cd}	[MPa]	20,8

Geometry

height	h	[cm]	**25**	Cross sec. area of reinf.	$A_s=A'_s$	[cm²]	**9,76**
width	b	[cm]	**100**	edge distance	d_1, d_2	[cm]	**2,5**
area	A	[m²]	0,25	reinforcement ratio	ρ_s	[-]	0,8%

Calculation values

ω_{tot}	[-]	0,187	$N_{c,Rd}$	[kN]	2534,0
d_1/h	[-]	0,08	$N_{s,Rd}$	[kN]	976,3
κ	[-]	0,416	$M_{c,Rd}$	[kNm]	158,4
β	[-]	0,810	$M_{s,Rd}$	[kNm]	102,5
x	[cm]	15,03			

Diagram

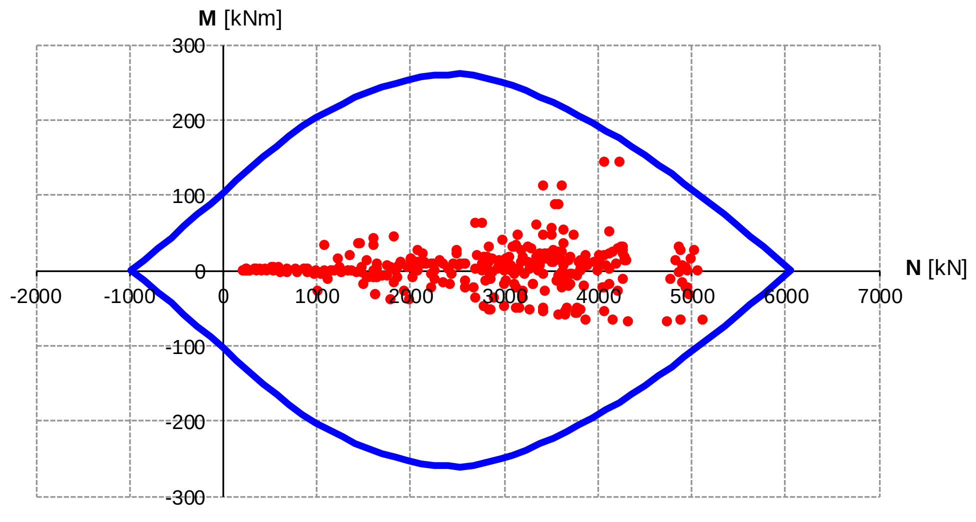
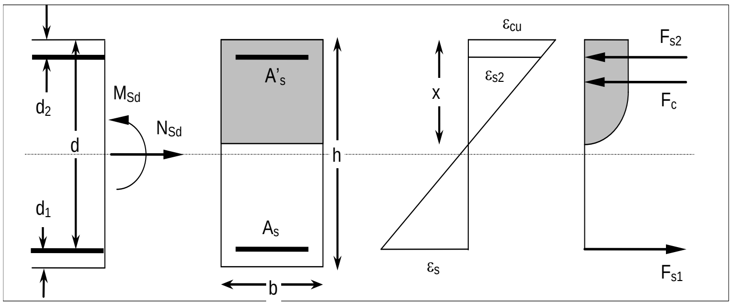


Fig. 52 Structural design, primary lining, main tunnel, SC E, exceptional load case

Project no..:	**8482B**	Editor:	MBu
Project:	Zojila Tunnel	Date:	26.04.2013
Position:	**Egress Tunnel SC-E/E**	Notes:	acc. num. analysis 2

Design for bending and axial force according EC 2 (2004) rectangular section with symmetrical reinforcement



Materials

Reinforcing steel			**S 500**	Strength class for concrete			**C 25/30**
characteristic yield strength of reinforcement	f_{yk}	[MPa]	500	characteristic compressive cylinder strength of concrete	f_{ck}	[MPa]	25
partial factor	γ_s	[-]	1	partial factor	γ_c	[-]	1,2
design yield strength of reinf.	f_{yd}	[MPa]	500	coeff. for long term effects	α_{cc}	[-]	1
				design value of concr. comp.	f_{cd}	[MPa]	20,8

Geometry

height	h	[cm]	**25**	Cross sec. area of reinf.	A_s=A'_s	[cm²]	**9,76**
width	b	[cm]	**100**	edge distance	d_1, d_2	[cm]	**2,5**
area	A	[m²]	0,25	reinforcement ratio	ρ_s	[-]	0,8%

