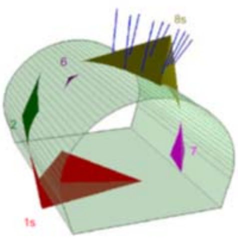
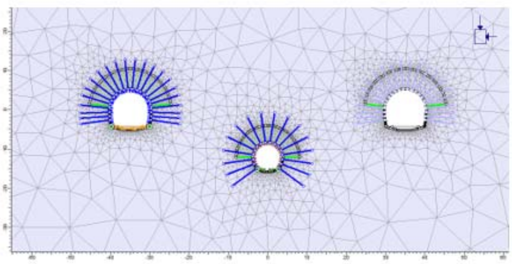
Brenner Base Tunnel, Italian Side: Mining methods stretches – Design procedures

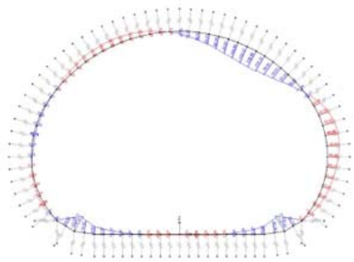
D. Marini & R. Zurlo
BBT SE, Bolzano, Italy

M. Rivoltini & E.M. Pizzarotti
Pro Iter, Milan, Italy

ABSTRACT: The Brenner Base Tunnel is mainly composed by two single track Railway Tunnels and a Service Tunnel; Lots Mules 1 and 2-3 involve a 22 km long stretch on the Italian side. They cross the South part of the mountainous dorsal between Austria and Italy, under overburdens up to 1850 m, consisting of rocks both of Southalpine and Australpine domains, separated by the major Periadriatic Fault. More than 30 km of tunnels must be carried out with Mining Methods, with average dimension ranging from 7 to 20 m and with a 200 years required service life. The paper describes the procedures implemented in the design: geological assessment; geomechanical characterization and selection of rock mass design parameters; analytic and numerical methods to forecast rock mass behaviour (varying from rock burst to squeezing); definition and sizing of rock enhancements, first phase and final linings, under a wide range of load conditions.







BBT Mules 2-3: Mining methods design procedure

D. Marini, R. Zurlo – M. Rivoltini, E.M. Pizzarotti

BBT SE, Bolzano, Italy – Pro Iter, Milan, Italy

1 GEOMECHANICAL CHARACTERIZATION

The initial analysis of the geological and geomechanical model allowed the definition of *Geomechanical Homogeneous Stretches* (GHS), that are a stretch with uniform lithology, overburden and rock mass quality.

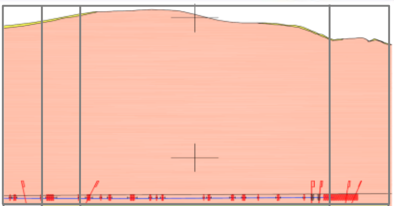
For each GHS, mean value and standard deviation of basic geomechanical parameters were assessed: γ σ_{ci} m_i E_i RMR (Bieniawski 1989), GSI (Hoek et al. 2002).

Characteristic (Design) Values of those parameter were defined based on statistical analysis:

- σ_{ci} m_i E_i : a conservative estimate of the representative value $X_K = X_M - \sigma_x / 2$
- RMR, GSI : minimum value of each Bieniawski class; class distribution defined by the area under the curve of the probability density

Starting from *Characteristic (Design) Values*, peak (Hoek et al. 2002, Hoek & Diederichs 2006) and post-peak (Cai et al. 2007) rock mass parameters were calculated.

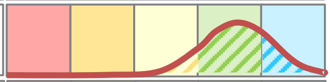
Mules Fault Line was treated with specific *GHSs;* instead, minor faults were characterized with conservative values of rock mass index parameters (GSI = 20 – 30 for the Core Zone and Damage Zone, respectively) and other parameters corresponding to the worst rock mass separated by the considered fault.



GHS	GB-G-GA-2	GB-G-GA-3	GB-G-GA-4
γ (*) [kN/m³]	26.7± 0.15	26.7 ± 0.15	26.7±0.15
σ_{ci} (*) [Mpa]	89±18	131 ± 27	90 ± 18
m_i (*)	22±4	24 ± 4	22 ± 4
E_i (*) [Gpa]	28±5	29 ± 5	28 ± 5
RMR (*)	52±14	72 ± 13	59 ± 8
GSI (*)	54±11	77 ± 14	57 ± 11

(*) MEAN VALUE ± STANDARD DEVIATION

CLASS	v	IV	III	II	I
GB-G-GA-3					
PROB.	0 %	0 %	17 %	55 %	25 %



2 FORECAST OF ROCK MASS BEHAVIOR

Empirical methods were applied to for a quick and qualitative evaluation of risks:

- Risk of squeezing / front face instability – Jehtwa et al. 1984; Bhasin 1994; Hoek & Marinos 2000; Panet 1995
- Risk of rock burst – Tao Z.Y., 1988; Hoek & Brown, 1980

Afterwards, Convergence – Confinement method was used to define rock mass class behavior during excavation, compared with stability criterions by Gamble 1971, Sakurai 1997.

Those methods, even if preliminary and often too conservative, were used for an initial definition of the geotechnical risk for each GHS and for each class.

ZONE	CLASS	EMPIRICAL METHODS						CONV. CONF.
		JETHWA	BHASIN	HOEK	PANET	TAO	HOEK	
GB-G-GA-3	I							
	II							
	III							

ZONE	CLASS	SQUEEZING	FACE STABILITY	ROCK BURST
GB-G-GA-3	I			
	II			
	III			

3 DEFINITION AND SIZING OF TYPICAL SECTIONS

From geotechnical risk matrix for each GHS, expected behaviors for each rock mass class are identified: elastic, slightly or strongly elastoplastic, squeezeng, rock burst.

From these behaviors, six typical sections were defined based on rock mass quality, overburden and lithology; support measures vary from radial bolting + shotcrete (section T2) to radial / ahead bolting and deformable lining with shotcrete and yielding steel ribs.

For the sizing of 1st stage lining two numerical methods were used:

- Only for elastic behavior, analysis of potentially unstable blocks
- FDM modeling in plane strain

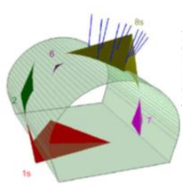
Instead, final linings were analyzed with specific FEM analyses

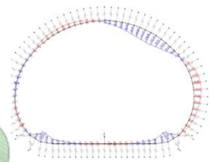
All SLS and ULS structural checks were performed in compliance with the requirements of EC 2, that is actions were amplified and combined using prescribed partial factors.

Because of remaining uncertainty, a monitoring system was planned with the scope of a more efficient application of the excavation sections during advancement.

