Critical Aspects in the Design of the Chenani-Nashri Tunnel

L. FACIBENI, M. PALOMBA, G. CARRIERI, G. RUSSO[1] and A.H. GÜVENC[2]

[1]*Geodata Engineering S.p.A.*

[2]*Leighton Welspun Contractors Pvt. Ltd.*

(*lfi@geodata.it; mpl@geodata.it; gca@geodata.it; grs@geodata.it; Alihayri.Guvenc@lwin.co.in*)

ABSTRACT: The Chenani-Nashri Tunnel, the longest road tunnel in India with its 9 km length, currently under construction, is part of an ambitious four laning project of the NH-1A between Udhampur and Banihal on a DBFO pattern in the state of Jammu & Kashmir. The tunnel, bypassing the existing NH-1A from km 89 to km 130, crosses the sub-Himalayan Patnitop range through a flyshoid geological formation with maximum overburden of 1050 m. It includes a 2-lane main tunnel with parallel escape tunnel and is to be constructed with conventional methods (NATM) by Leighton Welspun Contractors. This paper illustrates some critical aspects affecting the design, from the time schedule constraints, through to the geomechanical characterization and the environmental constraints at large. In doing so, it presents the additional studies developed by Geodata Engineering for the Detailed Design of Main and Escape tunnels functional and support section types, integrated with risk analysis for the probabilistic evaluation of the main geological and geomechanical hazards for tunneling and the required mitigation measures, the design of the tunnel portals in challenging geotechnical context and the issues related to the design of muck disposal areas, partly used to reclaim flat ground for ancillary buildings and wayside amenities.

1 PROJECT DESCRIPTION

The new Highway link project between Udhampur and Banihal is one of the recent, most important Indian projects planned to connect the Kashmir valley with the rest of the Indian transportation network.

The Government of India has entrusted the National Highways Authority of India (NHAI) with the responsibility of the "Rehabilitation, Strengthening and four Laning of Chenani to Nashri Section of NH-1A, from 89.00km to 130.00km" in the state of J&K defined as "Project Highway" (Fig.1).

