High Speed Railway Milan – Genoa, Parametric analysis of rock stress-strain control during tunnel excavation in the “Argille a Palombini” formation.

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ABSTRACT: The Stretch of the Italian high-speed railway between Milan and Genoa is currently under construction and will be completed by 2021 with more than 90 km of tunnels. More than 60% of the tunnels will be carried out in the “Argille a Palombini” (aP) formation, characterized by an alternation of highly tectonized claystone schists with limestone lenses and squeezing behaviour. The paper describes the experience gained in the full-face excavation of more than 7 km of tunnel in aP, with overburden ranging from 5 to 450 m, extreme variability of rock mass quality and tunnel sections. Excavation data have been collected by an extensive monitoring system, compared with design forecast and interpreted by back analysis models. Such experience permitted us to define a parametric analysis for the control of rock mass deformations, according to ADECO-R.S. approach (Lunardi, 2006), as function of geomechanical and monitoring data gathered during the excavation.

1 INTRODUCTION

The high speed railway Milan-Genoa is one of the thirty European Priority Projects adopted by the European Union on 29th April, 2004 (Project n. 24 "Railway axis Lyon/Genova-Basel-Duisburg-Rotterdam/Antwerpen") as a new European project, the so-called "Bridge between Two Seas", i.e. a rail-link Genoa-Rotterdam. The line will improve the connection between the port of Genoa and the Po Valley inland Northern Europe giving rise to the increase in transport capacity needed to meet the growing traffic demand (Figure 1).

The line runs along the Genoa-Milan route reaching Tortona, and proceeds along the Genova-Alessandria-Turin route up to Novi Ligure, crossing the provinces of Genoa and Alexandria. The new line will be connected to the South at Voltri and Bivio Fegino through interconnections with the railway facilities at Genoa hub and with dock basins of Voltri and Porto Storico. The connection to the North will be ensured by the existing railway lines Genoa-Torino and Tortona-Piacenza-Milan (Pagani, 2006).

Figure 1. Railway axis Lion/Genoa–Basel–Duisburg–Rotterdam/Antwerpen

The total length of the alignment will be approximately 53 km. The project requires the construction of 36 km of tunnels running through the Apennine Mountains between Piedmont and Liguria. The full scope of

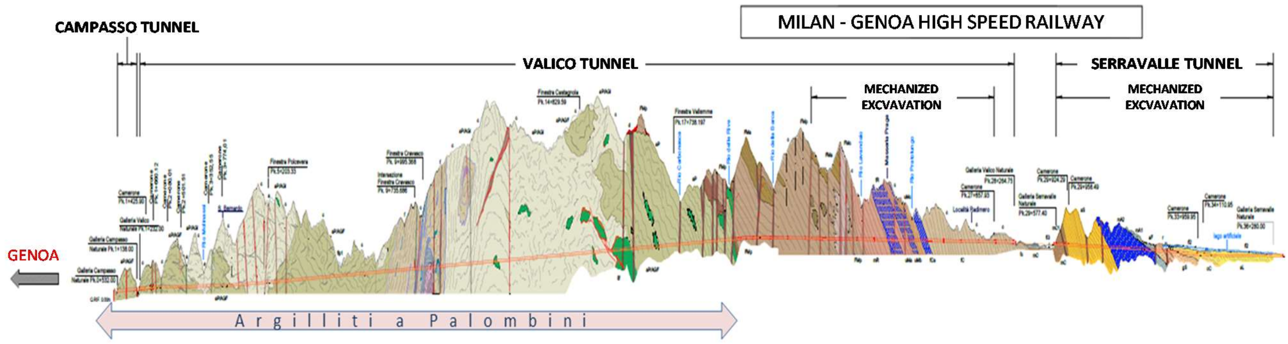


Figure 2. General profile along the line highlighting the stretch excavated in aP

underground works, including dual tube single-track running tunnels, accesses and interconnections, exceeds 90 km. About 60% of the design alignment crosses the lithogical unit defined as Argille a Palombini – claystone schist with limestone lens – (hereinafter aP), which therefore is particularly important for the implementation of the project (Figure 2).

The excavation intersects the aP at the southern part of the alignment, along the stretch Genoa-Voltaggio (AL), thus embracing the excavation of mainline tunnels, access tunnels and interconnections with the old Voltri railway. Excavations in aP are mainly designed and executed with conventional method by full face tunnel advance and cross sections area between 75 and 400 m^2 (Figure 3). The only exception is the Polcevera access, which was carried out by an EPB Tunnel Boring Machine, with an excavation diameter of approximately 10 m.