Calculation values

ω_{tot}	[-]	0,187	$N_{c,Rd}$	[kN]	2534,0
d_1/h	[-]	0,08	$N_{s,Rd}$	[kN]	976,3
κ	[-]	0,416	$M_{c,Rd}$	[kNm]	158,4
β	[-]	0,810	$M_{s,Rd}$	[kNm]	102,5
x	[cm]	15,03			

Diagram

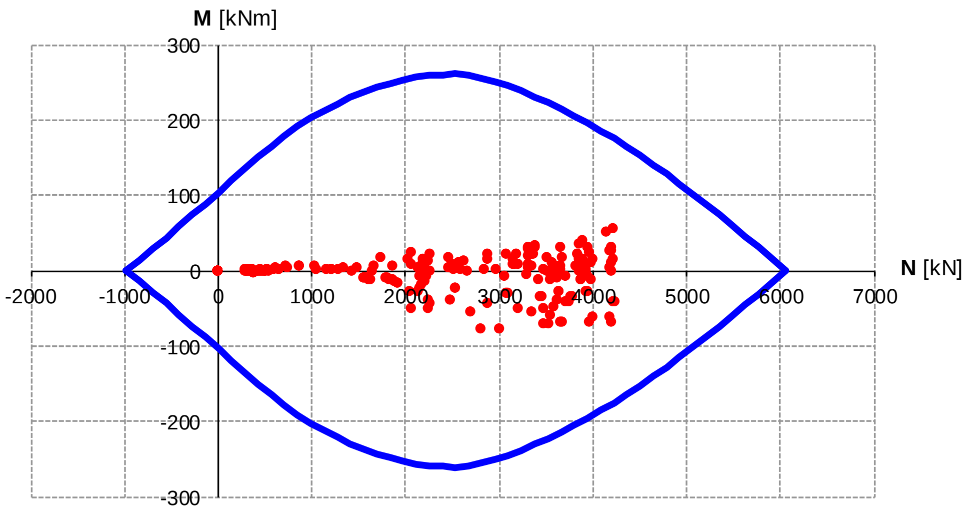
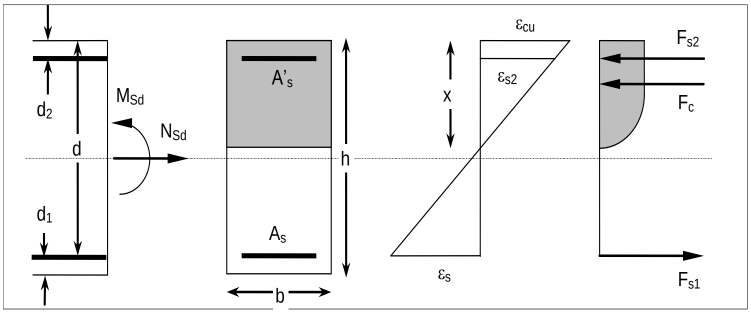


Fig. 53 Structural design, primary lining, egress tunnel, SC E, exceptional load case

Projekt Nr.:	8482B	Bearbeiter:	MBu
Projekt:	Zojila Tunnel	Datum:	26.04.2013
Bauteil:	Main Tunnel SC-G	Anmerkungen:	acc. num. analysis 3

Bemessung für Biegung mit Längskraft nach EC 2 (2003)
Rechteckquerschnitt mit symmetrischer Bewehrung



Baustoffe

Stahlsorte			S 500	Betonfestigkeitsklasse			C 25/30
Charakterist. Strecl	f_{yk}	[MPa]	500	Charakterist. Druckfestigkeit	f_{ck}	[MPa]	25
Teilsicherheitsbeiwe	γ_s	[-]	1,15	Teilsicherheitsbeiwert	γ_c	[-]	1,5
Bemess.-wert Strec	f_{yd}	[MPa]	435	Beiwert Langzeit	α_{cc}	[-]	1
				Bemess.-wert Druckfestigkeit	f_{cd}	[MPa]	16,7

Geometrie

Höhe	h	[cm]	30	Stahlfläche, je Bew.lage	A_s=A'$_s$	[cm²]	10,71
Breite	b	[cm]	100	Randabstand	d_1, d_2	[cm]	2
Fläche	A	[m²]	0,3	Bewehrungsgrad	ρ_s	[-]	0,7%

Berechnungsgrößen

ω_{tot}	[-]	0,186	$N_{c,Rd}$	[kN]	2432,7
d_1/h	[-]	0,07	$N_{s,Rd}$	[kN]	931,3
κ	[-]	0,416	$M_{c,Rd}$	[kNm]	182,4
β	[-]	0,810	$M_{s,Rd}$	[kNm]	121,1
x	[cm]	18,03			