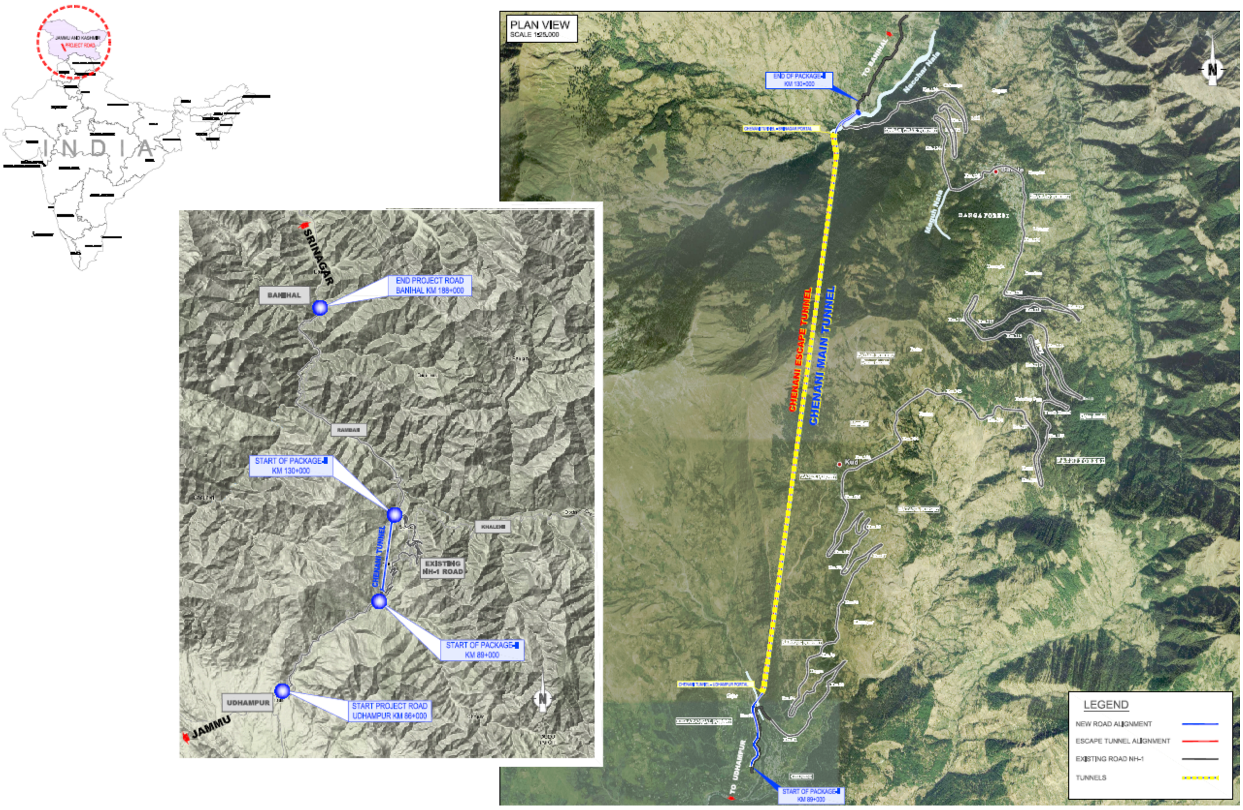


Fig. 1 Location of the "Project Highway.

The "Project Highway" (over a length of 10.9 km) aims at construction of:

- the Chenani-Nashri tunnel, 9km long, including the main tunnel single-tube, with two line bi-directional traffic and the parallel escape tunnel (schematic layout is shown in Figure 2);
- 40m single span bridge approaching the South portal of the tunnel;
- 50m single span bridge approaching the North portal of the tunnel;
- two rotary junctions with existing NH-1A at km 89 and km 130;
- surface works section: about 1.3 km of road approaching the South tunnel portal and about 0.6 km of road approaching the North tunnel portal (involves slope cuttings and embankments);
- service and technological buildings at the portals;
- toll plazas and other Project Facilities at portals;
- spoil dump areas located near South and North portals.