Tunnel	Total leght / Excavated lenght (m)	Section Type	Excavation Method	State of art 2016.9.15
Campasso Tunnel	606 606	*Excavation Face 130 m²*	*Conventional Full Face*	
Valico Tunnel *single tube - dual track*	194 194	*Excavation Face 130 m²*	*Conventional Full Face*	
Valico Tunnel *dual tube - single track*	16190 50	*Excavation Face 100 m²*	*Conventional Full Face*	
Valico Tunnel *emergency zone*	1488 0 *Ready to start*	*Excavation Face 120 m²*	*Conventional Full Face*	
Valico Tunnel *chambers*	1076 234	*Excavation Face up to 400 m²*	*Conventional Full Face*	
Voltri Interconnecting Tunnel *dual tube - single track - even*	1997 55	*Excavation Face 80 m²*	*Conventional Full Face*	
Voltri Interconnecting Tunnel *dual tube - single track - odd*	3 915 486	*Excavation Face 80 m²*	*Conventional Full Face*	
Polcevera Access Tunnel	1763 1763	*Excavation Face 75 m²*	*Mechanized*	
Polcevera Junction	117 117	*Excavation Face 180÷240 m²*	*Conventional Full Face*	
Castagnola Access Tunnel	2530 1226	*Excavation Face 100 m²*	*Conventional Full Face*	
Castagnola Junction	101 0 *Ready to start*	*Excavation Face 180÷240 m²*	*Conventional Full Face*	
Val Lemme Access Tunnel	1 738 1 738	*Excavation Face 130 m²*	*Conventional Full Face*	
Val Lemme Junction	188 28	*Excavation Face 210÷250 m²*	*Conventional Full Face*	
By-pass from Campasso T. to Voltri Interconnecting T.	462 462	*Excavation Face 130 m²*	*Conventional Full Face*	

Figure 3. Principal characteristics of tunnels designed and excavated or under excavation in aP

2 THE SURVEY STAGE

From a geological point of view, the aP is categorized as a lithostratigrafic complex composed of micaceous carbonate schists of dark grey colour, containing a very strong pervasive foliation and an abundance of intrafolial quartz and albite-bearing veins. The spacing of schistosity planes ranges from a few millimetres to several centimetres and locally the rock mass is strongly foliated (Figure 4.a).

The aP contains diffused layers of very compact microcrystalline limestone, called "Palombini", characterized by massive texture ranging in thickness from centimetre to decimetre scale and interbedded with phyllite (Figure 4.b) The calcareous intercalations are heterogeneously and discontinuously distributed and, therefore, their location is not predictable. Schists may also contain lenticular basaltic bodies, often very fractured, which can occur associated with banded jaspers.

a) b)

Figure 4. a) sample collected from a borehole; b) rocky outcrop

The permeability essentially depends on the degree of rock mass fracturing: outside the fault zones hydraulic conductivity mainly varies between 1×10^{-7} and 1×10^{-8} m/s. Along the fault damage zones, it may increase up to 1×10^{-6} m/s, while in the core areas (fault gouge zone) the permeability does not increase, due to the presence of clay filling material. Considering the standard time of primary lining construction, the rock mass can be considered, essentially, waterproof, with water seepage less than 0.16 l/s x 10 m. Despite, across fault zones the water flows can reach several litres per second, although it is often a temporary event (Lunardi, 2016).

The aP survey phase commenced with the preliminary design, and was further elaborated on the basis of the discovered evidence and data collected during subsequent design stages. Between 1996 and 1998, two exploratory tunnels Val Lemme and Castagnola were excavated with an overall length of 1200 m. The construction process was completed by several in-situ tests and installation of specific monitoring stations. The exploratory tunnels were evaluated as the most appropriate method to investigate geomechanical parameters of the aP and the rock mass response to excavation. The excavation of exploratory tunnels, in fact, permitted to overcome the main limitations of conventional survey due to high overburden and marked susceptibility of cores to damage during sampling (mainly caused by intrinsic fissility of the material).

The results of the investigations performed during the exploratory tunnels excavation were integrated during the next design phases by means over 5 km of geophysical lines (arrays), 400 in situ tests conducted in more than 100 boreholes, and 1200 laboratory tests. A survey campaign specifically dedicated to the evaluation of creep behaviour was carried out together with investigation of argillaceous schist drillability and conditioning to provide data necessary to the design of mechanized tunnelling.

2.1 *Geomechanical framework*

The survey phase highlighted the extreme variability of the stress-strain response during the excavation in aP. As results of investigation and parametric back-analyses performed during the exploratory tunnels excavation, such variability was connected with main factors detectable at the tunnel face. On the basis of this connection, it was possible to identify three geomechanical groups composing the rock mass and named Groups 1, 2 and 3. The main factors underlying the above groups were:

- lithological criteria (petrographical and mineralogical composition, percentage of Palombino with reference to the argillite matrix, the degree of alteration, possible presence of water);
- structural criteria (characteristics of joints, RDQ index, foliation intensity, tectonisation intensity, such as the presence of folded structures including also microscale folding);
- lithomechanical criteria (with reference to the first assessment of physical properties, strength and deformability).

The main characteristics of geomechanical groups are summarized hereunder:

• Geomechanical group 1: the rock mass exhibits adequate geomechanical properties. The RQD index on average exceeds 50÷60%; surfaces of joints are not altered, and water circulation is scarce or absent. The Palombini share can exceed 50% by weight i.e. increases up to the degree when it starts to completely determine the overall rock mass behaviour. Estimated strength of intact rock mass is close to 40 MPa, and operative value of elastic modulus is larger than 3 GPa. This lithological group represents about 28% of the whole rock mass intersected by exploratory tunnels.