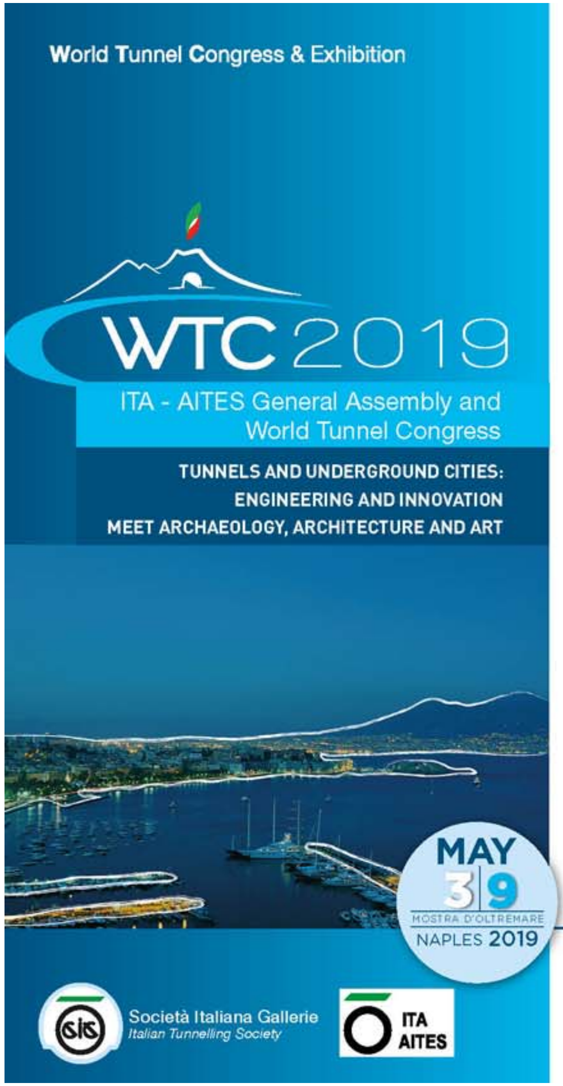
TYP. SECT.	SUPPORT MEASURES
2	RADIAL BOLTING + RIGID LINING (SHOTCRETE)
Rb (**)	RADIAL BOLTING + RIGID LINING (SHOTCRETE)
3	RADIAL BOLTING + RIGID LINING (SHOTCRETE)
4	POSSIBLE AHEAD BOLTING ON FACE/BOUNDARY + RIGID LINING (SHOTCRETE, STEEL RIBS)
5	AHEAD BOLTING ON FACE/BOUNDARY + RADIAL BOLTING + RIGID LINING (SHOTCRETE, STEEL RIBS)
6	AHEAD BOLTING ON FACE/BOUNDARY + RADIAL BOLTING + DEFORMABLE LINING (SHOTCRETE, YIELDING STEEL RIBS)

(**) Rb = SECTION FOR ROCK BURST BEHAVIOR






World Tunnel Congress & Exhibition
WTC 2019
MAY
3 9
MOSTRA D'OLTREMARE
NAPLES 2019
ITA - AITES General Assembly and World Tunnel Congress
TUNNELS AND UNDERGROUND CITIES: ENGINEERING
AND INNOVATIONMEET ARCHAEOLOGY, ARCHITECTURE AND ART
Società Italiana Gallerie
Italian Tunnelling Society
ITA
AITES



Long and deep tunnels

BBT, ITALIAN SIDE: MINING METHODS STRETCHES - DESIGN PROCEDURES

Authors: D. Marini, R. Zurlo
BBT SE, Bolzano, Italy

M. Rivoltini, E.M. Pizzarotti
Pro Iter, Milan, Italy

Presenting Author: E.M. Pizzarotti

Piazza Stazione 1
39100 Bolzano
info@bbt-se.com

PRO ITER Progetto Infrastrutture Territorio s.r.l.

Via Sammartini 5
20125 Milano
segreteria@proiter.it

Introduction – Why BBT?

The Brenner Base Tunnel (BBT) is the heart of the **Scandinavia-Mediterranean TEN Corridor** from Helsinki to La Valletta.

from:

Fortezza (Bz – Italy)

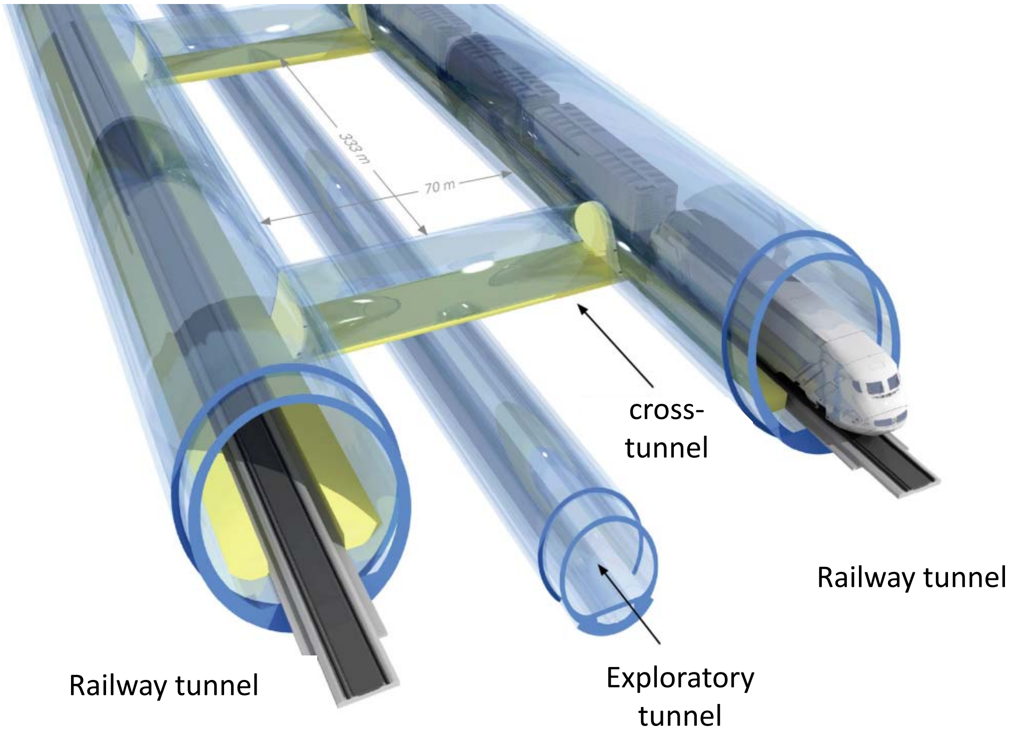
to:

Innsbruck (Austria)

Main goals:

- Freight transport: modal shift from road to rail
- Passenger transport: reduce travel time