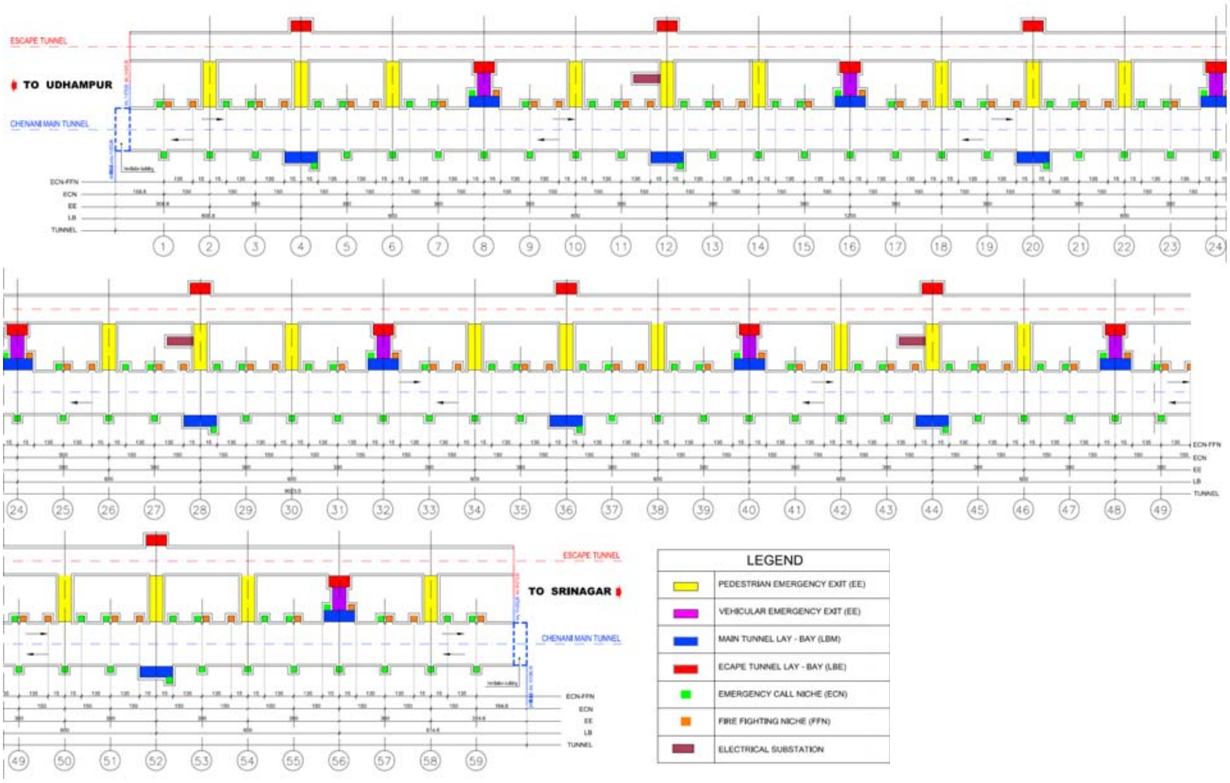


Fig. 2 Tunnel schematic layout.

2 TUNNEL TYPICAL CROSS SECTIONS

Particular attention has been given to the definition of the typical cross sections of the Main Tunnel (MT), Escape Tunnel (ET) and emergency exits (cross passages).

The MT Typical cross section (Fig. 3) has been defined based on the required clearance dimensions (H:9.35m x V:5.00m, plus 1.2m wide footpath each side) and on the M&E requirements (in particular ventilation ducts shape and dimensions for exhaust air extraction and fresh air inlet).

A significant reduction of the excavation area (compared to the Tender Design Solution) has been obtained which in turn has beneficial effects in limiting the disturbance of the rock mass in correspondence of critical geological conditions. This has been possible considering the changes listed below:

- modification of the intrados curvature radius, considering the same dimension of the ventilation ducts as per tender design solution and fully in compliance with the clearance requirements;
- different foundation level of left and right abutments of the final lining;
- optimization of the carriageway platform thickness;
- application of the TARS (Tunnel Axis Rotation System) which consists in excavating the tunnel considering an inclined axis of the final lining (but always perpendicular to the road platform).

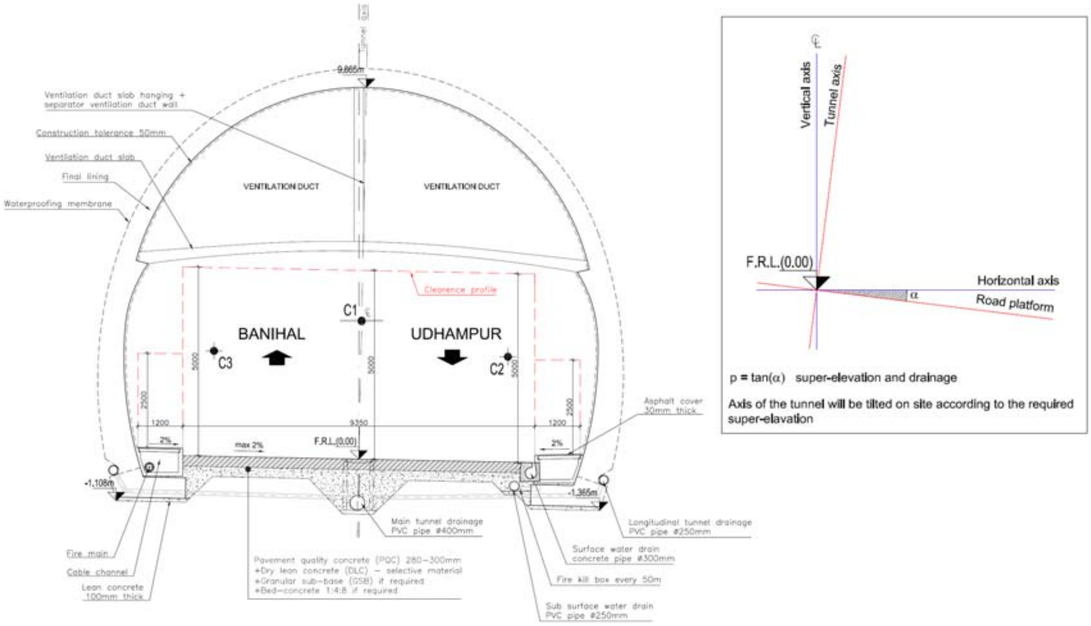


Fig. 3 Main tunnel typical cross section.