• Geomechanical group 2: the rock mass shows poor geomechanical properties. The RQD index varies on average between 20% and 50%; surfaces of joints vary from slightly altered to altered, and water circulation is scarse. The Palombini, locally altered, are less than 50% by weight, varying from fractured to strongly fractured. Estimated strength of intact rock is about 10 MPa, and operative values of elastic moduli range between 1 and 2 GPa. This lithological group represents about 55% of the whole rock mass intersected by exploratory tunnels.

• Geomechanical Group 3: the rock mass appears in particularly tectonized or altered zones and has very poor geomechanical properties. The RQD index on average is lower than 20%; the structure, when identifiable, is intensely folded maintaining its pattern at micro scale. Surfaces of joints vary from not altered to strongly altered, and water circulation can be significant. The Palombini, when present (not more than 30% by weight), are intensely fractured and altered. Estimated strength of intact rock mass is less than 5 MPa, and operative values of elastic moduli are also much lower than 1 GPa. This lithological group represents about 17% of the whole rock mass intersected by exploratory tunnels.

The evidences gathered during the excavation of exploratory tunnels were complemented by interpretation of in situ investigations and the results of laboratory tests. Classification of sampled material in terms of lithological composition, foliation pattern and recurring calcite veins has confirmed, also at the sample scale, the presence of three geomechanical groups characterized by different responses observed through uniaxial compressive test and strain controlled triaxial test (Figure 5). The stress – strain response revealed markedly fragile behaviour, with peak strength at low strain level (ε_a=0.2÷0.4%), and stabilization at residual strength values. It was possible to define confidence intervals for geomechanical parameters on a statistical basis, linking each value with its probability of occurrence.

The aP peculiarity identified during the investigation is the intrinsic variability of mechanical behaviour, mainly attributed to the foliated structure of the matrix and to the strongly fractured zones which significantly affect the rock mass stress-deformation response.

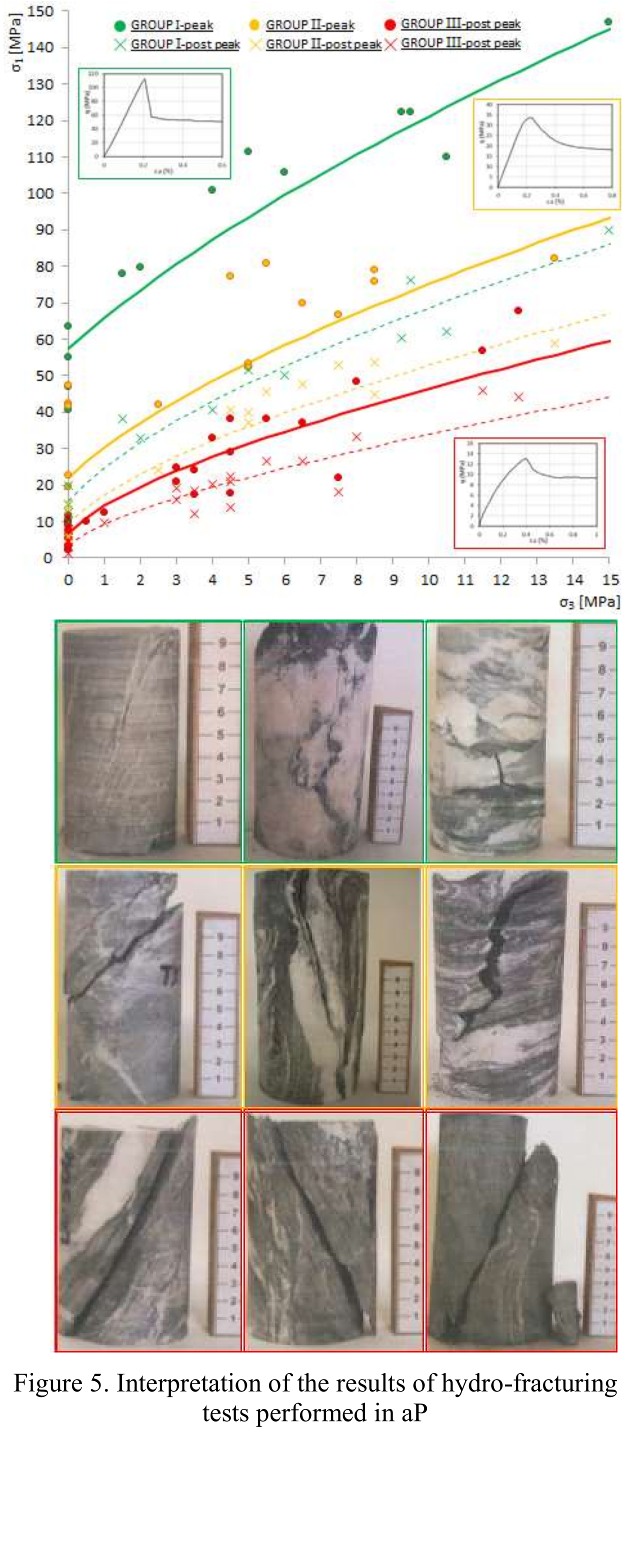


Figure 5. Interpretation of the results of hydro-fracturing tests performed in aP

