00	2013-03-31	Preliminary Tunnel Design Phase II	MUb, AGo	ASt, WSc	AGo
Rev	Date	Status	Prepared	Checked	Approved

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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 IV: Geotechnical Tunnel Design Report

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Contract No.:	CE (P) BCN/05/2009-10	File:	8482B_II-ZOT_rep-04-12-00
Document No.:	8482B_II-ZOT_rep-04-12-00	Rev.No.:	00

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1 REFERENCES

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[P2] Vayam Technologies Ltd.: Geological Map, Geological Cross and Longitudinal Sections of Zojila Tunnel, India, 2011, 2012.

[P3] Vayam Technologies Ltd.: Chemical Analysis Report of water samples from proposed Zojila tunnel area, Baltal, Minamarg (J&K), India, Apr. 2011.

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[P6] Vayam Technologies Ltd.: Laboratory testing of rock samples for Z-Morh Tunnel, Sonamarg (J & K), India, Apr. 2011.

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[P8] Vayam Technologies Ltd.: Drillings Zojila Tunnel area – Geological logs, drill core photos, India, 2010, 2011.

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1.2 Literature

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2 GEOLOGICAL – GEOTECHNICAL EVALUATION

The present evaluation of the geological and geotechnical conditions for Zojila Tunnel is based on all available data and reports of geological site investigation performed in the years 2009 to 2012 ([P1], [P2], [P3], [P4], [P5], [P6], [P7], [P8], [P9], [P10], [P11]).

2.1 Performed Geological Investigations

2.1.1 Remote Sensing

To identify important geological and geomorphological structures of larger scale (tectonic lineaments, fault zones, mass movements, etc.) satellite images (LANDSAT-TM) and DEM - Digital Elevation Models - were evaluated and analysed using GIS Software. The main results of these analyses are summarized in the report 'Remote Sensing Studies Zojila Pass Tunnel National Highway 1D' [P4].

2.1.2 Geological Field Mapping

Geological mapping of the surface was performed along the entire area of the Zojila Tunnel alignment (scale 1:1000). The results of geological mapping are summarized in the report 'Geological Investigations Zojila Tunnel Project' [P1] and graphically displayed as geological maps, cross and longitudinal sections [P2].

2.1.3 Drillings

19 core drillings with lengths between 21 and 385 m (approx. 2740 m in total) were performed in the years 2010 and 2011 in the area of Zojila Tunnel, including geological drill core logging, photo documentation [P8] and laboratory sampling.

2.1.4 Laboratory Tests

In a first step a few rock samples were collected in the field and tested in a rock mechanics laboratory for a rough estimation of the mechanical intact rock parameters. The following tests were performed: Unit Weight, Specific Gravity, Point Load Test, Brazilian Test, Uniaxial and Triaxial Compression Test. The results of these tests are compiled in [P6], [P7].

In 2012 an extensive laboratory testing program was performed with rock core samples from drillings in the area of Z-Morh and Zojila Tunnel. The following tests were performed: Bulk Density, Point Load Test, Uniaxial Compression Test, Triaxial Compression Test, Brazilian Test, Cerchar Tests and Petrographic Analysis of Mineral Thin Section. The results of these tests are compiled in [P9], [P10], [P11] and summarized in Chapter 2.4.

To evaluate the impact of ground water onto the structural elements of the tunnel and the suitability for construction and drinking purpose, water samples of the Zojila area were collected in the field and chemical analysis was carried out (see [P3]

2.2 Geological Overview

The project area of Zojila Tunnel is located in permo-carboniferous, metamorphic rocks of sedimentary and magmatic origin, which are regional-geologically related to the Tethyan facies [P1]. This facies is subdivided into five lithostratigraphic formations. Along the alignment of Zojila Tunnel three of these formations - 'Panjal Trap', 'Zojila Formation' and 'Agglomerate Slate' - are present (see Fig. 1).

The geological structure is dominated by large scale folding with an anticlinal fold structure in the eastern part and a syncline in the western part of the project area. Due to large scale folding the 'Panjal Trap' formation is encountered in three separate sections along the tunnel alignment (see Fig. 1).

The metamorphic bedrock is covered by recent sediments, mainly along the river beds (river deposits) and at the lower valley slopes below the steep rock cliffs of the mountain ridges (slope and rock fall debris).

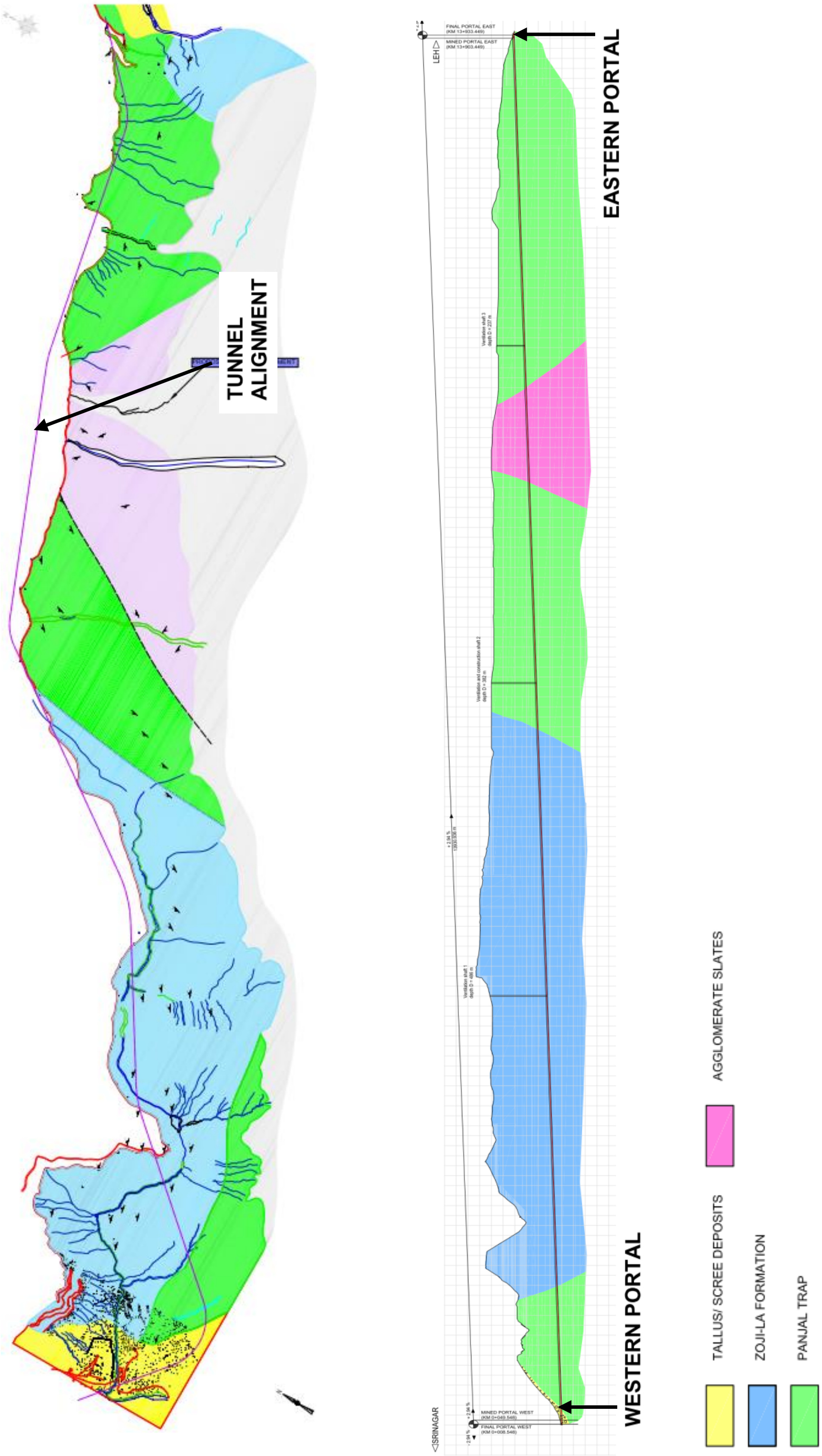


Fig. 1: Left: Geological map of Zojila tunnel from [P2], modified (actual tunnel alignment); Right: Geological longitudinal section of Zojila tunnel from [P12]

2.3 Geological Units

In the following subchapters the geological units which are relevant for the construction of Zojila Tunnel are summarized and characterized, basing on the results of performed geological investigations [P1], [P2], [P6], [P7], [P8].

2.3.1 Rock Fall and Slope Debris

Rock fall and slope debris (debris and talus material) are encountered at both portal areas of proposed Zojila Tunnel. The debris material consists mainly of angular rock boulders with varying block sizes (diameters of a few decimetres up to some meters), which are embedded in a (silty-) sandy-gravelly matrix. The rock mass can be described as a composition of ‘soil-like’ matrix and competent, intact rock blocks of bedrock (metabasite, phyllites, slates). The thickness of the debris accumulations ranges from a few meters up to several decametres at the toe of the valley slopes.

<u>Representative Investigation Drillings:</u> LA-11-03, LA-11-07, LA-11-19, LA-11-20

2.3.2 Zojila Formation

This geological unit is encountered in the western section of Zojila Tunnel and consists mainly of dark grey, fine-grained, finely foliated graphitic, partly carbonaceous, phyllites, slates, schists and subordinately marble (e.g. encountered in drilling LA-11-08; see [P11]). Laboratory tests performed with samples collected in the area of proposed Zojila Tunnel [P7], [P11] and simple manual field index tests indicate a dominance of strong uniaxial compressive strength. Due to intensive foliation and slaty cleavage the rock mass is highly anisotropic. The spacing of the foliation planes is very thinly to thinly (2-20 cm).

Tectonic shearing along fracture zones or shear zones increase the degree of fracturing (spacing of foliation planes < 2 cm) and result in clayey coatings or fillings on discontinuities (shear planes). Thick (several decimetres to several meters), continuous zones of fine-grained fault rock (fault gouge) were not encountered during performed site investigations. But due to morphological features the occurrence of bigger shear zones or faults within this geological unit can be assumed.

<u>Representative Investigation Drillings:</u> LA-10-02, LA-10-06, LA-11-03, LA-11-05, LA-11-08, LA-11-09, LA-11-10

2.3.3 Panjal Trap

This geological unit consists of metamorphic (greenschist facies) volcanic rocks of basaltic-andesitic origin (Metabasite). The mineralogical composition is dominated by Plagioclase and Pyroxene with Amphiboles and Olivine. Volcanic textures and structures like pillow lavas or porphyritic textures are preserved commonly. Younger, low temperature metamorphic conditions led to serpentinization which can be observed along discontinuities.

Generally the Metabasites are a greenish to dark grey, fine-grained, hard and compact rock. Laboratory tests [P7], [P11] and simple manual field index tests indicate a dominance of very strong uniaxial compressive strength. The rocks are fresh, with slight weathering phenomena (discoloration at surfaces and along discontinuities) near to the surface. The Metabasites are dominated by massive rock mass with thickly (60-200 cm) to very thickly (>200 cm) spaced foliation planes. Additionally, schistose types with thinly to medium spaced (6-60 cm) foliation planes are encountered.

Representative Investigation Drillings: LA-11-20 and drillings of 'Panjal Trap' formation in the area of Z-Morh Tunnel

2.3.4 Agglomerate Slate

The 'Agglomerate Slate' formation is found in the eastern part of the tunnel in an anticlinal core (see Fig. 1). This formation consists of greywacke, slates, phyllites and schists with angular fragments, mainly of quartz-porphyry, granite or slates. Due to its internal fabric the pyroclastic origin of the rocks is indicated.

The rock mass is mainly thinly foliated with foliation spacings varying between 6 and 20 cm. Laboratory tests [P7], [P11] and simple manual field index tests indicate a dominance of medium strong to strong uniaxial compressive strength.

Representative Investigation Drillings: LA-11-12, LA-11-11B

2.4 Laboratory Tests

2.4.1 Results of performed laboratory tests

In the following tables the results of the laboratory testing program 2012 [P10], [P11] are summarized and evaluated in relation to the different geological units. The compilation includes all tests performed with rock core samples of 'Panjal Trap', 'Zojila Formation' and 'Agglomerate Slate' from the area of Zojila tunnel and if in similar geological formation also Z-Morh tunnel.

Metabasites of 'Panjal Trap':

	Bulk Density	Point Load Test		Uniaxial Compression Test		Triaxial Compression Test		Brazilian Test	Cerchar Test
		Point Load Index $I_{S(50)}$	≈ UCS (= $22*I_{S(50)}$)	Density	UCS	c	phi	Tensile Strength	CAI
	[g/cm³]	[MPa]	[MPa]	[g/cm³]	[MPa]	[MPa]	[°]	[MPa]	[-]
mean	2,90	6,24	137,22	2,90	121,94	34,73	49,22	24,40	0,81
min	2,84	2,76	60,72	2,84	66,40	19,03	45,10	15,14	0,69
max	2,98	9,43	207,46	5,80*	215,2	43,97	52,30	33,17	0,97
number	14	15	18	12	13	5	5	15	7

*…outlier (not considered for calculation of mean)

From rock core samples of 'Panjal Trap' formation six mineral thin sections were prepared and analysed petrographically. The samples consist of fine-grained, hard and compact rocks of volcanic origin. The mineral assemblages are dominated by feldspar (plagioclase) and pyroxene subordinately with quartz, biotite, olivine, hornblende and iron oxides (for detailed results see [P10]).

Phyllites, Slates and Schists of 'Zojila Formation':

	Bulk Density	Point Load Test		Uniaxial Compression Test		Triaxial Compression Test		Brazilian Test	Cerchar Test
		Point Load Index $I_{S(50)}$	≈ UCS (= $22*I_{S(50)}$)	Density	UCS	c	phi	Tensile Strength	CAI
	[g/cm³]	[MPa]	[MPa]	[g/cm³]	[MPa]	[MPa]	[°]	[MPa]	[-]
mean	2,70	4,11	90,38	2,70	55,78	17,24	52,53	12,36	0,79
min	2,61	1,28	28,16	2,62	22,10	13,32	47,54	7,45	0,61
max	2,79	8,57	188,54	2,79	105,10	20,77	54,37	17,45	1,32
number	44	60	60	34	34	8	8	42	33

From core samples of rocks of 'Zojila Formation' twelve mineral thin sections were prepared and analysed petrographically. Most samples consist of fine- to medium grained schists and phyllites. The mineral assemblages are dominated by quartz, feldspar and mica (muscovite, biotite and sericite), subordinately with chlorite, chloritoid and iron oxides. An exception of this are samples of marble in drillings LA-10-04 and LA-11-08 which mainly consist of calcite, dolomite and wollastonite (for detailed results see [P11]).

Greywackes, Slates, Phyllites, Schists of 'Agglomerate Slate'

	Bulk Density	Point Load Test		Uniaxial Compression Test		Triaxial Compression Test		Brazilian Test	Cerchar Test
		Point Load Index $I_{S(50)}$	≈ UCS (= 22*$I_{S(50)}$)	Density	UCS	c	phi	Tensile Strength	CAI
	[g/cm³]	[MPa]	[MPa]	[g/cm³]	[MPa]	[MPa]	[°]	[MPa]	[-]
mean	2,90	2,60	57,21	2,90	60,64	16,97	51,65	11,53	0,79
min	2,75	1,32	29,04	2,77	35,94	14,49	49,00	9,94	0,66
max	2,97	4,22	92,84	2,97	71,90	18,37	53,78	14,63	0,96
number	14	24	24	10	10	3	3	13	14

From core samples of rocks of 'Agglomerate Slate' five mineral thin sections were prepared and analysed petrographically. Most samples consist of fine- to medium grained schists. The mineral assemblages are dominated by quartz, feldspar, chlorite, chloritoid and mica (muscovite and biotite), subordinately with iron oxides (for detailed results see [P11]).

2.4.2 Discussion of laboratory results

The results of the performed Bulk Density Tests, Point Load Tests, Uniaxial Compression Tests, Triaxial Compression Tests and Brazilian Tests provide reliable data basis for the evaluation of the intact rock parameters for the different geological units.

The CAI-values of the performed Cerchar Abrasivity Tests for samples of Metabasites ('Panjal Trap') seem to be too low (CAI 0.69 – 0.97: "slightly abrasive"). According to experience from other projects as well as data from literature the CAI-values of such type of rocks should be approximately in the range of 2.0 to 4.0 ("very abrasive").

2.5 Discontinuity and Tectonic Structure

The evaluated discontinuity pattern of the project area of Zojila Tunnel (see Fig. 2) is based on the discontinuity data gained during geological field mapping.

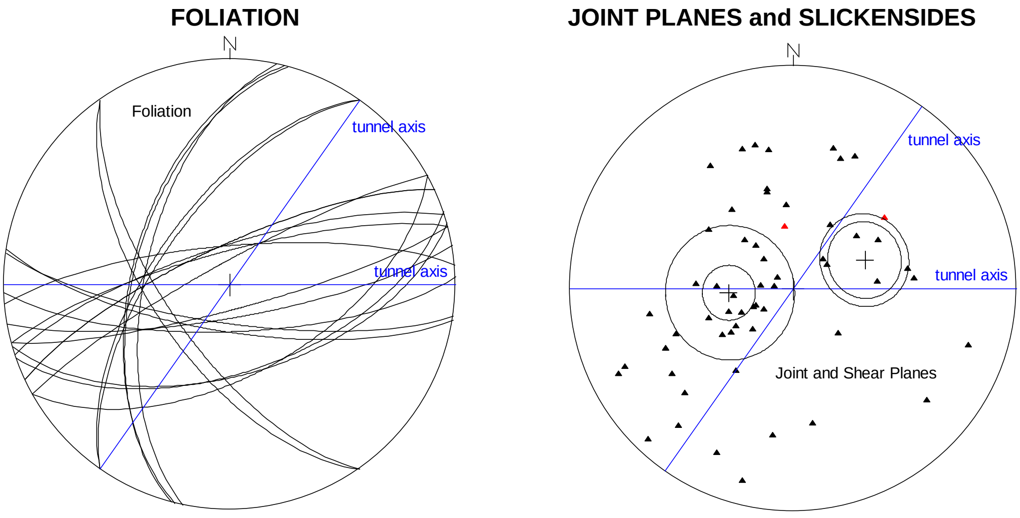


Fig. 2 Discontinuity pattern of Zojila project area (data taken from [P1]); Left plot: foliation orientations displayed as great circles; Right plot; Orientation of Joints (black) and slickensides (red) displayed as pole points

The main discontinuity set, especially in schistose Metabasites, Phyllites and Greywackes, is the foliation. The foliation planes are steeply dipping and strike ENE-WSW to NNE-SSW. Due to overall fold structure the foliation planes are dipping towards SW to SSW or towards NNW to WNW. The spacing of the foliation planes is related to the rock type, thickly to very thickly (60-200 cm, >200 cm) in massive Metabasite, thinly to medium (6-60 cm) in schistose Metabasites and very thinly to thinly in Phyllites and Greywackes (2-20 cm).

In addition to the foliation two to four joint-sets are developed in the rock mass. The two dominating joint sets (see Fig. 2, right plot) are striking N-S and NNW-SSE and dipping gently to medium steeply towards E and towards WSW. The spacing of the joint planes is predominantly in the range of a few meters. The discontinuity surfaces are mainly planar to undulating and rough. Tectonic shearing leads to formation of slickensided shear planes, which are partly filled with fault gouge. Shear zones filled with 10 to 15 cm fault gauge were recorded during geological mapping of the project area.

During geological mapping and core drilling no faults with thicknesses of several metres or more were investigated. But due to regional geological background and due to morphological features the occurrence of faults with relevant amount of fault rocks and fracture zones can be expected.

2.6 Ground Water Conditions

The bedrock is basically nonporous and impermeable but the joints, fractures and shear zones provide considerable permeability. Due to this water inflows along open joints or zones of higher degree of fracturing can be expected.