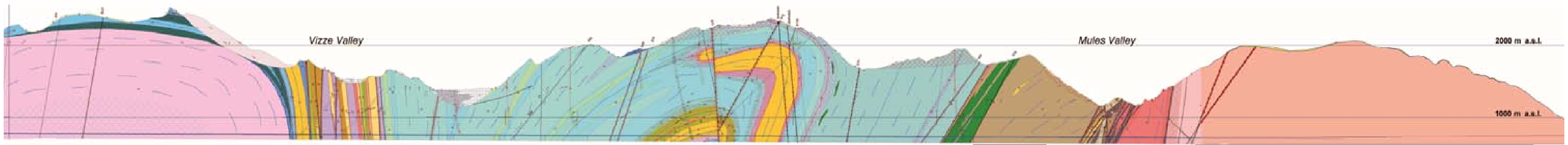
Introduction – The system



Main system components:

- 2 Railway Tunnels, generally single track, 8.8 m wide (internal), running 40-70 m apart from one another.
- 1 Exploratory Tunnel between the two main tunnels and about 12 m below them, 5.0 m wide, aiming to provide information on the rock mass, to drain rock mass water and working as service tunnel during BBT operation .
- Cross-tunnels every 333 m, used in emergencies as escape routes and accommodating plants facilities.
- 3 Emergency Stops (ES) for train halt in case of unforeseen events.
- 4 Access Tunnels (AT) to connect the system with the outside.

Introduction – Geology



Kernel of Tux Central Gneiss	Granite gneiss / granodiorite - diorite gneiss
Trias at the base of Vizze nappe	Metasedimentary mesozoic rocks (marble, quarzite, anhydrite and gypsum, schist and micaschist, phyllite)
Vizze nappe	Calcareous calc-schists, grey marbles and calcareous schists
Zillertal Antiform	Calcareous calc-schists, grey marbles and calcareous schists
Vizze nappe and Glockner nappe	Calc-schists (both calcareous and not), marbles
Austro alpine Crystalline Basement	Paragneiss and micaschist, amphibolite
Mules Fault Line	Schist and micaschist, phyllite, fault rocks
Mules Tonalitic lamellae	Tonalite
Brixen Granit	Granite and granodiorite

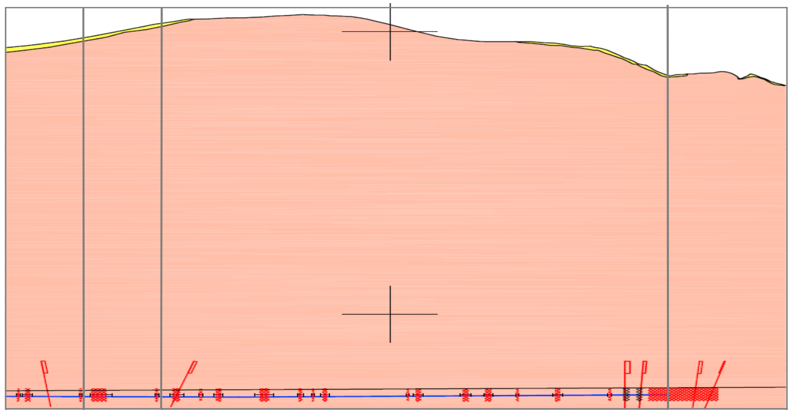
Geomechanical characterization

Analysis of geological and geomechanical model

- Critical examination of the documents developed during the previous phases of in-depth design study
- Experience acquired during the excavation of Lot Mules 1

Identification of Geomechanical Homogeneous Stretches (GHS)

- GHS = stretch with uniform lithology, overburden and rock mass quality
- For each GHS, definition of basic geomechanical parameters (mean value and standard deviation):
  - Intact rock: natural unit weight of the intact rock γ; uniaxial matrix compression strength σ_{ci}; parameter of failure envelope m_i; modulus of deformation of intact rock E_i
  - Rock mass: Rock Mass Rating (*RMR* – Bieniawski 1989); Geological Strength Index (*GSI* – Hoek et al. 2002)



GHS	GB-G-GA-2	GB-G-GA-3	GB-G-GA-4
γ (*) [kN/m³]	26.7 ± 0.15	26.7 ± 0.15	26.7 ± 0.15
σ_{ci} (*) [Mpa]	89±18	131 ± 27	90 ± 18
m_i (*)	22±4	24 ± 4	22 ± 4
E_i (*) [Gpa]	28±5	29 ± 5	28 ± 5
RMR (*)	52±14	72 ± 13	59 ± 8
GSI (*)	54±11	77 ± 14	57 ± 11

(*) Mean value ± standard deviation

Geomechanical characterization

Major Fault zones

- Pusteria Fault Line (30 m) and Mules Fault Line (760 m)
- Treated with specific GHSs, defined from the results of Exploratory and Railways Tunnels excavation in Lot Mules 1

Minor fault zones

- *GSI* = 30 for the Damage Zone (fractured rock mass)
- *GSI* = 20 for the Core Zone (fault gauge, cataclasite)
- Other parameters: corresponding to the worst rock mass separated by the considered fault

Front face of Exploratory Tunnel in Mules Fault Line (schist and micaschist, phyllite).

Geomechanical characterization

Peak parameters

- Hoek & Brown failure criterion (*Hoek et al. 2002*)

$$\sigma_1' = \sigma_3' + \sigma_{ci} \cdot \left[\left(m_b \cdot \frac{\sigma_3'}{\sigma_{ci}}\right) + s\right]^a$$

- c', φ' from the linearization of the criterion within the stress field of reference

$$\varphi' = \sin^{-1}\left[\frac{6am_b(s + m_b\,\sigma'_{3n})^{a-1}}{2(1+a)(2+a) + 6am_b\,(s + m_b\,\sigma'_{3n})^{a-1}}\right]$$

$$c' = \frac{\sigma_{ci}[(1+2a)s + (1-a)m_b\sigma'_{3n}](s + m_b\sigma'_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1 + (6am_b(s + m_b\sigma'_{3n})^{a-1})/((1+a)(2+a))}}$$

- Rock mass deformation modulus from *Hoek & Diederichs 2006*

$$E_m = E_i \cdot \left(0.02 + \frac{1 - D/2}{1 + e^{(60+15\cdot D - GSI)/11}}\right)$$

Post-peak parameters

- c', φ' : the same as per peak parameters, but with reduced $GSI = GSI_{res}$ (*Cai et al. 2007*)

$$GSI_{res} = GSI \cdot e^{-0.0134 \cdot GSI}$$

- Dilation angle: depending on the difference between friction angles in peak and post-peak conditions (*Rowe 1962*), divided by 1.5 instead of 2.0 to take into account phenomena of failure of rock joint asperity

$$\psi = \frac{\varphi'_{peak} - \varphi'_{post\ peak}}{1.5}$$

Geomechanical characterization

Design values of rock mass parameters

- NTC (Italian Technical Building Regulations) and Eurocode 7: in geotechnical design, **the characteristic values of geotechnical materials strength and deformability parameters** must be used, defined as a **reasoned and precautionary estimate of the parameter value in the considered limit condition**:
  - When a small volume of ground is involved and stresses can concentrate in weaker areas (e.g. shallow foundations) ⇒ Characteristic values are similar to minimum values
  - When a great volume of ground is affected by the construction and stresses can be distributed from weaker to stronger areas (as in tunneling) ⇒ Characteristic values are similar to mean values