With the application of the TARS system the MT typical cross section maintains the same structural and functional characteristics, although this solution requires more attention during the design and construction stage regarding, in particular:

- structural verification of the ventilation ceiling;
- the drainage system (longitudinal PVC pipes) for the rock mass inflow groundwater;
- different foundation levels between left and right sides of steel ribs and of the abutments of the final lining; this could lead to problems with the installation of ribs and the correct position of the formwork prior to final lining concreting.

The ET typical cross section, illustrated in Figure 4 below, has been defined on the basis of a detailed study of the movements of the available construction equipment (jumbos, excavators, dumpers and ventilation ducts) in cooperation with Leighton Welspun Contractors. The ET is fully in compliance with the required clearance dimensions for the operative stage (H:5.00m x V:2.50m) with footpath fully motorable on each side.

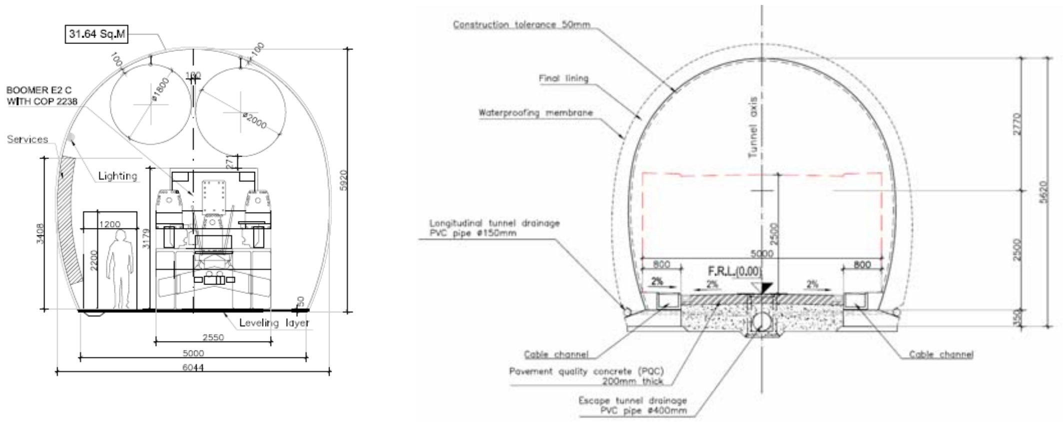


Fig. 4 Escape tunnel typical cross section (during construction and operative stage).

Two types of cross passages have been provided: pedestrian and vehicular emergency exits at 300m centers and 1200m centers respectively. The typical section of Pedestrian cross passage has been designed considering the same size of the ET section; vehicular cross passages (Fig.5) are larger, with the clearance requirements of H:7.50m x V:4.50m. The adoption of two different types of cross passages is the most suitable for the reason listed below:

- clearance dimensions of the pedestrian cross passage allows at the same time evacuation of pedestrians as well as the transit of rescue vehicles in emergency conditions: hence the safety concept for the tunnel complex results enhanced;
- the junction between the escape tunnel and pedestrian cross passage is geometrically smooth, with no corner points where stresses concentrate;
- both pedestrian and vehicular cross passages allow the transit of all equipments (dumpers, jumbos and excavators) during construction stage. This means that in case of unexpected variation of the construction program it will be possible to use also the pedestrian cross passage to open additional excavation face for the construction of the MT.