The coarse-grained slope debris encountered at the portal areas and shaft heads of Zojila Tunnel are characterised by higher permeability compared to the underlying bedrock. Therefore water inflows during excavation within debris material as well as at the border between slope debris and bedrock can be expected. The occurrence and amount of these water inflows are also controlled by seasonal fluctuations and weather conditions (heavy rainfalls, snow melting season, etc.).

2.7 Seismicity

The project area of Zojila Tunnel is situated in the seismically active mountain range of the Himalaya and is influenced by active faulting associated with main tectonic features of the Himalayan mountain belt (e.g. Main Central Thrust, Main Boundary Thrust, Kishtwar Fault, etc.). Historical and instrumental data reveal that at least eleven earthquakes with magnitudes ≥ 6.0 were recorded in this region from around 1800 to present date [P5].

According to seismic zoning map of India (BIS Code IS 1893: 2002, Part I) the tunnel site is situated in Zone IV, which represents the second most vulnerable zone category. Seismic design parameters relevant for the project area related to Maximum Credible Earthquake (MCE) and Design Basis Earthquake (DBE) are accelerations of 0.24 g and 0.12 g [P4].

2.8 In-situ Stress Condition

Zojila tunnel runs basically parallel to the river valley up to Zojila pass with high mountains ranges on both sides. Due to the given topographical conditions a significant effect on the initial stress state is assumed. A numerical model is developed for the evaluation of the in situ stress condition at the level of Zojila tunnel for the given topography and for different locations of the tunnel alignment in respect to the valleys course.

2.8.1 Numerical Model

Numerical analyses were performed by the finite element code PLAXIS on a 2-dimensional model assuming plane strain conditions. The underground conditions were assumed “homogeneous”. The tunnel was not considered in this model as it does not have an effect on the initial stresses.

As representative cross section the narrow valley between two mountain ranges with steep slopes at tunnel chainage approx. km 5+000 was modelled. The tunnel is located at the northern slope toe about 550 m below ground surface and about 450 m below valley bottom. The valley bottom is at a level of approx. 3500 m, the mountains are approx. 4600 m high at both sides.

The model boundaries were determined by the distance of the mountain tops at both sides and about 550 m (≈ overburden) below the tunnel. The model was fixed at the lower boundary in x- and y-direction and at the lateral boundaries in x-direction. The upper boundary was free to move.

The mesh generation was done automatically by the finite element program with 15-noded triangular shaped elements.

2.8.2 Modelling of In-situ Stress Condition

The initial stresses in a rock mass are influenced by the weight of the material and the history of its formation. In PLAXIS the initial stresses can be generated in two ways:

- K_0-Procedure: initial vertical stresses due to self-weight of material; lateral effective stresses based on vertical effective stresses and specified K_0-value; should only be used in cases with horizontal ground surface and soil layers (Coulomb's criterion can be violated, equilibrium is not guaranteed)
- Gravity loading: initial stresses due to self-weight of material and Poisson's ratio; should be used in all other cases; only normally consolidated stress states can be created

For the generation of the initial stresses the Gravity Loading procedure was used.

2.8.3 Simulations and results

Main goal was the evaluation of the initial stresses. In total 5 calculations were performed varying the Poisson's ratio in the given ranges for GT 2 and GT 7. For the other rock mass input parameters the expected mean values were used. The results for the vertical stresses do not differ significantly. Following Fig. 1 shows exemplarily the results of one simulation.

The results in Fig. 1 show that beneath valley bottom the vertical stresses increase faster with depths as they do at the lateral model boundaries because of the redistribution of stress towards the valley due to the topographical conditions. Following Tab. 2 shows the difference between the classical approach for overburden stresses of γ x h compared to the results of the numerical analyses depending on the selected profile in the numerical model at tunnel level.

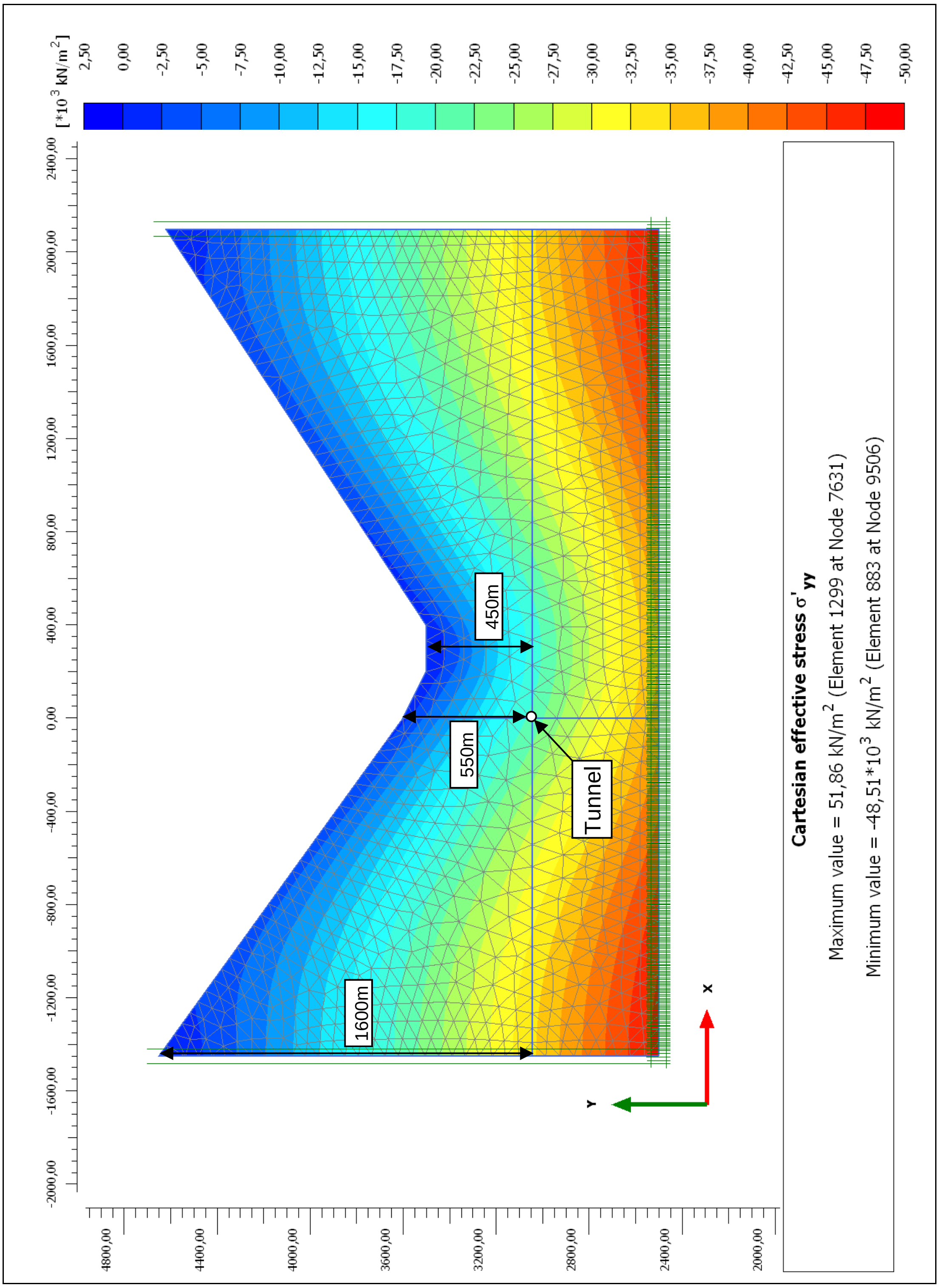


Fig. 3 Example for the results of the initial vertical stress field

Tab. 1 Comparison of classical and numerical calculated vertical stresses (GT 2)

Selected vertical profile	γ_{mean} [kN/m³]	**h** [m]	$\sigma_{y,class}$ [kPa]	$\sigma_{y,num}$ [kPa]	$\Delta\sigma_y$ [kPa]	$\Delta\sigma_y$ [%]
Lateral boundaries	28,5	1600	45.600	34.000	-11.600	-25
Tunnel axis in CS 1-1	28,5	550	15.680	21.000	5.320	+34
Valley bottom	28,5	450	12.830	18.500	5.670	+44

γ_{mean} Specific weight (mean value)
h vertical distance from ground surface to tunnel level
$\sigma_{y,class}$ Effective vertical stress at tunnel level from classical approach (= γ x h = 100 %)
$\sigma_{y,num}$ Effective vertical stress at tunnel level from numerical analyses Fig. 1

Tab. 2 shows that – assuming the tunnel level as constant in the cross section – the highest increase of the vertical stresses in relation to the calculated overburden stress values occurs if the tunnel runs below valley bottom. In this case an increase of the calculated vertical overburden stress value in the order of 50 % is adequate.

In Tab. 3 the results of the performed numerical simulations with varying Poisson's ratios in the given ranges for GT 2 and GT 7 are presented. From the computed vertical and horizontal stresses at the designed tunnel location in cross section CS 1-1 depending on the selected Poisson's ratio, the coefficient of lateral earth pressure can be back-calculated.

Tab. 2 Initial stresses for Ground Type GT 2 and GT 7 at tunnel level in CS 1-1

Ground Type	**Initial Stress Generation**	γ_{mean} [kN/m³]	ν [-]	σ_y [kPa]	σ_x [kPa]	$K_{0,x}$ [-]
GT 2	Gravity loading	28,5	0,10	21.000	4.470	0,21
	Gravity loading	28,5	0,20	21.060	8.400	0,40
	Gravity loading	28,5	0,40	21.150	20.100	0,95
GT 7	Gravity loading	27,0	0,15	20.400	13.750	0,67
	Gravity loading	27,0	0,30	20.040	14.440	0,72
	Gravity loading	27,0	0,40	19.800	19.300	≈ 1,0

γ_{mean} Specific weight (mean value)
ν Poisson's ratio
σ_y Effective vertical stress at tunnel level
σ_x Effective vertical stress at tunnel level
$K_{0,x}$ Coefficient of lateral earth pressure (back-calculated from initial stress state)

In DPR Volume V: Primary Lining Design Report, numerical and analytical calculations are carried out based on the above investigations on the initial stress conditions. The initial stresses are increased of approx. 50 % in analytical analyses to consider the higher overburden stresses in the valley bottom. In the numerical analyses the actual ground surface of each calculation section is modelled to consider the influence of the ground surface onto the initial stress state. The lateral earth pressure coefficient at rest k_0 of 1.0 is applied in the numerical and analytical analyses in DPR Volume V: Primary Lining Design Report to consider a conservative stress state condition and consequently higher internal forces in the lining.

2.9 In-situ Temperature

No in-situ temperature tests were performed. The Zojila Tunnel is not very deep, the maximum distance to surface is approximately 600 m. Due to this the increase of ground temperature along the tunnel alignment is assumed to follow a typical ground gradient and being not relevant for the tunnel design.

3 ROCK MASS CLASSIFICATION

3.1 Methodology

One of the essential principles of the NATM geotechnical design procedure (rock mass classification) is to identify hazards, which might arise during excavation of an underground structure, then devise methods to mitigate those hazards, and arrive at a safe and economical construction. To be able to identify the potential hazards (or modes of failure), a geological model has to be established, including distinguishing rock types as well as structural characteristics and singularities (joint setup, faults, etc.), and the determination of factors influencing the behaviour (stresses, water, size and shape of underground opening, etc.). In a next step, analyses for an unsupported tunnel are done to identify the potential hazards. Depending on the type of potential failure mode of the unsupported tunnel a construction concept is devised with the aim of optimally applying construction measures to mitigate the expected hazards. This allows addressing local and project or site specific problems and requirements.

This common practice of design procedure in Austria has been summarized and published in a guideline in 2001, which has been revised in 2008 and translated to English in 2009 [L1]. In the following the basic design procedure according to the guideline is briefly outlined.

The geotechnical design, as part of the tunnel design, serves as a basis for approval procedures, the tender documents (determination of excavation classes and their distribution) and the determination of the excavation and support methods used on site [L2]. The flow chart in Fig. 4 shows the basic procedure to develop the geotechnical design, beginning with the determination of the ground types and ending with the definition of excavation classes.

The procedure incorporates following steps:

Step 1 – Establishment of geological model and determination of Ground Types

The first step starts with the establishment of the geologic model and proceeds by defining geotechnically relevant parameters for each Ground Type. The key parameters values and distributions are determined from available information and/or estimated with engineering and geological judgment. Ground with similar properties is classified into Ground Types (GT). The number of Ground Types elaborated depends on the project specific geological conditions.

Step 2 – Determination of Ground Behaviour and assignment to Behaviour Types

The second step involves evaluating the potential ground behaviours considering each Ground Type and local influencing factors, including the relative orientation of relevant discontinuities to the excavation, ground water conditions, stress situation, etc. For each section, which has similar ground properties and influencing factors, the Behaviour Type is determined.

The ground behaviour has to be evaluated for the full cross sectional area without considering any modifications including the excavation method or sequence and support or other auxiliary measures.

The evaluated project specific ground behaviours shall be assigned to basic Behaviour Types (Tab. 3). Project specific conditions may require a further subdivision of the Ground Behaviour Types as well as a detailed description of the single expected behaviours.

Basically this step shall identify potential hazards, like type of possible failure mode, magnitude and characteristics of displacements, amount of water inflows and its effects on ground stability. The knowledge of the potential hazards is an important basis for the selection of the construction concept.

Step 3 – Selection of construction concept

Based on the ground characteristics and the determined ground behaviour for each characteristic situation a feasible construction concept is chosen, consisting of excavation method, sequence of excavation, support and auxiliary methods.

The target of the selected construction concept is to mitigate the hazards identified in step 2 in an efficient and economical way.

Step 4 – Assessment of system behaviour in the excavation area

Under consideration of the construction concept, including sequence of construction, stability of the face and perimeter, and the spatial stress distribution, the system behaviour in the excavation area is assessed.

Step 5 – Detailed determination of the excavation and support method and evaluation of system behaviour in the supported area

The excavation and support methods are fixed in quality and quantity, considering probable further excavation steps, and the system behaviour is determined. The evaluated system behaviour is then compared to the requirements. In case the system behaviour does not comply with the requirements, excavation and support methods have to be modified, and the system behaviour evaluated again.

Step 6 - Geotechnical report - baseline construction plan

Based on steps 1 through 5 the alignment is divided into sections with similar excavation and support requirements. The baseline construction plan (e.g. geotechnical longitudinal section) indicates the excavation and support methods recommended for each section, and contains limits and criteria for possible variations or modifications on site if necessary.

Step 7 - Determination of excavation classes

In the final step of the design process excavation classes are defined, based on the evaluation of the excavation and support measures. The excavation classes form a basis for compensation clauses in the tender documents.

It should be mentioned that the selection of an appropriate construction concept besides pure geotechnical aspects depends on a number of factors, like requirements the underground structure has to meet, site conditions, contractors experience, legal regulations and environmental requirements. As each project to a certain extent is a prototype, those factors have to be carefully assessed to arrive at a safe, economical and sustainable design, which meets all the requirements of users, authorities and the public.

As could be seen from the procedure outlined above, with the NATM design approach each case is treated separately, by identifying the inherent hazards and designing appropriate mitigation measures under consideration of project specific requirements and boundary conditions. Simplifications have to be limited to the absolutely necessary minimum, and are allowed only, if they do not influence dominant mechanisms.

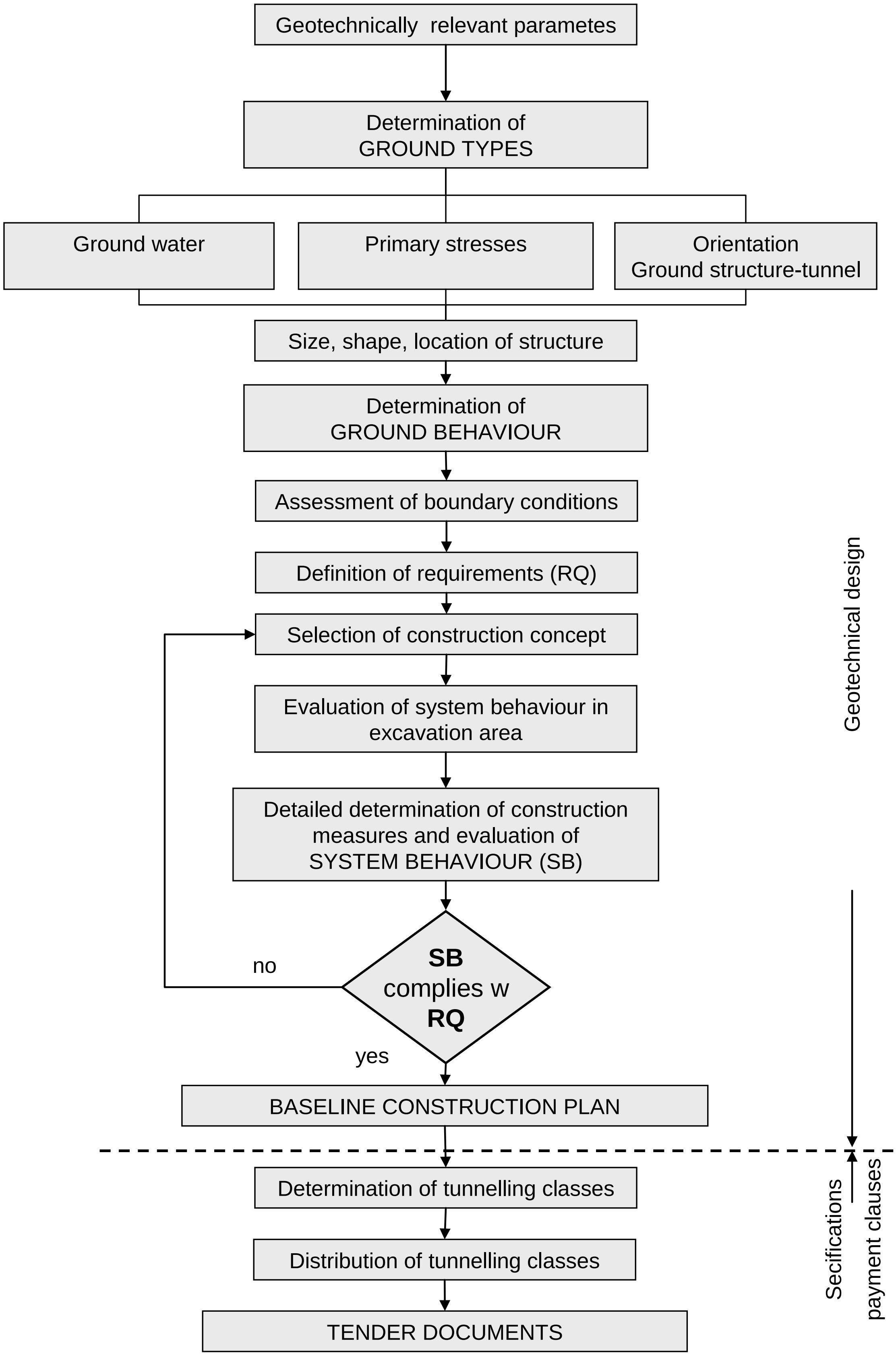


Fig. 4 Basic procedure for the geotechnical design of underground openings from [L1]

Tab. 3 General categories of different Ground Behaviours from [L1]

Basic categories of Behaviour Types (BT)		**Description of potential failure modes/mechanisms during excavation of the unsupported ground**
1	Stable	Stable ground with the potential of small local gravity induced falling or sliding of blocks
2	Potential of discontinuity controlled block fall	Voluminous discontinuity controlled, gravity induced falling and sliding of blocks, occasional local shear failure on discontinuities
3	Shallow failure	Shallow stress induced failure in combination with discontinuity and gravity controlled failure
4	Voluminous stress induced failure	Stress induced failure involving large ground volumes and large deformation
5	Rock burst	Sudden and violent failure of the rock mass, caused by highly stressed brittle rocks and the rapid release of accumulated strain energy
6	Buckling	Buckling of rocks with a narrowly spaced discontinuity set, frequently associated with shear failure
7	Crown failure	Voluminous overbreaks in the crown with progressive shear failure
8	Ravelling ground	Flow of dry or moist, intensely fractured, poorly interlocked rocks or soil with low cohesion
9	Flowing ground	Flow of intensely fractured, poorly interlocked rocks or soil with high water content
10	Swelling ground	Time dependent volume increase of the ground caused by physical-chemical reaction of rock and water in combination with stress relief, leading to inward movement of the tunnel perimeter
11	Ground with frequently changing deformation characteristics	Combination of several behaviours with strong local variations of stresses and deformations over longer sections due to heterogeneous ground (i.e. in heterogeneous fault zones; block-in-matrix rock, tectonic melanges)

3.2 Ground Types (GT)

For the geotechnical design the rock mass relevant for Zojila Tunnel has been classified into Ground Types. 7 Ground Types with similar geotechnical properties have been developed based on the existing geological and geotechnical data. The values for the key parameters and the additional parameters of each Ground Type are evaluated from available information of performed geological site investigation and/or estimated based on experience from other tunnel projects in comparable ground conditions as well as from geotechnical literature.