- Intact rock parameters, used in Hoek & Brown failure criterion (γ, σ_{ci}, m_i, E_i): **characteristic value is a conservative estimate of the mean value**

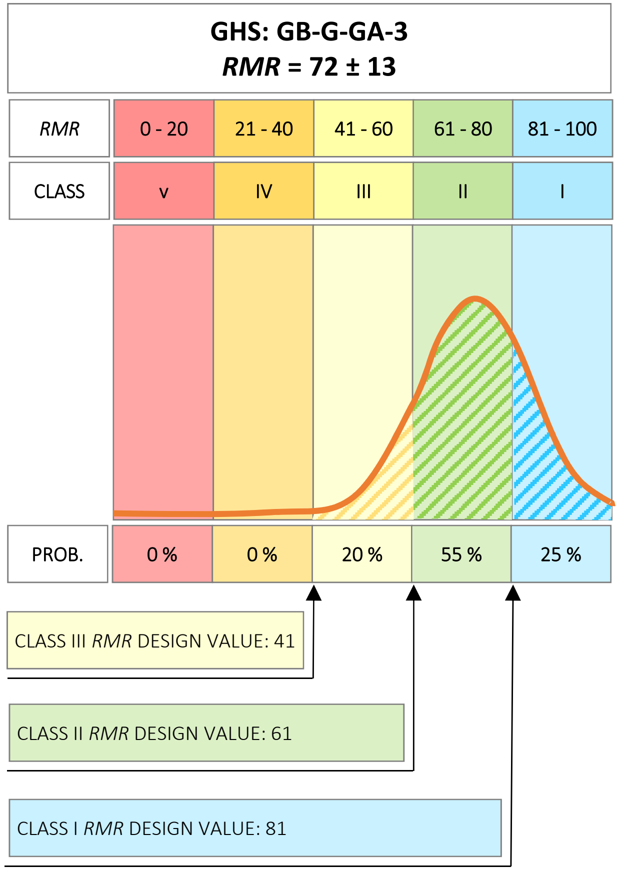
$$X_k = X_m - \sigma_X/2$$

X_k , X_m , σ_X = characteristic and mean values and standard deviation of generic parameter X

Geomechanical characterization

Design values of rock mass parameters

- *RMR* index : to size the different excavation sections and assign them an implementation rate, the distribution of quality classes in each GHS (according to Bieniawski 1989) was needed
- Within a single GHS, each class was given a probability equal to the area under the curve of the probability density in the corresponding *RMR* interval, regardless of the existence of a class when its probability was lower than 5%
- Once the actual existing classes were identified, the design value for each class was assigned; on the conservative side, the lower *RMR* value of each class was adopted as design value
- For GHS shorter than 100 m, a single *RMR* value (equal to mean value) was adopted in design



Geomechanical characterization

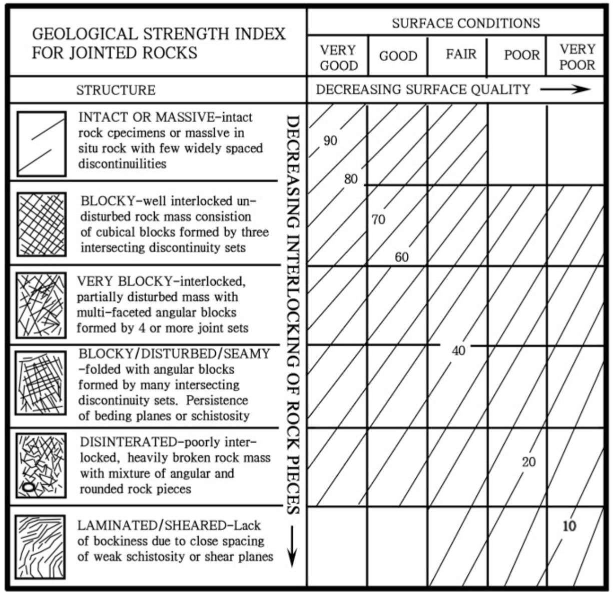
Design values of rock mass parameters

- *GSI* index : as reported in literature, there can be a linear link between *RMR* and *GSI*, net of hydraulic conditions and orientation of discontinuities (not considered in *GSI*)
- Assuming that within each GHS the hydraulic conditions and orientation of discontinuities are approximately constant, the difference between *RMR* and *GSI* in the stretch would also be constant

  This difference was assumed equal to the difference between the average values of the two indexes
- It was therefore decided to use as design *GSI* value for each class of each GHS the value:

$$GSI_{design} = RMR_{design} - (RMR_{mean} - GSI_{mean})$$

- For GHS shorter than 100 m, a single *GSI* value (equal to mean value) was adopted in design



Rock mass behavior forecast

Empirical methods – Risk of squeezing / front face instability

- Scope: quick and qualitative evaluation of squeezing risk
- Conservatively, the maximum overburden in each GHS was considered

Jehtwa and Singh 1984:

$$N_c = \frac{\sigma_{cm}}{P_0}$$

σ_{cm} = rock mass compressive strength
P_0 = lithostatic pressure

Bhasin 1994:

$$N_t = \frac{2P_0}{\sigma_{cm}}$$

Hoek & Marinos 2000:

$$\varepsilon = 0.2(\sigma_{cm}/P_0)^{-2}$$

Panet 1995:

$$N = \frac{2P_0}{\sigma_{cm}} \qquad \lambda_e = \frac{1}{4N}\left(\sqrt{m_b^2 + 8m_bN + 16s} - m_b\right)$$

m_b, s = parameters of Hoek & Brown failure criterion

CONDITION	JEHTWA	BHASIN	HOEK
NO SQUEEZING	$N_c > 2.0$	$N_t < 1.0$	$\varepsilon < 1.0$ %
MILDLY SQUEEZING	$N_c = 0.8 - 2.0$	$N_t = 1.0 - 5.0$	$\varepsilon = 1.0 - 2.5$ %
MODERATELY SQUEEZING	$N_c = 0.4 - 0.8$		$\varepsilon = 2.5 - 5.0$ %
HIGHLY SQUEEZING	$N_c < 0.4$	$N_t > 5.0$	$\varepsilon > 5.0$ %

FACE BEHAVIOR	PANET
ELASTIC	$N < 2.0$
PARTIALLY PLASTIC	$N = 2.0 - 5.0$
PLASTIC	$N > 5.0$

FACE CONDITION	PANET
STABLE	$\lambda_e = 0.6 - 1.0$
STABLE (SHORT TERM)	$\lambda_e = 0.3 - 0.6$
UNSTABLE	$\lambda_e < 0.3$