In order to make the geomechanical context even more demanding, some further elements that emerged during the investigations should be taken into account, both at the scale of the material and at the scale of rock-mass, such as:
• ratio of average horizontal stress to vertical stress (K_0), predominantly greater than one up to 200÷250 m below ground level (Figure 6);
• high probability of squeezing behaviour when overburden is deeper than 250 m in geomechanical conditions of the Group 3;
• presence of potentially swelling clay minerals.

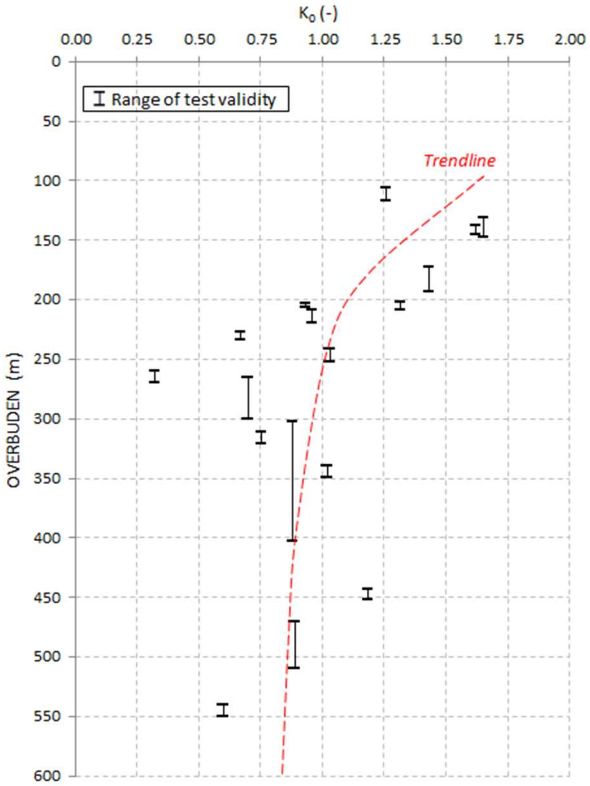


Figure 6. Interpretation of the results of hydro-fracturing tests performed in aP

3 THE DESIGN STAGE

The inherent variability of aP made it necessary to divide the rock mass into "geomechanical groups" at the survey phase. Each group is characterized by a specific set of strength and deformability parameters that determine its stress-strain response to the excavation. During the excavations the explorative tunnels revealed continuous variations in rock-mass conditions, and consequently the inability to predict the occurrence of a particular representative geomechanical group even for short route sections. Therefore, principal geomechanical parameters of each group allowed a certain range of variability thus specifying the “intrinsic variability band”, which will determine the stress-strain behaviour of the examined group during excavation in all possible geomechanical scenarios. It means that the rock-mass conditions are verified at the construction stage through direct survey of geostructural, geomechanical and hydrogeological features of the excavation face, thus identifying the geomechanical group it belongs to.

In view of these assumptions, it became necessary at the design stage to determine a full set of section types, verified through all possible scenarios (depending on the overburden and on the variability of geomechanical parameters valid for statistically defined intervals). A design and contract instrument, called *Guideline*, has been defined in order to manage the application of these section types. For each section type, a specific range of application was defined together with a variety of stabilization interventions assigned to it. In this way each section type, without changing the final structural characteristics of the tunnel, can fit to actual geomechanical conditions, the rock mass hydrogeological properties, tunnel face extrusion pattern and cavity deformation type (Figure 7).

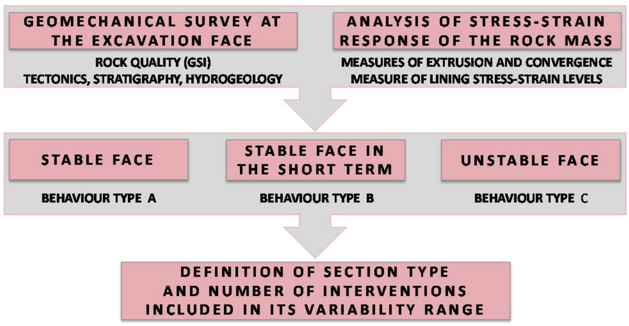


Figure 7. *Guidelines* logical scheme

The *Guidelines*, managing section type application, allowed to define the criteria that the designer can rely upon during the construction works in order to:
• confirm the most applicable section type, selecting it from among those already assigned to a particular tunnel stretch;
• define the section type most suitable for the actual geomechanical context, according to the designed variability;
• identify a different section type from those assigned to each particular stretch or anyway

envisaged in the design for the same formation, in case actual conditions encountered during the excavation differ from those predicted.