The Ground Types can be separated into two basic groups, the rock mass consisting of metamorphic bedrock and the overlaying debris material.

For the definition of the Ground Types within the metamorphic bedrock following key parameters are defined:

- Lithology
- Uniaxial compression strength of the intact rock
- Spacing of dominating discontinuity set

In addition to the key parameters the following additional parameters for intact rock and discontinuity properties are given to complete the information:

- Intact rock parameters (Density weight, Hoek constant mi, Poisson's ratio, Young's modulus)
- Discontinuity properties (Joint Roughness Coefficient - JRC)

Based on above parameters the following rock mass parameters are determined:

- GSI
- Rock mass strength (UCS, cohesion, friction)
- Young's modulus

For the definition of the Ground Types within the debris material the following key parameters are defined:

- Rock Type
- Block size
- Grain size of Matrix

The following rock mass parameters for the debris material are determined:

- Block properties (UCS, cohesion, friction, Young's modulus)
- Properties of block contact planes (friction)
- Matrix properties (friction, cohesion, Young's modulus)

Ground Type GT1 – Rock Fall and Slope Debris

Description

Rock Fall and Slope Debris;

The debris consist mainly of angular rock boulders with varying block sizes (diameters of a few decimetres up to some meters), which are embedded in a (silty-) sandy-gravelly matrix.

The matrix can be characterized as “soil-like” material while the blocks can be described with intact rock parameters.

Block-dominated, as well as matrix-dominated types are possible.

Key Parameters

Rock Type		Rock fall and slope debris
Block size (diameter)		some decimetre up to meter
Matrix (grain size)		(silty-)sandy-gravelly
Determined rock mass parameters		
Blocks	UCS [MPa]	>50
	Cohesion [MPa]	>10
	Friction angle [°]	>35
	Young modulus [GPa]	>10
Block Contact Planes – Friction angle [°]		30 - 40
Matrix	Cohesion [MPa]	0 - 0.1
	Friction angle [°]	30
	Young modulus [GPa]	0.1 - 0.2

Example:

Debris/Talus material at a slope-cut in the area of the eastern portal of Zojila Tunnel

Ground Type GT2 – Massive Metabasites	
Description	
Massive, competent rock mass consisting of very strong metabasic rocks (Metabasite of 'Panjal Trap' - Formation) with low degree of fracturing. Foliation planes are thickly (60-200 cm) to very thickly (>200 cm) spaced. The spacing of the joint planes is > 60 cm, mainly in the range of a few meters. Discontinuity surfaces are mainly undulating to planar and rough. The rock mass is generally fresh, respectively slightly weathered near to the ground surface.	
Key parameters	
Rock Type	Metabasite
UCS $_{intact}$ [MPa]	80 - 200
Spacing of dominating discontinuity set [cm]	> 60
Additional parameters	
Density weight [KN/m³]	27 – 30
m_i [-]	25 (20 – 30)
Poisson's ratio [-]	0.1 – 0.2
Young modulus [GPa]	130 (100 – 200)
JRC	10 - 14
Determined rock mass parameters	
GSI [-]	80 (70 – 95)
UCS rock mass [MPa]	80 (30 – 170)
Cohesion [MPa]	15 (7 – 30)
Friction angle [°]	45 (40 – 50)
Young modulus [GPa]	110 (70 – 180)
Example	

Massive Metabasites near Gumri

Ground Type GT3 – Jointed/Foliated Metabasites	
Description	
Jointed/Foliated Metabasites ('Panjal Trap') with - compared to GT2 - increased schistosity and higher degree of fracturing (closer spacing of discontinuities). Foliation planes are thinly to medium spaced (6-60 cm). The spacing of the joint planes varies from 6-60 cm up to a few meters. Discontinuity surfaces are predominately undulating to planar and rough. Occasionally single, slickensided shear planes appear. The rock mass is generally fresh, respectively slightly weathered near to the surface.	
Key parameters	
Rock Type	Metabasite
UCS $_{intact}$ [MPa]	80 - 200
Spacing of dominating discontinuity set [cm]	6 – 60
Additional parameters	
Density weight [KN/m³]	27 - 30
m_i [-]	25 (20 – 30)
Poisson's ratio [-]	0.1 – 0.2
Young modulus [GPa]	130 (100 – 200)
JRC	10 - 14
Determined rock mass parameters	
GSI [-]	60 (50 – 75)
UCS rock mass [MPa]	50 (20 – 100)
Cohesion [MPa]	10 (5 – 20)
Friction angle [°]	40 (35 – 45)
Young modulus [GPa]	70 (20 – 160)

Example

Jointed Metabasites in the area of Zojila Tunnel

Ground Type GT 4 – Faulted Metabasites	
Description	
Metabasites ('Panjal Trap'), heavily fractured, sheared, faulted; Increasingly fractured due to tectonic shearing. Spacing of dominating discontinuity set in mm- to cm-range. Shear planes with slickensided surfaces, occasionally with clayey coatings and fillings in mm- to cm-range. Potential appearance of "soil-like" fault rocks (cataclasite, fault breccia, fault gouge) within shear zones or faults. In bigger fracture or shear zones Ground Type GT 4 is typically associated with GT 3, respectively GT 2. This results in heterogeneous rock mass with rapid variations of rock mass parameters (e.g. degree of fracturing) within short distance.	
Key parameters	
Rock Type	Metabasite, (Fault Rocks)
UCS $_{intact}$ [MPa]	50 - 100, (< 50)
Spacing of dominating discontinuity set [cm]	< 6
Additional parameters	
Density weight [KN/m³]	27 – 29
m_i [-]	20 (15 – 25)
Poisson's ratio [-]	0.15 – 0.30
Young modulus [GPa]	100 (50 – 150)
JRC	6 - 12
Determined rock mass parameters	
GSI [-]	35 (25 – 55)
UCS rock mass [MPa]	8 (5 – 20)
Cohesion [MPa]	2.5 (1.5 – 5)
Friction angle [°]	27 (23 – 35)
Young modulus [MN/m²]	6 (2 – 20)

Ground Type GT5 – Jointed/Foliated Phyllites and Slates

Description

Jointed/Foliated slates, phyllites, schists and greywackes of 'Zoijla Formation' and 'Agglomerate Slate'.

Foliation planes are mainly thinly to medium spaced (6-60 cm). The spacing of the joint planes varies from 6-60 cm up to a few meters. Discontinuity surfaces are predominately undulating to planar and rough. Occasionally single, slickensided shear planes appear.

The rock mass is generally fresh, respectively slightly weathered near to the surface.

Key parameters	
Rock Type	Slates, Phyllites, Schists, Greywackes
UCS_{intact} [MPa]	40 – 100
Spacing of dominating discontinuity set [cm]	6 – 60
Additional parameters	
Density weight [KN/m³]	26 - 29
m_i [-]	10 (7 – 13)
Poisson's ratio [-]	0.1 – 0.2
Young modulus [GPa]	70 (35 – 100)
JRC	10 - 14
Determined rock mass parameters	
GSI [-]	60 (50 – 75)
UCS rock mass [MPa]	15 (6 – 35)
Cohesion [MPa]	4 (2 – 8)
Friction angle [°]	33 (27 – 40)
Young modulus [GPa]	35 (10 – 80)

Example

Jointed/Foliated 'Agglomerate Slate' at Gumri nallah

Jointed/Foliated 'Zojila Formation' near Zojila Pass

Ground Type GT6 – Intensively Foliated Phyllites and Slates

Description

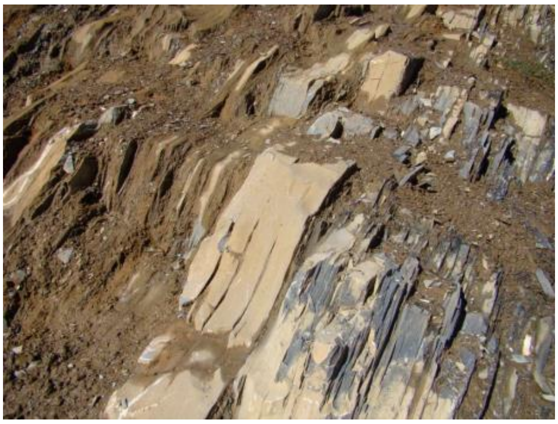
Intensively foliated phyllites, slates and greywackes. Highly anisotropic due to dominance of foliation and slaty cleavage.

Foliation planes are mainly very thinly to thinly spaced (2-20 cm). The spacing of the joint planes varies between cm-range up to a few meters. Discontinuity surfaces are predominately planar and rough to smooth. Occasionally slickensided shear planes appear.

The rock mass is generally fresh, respectively slightly weathered near to the surface.

Key parameters	
Rock Type	Slates, Phyllites, Greywackes
UCS $_{intact}$ [MPa]	25 – 70
Spacing of dominating discontinuity set [cm]	2 – 20, (<2)
Additional parameters	
Density weight [KN/m³]	26 – 29
m_i [-]	10 (7 – 13)
Poisson's ratio [-]	0.15 – 0.25
Young modulus [GPa]	55 (25 – 80)
JRC	8 – 12
Determined rock mass parameters	
GSI [-]	40 (35 – 55)
UCS rock mass [MPa]	6 (3 – 15)
Cohesion [MPa]	2 (1 – 4)
Friction angle [°]	27 (23 – 35)
Young modulus [GPa]	6 (2 – 20)

Examples



Dark grey slates of 'Zojila-Formation'

Ground Type GT 7 – Faulted Phyllites and Slates	
Description	
Heavily fractured, sheared, faulted phyllites and slates; Increasingly fractured due to tectonic shearing. Spacing of dominating discontinuity set in mm- to cm-range (e.g. extremely thin foliated/sheared phyllites). Shear planes with slickensided surfaces, occasionally with clayey coatings and fillings in mm- to cm-range. Potential occurrence of "soil-like" fault rocks (cataclasite, fault breccia, fault gouge) within shear zones or faults. In bigger fault zones Ground Type GT 7 is typically associated with GT5 and GT6 resulting in heterogeneous rock mass with rapid variations of rock mass parameters (e.g. degree of fracturing) within short distance.	
Key parameters	
Rock Type	sheared Slates, Phyllites, Greywackes, Fault Rocks
UCS $_{intact}$ [MPa]	< 50
Spacing of dominating discontinuity set [cm]	< 6
Additional parameters	
Density weight [KN/m³]	26 – 28
m_i [-]	8 (7 – 10)
Poisson's ratio [-]	0.15 – 0.30
Young modulus [GPa]	15 (10 – 20)
JRC	4 – 8
Determined rock mass parameters	
GSI [-]	30 (20 – 40)
UCS rock mass [MPa]	3.5 (1.5 – 7)
Cohesion [MPa]	1 (0.5 – 2)
Friction angle [°]	23 (20 – 27)
Young modulus [MN/m²]	1.5 (0.5 – 3)

Examples

Thinly foliated/sheared phyllites of 'Zojila Formation'

3.3 Behaviour Types

3.3.1 General

The Ground Behaviour represents the determined behaviour of the rock mass due to the excavation of the tunnel without any support measures. To determine the ground behaviour the Ground Types are combined with the predicted ground conditions represented by the influencing factors which are the primary stress condition, the water condition and the orientation of discontinuities.

The influencing factors are described more detailed in the geological and geotechnical investigation reports and the geological and geotechnical evaluation in chapter 2. All relevant data (Ground Type and influencing factors), the resulting rock mass behaviour, typical failure modes and accompanying information are presented systematically in tables. A drawing illustrates the typical behaviour of the rock mass. Additionally the behaviour of the face without support measures is given. The classified Behaviour Types are the basis for the design of appropriate measures to achieve stable tunnel conditions.

In this design phase the Behaviour Types for the main and egress tunnel geometry are determined. The behaviour for the other tunnel geometries such as cross passages, lay-by, ventilation shafts and ventilation caverns shall be investigated in the detailed design.

3.3.2 Criteria for Behaviour Types

Different failure modes occur under different conditions. Different criteria need to be established to be able to identify the applicable Behaviour Types. Considering the geological ground condition, the stress situation, the geometry and size of the opening, and other influencing factors like ground water, the criteria are used to identify different behaviour Types. In the following these criteria, which are used for the identification of different Behaviour Types, are briefly described. Note that combinations of different Behaviour Types can occur, for example stress induced failure of the ground in combination with discontinuity controlled overbreak.

Behaviour Type 1: Stable

“Stable rock mass with the potential of small local gravity induced falling or sliding of blocks”. No failure except minor gravity induced falling of blocks with a volume of less than 0,2 m³.

Behaviour Type 2: Stable with the potential of discontinuity controlled block fall

“Deep reaching discontinuity controlled, gravity induced falling and sliding of blocks, occasional local shear failure. A minimum of 3 joint sets is required to allow the falling or sliding of blocks kinematically. The joint spacing has to be smaller than the tunnel diameter. The block volume has to be more than 0,2 m³.

Influencing factors are joints' orientation, persistence and spacing. Analytical analyses are used to estimate the volume of gravity induced falling and sliding of blocks.

Behaviour Type 3: Shallow shear failure

"Shallow stress induced shear failures in combination with discontinuity and gravity controlled failure of the rock mass". The depth of shear failure or plastic zone is investigated with analytical and numerical analysis with/without fault zones. Main influencing factors are the strength parameters of the rock mass and the primary stress condition. If the plastic zone outside the underground excavation is smaller than 25% of the tunnel diameter (~ 3 m for main tunnel and ~ 1.5 m for egress tunnel) Behaviour Type 3 is assigned.

Behaviour Type 4: Deep seated shear failure

"Deep seated stress induced shear failures and large deformation". The depth of shear failure or plastic zone is investigated with analytical and numerical analysis with/without fault zones. The main influencing factors are the strength parameters of the rock mass and the primary stress condition. If the plastic zone outside the excavation exceeds 25% of the tunnel diameter (~ 3 m for main tunnel and ~ 1.5 m for egress tunnel) Behaviour Type 4 is assigned.

Behaviour Type 5: Rock burst

"Sudden and violent failure of the rock mass, caused by highly stressed brittle rocks and the rapid release of accumulated strain energy". To evaluate the potential for rock burst the approach by Wang & Park [L3] is used, including four conditions for the development of rock burst. The rock mass must have a potential for storing considerable elastic strain energy as well as brittle post peak behaviour, and the stresses must be near the peak strength of the rock mass.

Behaviour Type 6: Buckling failure

"Buckling of rocks with a narrowly spaced discontinuity set, frequently associated with shear failure". Feder and Arwanitakis 1976 [L4] proposed a solution for the determination of buckling failure. The proposed criterion can be used to assign Behaviour Type 6 (buckling failure), based on the bedding plane thickness. The solution is based on the primary stress condition and the friction between the buckling planes. Buckling will occur if the actual thickness of the bedding planes is smaller than the critical thickness t. Main influencing factors are the primary stress state, joint orientation and spacing (thickness of bedding planes), strength parameters of joints, elastic parameters of the rock mass and strength parameters of the rock mass.

Behaviour Type 7: Shear failure under low confining pressure

"Potential for excessive overbreak and progressive shear failure with the development chimney type failure, caused mainly by a deficiency of side pressure".

The solution is based on the comparison of the block weight to the resisting shear forces along the vertical sliding planes. If the weight of the block is higher than the sum of resisting forces in the two sliding planes, a chimney type failure will occur.

The criterion is expressed as follows:

$$G = \gamma_{RM} \times h \times D \qquad\qquad S = H \times \tan(\varphi_{RM}) = \frac{\gamma_{RM} \times h^2}{2} \times K_0 \times \tan(\varphi_{RM})$$

$$G \overset{!}{\leq} 2 \times S \rightarrow stable$$

S is the shear force in the sliding plane [MN], G is the weight force of the block [MN], H is the horizontal primary stress [MN], φ_{RM} is the friction angle of the rock mass [°], h is the overburden [m], γ_{RM} is the specific weight of the rock mass [MN/m³], K_0 is the lateral confining coefficient [-] and D is the diameter of the silo [m].

Main influencing factors are overburden, lateral confining coefficient and the friction angle of the rock mass.

Behaviour Type 8: Ravelling ground

“Flow of cohesionless dry or moist, intensely fractured rocks or Soil”. Influencing parameters for the determination of ravelling ground conditions are the size and shape of the ground particles as well as the grain size distribution. Additionally, the joints’ cohesion of the highly fractured rock affects the potential for failure. For ravelling ground the spacing of discontinuities must be smaller than 10 cm and the block volume of the loose material must be smaller than 0,001 m³. Additionally the cohesion of the joints c_j must be less than c_{limit} = 50 kPa. If both criteria are fulfilled, potential for ravelling ground can is assigned. Main influencing factors are block size of the loose material, and the cohesion of the joints between the blocks.

Behaviour Type 9: Flowing ground

“Flow of intensively fractured rocks or soil with high water content”. Important factors for the determination of flowing ground conditions are the existence of water in the rock mass, the size and shape of single ground particles as well as the grain size distribution. Additionally, the ground permeability around the excavation affects the potential for failure. The critical grain size distribution is characterised with two parameters, d_{90} and d_{10}. These parameters represent the grain size for 90 and 10 percent of grains (weight proportion) smaller than d_{90} and d_{10}, respectively. The following two values have been defined: d_{90} = 20 mm and d_{10} = 0.2 mm. Ground conditions with the associated characteristic values d_{10} and d_{90} within the limits mentioned above as well as ground water up to the tunnel crown (or higher) will result in the assignment of Behaviour Type 10. The limits for both values were chosen to characterise medium sand to medium gravel, which generally has a high potential for flowing ground.

Influencing factors are the grain size of the rock mass material and existence of high water pressure.

Behaviour Type 10: Swelling

"Time dependent volume increase of the rock mass caused by physical-chemical reaction of rock and water in combination with stress relief, leading to inward movement of the tunnel perimeter". A swelling potential can be identified, if the rock mass contains a certain percentage of swelling minerals (for example clay minerals) and water is present. An additional condition is a stress release, which is always associated with tunnel excavation.

Behaviour Type 11: Heterogeneous rock with frequently changing deformation characteristics

"Rapid variations of stresses and deformations (for example in heterogeneous fault zones; block-in-matrix rock, tectonic melanges)". Basic requirements for this Behaviour Type are frequently changing ground conditions which result in heterogeneous displacements of the excavation. Presence of heterogeneous fault zones or "block-in-matrix structured rock" forms the basis for the criterion of this Type. This Behaviour Type is assigned if it is not possible to assign specific Behaviour Types due to the rapid variation of ground conditions.