Rock mass behavior forecast

Empirical methods – Risk of rock burst

- Scope: quick and qualitative evaluation of rock burst risk
- Conservatively, the maximum overburden in each GHS was considered

Tao Z.Y. 1988:

$$\frac{\sigma_{ci}}{\sigma_1}$$

σ_{ci} = intact rockcompressive strength
σ_1 = principal geostatic stress

Hoek & Brown 1980:

$$\frac{P_0}{\sigma_{ci}}$$

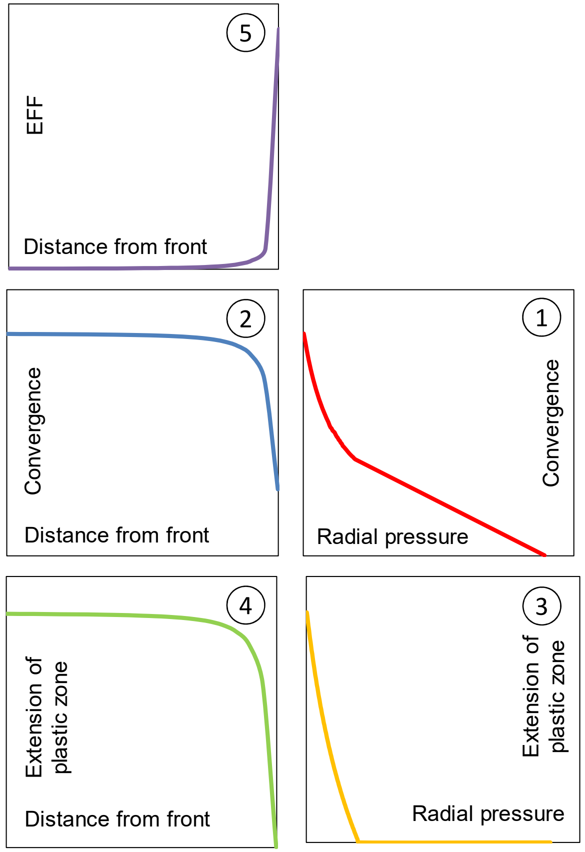
P_0 = lithostatic pressure

CONDITION	TAO Z.Y. σ_{ci} / σ_1	HOEK & BROWN P_0 / σ_{ci}
NO ROCK BURSTING	> 13.5	0.1
LOW ROCK BURST ACTIVITY	5.5 – 13.5	0.2
MODERATE ROCK BURST ACTIVITY	2.5 – 5.5	0.3 – 0.4
HIGH ROCK BURST ACTIVITY	< 2.5	0.5

Rock mass behavior forecast

Convergence – Confinement method

- Definition of rock mass behavior during excavation, analyzing five curves:
    1) Radial stress vs Convergence (Mohr-Coulomb elastoplastic constitutive model with softening and non-associated flow rule – *Ribacchi & Riccioni 1977*)
    2) Convergence vs Distance from the Front (simplified analytical procedure – *Nguyen, Minh & Guo 1996*)
    3) Radial stress vs Extension of the Plastic Zone (Mohr-Coulomb elastoplastic constitutive model with softening and non-associated flow rule – *Ribacchi & Riccioni 1977*)
    4) Distance from the Front vs Extension of the Plastic Zone (derived by extrapolation from curves 2 and 3)
    5) Distance from the Front vs Excavation Fictitious Forces EFF (derived by extrapolation from curves 2 and 1)



Rock mass behavior forecast

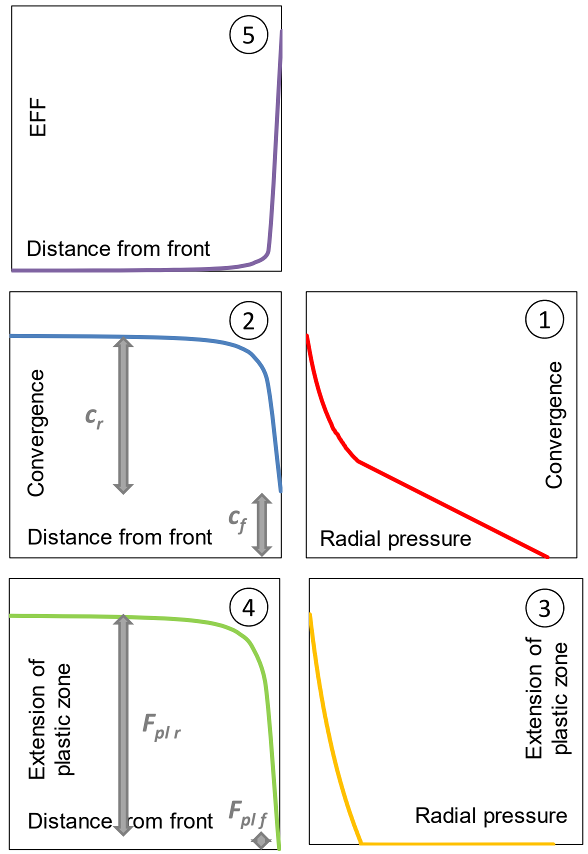
Convergence – Confinement method

- Criteria of stability – Front (*Gamble 1971 & Sakurai 1997*)

FACE CONDITION	c_f / R	$F_{pl\,f} / R$
STABLE	< 1.0 %	0.1
STABLE (SHORT TERM)	1.0 – 2.0 %	0.2
TENDENCY TO INSTABILITY	2.0 – 3.0 %	0.3 – 0.4
UNSTABLE	> 3.0 %	0.5

- Criteria of stability – Cavity

CAVITY CONDITION	c_r / R	$F_{pl\,r} / R$
STABLE	< 1.0 %	0.1
STABLE (SHORT TERM)	1.0 – 2.0 %	0.2
TENDENCY TO INSTABILITY	2.0 – 3.0 %	0.3 – 0.4
UNSTABLE	> 3.0 %	0.5