During the design phase the following set of components were defined:

• 12 section types each with specific interventions to control the aP deformation in the various geomechanical contexts specified at the survey phase;

• uniform sections of underground alignment, in terms of deformation response of rock mass to the excavation, together with a set of section types assigned to them; each section type is characterized by a percentage of its application defined by a special parametric analysis;

• alert and alarm threshold values set for rock mass and lining deformation parameters to be monitored during construction works by means of the instruments specified in the monitoring plan;

• three levels of variability established for the designed interventions (minimum, medium and maximum) defined for each section type, and introduction of *Guidelines* as a management tool operating with a complete set of 36 different excavation sections (Figure 8).

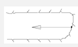
Geomech. conditions.	Overburden (m)		Designed section type				Predicted deformation response		Variability		
			Interventions						Interventions		
			Type	1° Phase	Final				1° Phase	Final	
					invert (th./D)	crown (th./D)				invert (th./D)	crown (th./D)
	from	to		Preconfinement and presupport			(cm)	(cm)	Preconfinement and presupport		
GR1 GSI=45÷55	0	300	**B0L**	2IPN160/1.4 th.shotcr. 20	80/12Φ	60/15Φ	<5	-	2IPN160/1.2÷1.5 th.shotcr. 20	9÷15Φ	12÷18Φ
GR1 GSI=45÷55 mixed core-face	300	500	**B0V**	HEB200/1.2 th.shotcr. 30	100/12Φ	60/15Φ	<6÷8	-	HEB200/1.0÷1.5 th.shotcr. 25÷35	9÷15Φ	12÷18Φ
				25 reinf. contour overlapp. 3m					25 reinf. contour overlapp.. 3m		
GR1 GSI=45÷55	300	500	**B0/1**	HEB180/1.2 th.shotcr. 30	90/12Φ	70/15Φ	<6÷8	-	HEB180/1.0÷1.4 th.shotcr. 25÷35	9÷15Φ	12÷18Φ
GR1 GSI=45÷55	500	600	**B0/2**	HEB200/1.2 th.shotcr. 30	100/12Φ	80/15Φ	<6÷8	-	HEB200/1.0÷1.4 th.shotcr. 25÷35	9÷15Φ	12÷18Φ
GR1 GSI=40÷45 anisotropy	0	600	**B1**	2IPN220/1.2 th.shotcr. 30	100/12Φ	80/15Φ	<6÷8	<5	2IPN220/1.0÷1.4 th.shotcr. 25÷35	9÷15Φ	12÷18Φ
				25 Bolts Φ24/1.2x2 L=5.5÷6.5					25 Bolts Φ24/1.2x2m L=5.5÷6.5m		
GR2 GSI=40÷45 mixed core-face	0	600	**B2V**	HEB240/1.0 th.shotcr. 30	100/5Φ	80/9Φ	<6÷8	<5	HEB240/0.8÷1.2 th.shotcr. 25÷35	3÷6Φ	7÷11Φ
				25 reinf. contour overlapp. 3m					25 reinf. contour overlapp.. 3m		
GR2 GSI=40÷45	0	500	**B2/1**	HEB200/1.0 th.shotcr. 30	100/5Φ	80/9Φ	<6÷8	<5	HEB200/0.8÷1.2 th.shotcr. 25÷35	3÷6Φ	7÷11Φ
				60 FbGlcem. face overlapp. 9					40÷80 FbGlcem. face overlapp.6÷12		
GR2 GSI=40÷45	500	600	**B2/2**	HEB240/1.0 th.shotcr. 30	100/5Φ	90/9Φ	6÷8	<5	HEB240/0.8÷1.2 th.shotcr. 25÷35	3÷6Φ	7÷11Φ
				60 FbGlcem. face overlapp. 9					40÷80 FbGlcem. face overlapp.6÷12		
GR2 GSI=35÷40 anisotropy	0	500	**B4/1**	2IPN240/1.2 th.shotcr. 30	100/5Φ	80/9Φ	<8÷9	<7	HEB240/0.8÷1.2 th.shotcr. 25÷35	3÷6Φ	7÷10Φ
				60 FbGlcem. face overlapp. 9 25 Bolts Φ24/1x1.2 L=5.5÷6.5					40÷80 FbGlcem. face overlapp.6÷12 25 Bolts Φ24/1x2 L=5.5÷6.5		
GR2 GSI=35÷40 anisotropy	500	600	**B4/2**	2IPN240/1.2 th.shotcr. 30	100/5Φ	90/9Φ	<8÷9	<7	2IPN220/1.0÷1.4 th.shotcr. 25÷35	3÷6Φ	7÷10Φ
				70 FbGlcem. face overlapp. 9m 25 Bolts Φ24/1x1.2m L=5.5÷6.5m					55÷90 FbGlcrfm. face overlapp.6÷12 25 Bolts Φ24/1x2m L=8m		
GR3 GSI=25÷30 / interference	0	600	**C2**	HEB240/1.0 th.shotcr. 30	100/3Φ	90/9Φ	<12	<10	HEB240/0.8÷1.2 th.shotcr. 25÷35	1.5÷3Φ	5÷9Φ
				55 FbGlcem. face overlapp. 9 55 FbGlvalv. contour overlapp. 9					40÷80 FbGlcem. face overlapp.6÷12 40÷70 FbGlvalv. contour overlapp.6÷12		
GR3 GSI=30÷35 / interference	0	600	**C4**	HEB240/1.0 th.shotcr. 30	100/3Φ	90/9Φ	<10÷12	<10	HEB240/0.8÷1.2 th.shotcr. 25÷35	1.5÷3Φ	5÷9Φ
				70 FbGlcem. face overlapp.9 71 FbGlcem. contour overlapp.9					55÷90 FbGlcem. face overlapp.6÷12 55÷85 FbGlcem. contour overlapp.6÷12		