3.3.3 Behaviour Types

Main Tunnel

According to the defined criteria the following Behaviour Types are expected in the project area for the main tunnel. In the portal areas debris material (GT1) can be expected which is evaluated as the most difficult concerning the ground condition. Debris material is expected in combination with overburden of only a few metres. Additionally water is expected to flow through the debris material during rain falls. These influencing factors lead to BT7 (Crown Failure), BT8 (Ravelling Ground) and predominant BT9 (Flowing Ground). In the area of the massive basic rock (GT2) generally BT1 (stable) and BT2 (Potential of Discontinuity Controlled Overbreak) are predicted. The joint sets are oriented unfavourably to allow kinematical freedom for potential block failure.

Metabasites: In the region with higher fractured basic rock (GT3) mainly BT2 and BT3 (shallow stress induced failure) in case of increasing overburden (approx. more than 500 m) are predicted. With higher degree of fracturing of (GT4) the shear failure occurs with lower overburden at approx. 200 m (shallow stress induced failure). An increase of the overburden in GT4 to approx. 400 m leads to deep stress induced failure (BT4).

Phyllite and Slates: In the region with jointed/foliated rock (GT5) discontinuity controlled overbreak is predicted up to an overburden of approx. 200 m. Higher overburden lead to shallow stress induced failure (BT3). BT4 (deep stress induced failure) is predicted with higher degree of fracturing (see GT6) and for GT7 at even low overburden of approx. 80 m.

Based on analytical and numerical analysis the following criteria for the assignment of Behaviour Types concerning stress induced failure mechanisms can be defined.

Tab. 4 Criteria for the assignment of stress induced Behaviour Types for main tunnel cross section

		GT1	GT2	GT3	GT4	GT5	GT6	GT7
Overburden Range	0-50 m	SC H	SC A,B,C	SC B,C	SC B,C	SC B,C	SC B,C	SC H
	50-80 m	-	SC A,B,C	SC B,C	SC B,C	SC B,C	SC B,C	SC D
	80-150 m	-	SC A,B,C	SC B,C	SC B,C	SC B,C	SC D	SC E
	150-200 m	-	SC A,B,C	SC B,C	SC D	SC B,C	SC D	SC F
	200-400 m	-	SC A,B,C	SC B,C	SC D	SC D	SC E	SC F, G
	400-500 m	-	SC A,B,C	SC B,C	SC E	SC D	SC E	SC G
	500-650 m	-	SC A,B,C	SC D	SC E	SC D	SC F	SC G

Egress Tunnel

The Behaviour Types predicted in egress tunnel excavation are very similar to the main tunnel. Analogous to the main tunnel, in the portal areas debris material (GT1) can be expected resulting in BT7 (Crown Failure), BT8 (Ravelling Ground) and predominant BT9 (Flowing Ground). BT1 (stable) and BT2 (Potential of Discontinuity Controlled Overbreak) are expected in areas with GT2 (massive basic rock).

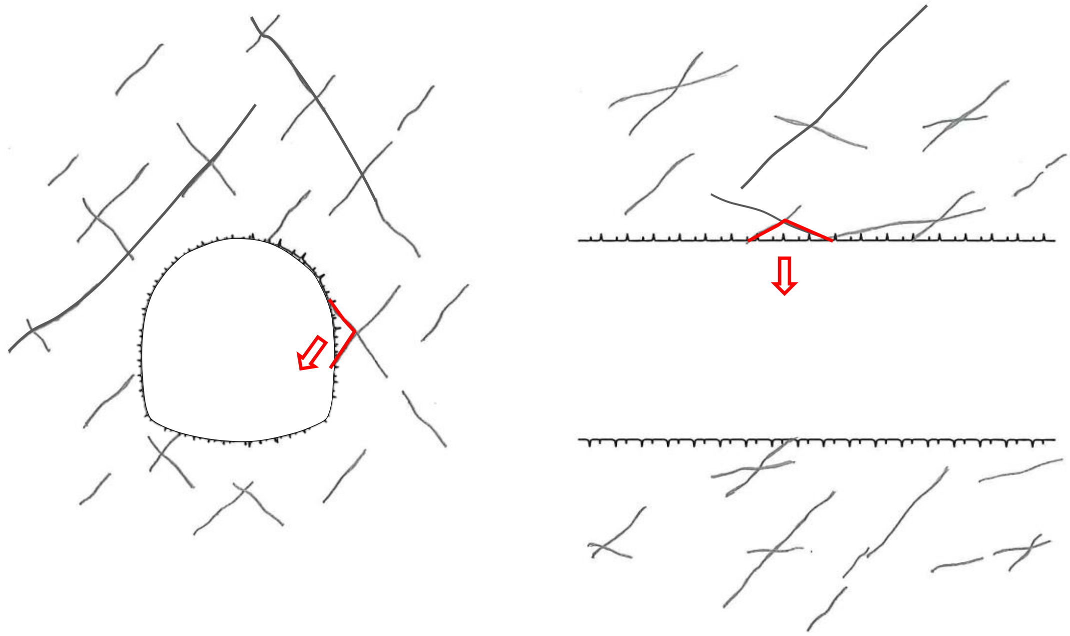
Based on analytical and numerical analysis the following criteria for the assignment of Behaviour Types concerning stress induced failure mechanisms can be defined.

Tab. 5 Criteria for the assignment of stress induced Behaviour Types for egress tunnel cross section

		GT1	GT2	GT3	GT4	GT5	GT6	GT7
Overburden Range	0-50 m	SC F	SC A,B,C	SC B,C	SC B,C	SC B,C	SC B,C	SC F
	50-80 m	-	SC A,B,C	SC B,C	SC B,C	SC B,C	SC B,C	SC D
	80-150 m	-	SC A,B,C	SC B,C	SC B,C	SC B,C	SC D	SC D
	150-200 m	-	SC A,B,C	SC B,C	SC D	SC B,C	SC D	SC E
	200-400 m	-	SC A,B,C	SC B,C	SC D	SC D	SC D	SC E
	400-500 m	-	SC A,B,C	SC B,C	SC D	SC D	SC D	SC E
	500-650 m	-	SC A,B,C	SC D	SC D	SC D	SC E	SC E

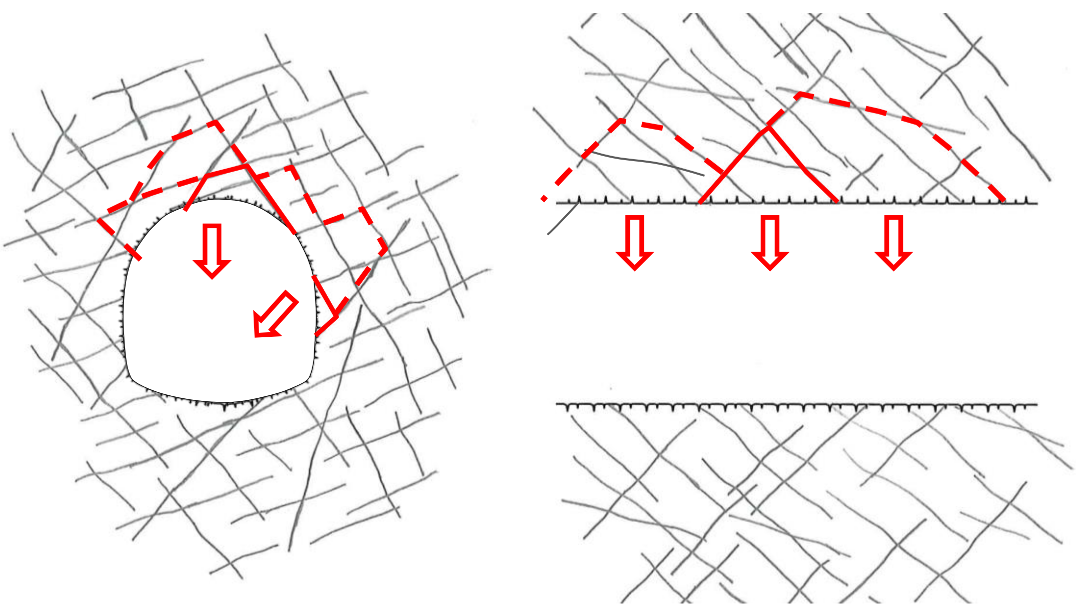
BEHAVIOUR TYPE 1 - STABLE	
Ground Type	GT2
Orientation of main joint sets	Various, moderately to steeply inclined
Stress conditions	The stresses are below the rock mass strength
Ground water	Predominantly dry conditions with local inflows along major discontinuities (low water pressure)
Rock mass behaviour (failure mechanism and behaviour during failure)	Stable ground with the potential of small local gravity induced falling or sliding of blocks from crown or side wall
Radial displacements	Generally small displacements in the range of a few millimetres
Face stability	Stable with potential of small local gravity induced sliding of blocks along unfavourably orientated discontinuities

Example of typical ground behaviour (for GT2)



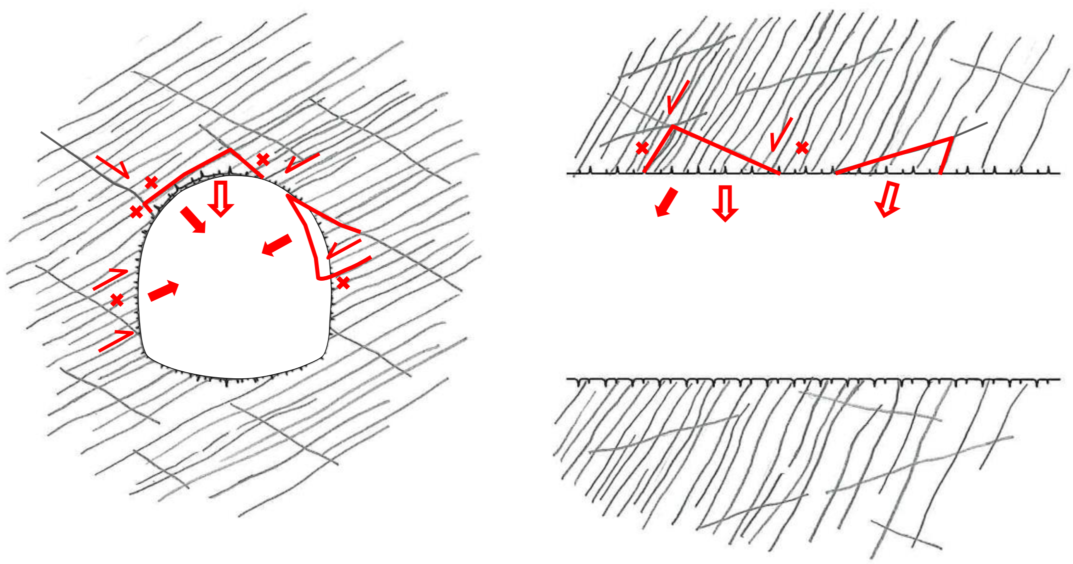
BEHAVIOUR TYPE 2 – DISCONTINUITY CONTROLLED OVERBREAKS	
Ground Type	GT2, GT3, GT4, GT5
Orientation of main joint sets	Foliation moderately to steeply inclined, different striking directions (perpendicular to subparallel to tunnel axis due to varying orientation of tunnel and of foliation) 2-4 moderately to steeply dipping joint-sets (striking subparallel to obtuse-angled to tunnel axis)
Stress conditions	The stresses are below the rock mass strength. Locally exceeding of shear-strength along discontinuities
Ground water	Predominantly dry conditions with local inflows along major discontinuities (low water pressure)
Rock mass behaviour (failure mechanism and behaviour during failure)	Voluminous discontinuity controlled, gravity induced falling and sliding of blocks from crown and side walls. Occasional local shear failure on discontinuities
Radial displacements	Generally small displacements in the range of millimetres to some centimetres
Face stability	In case of unfavourable orientation of major discontinuities (dipping into excavation) potential of voluminous discontinuity controlled sliding of blocks and wedges.

Example of typical ground behaviour (for GT3)



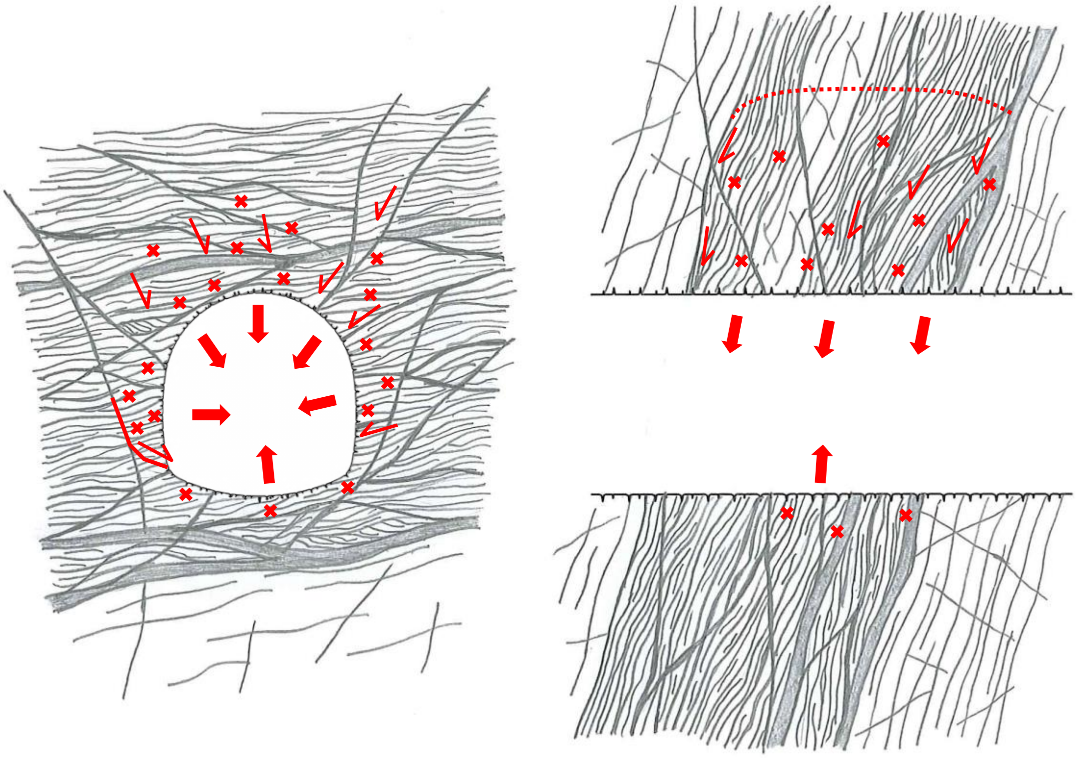
BEHAVIOUR TYPE 3 – SHALLOW SHEAR FAILURE	
Ground Type	GT3, GT4, GT5, GT6, GT7
Orientation of main joint sets	Foliation moderately to steeply inclined, different striking directions (perpendicular to subparallel to tunnel axis because of varying orientation of tunnel and of foliation) 2-4 moderately to steeply dipping joint-sets (striking subparallel to obtuse-angled to tunnel axis)
Stress conditions	The stresses at the tunnel perimeter exceed the rock mass strength.
Ground water	Predominantly dry conditions with local inflows along major discontinuities or fracture zones (low water pressure)
Rock mass behaviour (failure mechanism and behaviour during failure)	Stress induced failure of the rock mass at the tunnel perimeter in terms of brittle failure of the rock or shear failure along discontinuities, resulting in a failure zone of limited depth (up to ¼ of tunnel diameter). Usually combined with discontinuity and gravity controlled falling and sliding of blocks.
Radial displacements	Radial displacements in the range of several centimetres until stabilization.
Face stability	Favourable face stability for foliation dipping into the face, potential for block slides for foliation dipping into the excavation (depending on excavation direction)

Example of typical ground behaviour (for GT4)



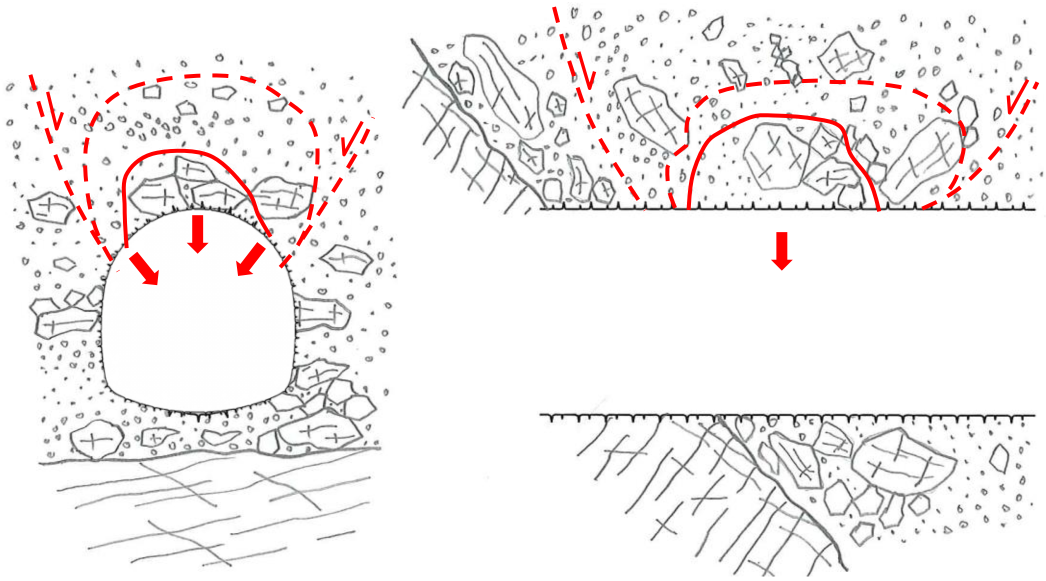
BEHAVIOUR TYPE 4 – DEEP SEATED SHEAR FAILURE	
Ground Type	GT4, GT6, GT7
Orientation of main joint sets	Fracture zones, shear zones and faults orientated parallel to foliation respectively steep inclined and obtuse-angled to tunnel axis.
Stress conditions	The stresses exceed the rock mass strength significantly.
Ground water	Predominantly dry. Potentially local inflows along fracture zones of higher permeability (low water pressure).
Rock mass behaviour (failure mechanism and behaviour during failure)	Stress induced, progressive shear failure, involving large ground volumes. Deep reaching failure zone in combination with long lasting stress redistribution and large deformations. Additionally gravity induced failure due to falling or sliding of sheared and heavily fractured rock mass from crown, face and side walls. Potential for deep reaching, voluminous overbreaks.
Radial displacements	Long-lasting displacements in the range of decimetres up to collapse of tunnel.
Face stability	Spalling and falling of sheared, fractured rock mass from face and sliding along major shear planes

Example of typical ground behaviour (for GT5)



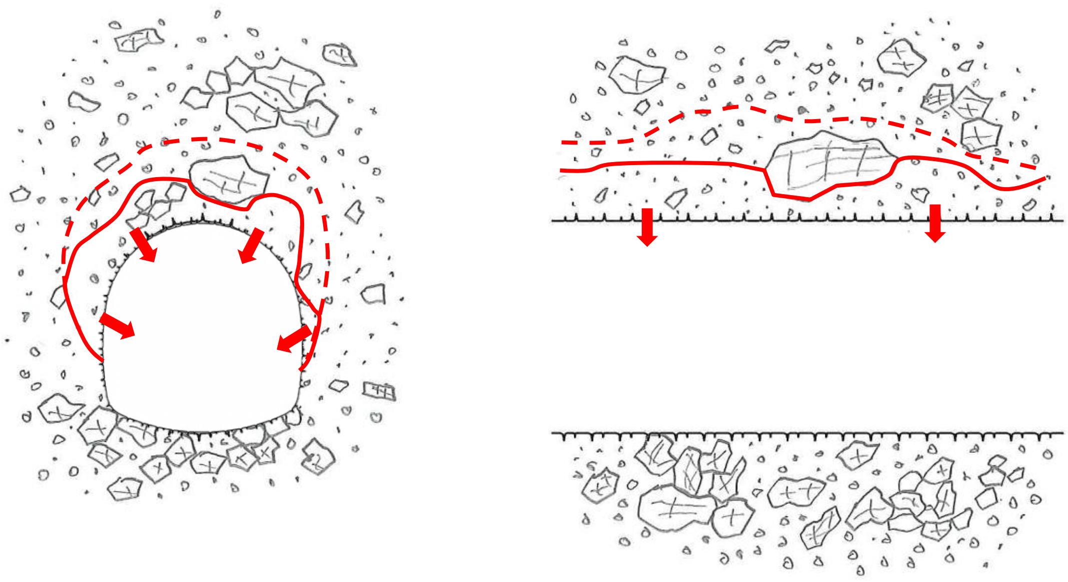
BEHAVIOUR TYPE 7 – CROWN FAILURE	
Ground Type	GT1
Orientation of main joint sets	not relevant
Stress conditions	Low stress level (stresses lower or in the range of rock mass strength) in combination with low horizontal stresses.
Ground water	Predominately dry.
Rock mass behaviour (failure mechanism and behaviour during failure)	Voluminous overbreaks at crown and progressive shear failure induced by low confining pressure. Excessive overbreaks can propagate up to the surface (daylight collapse).
Radial displacements	Failure model will lead to collapse of tunnel.
Face stability	Potential for voluminous overbreaks and progressive shear failure.