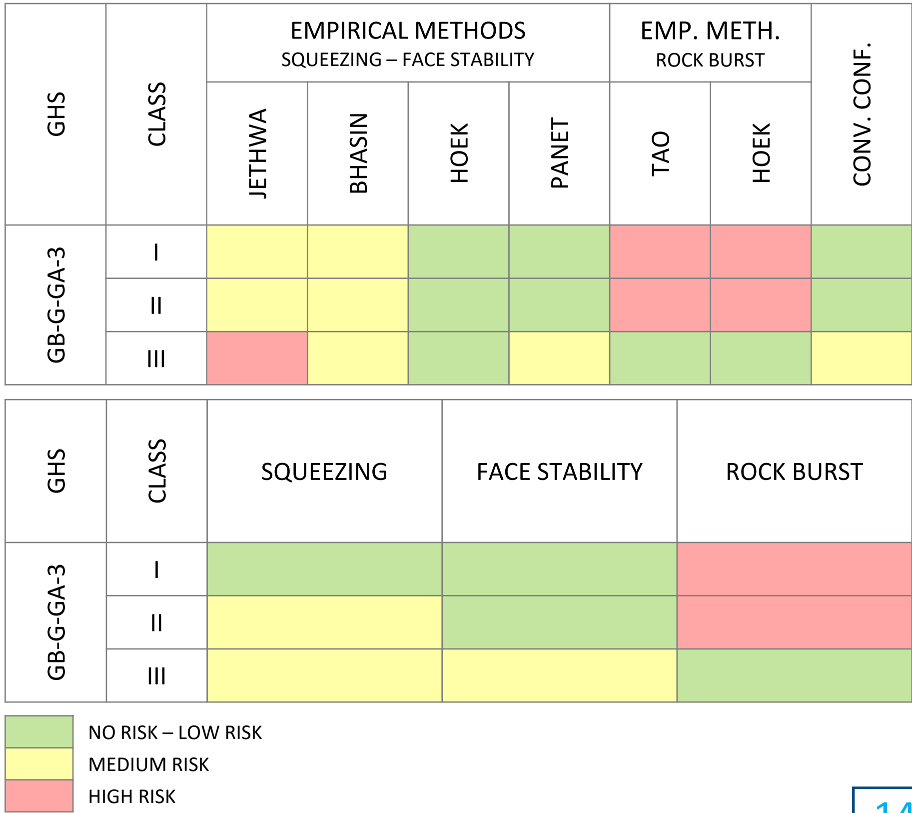
Rock mass behavior forecast

Results and expected behavior

- For each quality class in each GHS:

  1. Identification of forecasted behavior from empirical methods and from Convergence – Confinement method

  2. Definition of the risk matrix for the three principal risks:
     - Squeezing
     - Face stability
     - Rock burst



Definition of excavation typical sections

Adopted excavation typical sections

- Analysis of geological, hydrogeological and geomechanical data
- Estimates of geomechanical behavior (empirical methods, Convergence – Confinement method)
- Experience gained during the excavation of previous lots

⇩

- Identification of 4 possible rock mass behavior:
  - High resistance, reduced fracturing: almost elastic behavior, with very modest convergences and plastic area. Rock bursts possible for overburden over 1000 m.
  - Medium-low resistance, moderate fracturing: medium elastoplastic behavior; for poorest materials significant convergences and thicknesses of plastic area. Possible swelling, depending on lithology.
  - Fault zones (no Mules Fault Line): elasto-plastic behavior; significant convergences and thicknesses of plastic area.
  - Mules Fault Line: significant deformation, strongly anisotropic (1:10 ratio between maximum radial convergences, in the order of cm, and the maximum extrusions, in the order of dm)

Definition of excavation typical sections

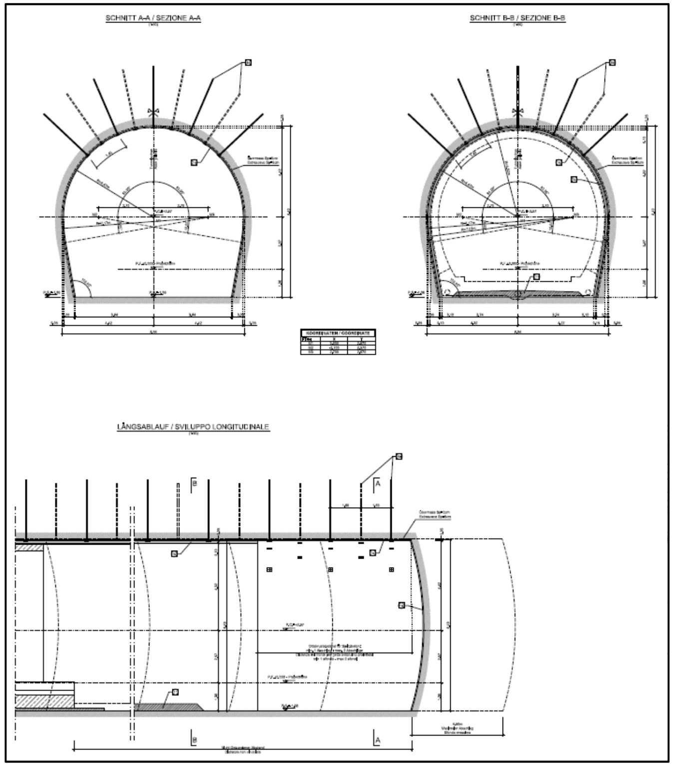
Adopted excavation typical sections

- Definition of 6 Excavation Sections basing on the forecasted behaviors previously described
- Application linked to three parameters: rock mass quality, overburden, lithology

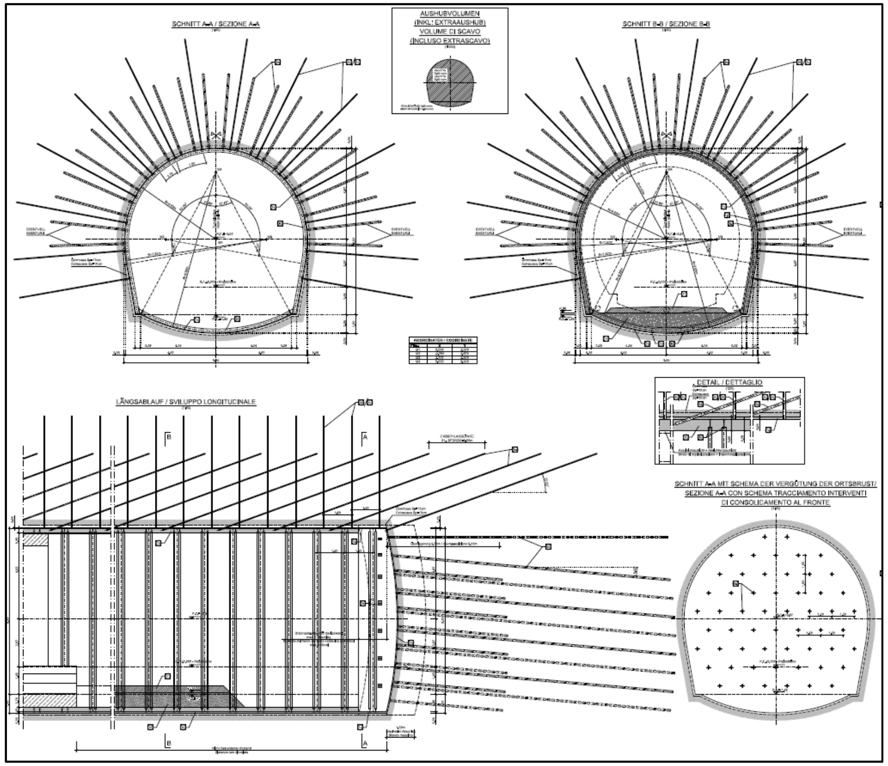
TYPICAL SECTION	APPLICATION	SUPPORT MEASURES
T2	*RMR* > 60, OVERBURDEN < 1000 m	RADIAL BOLTING + RIGID LINING (SHOTCRETE)
TRb	*RMR* > 60, OVERBURDEN > 1000 m	RADIAL BOLTING + RIGID LINING (SHOTCRETE)
T3	*RMR* = 41 - 60	RADIAL BOLTING + RIGID LINING (SHOTCRETE)
T4	*RMR* < 41 IN MULES FAULT LINE (BETTER LITHOLOGIES)	POSSIBLE AHEAD BOLTING ON FACE/BOUNDARY + RIGID LINING (SHOTCRETE, STEEL RIBS)
T5	*RMR* < 41 IN MULES FAULT LINE (MEDIUM LITHOLOGIES)	AHEAD BOLTING ON FACE/BOUNDARY + RADIAL BOLTING + RIGID LINING (SHOTCRETE, STEEL RIBS)
T6	*RMR* < 41 IN MULES FAULT LINE (WORSE LITHOLOGIES) OR SWELLING ROCK MASSES	AHEAD BOLTING ON FACE/BOUNDARY + RADIAL BOLTING + DEFORMABLE LINING (SHOTCRETE, YIELDING STEEL RIBS)