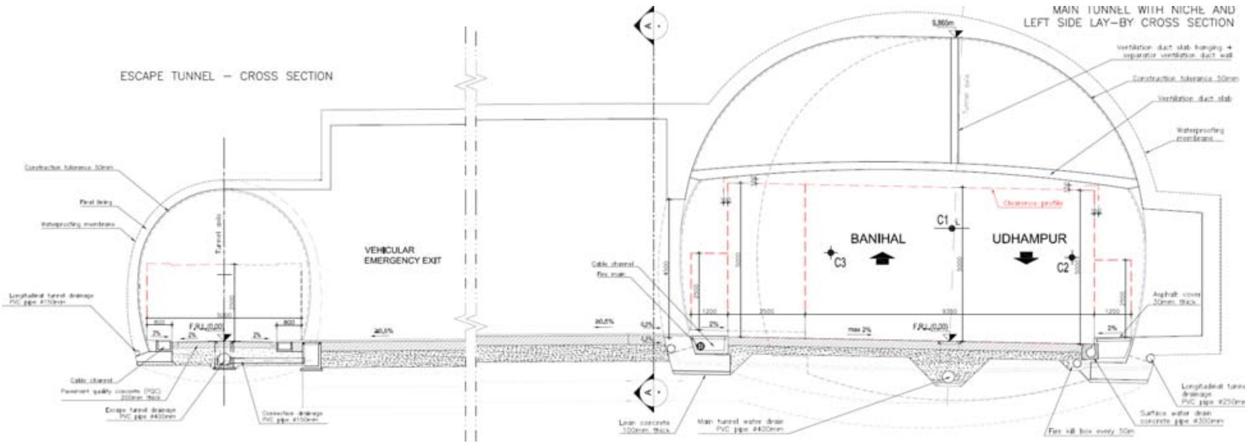


Fig. 5 Vehicular cross passage longitudinal profile.

All tunnels will be excavated using conventional tunnelling techniques (NATM). Excavation will involve the use of drill and blast methods by top heading, bench and invert for the MT and vehicular cross passage while full face excavation will be adopted for the ET and pedestrian cross passages. The tunnel will be supported by a primary lining (shotcrete, steel ribs and rock-bolts) and completed with a cast-in-place concrete final lining (reinforced for certain typical sections) designed to withstand all predicted long-term loads and seismic loads close to the portal areas. A waterproofing membrane paired with geotextile protective felt will be installed all around the tunnel section (except at the invert) for the complete length of the tubes.

3 GEOLOGICAL, HYDROGEOLOGICAL AND GEOMECHANICAL ASPECTS

The project area lies in Western Himalayan region in a sector of collisional belt known as sub-Himalayas. This tectonic domain is bounded toward south by the Himalayan Frontal Thrust or Main Frontal Thrust (HFT or MFT) and the Main Boundary Thrust (MBT) to the North (Figure 6).

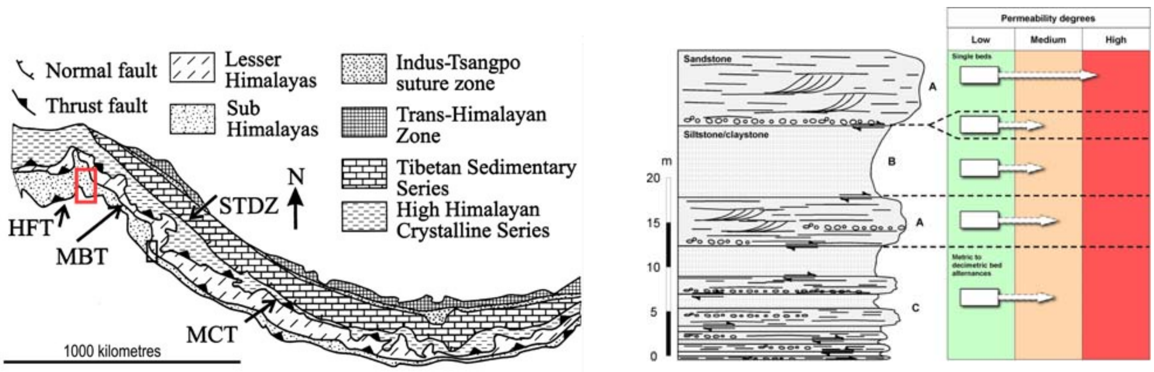


Fig. 6 Himalayan orogenic belt (in red square emphasized the studied area) and conceptual hydrological schematic model.

The rock masses along the project of the Chenani-Nashri tunnel belong to the Lower Murree Formation that includes a sequence of interbedded sandstones, siltstone/claystone layers (flysh) with thickness ranging from a few metres up to 10m.