Example of typical ground behaviour (for GT1)



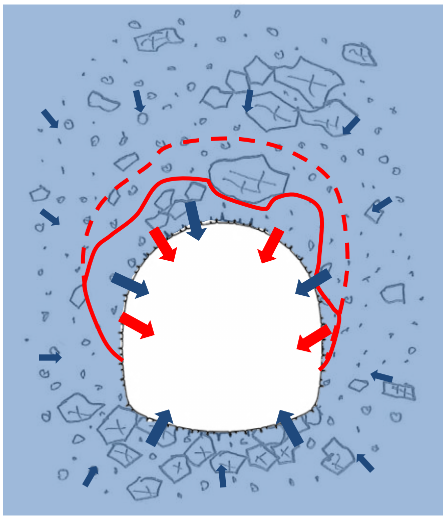
BEHAVIOUR TYPE 8 – RAVELLING GROUND	
Ground Type	GT1
Orientation of main joint sets	not relevant
Stress conditions	Low stresses
Ground water	dry to moist
Rock mass behaviour (failure mechanism and behaviour during failure)	Ravelling of ground due to lack of cohesion or interlocking within the rock mass.
Radial displacements	Not relevant, because failure model will lead to collapse of tunnel.
Face stability	Potential for ravelling of ground into the excavation.

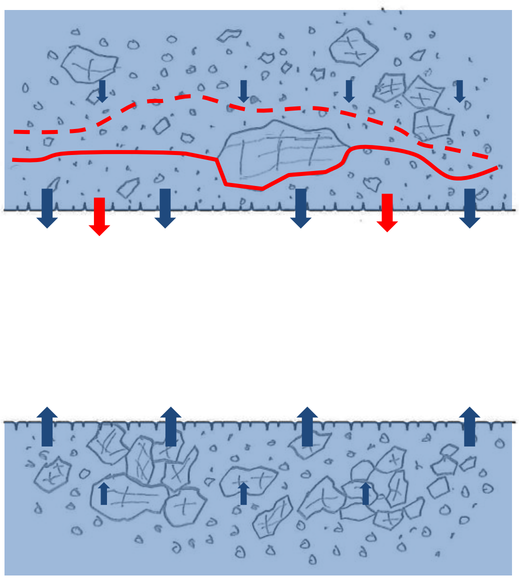
Example of typical ground behaviour (for GT1)



BEHAVIOUR TYPE 9 – FLOWING GROUND	
Ground Type	GT1
Orientation of main joint sets	not relevant
Stress conditions	Low stresses
Ground water	Water inflows due to high permeability.
Rock mass behaviour (failure mechanism and behaviour during failure)	Flowing of ground due to lack of cohesion or interlocking within the rock mass in combination with ground water.
Radial displacements	Not relevant, because failure model will lead to collapse of tunnel.
Face stability	Potential for flowing of ground into the excavation.

Example of typical ground behaviour (for GT1)





4 EXCAVATION DESIGN

4.1 General

Tunnel excavations can be divided into two main groups with respect to the construction procedure of the tunnel excavation.

- Continuous tunnel excavation: tunnel excavated by a tunnel boring machine (TBM). The face of the tunnel excavation is advancing continuously. The full face is excavated at once with the TBM.
- Cyclic tunnel excavation: tunnel construction with New Austrian Tunnelling Method (NATM). The face of the tunnel excavation is advancing cyclically. When required the tunnel excavation can be sub-divided into different excavation sequences.

In the following the two different excavation methods are discussed concerning their application at Zojila tunnel and then selected NATM excavation concept is described more detailed.

4.2 Comparison between NATM and TBM

A more detailed comparison between conventional tunnel excavation and excavation with tunnel boring machines is given in DPR Volume II: Tunnel Design Report, Addendum 2. The comparison given hereafter is a summary of the main aspects presented in the aforesaid document.

Tunnel boring machines (TBM) combine the excavation and the support installation in one working process with a high degree of automation, which significantly increases the productivity and the construction speed. On the other hand, the necessary investments are very high and the excavation cross section cannot be changed, which reduces the flexibility of the construction method.

In the following a brief listing of the project specific pros and cons of a TBM compared to NATM is given according to DPR Volume II: Tunnel Design Report, Addendum 2.

Tab. 6 Listing of the project specific pros and cons of a TBM compared to NATM as per DPR Volume II: Tunnel Design Report, Addendum 2

Item	Pro	Con
Tunnel cross section		The tunnel diameter is very large for a TBM tunnel. The tunnel diameter cannot be adjusted. The TBM can only excavate in one cross section, niches have to be constructed later on
Excavation material		More material is to be disposed due to higher TBM excavation cross section.
Ground condition		Tunnel sections in Zojila formation (approx 5 km) are not favourable TBM system. It might be necessary either of excavate this tunnel stretch with NATM or use more complex double shield TBM system.
Niches, cavern, ventilation tunnel	TBM is not able to excavate Niches and caverns. High number of niches leads to significant additional effort. Conventional excavation equipment must be provided anyway.	
Construction logistics	Excavation and main supply is done from only one tunnel portal	Big volumes of construction materials and supply have to be provided and hold available. High transportation effort of main TBM parts only with special trucks.
Construction time	Construction time will be shorter due to all year construction from western portal. Reduction of approx. 3 months based on very preliminary construction time estimation of TBM excavation. The mean advance rate is higher.	Any problem of the TBM directly results in overall project delays Only one tunnel drift from western portal. Long lead time must be considered.
Construction costs	Relatively low wage cost during excavation.	Very high investment costs (two different TBM are required for main and egress tunnel). Blockage of TBM due to squeezing or ravelling ground will lead to high additional costs.

Based on the above given comparison the excavation by conventional drill and blast methods is deemed to be preferably for Zojila Tunnel construction.

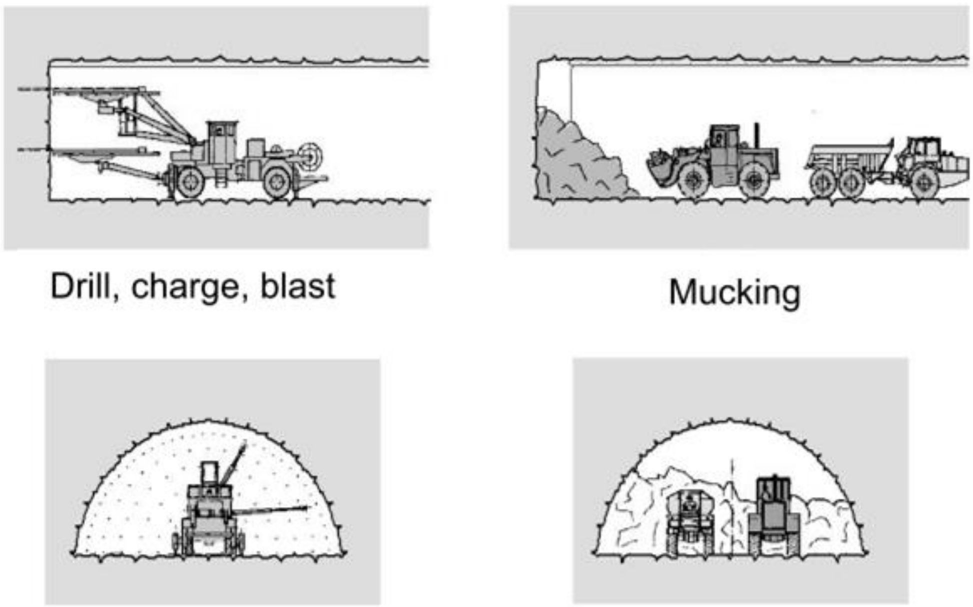
4.3 NATM – Excavation Method

The Zojila tunnel is designed to be constructed with conventional excavation in accordance with the principles of NATM based on the results of the comparison presented in DPR Volume II: Tunnel Design Report, Addendum 2. The excavation will be carried out by drill and blast or tunnel excavator with a subdivision of the tunnel cross section into top heading, bench and if required with invert. To increase the face stability the tunnel face excavation will be subdivided according to the geotechnical situation.

Each excavation sequence can be sub-divided in the following main construction steps:

- Excavation (either drilling, loading and blasting or excavation with excavator)
- Ventilation
- Scaling and spoil removal
- Installation of primary support measures
- Monitoring

In the following figure the basic construction sequence of a typical NATM tunnel in hard rock is shown schematically for the excavation of top heading, bench and invert.



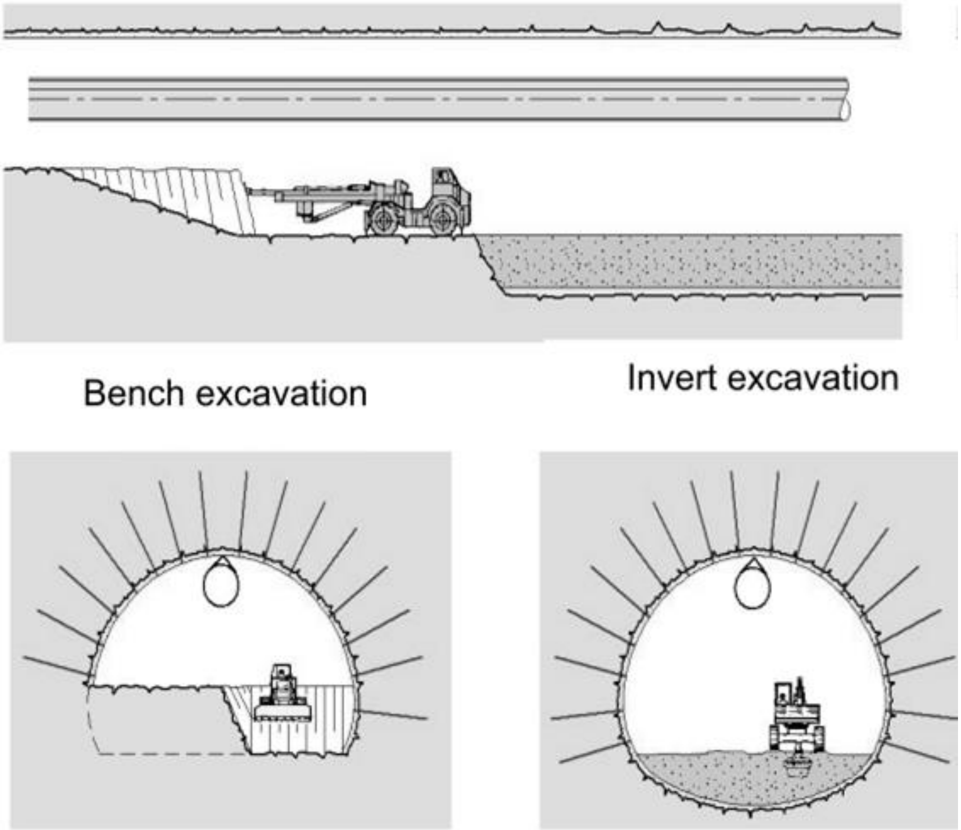


Fig. 5 Schematic construction sequence of a typical NATM tunnel in hard rock, from [L5]

In the Zojila Tunnel sections with brittle hard rock mass the excavation shall be done with drill and blast. Drill holes shall be drilled in the tunnel face with a length of approximately 1.5-4 m according to the round length. Then these drill holes will be loaded with explosives. The blasting will create a controlled excavation of the rock with a reasonable geometry according to the excavation and support classes. The blasting has to be designed in detail, including the number, pattern and distance of the drill holes as well as the loading with explosives per drill hole and the blasting sequence.

In the Zojila Tunnel sections with weak ground such as portal areas or in fault zones the ground can be excavated by a tunnel excavator. The careful excavation with a depth of 1-2 m according to the round length of the support category can be done for the entire excavation cross section (e.g. top heading) or in partial excavation areas in case of very loose ground and face instabilities.

In tunnel sections with mixed ground conditions such as big hard blocks in weak matrix material a combination of drill and blast and a tunnel excavator can be used. The blocks will be cracked by local blasting and excavated by the tunnel excavator together with the weak matrix material.

The mucking of the excavation material will be done by loaders and trucks from the tunnel face out of the tunnel. The muck material can either be stored temporarily or transported directly to the final disposal area.

4.4 Excavation Cross Section

The excavation will be separated into partial excavation areas to reduce the excavation height (for use of typical machinery) and to increase the stability of the excavation face. A change to full face excavation is not planned (even in case of very good ground conditions in some tunnel section), because significantly bigger machinery would be required in case of full face excavation.

The typical cross section is separated into a top heading with a height of up to approximately 6 m, a bench excavation with a height of up to approximately 4 m and, if required, an invert excavation with a height of approximately 2,5 m. Geometrical details are given in the drawings 8482B_II-ZOT_EXCA-01-12-00 to 8482B_II-ZOT_EXCA-16-12-00 for main and egress tunnel excavation respectively.

In tunnel sections with generally stable and hard rock conditions (Support Category SC A, B and C) the top heading and the bench is excavated independently with a minimum distance of 30 m. A maximum distance between top heading and bench excavation is not defined.

In case of potential long term instabilities of the tunnel (SC D) the top heading, the bench and the invert are excavated independently with a minimum distance of 30 m between top heading and bench and a minimum distance of 30 m between bench and invert. A maximum distance between top heading and bench, respectively between bench and invert is not defined.

In tunnel sections with generally unstable and weak conditions (SC E-H) the excavation is separated into a top heading with a temporary invert, a bench and an invert. The temporary invert with a height of approximately 3 m must be constructed within a distance of 2-6 round lengths behind the top heading face. The bench with a height of approximately 4 m is excavated independent to the top heading with a minimum distance of 30 m. The invert with a height of approximately 2.5 m must be constructed within a distance of 2-6 round lengths behind the top heading face.

Geometrical details to support categories A-H are given in the drawings 8482B_II-ZOT_EXCA-01-12-00 to 8482B_II-ZOT_EXCA-09-12-00 for main tunnel and support categories A-F in drawings 8482B_II-ZOT_EXCA-10-12-00 to 8482B_II-ZOT_EXCA-16-12-00 for egress tunnel excavation.

4.5 Excavation Steps

4.5.1 Drilling & Blasting

Currently, computerised hydraulic drilling jumbos with different level of automation are widely used in drill and blast tunnelling. The current generation of drilling jumbos is designed for high productivity, quality drilling, and comfortable working conditions for the operators. Fig. 6 gives an overview of the advance rates of different drilling equipment developed over the last century.

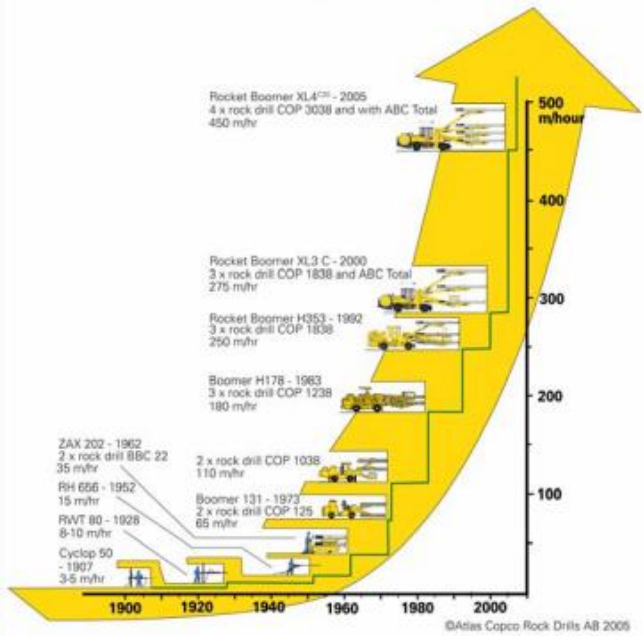


Fig. 6: Advance rate development within the last century (Atlas Copco 2006)

Currently drilling jumbos with two or three hydraulic drilling rods and a working platform are used. The operating area of one drilling rod and of the drilling jumbo is shown exemplarily in Fig. 7.

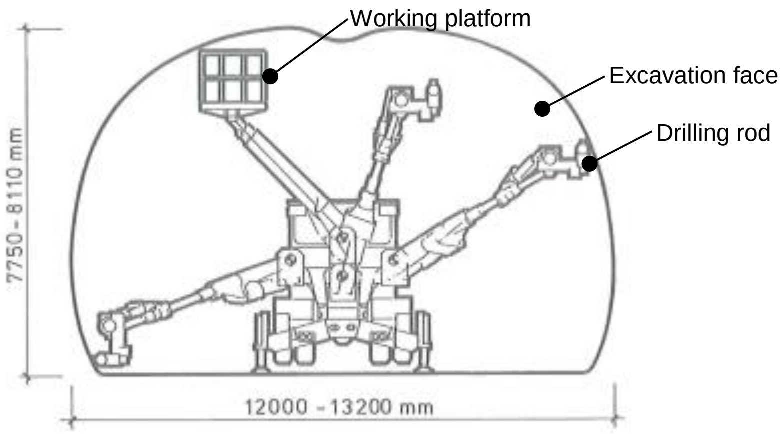


Fig. 7: Operating area of the drilling jumbo with three drilling rods and a working platform [L6]

Drilling is conducted with rotary hydraulic drilling rods. Usually the drilling head is flushed with water to secure the removal of the loosened material. The flushing water can exceed through the annular space of the drilling rod. The diameters of the drillings usually differ between 36 and 42 mm. In some cases diameters up to 45 mm are used. No use of water is permitted if squeezing ground conditions are predicted.

The blasting holes are drilled with a specific pattern to provide an excavation with a minimal over- and undercut and to secure a minimum damage of the rock in the vicinity of the excavation (smooth perimeter blasting). An exemplary blasting pattern is shown in Fig. 8.

The drilling pattern is governed by the ground conditions and the explosives used. The pattern shall be adjusted for each next round length based on the previous blasting results and experience gathered.