Definition of excavation typical sections

Typical section GL-T2



Typical section GL-T5



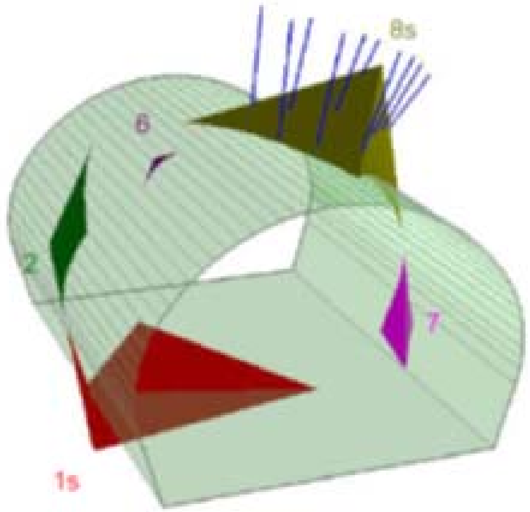
Definition of excavation typical sections

Adopted excavation typical sections

- Some significant variables on the responses of the rock mass to excavation can still exist during excavation. Therefore, a regular monitoring of the works was planned:
    1. Identification of Key Performance Indicators (KPI), to which the threshold values are linked
    2. Definition of Attention Threshold = value of any KPI, at which all the means and materials must be set up to allow a timely passage to an heavier excavation section, according to the sequence T2 / TRb, T3, T4, T5, T6
    3. Alarm Threshold = value of any KPI, at which the Excavation Section heavier than the one currently applied must immediately be adopted according to the same sequence

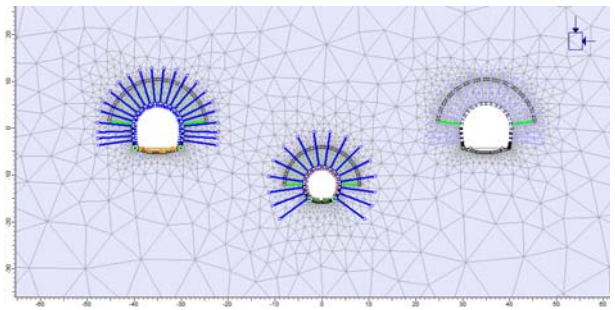
KPI	ATTENTION THRESHOLD	ALARM THRESHOLD
RMR	SEE APPLICATION OF TYPICAL SECTIONS	
RADIAL CONVERGENCE	1 % R	2 % R
FRONT FACE EXTRUSION	1 % R (2 % R IN MULES FAULT LINE)	2 % R (4 % R IN MULES FAULT LINE)
CONVERGENCE RATE	1.5 cm/m	2.0 cm/m
STRESS / STRAIN IN FIRST STAGE LINING	77 % OF DESIGN STRENGTH	100 % OF DESIGN STRENGTH
ACOUSTIC EMISSION	TO BE DEFINED ACCORDING TO EXCAVATION EVIDENCE	

Sizing of first stage and final lining



First stage lining

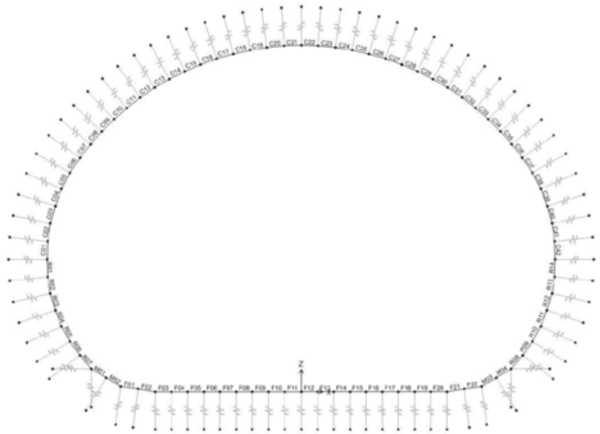
- Almost elastic behavior (Class I - II): loads on the lining were determined by analyzing potentially unstable blocks
- Elastoplastic behavior (Class IV - V): interaction between rock mass and lining was evaluated with plane strain FDM numerical modeling:
  - The rock mass was modeled as a perfectly elasto-plastic continuous mass with Mohr-Coulomb failure criterion with softening and non-associated flow rule.
  - Rock mass enhancements were schematized in FEM analysis by an increase in core cohesion depending on the pressure conveyed to the excavation front (for front enhancements) or by an increase of rock mass properties depending on area ratio (for enhancements on cavity contour)
- Intermediate condition (Class III): both analyses (blocks stability and FDM) have been carried out considering the most severe results for the sizing of the lining



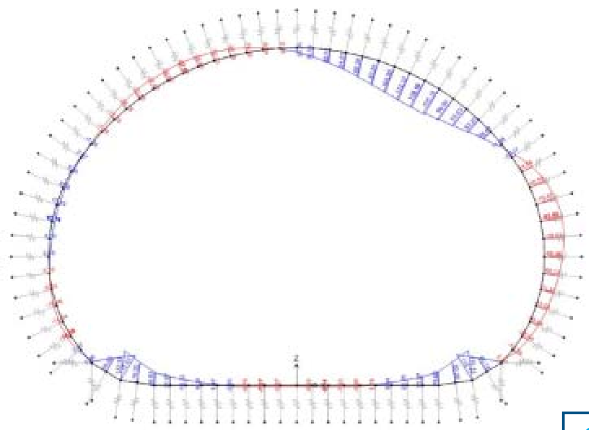
Sizing of first stage and final lining

Final lining

- FEM analyses, modeling the structure by means of beam elements on which the following loads were applied:
  1. Structure weight
  2. Rock mass loads (from first stage lining analyses)
  3. Railway operations
  4. Differences in temperature
  5. Concrete viscosity and shrinkage
  6. Earthquakes
  7. Aerodynamic pressure due to train transit
  8. Impact
  9. Fire
- Soil-structure interaction: linking elements at the model nodes that, when compressed, send to the structure a reaction equal to the soil-structure contact pressure. Stiffness of the links from parameters of rock mass and waterproofing layer