Figure 8. Interventions variability for each section type foreseen during the design phase to excavate Valico tunnel - single track in aP

4 CONSTRUCTION STAGE

To date, more than 7000 m of tunnels have been excavated in the aP with the overburden varying between 5 m and 450 m.

Geomechanical surveys of the excavation face showed a strongly dispersed distribution of the GSI index, especially under shallow overburden (Figure 9a). Such result has confirmed the design predictions: 10% of tunnel faces belong to the Geomechanical group 3; 50% of them belong to the Geomechanical group 2, and the remaining 40% belong to the Geomechanical group 1 (Figure 9b).

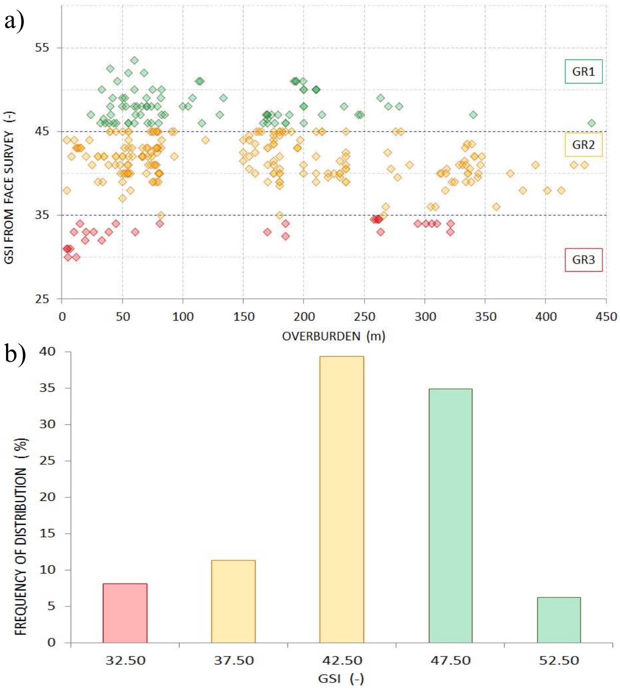


Figure 9. GSI determined from geomechanical survey of the tunnel face: a) distribution by depth; b) frequency histogram

There is substantial consistency between the percentages foreseen in the design stage for each section type (Figure 10a) and those applied in the construction stage (Figure 10b).

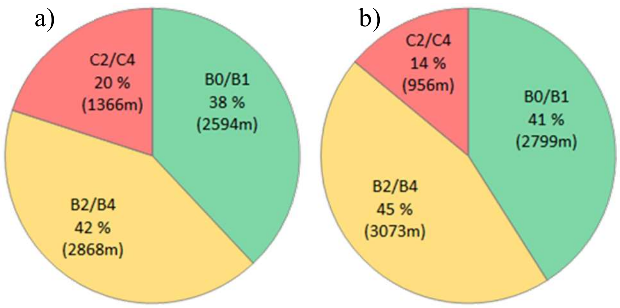


Figure 10. Comparison between the percentage of section types: a) foreseen in design stage; b) actually applied

The marked variability of the rock mass conditions are further highlighted by the time (Ts) needed for 1 meter advance (Figure 11). Ts varied from several tens of minutes up to hours, even on short distances in the same tunnel.

Figure 11. Ts needed for 1 metre advance with hydraulic hammer in the Castagnola access (excavation cross section about 100 m^2)

The *Guidelines*, being a design and contractual tool, made it possible to cope with a succession of rapidly changing geomechanical conditions by applying appropriate section types or varying interventions intensities of the same section type. The flexibility of applied operating procedures has required concerted action and synergies from all the involved operators, as well as adequately arranged construction site needed to quickly implement various types of intervention. These factors allowed to reach the average advance rate of more than 40 m/month with the trend maintained over time (Figure 12).