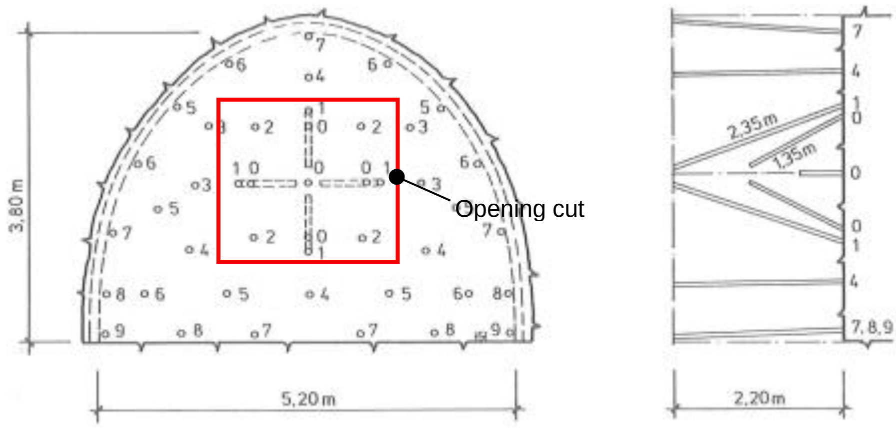


Fig. 8: Blasting pattern with V – Cut (opening cut red marked zone) [L6]

The so called opening cut is blasted first and provides enough space for rock loosening of the subsequently blasted holes. Different opening cuts may be applied in drill & blast tunnelling construction. The most commonly used opening cut in tunnelling is the parallel hole cut, or large hole cut. All holes in the large hole cut are drilled parallel to each other, and the blasting is carried out towards one or more empty large drill holes, which act as an opening. In the opening cut one or more holes with a big diameter are not charged and the surrounding blasting holes are fully charged (Fig. 9). The opening cut may be placed at any location of the excavation face. For less throw and explosive consumption the opening cuts may be placed in the center of the excavation face. There may be the requirement to place the opening cut in a different location (e.g. placement in relative undisturbed rock). If more empty blasting holes are conducted an fictitious equivalent diameter D can be estimated with the hole diameter d and the number of uncharged holes which are used (Equ. 1).

$$D = d \cdot \sqrt{n} \qquad \text{Equ. 1}$$

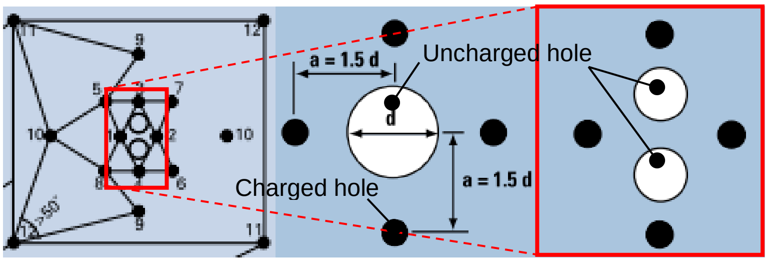
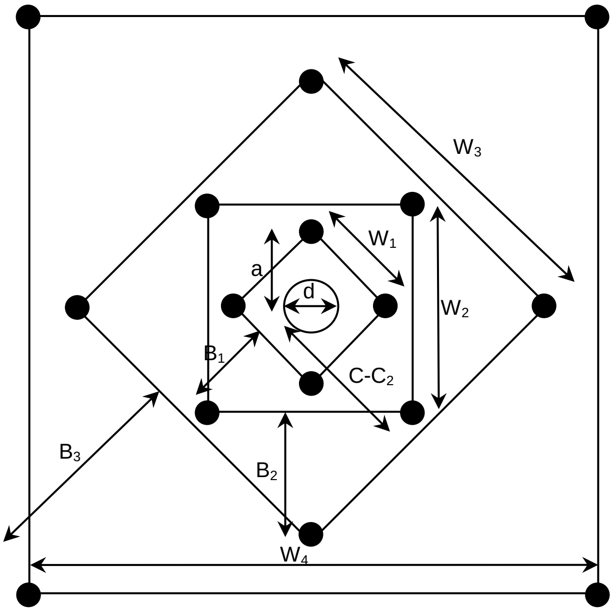


Fig. 9: Opening cut General Layout

The opening cuts can be designed with one large hole diameters as follows (Fig. 10 and Tab. 7).



a = Distance blasthole-large hole

W_i = Distance of blastholes in square i

B_i = Burden of square I

$C\text{-}C_i$ = Distance of blasthole in square i to large hole

Fig. 10: Opening cut detailed drilling pattern for one large hole

$$W_1 = a \cdot \sqrt{2} \qquad \text{Equ. 2}$$

$$W_1 = B_1 \qquad \text{Equ. 3}$$

$$C - C_i = 1{,}5 \cdot B_{i-1} \qquad \text{Equ. 4}$$

$$W_{i+1} = 1{,}5 \cdot \sqrt{2} \cdot W_i \qquad \text{Equ. 5}$$

Tab. 7: Distance of open cut drillings pattern with respect to large hole diameter d [L7]

d [mm]	76	89	102	127	154
a [mm]	110	130	150	190	230
B_1=W_1 [mm]	150	180	210	270	320
C-C_2	225	270	310	400	480
B_2=W_2 [mm]	320	380	440	560	670
C-C_3	480	570	660	480	1000
B_3=W_3 [mm]	670	800	930	1180	1400
C-C_4	1000	1200	1400	1750	
B_4=W_4 [mm]	1400	1700	1980	2400	

The described distances are derived for a blasthole diameter of 38 mm. If larger blastholes are applied, the distances may be adjusted.

The drill holes are charged with explosive. Currently the most commonly used explosive are:

- Explosive gelantines: Mainly used as booster or for opening the cut slot
- ANFO (Amonnium Nitrate Fuel Oil): Cheap and widespread but not water resistant
- Emulsion (slurries): Pumpable, water resistant, widespread

The different blasting holes are not charged with the equal amount of explosives. The opening cut (red marked zone in

Fig. 11) is charged with the highest amount of explosive and the contour holes parallel to the roof and side wall are charged with the lowest amount of explosives to provide an excavation with minimal rock damage outside of the excavation line (smooth perimeter blasting).

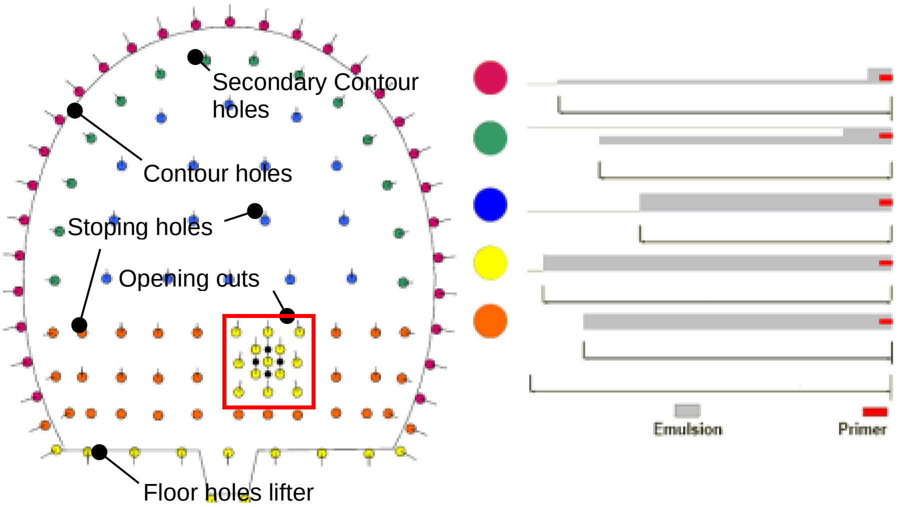


Fig. 11: Qualitatively distribution of charging of the blasting holes

Opening Cut: The opening cut can be charged according to

Fig. 12, where the maximum and minimum charge concentrations for the first and remaining squares are given, is depending on the C-C distances.

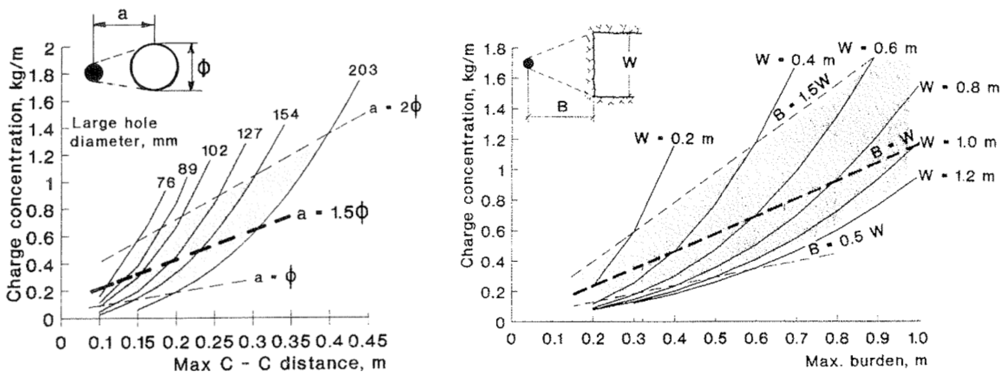


Fig. 12: Minimum and maximum charge concentration [kg/m³] and max. C-C distance for a) first square and b) remaining squares of opening cut [L7]

Stoping Blastholes: The burdens and charges of the stoping blastholes can be determined according Fig. 13 with respect to the blasthole diameter and the explosive.

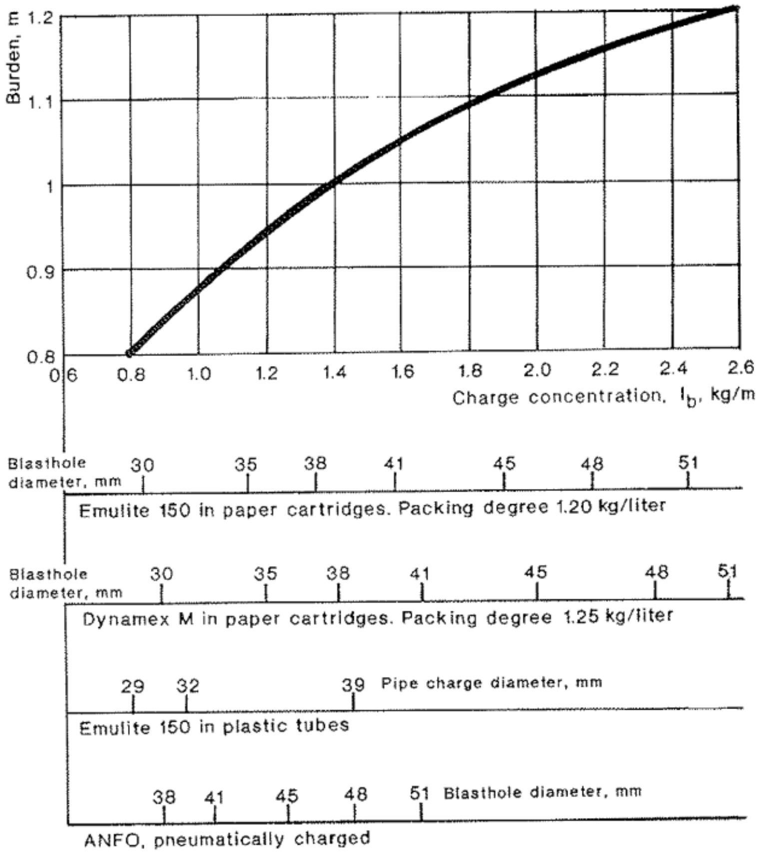


Fig. 13: Charge concentration [kg/m³] and burden B for stoping blastholes with respect to the blasthole diameter and the explosive [L7]

Contour Blastholes: The contour holes are divided into floor holes, roof and wall holes (primary and secondary). The floor holes are more charged as the roof and wall holes to compensate for the dead weight of the rock mass. The charging and pattern can be determined with Fig. 13. It can be differentiated between normal profile blasting and smooth profile blasting. The smooth profile blasting has been developed to obtain a smaller weakness zone of the surrounding rock mass and a smoother excavation profile. Smooth blasting is carried out with closely spaced blastholes and special explosives. Recommended drilling patterns and charging concentration are given in Tab. 8.

Tab. 8: Charging of roof and wall holes for smooth profile blasting [L7]

Blasting hole diameter D [mm]	Charge concentration [kg/m]	Explosive Type	Burden [mm]	Spacing [mm]
25-32	0,11	11 mm Gurit	300-500	250-350
25-48	0,23	17 mm Gurit	700-900	500-700
51-64	0,42	22 mm Gurit	1000-1100	800-900
51-64	0,45	22 mm Emulite	1100-1200	800-900

The blasting of each blasting area is conducted temporal subsequently hence the opening cut requires enough time to loosen the material so the force of the subsequent blasting is directed into the provided surface of the opening cut detonation. Therefore it is important in tunnel blasting to have sufficient time delay between the holes. In the opening cut area, the delay between the holes must be long enough to allow time for breakage, and throw of rock through the narrow empty hole, which takes place at a velocity of 40 to 60 m/sec. Normally, delay times of 75 to 100 milliseconds are used in the cut. In the stoping area (Fig. 11) the delay time shall be as high to allow movement of the rock to generate place for the adjacent rock to be loosened (normally between 100 and 500 msec). Electric and non electric detonators are used to provide an as small as possible scatter in the delay time (Fig. 14).

Properties MSDS #1076

Shell Material	Aluminum Copper also available (custom product)
Shell Length (range)	63.5 - 94 mm 2.5 - 3.7 in
Legwire Material	Copper wire (single)
Legwire Color	Red & Yellow
Maximum Water Depth	76 m (250 ft)
Shelf Life Maximum	3 years (from date of production)
Maximum Usage Temperature	+ 66°C (150°F)
Net Explosive Content per 100 units	0.0885 kg 0.1947 lb

Delay Period	Nominal Firing Time (msec)	Delay Period	Nominal Firing Time (msec)	Delay Period	Nominal Firing Time (msec)
1	25	7	1200	13	2900
2	200	8	1400	14	3300
3	400	9	1600	15	3800
4	600	10	1900	16	4400
5	800	11	2200	17	5100
6	1000	12	2500		

a)

Properties MSDS #1122

Net Explosive Content per 100 units 0.0885 kg 0.1947 lbs

Delay Time (msec)	Delay Tag Color	Delay Time (msec)	Delay Tag Color	Delay Time (msec)	Delay Tag Color
0	Orange	225	White	450	Orange
25	Red	250	Red	475	White
50	Tan	275	White	500	Purple
75	Green	300	Tan	525	White
100	Blue	325	White	600	Gray
125	Orange	350	Green	700	Lt Blue
150	Purple	375	White	800	Red
175	Gray	400	Blue	900	Tan
200	Lt Blue	425	White	1000	Green

b)

Fig. 14: a) Electrical and b) non electrical detonators (NONEL) as per Technical Specifications from Dyno Nobel Inc.

A schematic layout of a detonator is given in Fig. 14.

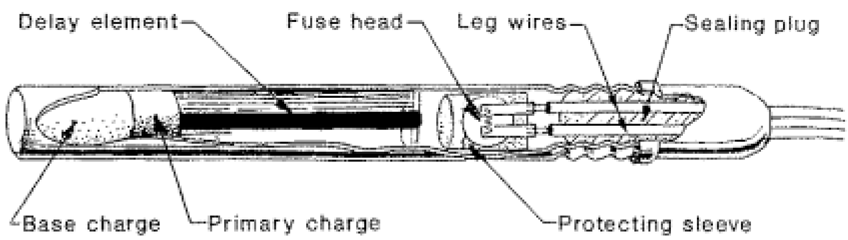


Fig. 15: Layout of detonators [L7]

4.5.2 Temporary Ventilation

Subsequently to blasting ventilation of the excavation face area is required due to firing fumes, gases and dust. High temperatures and the concentration of harmful gasses (e.g. Carbon Monoxide CO, Carbon Dioxide CO_2, Nitrogene Dioxide NO_x etc.) shall be decreased to specified concentrations as per National or International

Standards, Guidelines and the Technical Specifications of the Detailed Project Report.

Pits, shaft tunnels and headings shall at be kept ventilated all times to maintain an atmosphere fit for respiration and free from oxygen deficiency, potentially explosive or noxious gases and dust, whether present naturally or otherwise. Ventilation shall also be used to maintain a safe working temperature.

The permanent air supply shall not be less than two cubic meters per minute for each employee working under surface and four cubic meters for each kW power for all diesel units operating underground. Additionally a linear velocity of 0,3 m/s, provided for the largest cross section is required.

If natural deposits of harmful gases (e.g. methane CH_4, hydrogen sulfide H_2S, carbon dioxide CO_2 etc.) are found additionally effort for ventilation during construction is required.

The ventilation during construction will be done with simple jet fans installed outside the tunnel in the portal area. Through the pipe (installed in the upper part of the top heading) fresh air is pressed into the tunnel to the working face. One pipe with a diameter of approx. 2 m will be sufficient for the maximum excavation length of 2 km for one tunnel drift.

4.5.3 Soil Disposal

Subsequently to ventilation the loading and mucking procedure can be started. The mucking procedure can be divided into the loading, transport and disposal.

4.5.3.1 Loading

The loading of the loosened material can be conducted with excavators or loaders. Both types can be adopted as wheeler or as crawler excavator (Fig. 16).

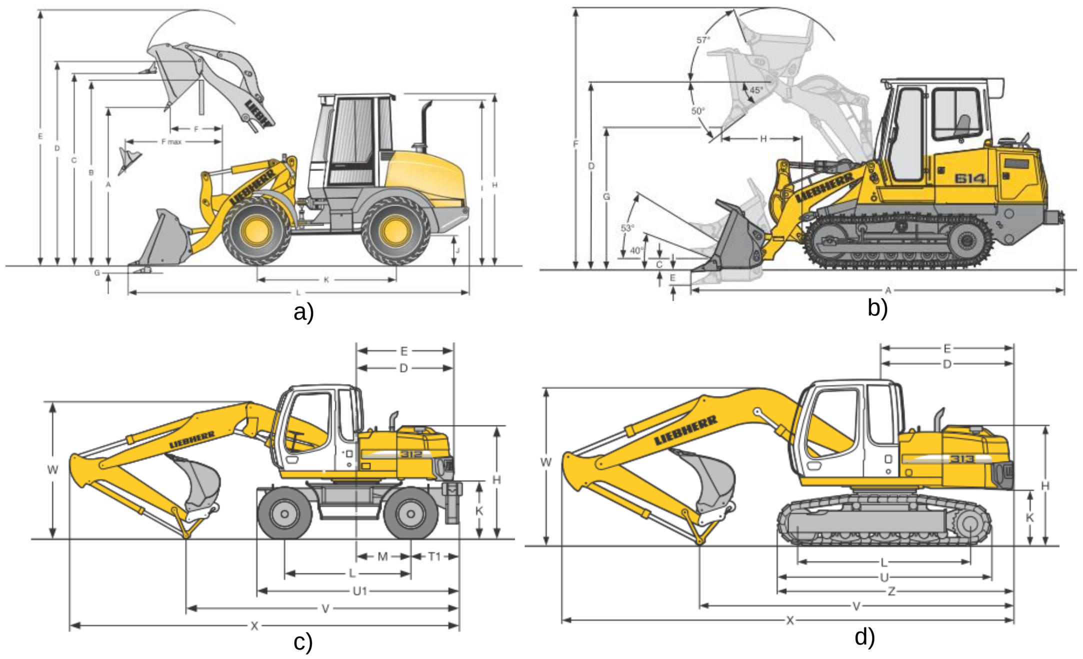


Fig. 16: a) Wheeled loader, b) crawler loader, c) wheeled excavator and d) crawler excavator from Technical Specifications of Liebherr International Germany GmbH

Crawler excavator and loaders compared to wheeled types have following advantages:

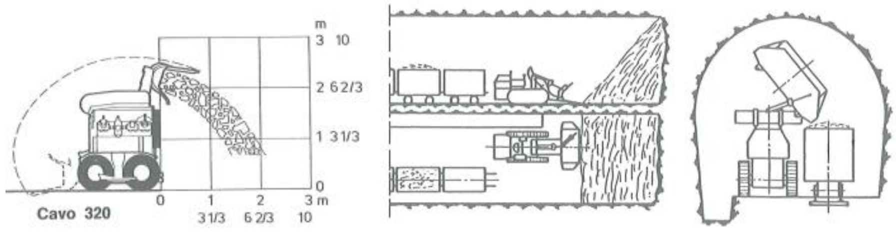
- low pressure on ground due to higher contact area
- high stability
- high inclination drivable, especially required for ramps providing accessibility of different faces when excavation sequences are applied (top heading, bench and invert)

In general wheeled excavator and loaders are applied if the ground bearing capacity is high enough to withstand the pressure, due to the fact that the driving velocity is higher.

Special crawler excavators have been developed for tunnel construction with a higher operating range and mobility of the boom.