Regarding the hydrogeological aspects, a schematic conceptual model of the groundwater circulation is shown in Figure 6: in the sandstone beds with a limited amount of joints (A) the permeability is low and increases, as secondary permeability, with the length and spacing of joint sets, becoming higher along very jointed bands (i.e. faults); in the siltstone and claystone (B) the permeability is low because of argillaceous filling in joints and, in any case, degree of fracturing does not result in higher permeability; along the siltstone and claystone interface (C), again because of the argillaceous filling, the permeability expected is low to medium; fault planes and primary joints can be characterized by higher permeability.

Specific geostructural survey of rock outcrop have been undertaken for the geomechanical characterization of the rock masses and additional investigations (boreholes, laboratory tests and geophysics investigations) have been carried out, mainly at portals area. On the basis of the field geological, geostructural data, the resulting geomechanical classification (performed using a probabilistic approach that takes into account the variability of parameters) and also considering previous experiences of long deep tunnel projects designed by Geodata Engineering in the same geological context, the longitudinal geological profile represented in Figure 7 has been prepared.

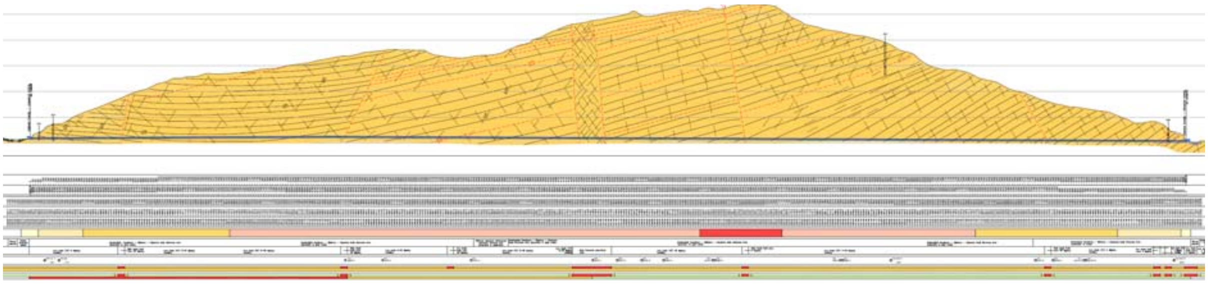


Fig. 7 Longitudinal Geological profile along the tunnel.

Considering the high overburden, the main critical aspect is to define at the scale of the tunnel the relative predominance of one lithology over the other. It's reasonable to expect some different lithostratigaphic configurations at the tunnel face, represented by:

- prevalence of plurimetric sandstone layers across the tunnel face (most representative GSI intervals between 45-65, secondarily >65);
- prevalence of heterogeneous rock mass (decimetric to metric interbedded siltstone/sandstone/claystone in different percentages) across the tunnel face (most representative GSI intervals between 25-45 and secondarily between 45-65);
- an intermediate condition representing the transition between the previous two.

The length of each one of the above described conditions along the excavation axis depends on the angle of intersection between the dip direction of the bedding planes and the tunnel direction. Faults and joints are frequently preferred paths for groundwater movement and consequently internal erosion or significant wall-rock alteration can occur. So along the expected fault areas or the main high fractured sectors GSI values <25 are expected.

In accordance with the method suggested by Russo (2009), four GSI groups' variability have been distinguished: 1st group (GSI>65): undisturbed rock mass consisting of cubic blocks with few fresh or slightly weathered discontinuities (Structure Blocky/Massive); 2nd group (GSI=45-65): partially disturbed rock mass with multi-faceted angular blocks formed by 4 or more joint sets; the joint surfaces are slightly/moderately weathered (Structure Blocky/ Very Block); 3rd group (GSI=25-45): tectonic disturbed rock mass with many intersecting joint sets: the joint surfaces are moderately/high weathered (Structure Blocky-Seamy); 4th group (GSI < 25) (Poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces; highly weathered joint surfaces with compact or soft fillings).

4 TUNNEL DESIGN CRITERIA

4.1 Excavation Behaviors, Correlation with NATM Classes and Support Classes Definition

The behaviour classification of the rock mass has been performed using the quantitative approach proposed by Russo and Grasso (2007). The main advantage of the proposed probabilistic approach, whose steps are schematically illustrated in Figure 8, is that categorization of the deformational behavior is based on the results of the ground convergence curve with a relatively high degree of reliability.