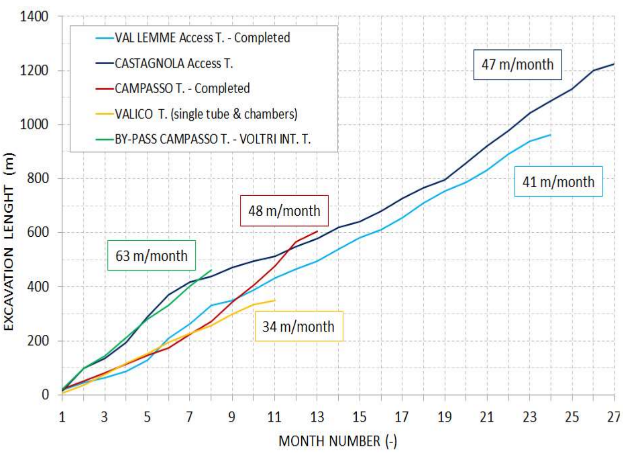


Figure 12. Advance rate reached in tunnels excavated in aP

Tunnel excavation production rate ranging from 30 and 40 m/month was reached at double track stretches and chambers (with cross-section area up to 255 m^2), excavated in difficult geomechanical conditions and low overburden interfering with buildings. Production rate higher than 30 m/month have been achieved in 80% of cases (Figure 13).

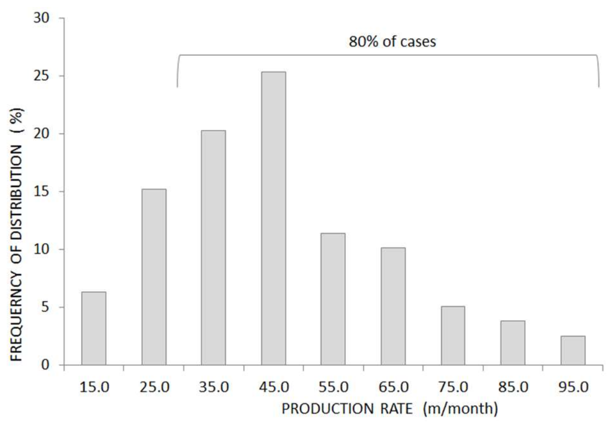


Figure 13. Advance rate reached in tunnels excavated in aP (frequency histogram)

The possibility to quickly and timely modify the section type and the intensity of the interventions operating within the designed variability, allows also to achieve an important goal in underground construction: the deformation control. Despite a marked variability of overburden and difficult rock mass conditions, the convergences observed at the springline were lower than 1.3% of the tunnel diameter along more than 90% of the stretches excavated in aP (Figure 14).

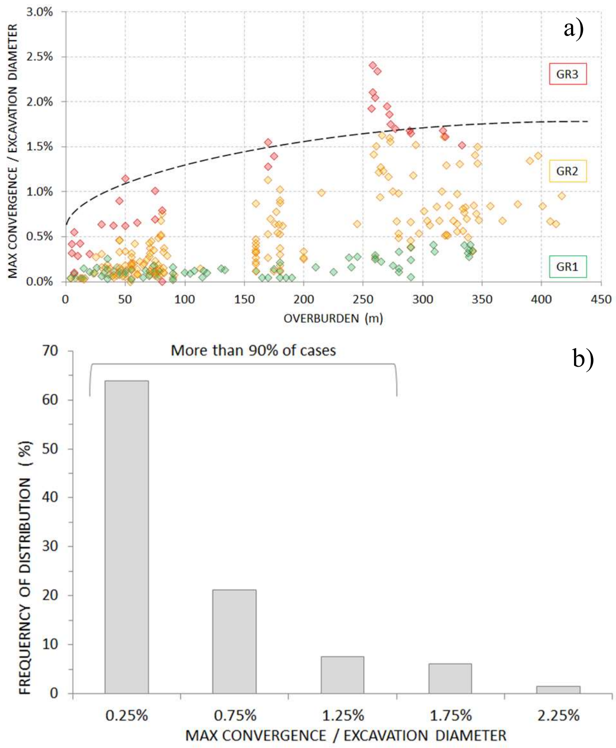


Figure 14. Ratio between maximum convergence and the excavated diameter: a) distribution by depth; b) frequency histogram

The ratio between the maximum horizontal displacement and settlement (Figure 15) suggests a marked anisotropy of stress state, occurred particularly in shallow overburden, as already highlighted in the survey phase (see Figure 6). Maximum convergence values greater than 1.5% are related to high overburden (between 200 m and 250 m), with sudden and marked worsening in geomechanical conditions: in these circumstances the lack of sufficient core face preconfinement immediately caused high strains that trigger the rock mass softening process, already highlighted in the survey stage (see Figure 5) with displacement increasing over time. The introduction of a new section type and the modulation of the intensity of interventions allowed the progress of the excavations in persisting poor geomechanical conditions.