Wheeled loaders are preferred for spoil loading in rock excavations, where drill and blast methods are applied. Due to the higher driving velocity of loaders, they can also be used for small transport ranges (e.g. when a temporary disposal area behind the face is proposed or the loosened rock mass is required elsewhere in the tunnel).

In excavation faces where less working space is available loaders are developed with special shovels for spoil loading. These developments allow higher mobility of the shovels (e.g. traversing shovels, side-dump truck, overhead loader etc.) hence the driving distance to the transporter can be minimized. A summary of the different types of loaders with its shovel movement is summarized in Fig. 17 and Fig. 18.



a) b)

Fig. 17: a) overhead loader and b) side-dump loader [L6]

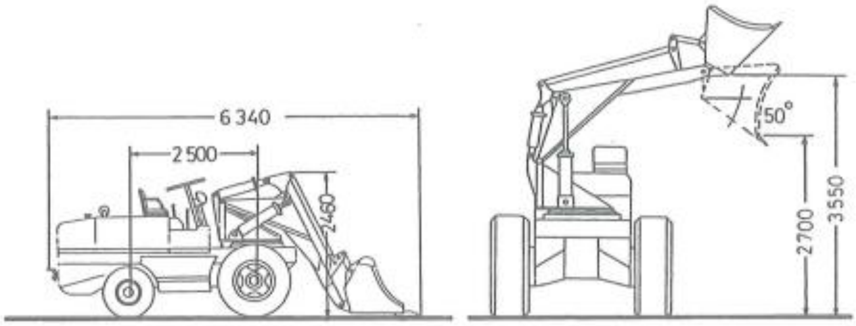


Fig. 18: Loader with a traversing shovel [L6]

The proposed excavation cross section of the Zojila Tunnel excavation provides enough space for wheeled loaders and does not require the application of the above mentioned special shovels.

4.5.3.2 Transport

The transport of the material can be categorized into the following main options:

- Transport without rails
- Transport on rails
- Special transportation logistics (e.g. band-conveyor)

Tab. 9 provides a comparison of advantages between the transportation of spoil material on a railway track and without it.

Tab. 9: Comparison between transport on rails and without rails

Advantages of transport without rails	Advantages of transport on rails
high mobility of transport equipment easy applicable on other construction sites higher transport capacity for wide diameter excavation higher inclinations are drivable	smaller ventilation during construction required low maintenance and operating costs low personal costs higher transport capacity for small diameter excavation

Trucks, dumper and small transport vehicles can be used for the transport of spoil. Small transport vehicles are only applied in small cross section tunnels with short length. For other long small cross sections transport on rails are applied due to the higher transport capacity (see Tab. 9).

Special transport logistics can be applied, for example band-conveyor, which are constructed on the top of the tunnel excavation. These applications can also be used in combination with transport options with or without rails.

For tunnel excavation in Zojila Tunnel only the conventional transport method on trucks is proposed due to the high mobility and flexibility of this option.

4.5.3.3 Disposal

Disposal sites are required for the excavated material. It can be differed between temporary and permanent disposal sites. Excavation material which can be recycled and reused in the construction process can be stored in temporary disposal sites. In contrast excavation materials which cannot be reused have to be disposed.

Slope debris material and poor quality rock material may only be used for unloaded/uncharged landfills. Good quality rock materials can probably be conditioned for other purposes. The use as backfill material, material for road dams as well as material for filter and drainage layers is most probable. Sufficient high quality of material for the use in sub-base layers or as mineral aggregates for concrete shall be secured by further investigations.

For final material disposal convenient locations have to be found with respect to environmental aspects. Preferably the disposal areas shall be located close to the site installations and in areas where sufficient space is available.

The excavation material is determined with respect to the location of its occurrence. It is differentiated between the excavation at the eastern and western portal and the drifts at the intermediate construction/ventilation shafts.

The volume of the excavation material is determined considering the Support Categories as defined in the Geological Evaluation Report, including the distribution of Support Categories over the tunnel length. The Support Categories have different excavation cross section (see Tab. 10.)

Tab. 10: Volume of excavation material Zojila Tunnel for different support categories and main construction elements

Tunnel	Support category and the corresponding excavation area [m²]							
	A	B	C	D	E	F	G	H
Main tunnel	116.2	117.50	119	140.7	143.6	145.8	145.8	153.3
Egress tunnel	39.7	40.5	41.4	51.4	52.7	61	-	-
Shaft 1 & 2	130.7	132.7	134.8	136.7	141	-	-	-
Shaft 3	28.3							

The mean excavation cross section for the ventilation cavern is determined to approx. 185 m² and a length of approx. 30 m.

The excavation material conveyed from each construction face is summarized in Tab. 11. A loosening coefficient of 1.3 is considered, meaning the volume of the blasted material is 1.3 times higher than the compact rock mass volume. An overcut and undercut respectively is not considered in the estimation of the spoil volume. A compaction factor on the disposal site is not considered in the determination of the disposal volume. A total amount of 3.4 million m³ of excavation material is estimated for the excavation of the Zojila Tunnel.

Tab. 11: Volume of excavation material Zojila Tunnel

Construction Face	Excavated Material [m³]
Portal East	699000
Shaft 1	730800
Shaft 2	982300
Shaft 3	15800
Portal West	956300
Sum	3384200

5 PRIMARY SUPPORT DESIGN

The Zojila tunnel is designed to be constructed with conventional excavation in accordance with the principles of NATM. The excavation will be carried out by drill and blast or tunnel excavator with a subdivision of the tunnel cross section into top heading, bench and if required into invert. To increase the face stability the tunnel face excavation shall be subdivided according to the actual geotechnical condition.

The tunnel support system consists of two generally independent lining systems:

- The primary (outer) lining consisting of shotcrete if necessary reinforced with wire mesh, lattice girder and rock bolts. All support measures are installed each round immediately after tunnel excavation. The primary lining is designed to provide immediate support and stability of the excavation until the inner lining is installed. The primary (outer) lining is designed in DPR Volume V: Primary Lining Design Report.
- The final (inner) lining, constructed of plain or reinforced concrete, is designed to sustain all internal and external forces without considering the bearing capacity of the primary lining. The final (inner) lining is designed in DPR Volume VI: Final Lining Design Report.

In the following figure the basic construction sequence of a typical NATM tunnel in hard rock is shown schematically.

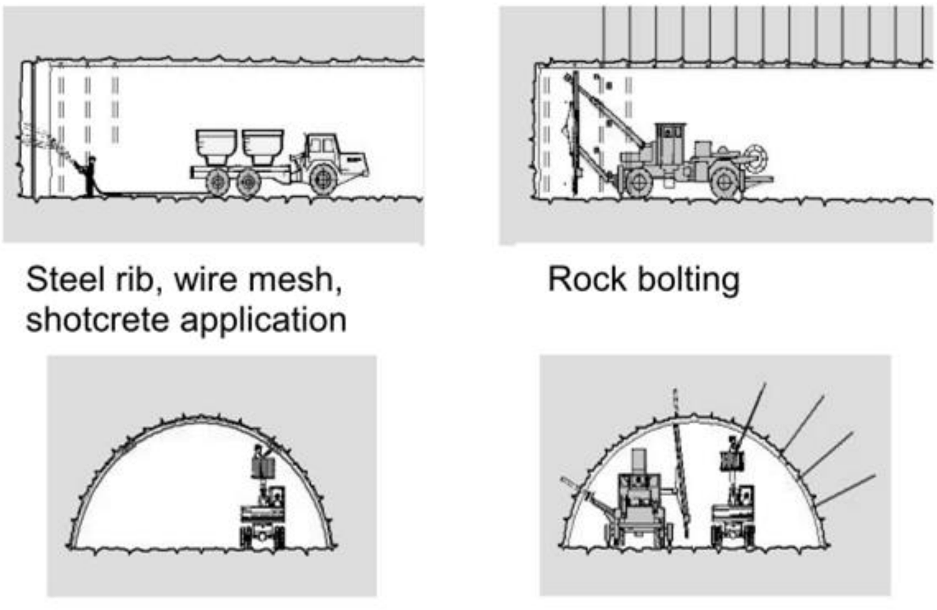


Fig. 19 Schematic construction sequence of a typical NATM tunnel in hard rock, from [L5]

In tunnel sections with squeezing ground conditions a deformable primary tunnel lining system is designed to allow controlled deformation of the tunnel lining to reduce the lining loads. The permanent concrete lining is constructed in sections of approximately 12.5 m and is either plain concrete or reinforced according to the structural requirements.

5.1 Primary Support Categories Main Tunnel

The preliminary excavation and support measures are categorised into eight Support Categories. The correlation between the Behaviour Types and the Support Categories is given in the following table.

Tab. 12 Basic 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

In the following a description of the support concept including the necessary support measures is given. This description also includes the Behaviour Types. The design of the support measures is mainly based on analytical and numerical analysis as well as experiences from comparable tunnel projects. The analysis of the primary lining is given in DPR Volume V: Primary Lining Design Report.

5.1.1 Support Category A

Designed for Behaviour Type 1 – Stable
For further details see drawing 8482B_II-ZOT_EXCA-01-12-00

<u>Excavation:</u>

The excavation is divided into top heading and bench excavation. The round length of the top heading can vary within a range from 3 to 4 m. The round length of the bench can vary within a range from 6 to 8 m. The distance between the top heading and the bench face shall be more than 30 m. The maximum distance between top heading and bench excavation is not limited.

<u>Primary lining:</u>

A 5 cm thick layer of steel fibre shotcrete and, if necessary, frictional bolts with immediate response, a capacity of min. 200 kN (Swellex or equivalent) and a length of 4 m provide the necessary support in the top heading and the bench. As a general estimation, 5 bolts are expected to be installed each round (in top heading and/or bench).

Additional measures:

Drainage drillings with a length of 12 m in case of water inflows shall be constructed.

5.1.2 Support Category B

Designed for Behaviour Type 2 – Discontinuity controlled overbreak
For further details see drawing 8482B_II-ZOT_EXCA-02-12-00

Excavation:

The excavation is divided into top heading and bench excavation. The round length of the top heading can vary within a range from 2 to 3 m. The round length of the bench can vary within a range from 4 to 6 m. The distance between the top heading and the bench shall be more than 30 m. The maximum distance between top heading and bench excavation is not limited.

Primary lining:

A 10 cm thick shotcrete layer with 1 layer of wire mesh and frictional bolts with immediate response, a capacity of min. 200 kN (Swellex or equivalent) and a length of 6 m at the crown and 4 m at the side walls provide the necessary support in the top heading and the bench. Every round 5/4 bolts with a length of 6 m and 2/4 bolts with a length of 4 m are installed in the top heading and 4/2 bolts with a length of 4 m in the bench.

Additional measures:

Drainage drillings with a length of 12 m in case of water inflows shall be constructed.

5.1.3 Support Category C

Designed for Behaviour Type 2 – Discontinuity controlled overbreak
For further details see drawing 8482B_II-ZOT_EXCA-03-12-00

Excavation:

The excavation is divided into top heading and bench excavation. The round length of the top heading can vary within a range from 1.5 to 2 m. The round length of the bench can vary within a range from 3 to 4 m. The distance between the top heading and the bench face shall be more than 30 m. The maximum distance between top heading and bench excavation is not limited.

Primary lining:

A 15 cm thick shotcrete layer with two layers of wire mesh, a lattice girder (e.g. LG 50/18/26) and grouted bolts with a capacity of min 300 kN and a length of 6 m provide the necessary support in the top heading and the bench. Every round, 9/10 bolts are installed in the top heading and 4/2 bolts in the bench.

Additional measures:

Drainage with a length of 12 m in case of water inflows drillings shall be constructed.

Forepoling with grouted steel bars with a diameter of 32 mm and a length of 6 m has to be installed to support the rock mass in the excavation roof, if necessary.

5.1.4 Support Category D

Designed for Behaviour Type 3 – Shallow stress induced failure
For further details see drawing 8482B_II-ZOT_EXCA-04-12-00

Excavation:

The excavation is divided into top heading, bench and invert excavation. The round length of the top heading can vary within a range from 1.25 to 1.75 m. The round length of the bench and the invert can vary within a range from 2.5 to 3.5 m. The distance between the top heading and the bench face as well as between the bench and the invert face shall be more than 30 m. The maximum distance between top heading and bench excavation and bench and invert excavation is not limited.

Primary lining:

A 20 cm thick shotcrete layer with two layers of wire mesh, a lattice girder (e.g. LG 70/26/34) and grouted bolts with a capacity of min. 300 kN and a length of 6/9 m provide the necessary support in the top heading and the bench. Every round, 7/6 bolts with a length of 6 m and 2/4 bolts with a length of 9 m are installed in the top heading and 4/2 bolts with a length of 9 m in the bench. The invert is supported with a 20 cm thick shotcrete layer with two layers of wire mesh.

Additional measures:

Drainage with a length of 12 m in case of water inflows drillings shall be constructed.

Forepoling with grouted steel bars with a diameter of 32 mm and a length of 6 m has to be installed to support the rock mass in the excavation roof, if necessary.

5.1.5 Support Category E

Designed for Behaviour Type 4 – Voluminous stress induced failure
For further details see drawing 8482B_II-ZOT_EXCA-05-12-00

Excavation:

The excavation is divided into top heading, temporary invert, bench and permanent invert excavation. The round length of the top heading can vary within a range from 1,0 to 1,5 m. The round length of the temporary invert, the bench and the permanent invert can vary within a range from 2,0 to 3,0 m. The distance between the top heading and the temporary invert face as well as the distance between the bench and the final invert face has a maximum of 6 round length of the top heading. The distance between the top heading and the bench face shall be more than 30 m. The maximum distance between the top heading and the bench excavation is not limited.

Primary lining:

A 25 cm thick shotcrete layer with two layers of wire mesh, a lattice girder (e.g. LG 95/26/34) and grouted bolts with a capacity of min. 350 kN and a length of 9 m provide the necessary support in the top heading and the bench. Every round, 13/12 bolts with a length of 9 m are installed in the top heading and 4/6 bolts with a length of 9 m in the bench. The temporary invert is supported with a 20 cm thick shotcrete layer with two layers of wire mesh, the permanent invert with a 25 cm thick shotcrete layer with two layers of wire mesh.

Additional measures:

Drainage with a length of 12 m in case of water inflows drillings shall be constructed.

Forepoling with grouted steel bars with a diameter of 32 mm and a length of 6 m has to be installed to support the rock mass in the excavation roof, if necessary.

5.1.6 Support Category F

Designed for Behaviour Type 4 – Voluminous stress induced failure in combination with displacements
For further details see drawing 8482B_II-ZOT_EXCA-06-12-00

Excavation:

The excavation is divided into top heading, temporary invert, bench and permanent invert excavation. The round length of the top heading is 1,0 to 1,5 m. The round length of the temporary invert, the bench and the permanent invert is 2,0 to 3,0 m. The distance between the top heading and the temporary invert face as well as the distance between the bench and the final invert face has a maximum of 6 round length of the top heading. The distance between the top heading and the bench face shall be more than 30 m. The maximum distance between the top heading and the bench excavation is not limited.

Primary lining:

A 30 cm thick shotcrete layer with two layers of wire mesh, a lattice girder (e.g. LG 95/26/34) and grouted bolts with a capacity of min. 350 kN and a length of 9 m provide the necessary support in the top heading and the bench. Every round, 13/10 bolts with a length of 9 m are installed in the top heading and 4/6 bolts with a length of 9 m in the bench. The temporary invert is supported with a 25 cm thick shotcrete layer with two layers of wire mesh, the permanent invert with a 30 cm thick shotcrete layer with two layers of wire mesh.

The primary lining of the top heading is divided by two horizontal gaps with a height of 50 cm each. These gaps allow large displacements of the tunnel lining. In the gaps yielding elements (lining stress controllers LSC or equivalent) shall be installed to control the deformational behaviour. Additional over-excavation provides space for the displacements.

Additional measures:

Drainage with a length of 12 m in case of water inflows drillings shall be constructed.

Forepoling with grouted steel bars with a diameter of 32 mm and a length of 6 m has to be installed to support the rock mass in the excavation roof, if necessary.

5.1.7 Support Category G

Designed for Behaviour Type 4 – Voluminous stress induced failure in combination with large displacements
For further details see drawing 8482B_II-ZOT_EXCA-07-12-00

Excavation:

The excavation is divided into top heading, temporary invert, bench and permanent invert excavation. The round length of the top heading is 1,0 to 1,5 m. The round length of the temporary invert, the bench and the permanent invert is 2,0 to 3,0 m. The distance between the top heading and the temporary invert face as well as the distance between the bench and the final invert face has a maximum of 6 round length of the top heading. The distance between the top heading and the bench face shall be more than 30 m. The maximum distance between the top heading and the bench excavation is not limited.

Primary lining:

A 30 cm thick shotcrete layer with two layers of wire mesh, a lattice girder (e.g. LG 95/26/34) and grouted bolts with a capacity of min. 350 kN and a length of 9 m provide the necessary support in the top heading and the bench. Every round, 13/8 bolts with a length of 9 m are installed in the top heading and 4/6 bolts with a length of 9 m in the bench. The temporary invert is supported with a 25 cm thick shotcrete layer with two layers of wire mesh, the permanent invert with a 30 cm thick shotcrete layer with two layers of wire mesh.

The primary lining of the top heading is divided by four horizontal gaps with a height of 50 cm each. These gaps allow large displacements of the tunnel lining. In the gaps yielding elements (lining stress controllers LSC or equivalent) shall be installed to control the deformational behaviour. Additional over-excavation provides space for the displacements.

Additional measures:

Drainage with a length of 12 m in case of water inflows drillings shall be constructed.

Forepoling with grouted steel bars with a diameter of 32 mm and a length of 6 m has to be installed to support the rock mass in the excavation roof, if necessary.

5.1.8 Support Category H

Designed for Behaviour Type 7 – roof failure, 8 – ravelling ground and BT 9 – flowing ground
For further details see drawings 8482B_II-ZOT_EXCA-08-12-00 and 8482B_II-ZOT_EXCA-09-12-00

Excavation:

The excavation is divided into top heading, temporary invert, bench and invert excavation. The round length of the top heading is 0,75-1,25 m. The round length of the temporary invert, the bench and the permanent invert is 1,5-2,5 m.

The distance between the top heading and the temporary invert face as well as between the bench and the final invert face has a maximum of 6 round length of the top heading. The distance between the top heading and the bench face shall be more than 30 m.

Primary lining:

A 30 cm thick shotcrete layer including two layers of wire mesh, a HEB 160 steel arch and self-drilling bolts with a capacity of min. 350 kN and a length of 9 m provide the necessary support. Every round, one steel arch in top heading and bench, 4/4 bolts with a length of 9 m in the top heading and 4/6 bolts with a length of 9 m in the bench are installed.

The footing of the shotcrete lining in the top heading is 75 cm thicker than the typical lining (elephant foot).

The temporary invert is supported with a 25 cm thick shotcrete layer with two layers of wire mesh, the permanent invert with a 30 cm thick shotcrete layer with two layers of wire mesh.

A pipe umbrella with 30 pieces of 114/101 mm steel pipes with a distance of 30 to 50 cm, a length of 12 m and an overlap of minimum 4,0 m is installed to improve roof and face stability of the excavation.

Additional measures if required:

Drainage drillings with a length of 12 m have to be installed in the tunnel face and in the tunnel lining in case of water inflows.

A face buttress, 5 cm shotcrete in the face, 10 cm shotcrete with 1 layer of wire mesh and/or 3-5 self-drilling bolts have to be installed to support the face, if necessary.

5.2 Primary Support Categories Egress Tunnel

The preliminary excavation and support measures are categorised into six Support Categories. The correlation between the Behaviour Types and the Support Categories is given in the following table.

Tab. 13 Basic 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 D and SC E
BT 7 Crown failure	SC F
BT 8 Ravelling ground	SC F
BT 9 Flowing ground	SC F

In the following a description of the support concept including the necessary support measures is given. This description also includes the Behaviour Types. The design of the support measures is mainly based on analytical and numerical analysis as well as experiences from comparable tunnel projects. The analysis of the primary lining is given in DPR Volume V: Primary Lining Design Report.