The quantitative approach adopted in this study takes into account the following input data:

- geomechanical parameters of rock mass and intact rock (GSI, RMR, σ_{ci}, m_i, etc.)

- other parameters as in situ stress, overburden and geometry of the cavity (K_0, H, R_0)

The processing of these input data through a Monte Carlo simulation allows definition of 6 types of Behavioral Categories (BC) as function of both the radial deformation at the excavation face (δ_0) and the normalized extension of the plastic zone around the cavity (R_P/R_0).

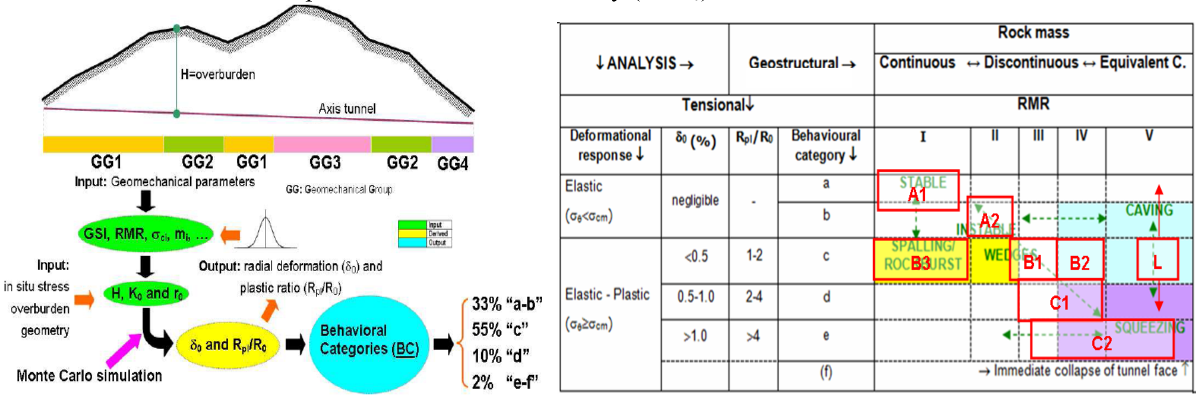


Fig. 8 Definition of the quantitative approach by Russo et al. (2007) and excavation behaviour obtained as a result of stress and geostructural analyses.

In the proposed quantitative approach, the potential geomechanical hazards occurring along the tunnel were identified on the basis of both the behavioral categorization (based on the quantification of indexes such as deformation at tunnel face and extent of the plastic zone) and the well-known RMR system (Bieniawski, 1989)., correlated with empirical relationship with the self supporting capacity of rock masses (Figure 8b). Starting from this matrix it's possible to define a correlation between the rock class description (A1 to L) as per NATM qualitative approach (Rabcewicz, 1964-65 and Pacher, et al., 1974).) and the behavior categories (a to f) obtained with the quantitative approach proposed by Geodata Engineering (Russo & Grasso, 2007), as shown in Table 1.

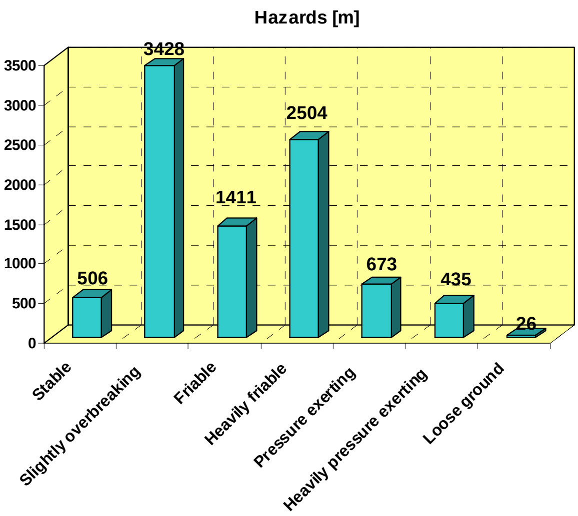
Table 1 Correlation among rock class description as per NATM and behavioural categories as per Geodata Engineering approach