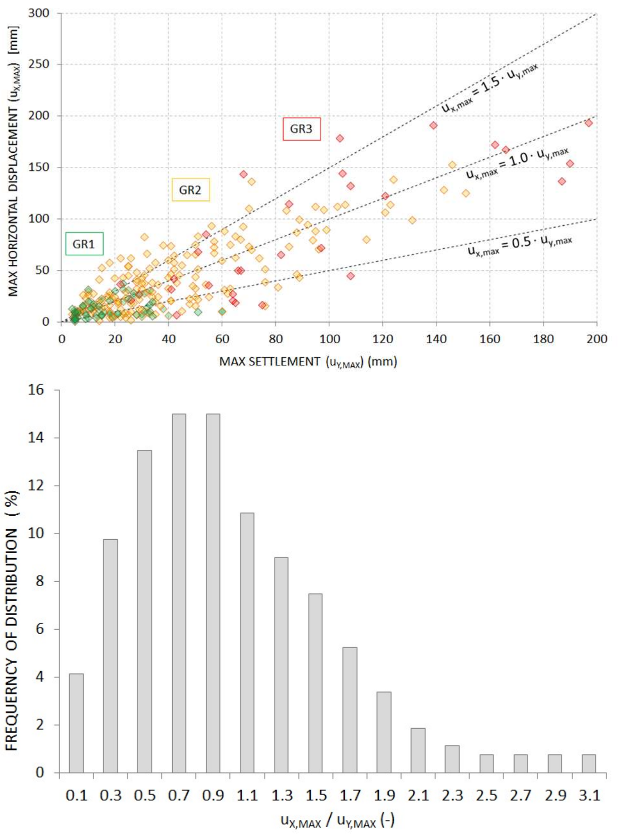


Figure 15. Maximum horizontal and vertical displacement: a) link between the displacements; b) frequency histogram of the relationship

The generalized convergence control was exerted within the limits specified at the design stage through implementation of core face and cavity preconfinement interventions with variable intensity depending upon the overburden and the encountered geomechanical group. Figure 16 shows the close connection of maximum values of extrusion and convergence measured at coupled monitoring stations.

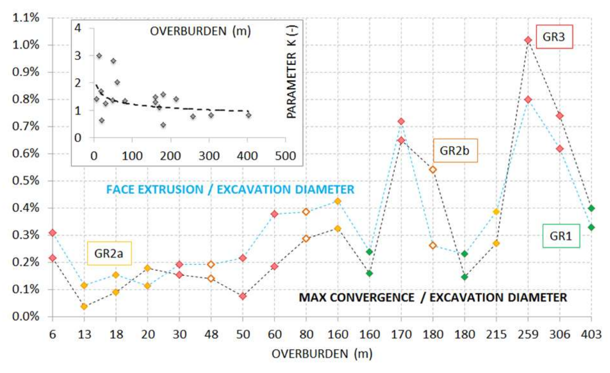


Figure 16. Deformation response: relation extrusion-convergence

As shown in Figure 16 the ratio extrusion/convergence (Parameter K) tends to 1 as the overburden tends to increase; under different geomechanical context, the measurements remain substantially constant because of the intensity control of the stabilization works.

Interpretation of the monitoring data made it possible to trace the extrusion curves inside the core which represent shallow and medium overburden for different geomechanical groups identified at the survey phase (Figure 17).

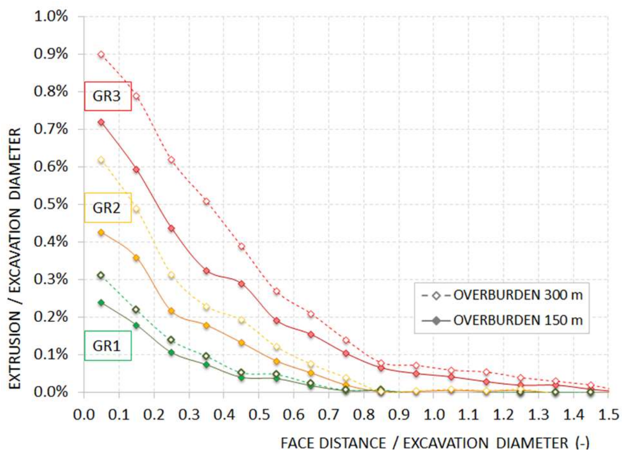


Figure 17. Deformation response: extrusion developed in the core-face

5 CONCLUSIONS

So far, the construction of the high-speed railway between Milan and Genoa has entailed the excavation of 7000 m of tunnels in an alternation of highly tectonized claystone schists with limestone lenses and characterized by squeezing behaviour, called Argille a Palombini.

The main excavation challenge is the managing of the extreme variability of the rock mass behaviour, characterized by sudden and severe deterioration of the rock mass quality, in wide ranges of overburden and cross section areas.

The paper described the experience gained during the tunnels excavation, highlighting the types of survey as well as the contractual instruments carried out during the design stage. Such instruments play a key role to manage the intrinsic variability of the encountered rock mass during the following construction stage.

The interpretation of data collected by an extensive monitoring system and supplemented by specific back analysis models has been used to analyse and understand the dependence between geomechanical condition at the face and the rock mass behaviour during the excavation. Such knowledge permitted us to evaluate the best sets of interventions to carry out, obtaining the control of the rock mass stress-strain response to the tunnel advance.

The progress of the excavations in the aP up to more than 90 km, which is planned to be completed within 2021, will allow to increase the available data base verifying and supplementing the obtained results.

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