5.2.1 Support Category A

Designed for Behaviour Type 1 – Stable
For further details see drawing 8482B_II-ZOT_EXCA-10-12-00

Excavation:

The excavation is divided into top heading and bench excavation. The round length of the top heading can vary within a range from 3 to 4 m. The round length of the bench can vary within a range from 6 to 8 m. The distance between the top heading and the bench face shall be more than 30 m. The maximum distance between top heading and bench excavation is not limited.

Primary lining:

A 5 cm thick layer of steel fibre shotcrete and, if necessary, frictional bolts with immediate response, a capacity of min. 200 kN (Swellex or equivalent) and a length of 3 m provide the necessary support in the top heading and the bench. As a general estimation, 4 to 5 bolts are expected to be installed each round (in top heading and/or bench).

Additional measures:

Drainage drillings with a length of 12 m in case of water inflows shall be constructed.

5.2.2 Support Category B

Designed for Behaviour Type 2 – Discontinuity controlled overbreak
For further details see drawing 8482B_II-ZOT_EXCA-11-12-00

Excavation:

The excavation is divided into top heading and bench excavation. The round length of the top heading can vary within a range from 2 to 3 m. The round length of the bench can vary within a range from 4 to 6 m. The distance between the top heading and the bench shall be more than 30 m. The maximum distance between top heading and bench excavation is not limited.

Primary lining:

A 10 cm thick shotcrete layer with 1 layer of wire mesh and frictional bolts with immediate response, a capacity of min. 200 kN (Swellex or equivalent) and a length of 4 m provide the necessary support in the top heading and the bench. Every round 7/8 bolts are installed in the top heading.

Additional measures:

Drainage drillings with a length of 12 m in case of water inflows shall be constructed.

5.2.3 Support Category C

Designed for Behaviour Type 2 – Discontinuity controlled overbreak
For further details see drawing 8482B_II-ZOT_EXCA-12-12-00

Excavation:

The excavation is divided into top heading and bench excavation. The round length of the top heading can vary within a range from 1.5 to 2 m. The round length of the bench can vary within a range from 3 to 4 m. The distance between the top heading and the bench face shall be more than 30 m. The maximum distance between top heading and bench excavation is not limited.

Primary lining:

A 15 cm thick shotcrete layer with two layers of wire mesh, a lattice girder (e.g. LG 50/18/26) and grouted bolts with a capacity of min 300 kN and a length of 4 m provide the necessary support in the top heading and the bench. Every round, 7/8 bolts are installed in the top heading.

Additional measures:

Drainage with a length of 12 m in case of water inflows drillings shall be constructed.

Forepoling with grouted steel bars with a diameter of 28 mm and a length of 4 m has to be installed to support the rock mass in the excavation roof, if necessary.

5.2.4 Support Category D

Designed for Behaviour Type 3 – Shallow stress induced failure and deep stress induced failure
For further details see drawing 8482B_II-ZOT_EXCA-13-12-00

Excavation:

The excavation is divided into top heading and invert excavation. The round length of the top heading can vary within a range from 1.0 to 1.5 m. The round length of the bench and the invert can vary within a range from 2.0 to 3.0 m. The distance between the top heading and the invert face shall be more than 30 m. The maximum distance between top heading and invert excavation is not limited.

Primary lining:

A 20 cm thick shotcrete layer with two layers of wire mesh, a lattice girder (e.g. LG 70/26/34) and grouted bolts with a capacity of min. 300 kN and a length of 4 m in the crown and 6 m in the side wall provide the necessary support in the top heading and the invert. Every round, 5/4 bolts with a length of 4 m and 6/6 bolts with a length of 6 m are installed in the top heading. The invert is supported with a 20 cm thick shotcrete layer with two layers of wire mesh.

Additional measures:

Drainage with a length of 12 m in case of water inflows drillings shall be constructed.

Forepoling with grouted steel bars with a diameter of 28 mm and a length of 4 m has to be installed to support the rock mass in the excavation roof, if necessary.

5.2.5 Support Category E

Designed for Behaviour Type 4 – Voluminous stress induced failure in combination with displacements
For further details see drawing 8482B_II-ZOT_EXCA-14-12-00

Excavation:

The excavation is divided into top heading and invert excavation. The round length of the top heading can vary within a range from 1.0 to 1.5 m. The round length of the invert can vary within a range from 2.0 to 3.0 m. The distance between the top heading and the invert face shall be more than 30 m. The maximum distance between the top heading and the invert excavation is not limited.

Primary lining:

A 25 cm thick shotcrete layer with two layers of wire mesh, a lattice girder (e.g. LG 95/26/34) and grouted bolts with a capacity of min. 300 kN and a length of 6 m provide the necessary support in the top heading and the invert. Every round, 10/8 bolts are installed in the top heading. The invert is supported with a 25 cm thick shotcrete layer with two layers of wire mesh.

The primary lining of the top heading is divided by two horizontal gaps with a height of 50 cm each. These gaps allow large displacements of the tunnel lining. In the gaps yielding elements (lining stress controllers LSC or equivalent) shall be installed to control the deformational behaviour. Additional over-excavation provides space for the displacements.

Additional measures:

Drainage with a length of 12 m in case of water inflows drillings shall be constructed.

Forepoling with grouted steel bars with a diameter of 28 mm and a length of 4 m has to be installed to support the rock mass in the excavation roof, if necessary.

5.2.6 Support Category F

Designed for Behaviour Type BT 7 – roof failure, BT 8 – ravelling ground and BT 9 – flowing ground
For further details see drawing 8482B_II-ZOT_EXCA-15-12-00 and 8482B_II-ZOT_EXCA-16-12-00

Excavation:

The excavation is divided into top heading and invert excavation. The round length of the top heading is 1.0 to 1.5 m. The round length of the temporary invert, the bench and the permanent invert is 2.0 to 3.0 m. The distance between the top heading and the invert face shall be more than 30 m. The maximum distance between the top heading and the invert excavation is not limited.

Primary lining:

A 25 cm thick shotcrete layer with two layers of wire mesh, a lattice girder (e.g. LG 95/26/34) and grouted bolts with a capacity of min. 300 kN and a length of 6 m provide the necessary support in the top heading and the invert. Every round, 6/6 bolts with a length of 6 m are installed in the top heading The invert is supported with a 25 cm thick shotcrete layer with two layers of wire mesh.

A pipe umbrella with 21 pieces of 114/101 mm steel pipes with a distance of 30 to 50 cm, a length of 12 m and an overlap of minimum 4.0 m is installed to improve roof and face stability of the excavation.

Additional measures:

Drainage with a length of 12 m in case of water inflows drillings shall be constructed.

A face buttress, 5 cm shotcrete in the face, 10 cm shotcrete with 1 layer of wire mesh and/or 3-5 self-drilling bolts have to be installed to support the face, if necessary.

6 SYSTEM BEHAVIOUR

The behaviour of the system ground-excavation-support shall be generally stable. Due to time dependency of stress redistribution in combination with construction process displacements will occur in the Support Categories dominated by stress induced failure modes of the rock mass. For a proper evaluation of the system behaviour during and after excavation and support installation the expected system behaviour is described in the following table. It is of essential importance to observe the system behaviour also in terms of 3D displacement monitoring and to compare with the predicted values of displacements. In case of deviation, especially in case of larger displacements than predicted counter measures have to be set immediately. Otherwise the displacements may exceed the values of over-excavation which would result in under-profile of the tunnel geometry.

In the following table the expected system behaviour of the main tunnel based on the analytical and numerical analyses are given for each support category. No voluminous overbreaks shall occur during excavation. Surface settlements are not limited.

Tab. 14 Expected radial displacements of the tunnel lining in main tunnel cross sections with respect to different support categories

	Expected total displacement [cm]	Expected displacement for top heading only [cm]
SC A	1	1
SC B	2	1
SC C	3	2
SC D	5	3
SC E	10	7
SC F	20	13
SC G	35	21
SC H	5	3

7 DISTRIBUTION OF GROUND TYPES (GT), BEHAVIOUR TYPES (BT) AND SUPPORT CATEGORIES

The following prediction of the distribution of Ground Types, Behaviour Types and Support Categories for the Zojila Tunnel is based on the geological-geotechnical investigation works as discussed in the geological and geotechnical evaluation in Chapter 2. The distributions are also given in the drawing 8482B_II-ZOT_GEN-04-12-00.

The mined Zojila Tunnel is divided into 6 sections, representing the main geological sections. The main geological sections are summarized as follows:

- Section 1: Slope Debris and Talus Material, GT 1
- Section 2: Metabasite (Panjal Trap), GT2 – GT4
- Section 3: Phyllites (Zojila Formation), GT4 – GT6
- Section 4: Metabasite (Panjal Trap), GT2 – GT4
- Section 5: Greywacke and Slate (Agglomerate Slate), GT2 – GT4
- Section 6: Metabasite (Panjal Trap), GT2 – GT4

The distribution is presented as a range and must be used as an estimation which will change with changes of ground conditions.

Tab. 15 Predicted distribution of Ground Types (GT), Behaviour Types (BT) and Support Categories (SC) for mined sections of Zojila tunnel

		Section					
		1	**2**	**3**	**4**	**5**	**6**
Length [m]		76	1640	5551	2217	1112	3487
Ground Types GT [%]	**GT 1**	100	0	0	0	0	0
	GT 2	0	65-85	0	65-85	0	65-85
	GT 3	0	15-30	0	15-30	0	15-30
	GT 4	0	0-5	0	0-5	0	0-5
	GT 5	0	0	30-50	0	50-70	0
	GT 6	0	0	40-60	0	25-45	0
	GT 7	0	0	5-15	0	0-10	0
Behaviour Types BT [%]	**BT 1**	0	30-45	0	30-45	0	30-45
	BT 2	0	50-60	2-10	50-70	10-20	50-70
	BT 3	0	5-10	25-45	0-5	45-60	0-5
	BT 4	0	0-5	50-70	0-2	25-40	0-2
	BT 7/8/9	100	0	0	0	0	0
Support Category Main Tunnel SC [%]	**SC A**	0	30-45	0	30-45	0	30-45
	SC B	0	30-45	0	30-45	0	30-45
	SC C	0	10-25	2-10	15-30	10-20	15-30
	SC D	0	2-8	25-45	0-5	45-60	0-5
	SC E	0	0-5	20-40	0-2	20-35	0-2
	SC F	0	0	15-25	0	0-10	0
	SC G	0	0	5-15	0	0	0
	SC H	100	0	0	0	0	0

Support Category Egress Tunnel SC [%]	**SC A**	0	30-45	0	30-45	0	30-45
	SC B	0	30-45	0	30-45	0	30-45
	SC C	0	15-25	2-10	15-30	10-20	15-30
	SC D	0	5-12	55-75	0-7	70-90	0-7
	SC E	0	0	25-35	0	0-10	0
	SC F	100	0	0	0	0	0

8 LINE OF EXCAVATION

Line of excavation means the line of excavation within, which no unexcavated ground material shall remain at any time. If due to additional displacements of the ground unexcavated material extend into the line of excavation, the Employer's Representative may order to excavate this material at no additional costs.

The line of excavation is calculated from the geometry of the inner lining by adding the thickness of the primary lining as required for each support category. Additionally space for displacements must be excavated if required for the Support Category and the Contractor has to excavate any space required for working tolerances for the primary and inner lining. The excavation quantities due to space for displacements and working tolerances are not included in the excavation volumes as given in DPR Volume VII: Cost Estimation.

In the following the geometrical conditions for the definition of line of excavation are given for the different Support Categories.

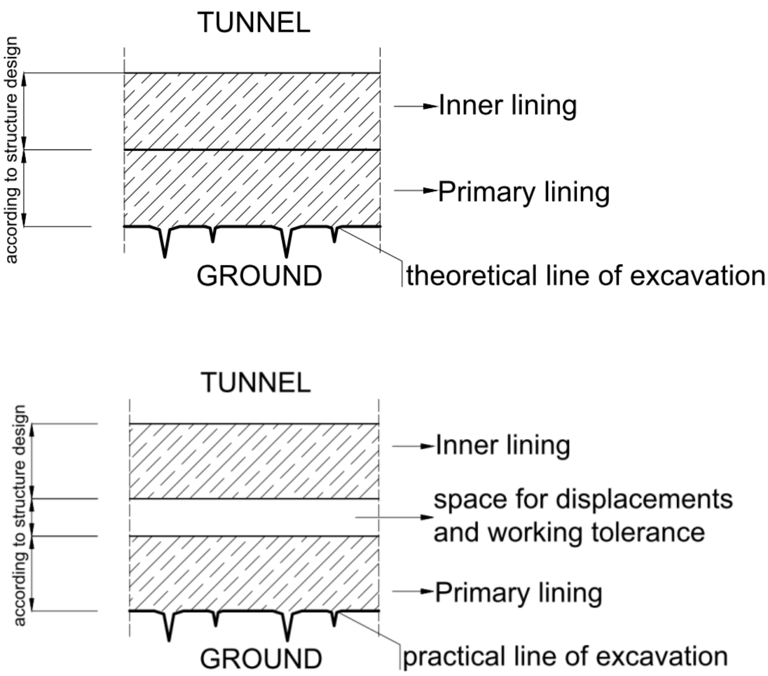


Fig. 20 Schematic sketch of the geometrical condition for the definition of the line of excavation

Tab. 16 Geometrical data of the different Support Categories for the definition of the line of excavation

	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

The working tolerances are estimated with 5 cm and shall be agreed between Contractor and Employer's Engineer.

9 MONITORING PROGRAM

The monitoring of the system behaviour is an important part of the application of the designed NATM support and shall cover

- displacements and stresses of the primary lining,
- forces of the anchors and
- development of the failure zone around the excavation.

Two different types of measuring sections shall be installed. In main measuring sections the displacements and the stresses of the shotcrete lining are measured as well as the forces in the anchors and the deformation of the rock mass (using extensometers). In subsidiary measuring sections the displacements and the stresses of the shotcrete lining are measured.

The distance between the measuring sections is defined in the following table.

Tab. 17 Distance between the measuring sections

Distance between measuring sections		
	Distance between main measuring sections	Distance between subsidiary measuring sections
Support Category A	-	25 m
Support Category B	100 m	25 m
Support Category C	60 m	15 m
Support Category D	35 m	10 m
Support Category E	25 m	7,5 m
Support Category F	25 m	7,5 m
Support Category G	25 m	7,5 m
Support Category H	20 m	4 m

Details concerning the installation of the monitoring equipment are given in drawings 8482B_II-ZOT_MON-01-12-00 to 8482B_II-ZOT_MON-03-12-00.

9.1.1 3D displacement measurement

The observation of the time dependent displacements of the tunnel lining is carried out by 3D measurements of reflection targets (geodetic points). The number of reflection targets in every measuring section depends on the Support Category and is given in the table below. The reflection targets have to be installed immediately after excavation. This means that the reflection targets have to be installed before shotcreting of the side walls and the roof of latest excavation step and have to be measured (zero-measurement) before the next excavation.

Tab. 18 Number of reflection targets in main and egress tunnel

Number of reflection targets			
Support Category	Main Tunnel		Egress Tunnel
	Top heading	Bench	Top Heading
SC A	3	2	3
SC B	3	2	3
SC C	3	2	3
SC D	5	4	3
SC E	5	4	3
SC F	5	4	3
SC G	5	4	n.a.
SC H	5	4	n.a.

The measuring bolt and the reflection targets have to be installed in a way not to be damaged during excavation and installation of support. The targets have to be removable without reducing the precision of the measurements. One system of blasting protection is given in the figure below; any equivalent system can be used.

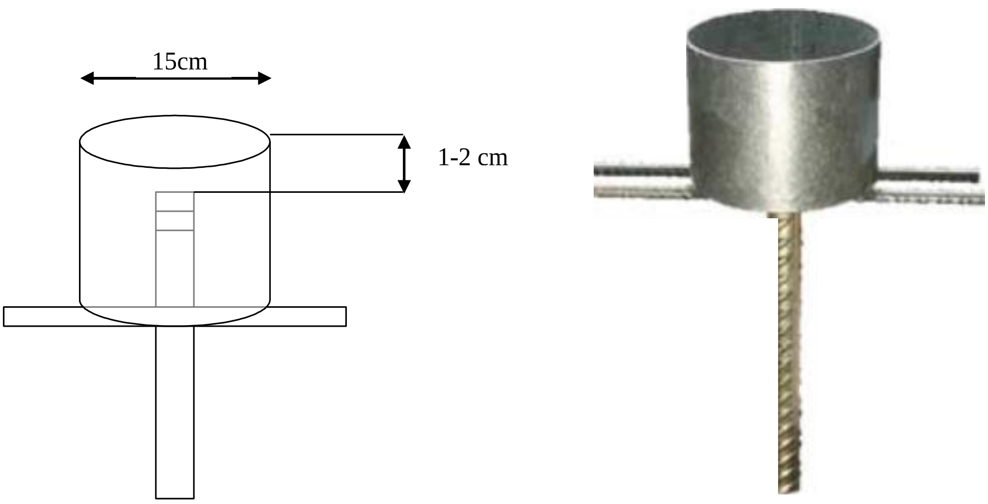


Fig. 21 Example for blasting protection

The 3D-measurements have to be taken on a daily basis. A detailed definition of the measurement frequency is given in Volume IX: Technical Specifications Civil Engineering.

In case of a stop of the tunnel advance over more than two days, 3 reflection targets on the tunnel face shall be installed additionally.

9.1.2 Extensometer

Extensometers are installed to gain detailed information on mechanisms in the rock mass by measuring the relative movements between anchor point and measuring head. The layout and lengths of the single extensometers should be adjusted to the anticipated mechanism. Generally three extensometers are installed in the top heading of all Support Categories and two more extensometers are installed in the bench of the Support Categories D, E, F, G and H in main tunnel. In egress tunnel two extensometers are installed in all Support Categories. The measuring distance of the extensometers has to be adapted to the predicted displacements in each support category.

The extensometers consist of three measuring bars with length of 3, 6 and 9 m.

The extensometers have to be installed directly after the excavation. A reflection target has to be placed and daily measured on the head of the extensometer.

9.1.3 Stress measurement in the shotcrete lining

Pressure pads are installed to measure the stress in the shotcrete. The filling fluid in the pressure pad is compressed and the pressure is indicated by either an installed manometer or by electronic sensors.

Cells are installed after excavating the tunnel but before applying the shotcrete. Radial cells are placed at the interface between the excavated ground surface and the shotcrete lining. Tangential cells are attached to the reinforcing cage or some other support so they will be embedded in the shotcrete lining. Once the cells are positioned, shotcreting takes place. After the shotcrete cures, tangential cells are pressurized by crimping the cell's pressurization tube, which forces oil from the tube into the cell therefore inflating the cell and providing firm contact with the surrounding shotcrete material. The quantity of filling fluid of the pressure pad and manometer is very small in order to keep temperature influences as small as possible. Influences caused by installation temperature are compensated by means of the repressurizing tube. Furthermore, an eventually arising shrinking gap in the concrete is overcome by this repressurizing tube. By expansion of the Bordon tube in the manometer and the thus arising loss in volume it is necessary to pre-stress the cell by the repressurizing tube.

The pressure cells are used for radial and tangential pressures in the shotcrete and joint pressure. The measuring range for the tangential stress is from 0 to 30 MPa, for the radial stress and joint pressure 0 to 5 MPa.

9.1.4 Anchor forces

Anchor load cells are installed to measure anchor forces. Anchor forces are distributed onto a tightly filled hydraulic cushion (pressure pad), which is sandwiched between load distribution plates at the anchor head. The pressure in this cushion is directly proportional to the anchor force.

The pressure is measured by a manometer and/or by an electrical pressure transducer. The manometer must be calibrated before installation to allow direct readings of the anchor force in kN.