Diagramm | **Diagramm**

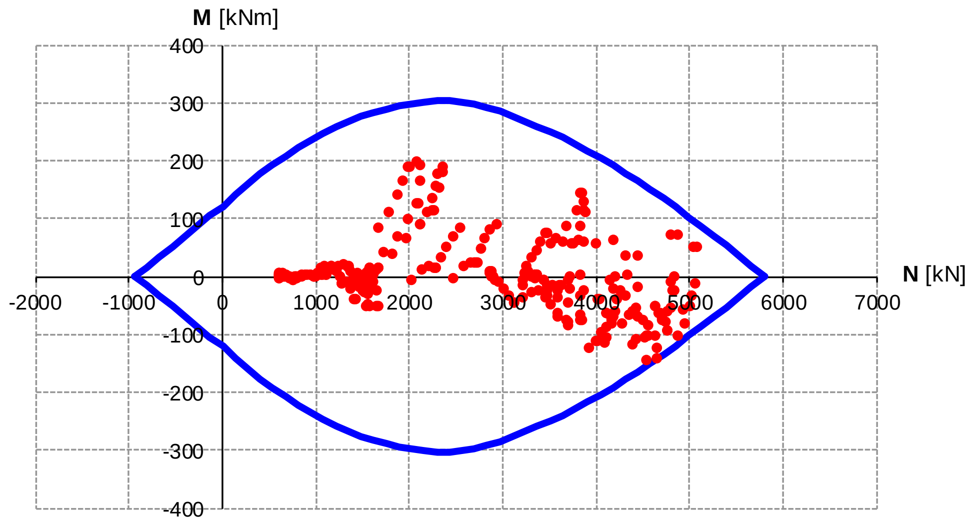
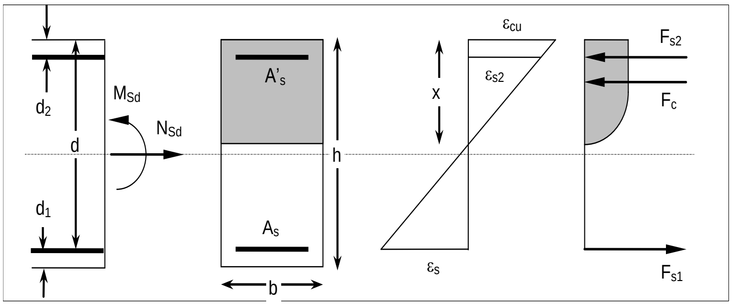


Fig. 54 Structural design, primary lining, main tunnel, SC G, normal load case

Project no..:	8482B	Editor:	MBu
Project:	Zojila Tunnel	Date:	26.04.2013
Position:	Egress Tunnel SC-E	Notes:	acc. num. analysis 3

Design for bending and axial force according EC 2 (2004) rectangular section with symmetrical reinforcement



Materials

Reinforcing steel			S 500	Strength class for concrete			C 25/30
characteristic yield strength of reinforcement	f_{yk}	[MPa]	500	characteristic compressive cylinder strength of concrete	f_{ck}	[MPa]	25
partial factor	γ_s	[-]	1,15	partial factor	γ_c	[-]	1,5
design yield strength of reinf.	f_{yd}	[MPa]	435	coeff. for long term effects	α_{cc}	[-]	1
				design value of concr. comp.	f_{cd}	[MPa]	16,7

Geometry

height	h	[cm]	25	Cross sec. area of reinf.	$A_s=A'_s$	[cm²]	9,76
width	b	[cm]	100	edge distance	d_1, d_2	[cm]	2,5
area	A	[m²]	0,25	reinforcement ratio	ρ_s	[-]	0,8%

Calculation values

ω_{tot}	[-]	0,204	$N_{c,Rd}$	[kN]	2027,2
d_1/h	[-]	0,08	$N_{s,Rd}$	[kN]	849,0
κ	[-]	0,416	$M_{c,Rd}$	[kNm]	126,7
β	[-]	0,810	$M_{s,Rd}$	[kNm]	89,1
x	[cm]	15,03			