FEM model

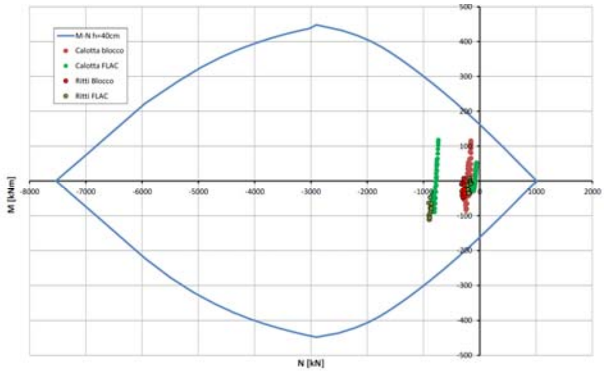


Resulting bending moment

Sizing of first stage and final lining

Structural checks

- For the SLS (Serviceability Limit State) and ULS (Ultimate Limit State) structural checks, the combinations of actions were considered in compliance with the requirements of Italian NTC.
- In order to comply with a service life of 200 years, some partial coefficients on strength were amplified compared to NTC requirement:
  - Reinforced concrete
    - Coefficient on concrete strength: standard conditions γ_c = 1.60 (instead of 1.50); exceptional conditions γ_c = 1.20 (instead of 1.00)
    - Coefficient on steel strength: standard conditions γ_s = 1.20 (instead of 1.15); exceptional conditions γ_s = 1.00 (as per NTC)
    - Reduction coefficient for long duration: α_{cc} = 0.85 (as per NTC)
  - Non reinforced concrete
    - Coefficient on concrete strength: standard conditions γ_c = 1.60 (instead of 1.50); exceptional conditions γ_c = 1.20 (instead of 1.00)
    - Reduction coefficient for long duration: α_{cc} = 0.80 (instead of 0.85)



VERIFICATIONS UNDER BREAK DUE TO SHEARING STRESS				
Material properties				
Concrete characteristic cube strength	R_{ck}	=	37	N/mm^2
Concrete characteristic cylinder strength	f_{ck}	=	31	N/mm^2
Concrete design compressive strength	f_{cd}	=	16.47	N/mm^2
Structural steel tensile strength	f_{yd}	=	375.00	N/mm^2
Design forces and moments (S.L.U.):				
Design shear force	V_{Ed}	=	328.00	kN
Design axial force corresponding to V_{Ed}	$N (V_{Ed})$	=	800.00	kN
Design bending moment corresponding to V_{Ed}	$M (V_{Ed})$	=	0.00	kNm
Section geometry				
Depth from rebar centroid to extreme compression fiber	d	=	672	mm
Section width (minimum)	b_w	=	1000	mm
Section reinforcement (subject to tensile stress):				
Rebar diameter	Ø	=	16	mm
Number of rebars in longitudinal reinforcement	n	=	6.7	--
Area of longitudinal reinforcement subject to tensile stress	A_{sl}	=	1340	mm^2
Longitudinal reinforcement ratio ρs ($\leq$ 0.02)	ρ_l	=	0.0020	--
VERIFICATION WITHOUT TRANSVERSAL REINFORCEMENT (§ 4.1.2.1.3.1)				
Factor dependent on d	k	=	1.55	--
Stress dependent on k and concrete strength	v_{min}	=	0.37	N/mm^2
Section mean compressive stress ($\leq 0.2 \times f_{cd}$)	σ_{cp}	=	1.19	N/mm^2
Minimum ultimate shear strength	$V_{Rd,min}$	=	371.62	kN
Ultimate shear strength ($V_{Rd} \geq V_{Rd,min}$)	V_{Rd}	=	**371.62**	**kN**

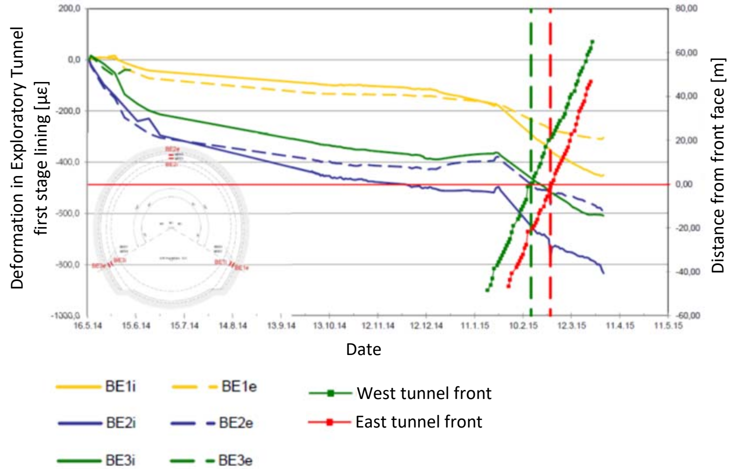
POSITIVE VERIFICATION RESULT:
no transversal shear reinforcement needed.

Sizing of first stage and final lining

Back analysis

- Data obtained from the Exploratory Tunnel crossing the Mules Fault Line, in previous Lot Mules 1, have been of great importance for design. Through a careful monitoring activity, it was indeed possible to observe the conditions of the crossed rock mass as well as of the strains in lining.
- These data allowed to evaluate the behavioral evolution over time, also with regard to the excavations of other works.
- The examination and interpretation of the Exploratory Tunnel monitoring data led to an update of the geomechanical-geological model, with identification of the actual geomechanical parameter.

  Following this back analysis, excavation Sections T4 and T5 were optimized; finally, application of these sections was redefined.



Deformations in the lining of the Exploratory Tunnel in Pusteria Fault Line.
Noteworthy is the increase upon passing of the Railway Tunnels (vertical dotted lines).

Conclusions

- The construction of a long railway tunnel provides the designer with a considerable amount of data for the design implementation and improvement.
- In particular, <u>a procedure was described for managing the statistical variability of the basic geomechanical data of Lot Mules 2-3</u>, the longest on the Italian side, with the aim of optimizing the design.
- Also, the back-analysis carried out in Lot Mules 1 based on the Exploratory Tunnel excavation results allowed to further improve the design and approach the actual conditions.

THANKS FOR YOUR ATTENTION!