Diagram

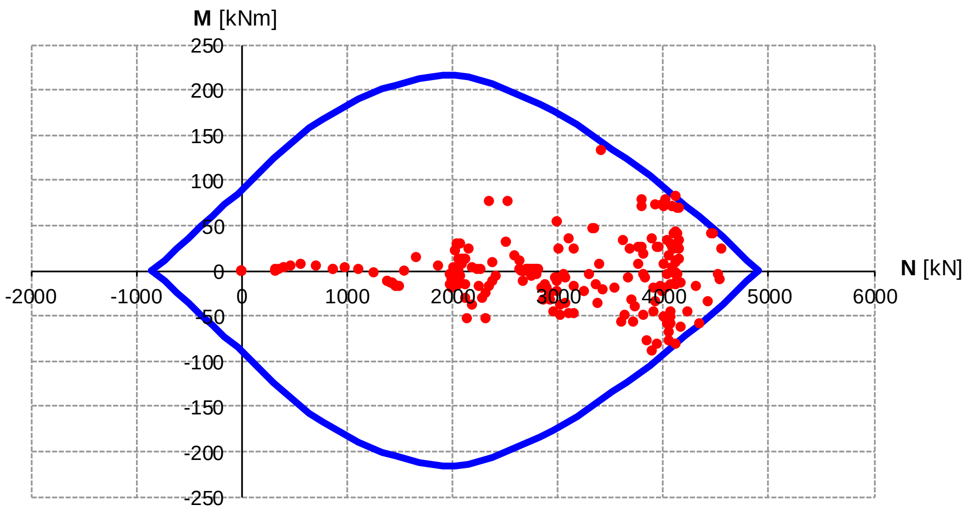
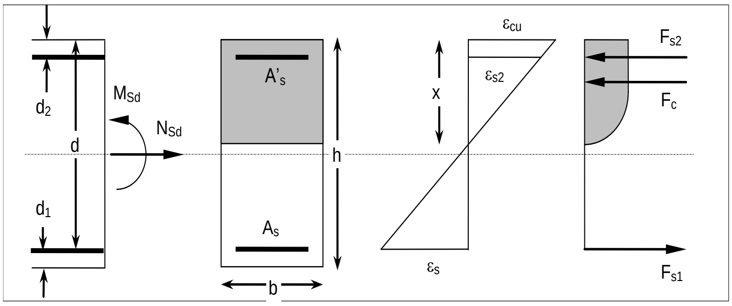


Fig. 55 Structural design, primary lining, egress tunnel, SC G, normal load case

Project no..:	8482B	Editor:	MBu
Project:	Zojila Tunnel	Date:	26.04.2013
Position:	Main Tunnel SC-G/EQ	Notes:	acc. num. analysis 3

Design for bending and axial force according EC 2 (2004)
rectangular section with symmetrical reinforcement



Materials

Reinforcing steel			S 500	Strength class for concrete			C 25/30
characteristic yield strength of reinforcement	f_{yk}	[MPa]	500	characteristic compressive cylinder strength of concrete	f_{ck}	[MPa]	25
partial factor	γ_s	[-]	1	partial factor	γ_c	[-]	1,2
design yield strength of reinf.	f_{yd}	[MPa]	500	coeff. for long term effects	α_{cc}	[-]	1
				design value of concr. comp.	f_{cd}	[MPa]	20,8

Geometry

height	h	[cm]	30	Cross sec. area of reinf.	A_s=A'$_s$	[cm²]	10,71
width	b	[cm]	100	edge distance	d_1, d_2	[cm]	2,5
area	A	[m²]	0,3	reinforcement ratio	ρ_s	[-]	0,7%

Calculation values

ω_{tot}	[-]	0,171	$N_{c,Rd}$	[kN]	3040,8
d_1/h	[-]	0,07	$N_{s,Rd}$	[kN]	1071,0
κ	[-]	0,416	$M_{c,Rd}$	[kNm]	228,1
β	[-]	0,810	$M_{s,Rd}$	[kNm]	139,2
x	[cm]	18,03			

Diagram

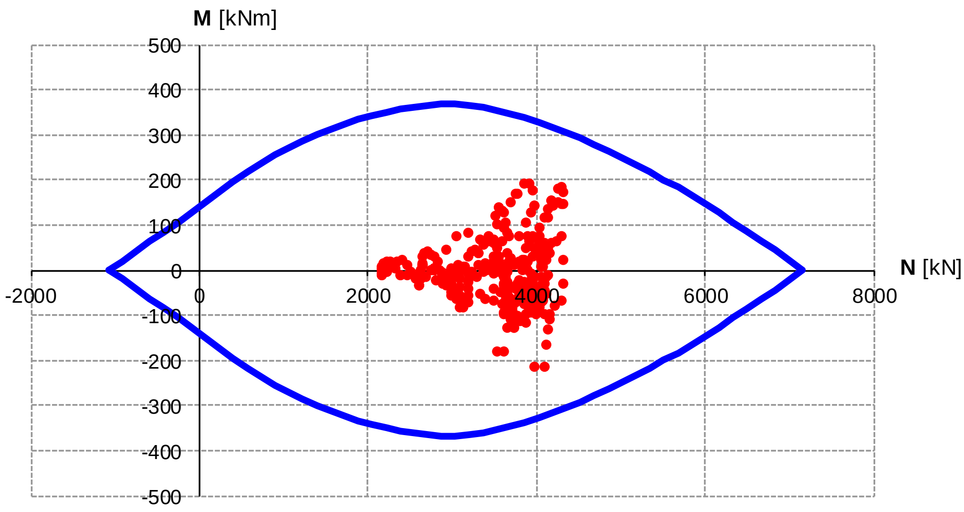
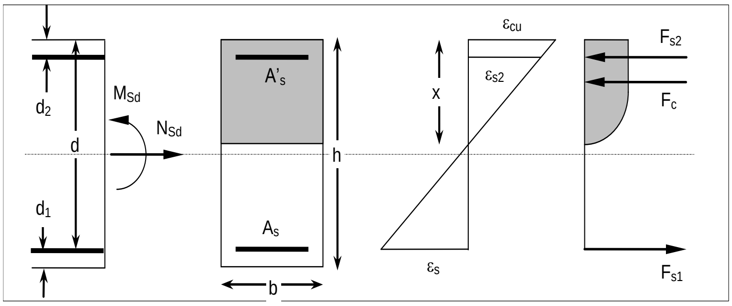


Fig. 56 Structural design, primary lining, main tunnel, SC G, exceptional load case

Project no..:	8482B	Editor:	MBu
Project:	Zojila Tunnel	Date:	26.04.2013
Position:	Egress Tunnel SC-E/E	Notes:	acc. num. analysis 3

Design for bending and axial force according EC 2 (2004)
rectangular section with symmetrical reinforcement



Materials

Reinforcing steel			S 500	Strength class for concrete			C 25/30
characteristic yield strength of reinforcement	f_{yk}	[MPa]	500	characteristic compressive cylinder strength of concrete	f_{ck}	[MPa]	25
partial factor	γ_s	[-]	1	partial factor	γ_c	[-]	1,2
design yield strength of reinf.	f_{yd}	[MPa]	500	coeff. for long term effects	α_{cc}	[-]	1
				design value of concr. comp.	f_{cd}	[MPa]	20,8

Geometry

height	h	[cm]	25	Cross sec. area of reinf.	A_s=A'_s	[cm²]	9,76
width	b	[cm]	100	edge distance	d_1, d_2	[cm]	2,5
area	A	[m²]	0,25	reinforcement ratio	ρ_s	[-]	0,8%

Calculation values

ω_{tot}	[-]	0,187		$N_{c,Rd}$	[kN]	2534,0
d_1/h	[-]	0,08		$N_{s,Rd}$	[kN]	976,3
κ	[-]	0,416		$M_{c,Rd}$	[kNm]	158,4
β	[-]	0,810		$M_{s,Rd}$	[kNm]	102,5
x	[cm]	15,03				

Diagram

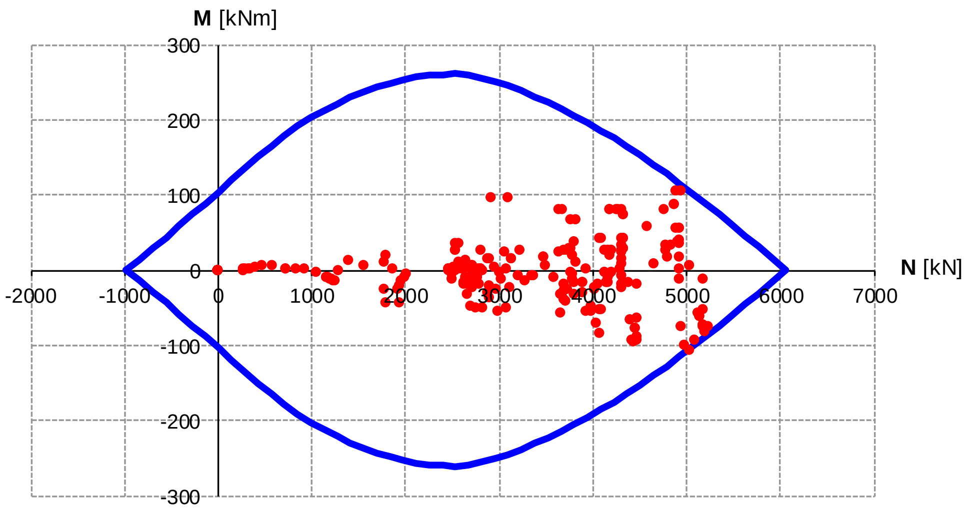


Fig. 57 Structural design, primary lining, egress tunnel, SC G, exceptional load case

Addendum 1

Analytical Analysis, Results