Rock class description (NATM) Qualitative approach		Russo & Grasso, 2007 (GEODATA) Quantitative approach	
Class	Description	Stress analysis (behavioural cat.)	Geostructural condition (typical RMR class)
A1	Stable	a/b	I
A2	Slightly overbreaking		II
B1	Friable	c	III
B2	Heavily friable		IV
C1	Pressure exerting	d	III-IV
C2	Heavily pressure exerting	e	III-IV-V
L	Loose ground	(f)	V

4.2 Definition of the Support Classes

The excavation behavior classes (related with the geomechanical groups) allow the selection of the appropriate support to be installed in the tunnel relying on previous experience in similar tunnlling conditions. Figure 9 shows the length of application of section types for each homogeneous geomechanical zone along the tunnel. Almost 80% of tunnel excavation could be affected by over-breaking and friable and heavy friable rock mass behaviors. In correspondence of the fault zones potential risk of squeezing (pressure exerting, first predicted with the method proposed by Hoek and Marinos, 2000) is expected and the excavation will be carried out implementing section types C1 and C2 with yielding support (lining stress controllers) as shown in Figure 10. It's clear that the behavior of the rock mass will be strictly connected to the prevalent lithology (sandstone ore siltstone/claystone) encountered during the excavation.

Chainage		Zone	D [-]	H [m]	Main lithology	Direction of excavation	Bedding planes in regard of longitudinal section of tunnel	Section type and behaviour classes - Rpl/R0 (m)							
from	to							A1	A2	B1	B2	C1	C2	L1	
								Stable	Slightly overbreaking	Friable	Heavily friable	Pressure exerting	Heavily pressure exerting	Loose ground	
								a/b		c		d	e	f	
1280	1355.94	1	76.3	0÷60	Soil / Loose material		favorable	0.0	0.0	0.0	59.3	0.0	0.0	17.0	
1356	1701.94	2	346	60÷200	Alternating sandstone-siltstone-claystone		favorable	76.8	99.3	72.7	97.2	0.0	0.0	0.0	
1702	1973.94	3	272	200÷310	Alternating sandstone-siltstone-claystone		favorable	56.3	77.8	65.3	72.6	0.0	0.0	0.0	
1974	2023.94	4	50	310÷325	Shear zone		favorable	0.0	0.0	4.3	3.1	16.5	26.2	0.0	
2024	2840.94	5	817	325÷500	Alternating sandstone-siltstone-claystone		irrelevant	85.0	306.4	201.0	213.2	11.4	0.0	0.0	
2841	3702.94	6	862	500÷630	Alternating sandstone-siltstone-claystone		irrelevant	33.6	391.4	173.3	222.4	41.4	0.0	0.0	
3703	3752.94	7	50	630	Shear zone		unfavorable	0.0	0.1	0.6	0.0	11.2	38.3	0.0	
3753	4529.94	8	777	630÷685	Alternating sandstone-siltstone-claystone		unfavorable	18.6	356.7	128.2	228.4	45.1	0.0	0.0	
4530	4579.94	9	50	625÷630	Shear zone		unfavorable	0.0	0.0	0.5	0.0	11.6	38.0	0.0	
4580	5499.94	10	920	625÷880	Alternating sandstone-siltstone-claystone		unfavorable	10.1	446.2	125.1	238.3	100.3	0.0	0.0	
5500	5803.94	11	304	880÷940	Shear zone		unfavorable	0.0	0.0	0.0	0.0	44.1	259.9	0.0	
5804	6488.94	12	685	940÷990	Alternating sandstone-siltstone-claystone		favorable	0.0	319.3	53.4	206.2	106.2	0.0	0.0	
6489	6815.94	13	327	990÷1055	Alternating sandstone-siltstone-claystone		favorable	0.0	154.0	21.3	94.2	57.6	0.0	0.0	
6816	6865.94	14	50	1050÷1055	Shear zone		favorable	0.0	0.0	0.0	0.0	5.3	44.7	0.0	
6866	7127.94	15	262	1000÷1050	Alternating sandstone-siltstone-claystone		favorable	0.0	119.2	16.0	81.7	45.1	0.0	0.0	
7128	8627.94	16	1500	500÷1000	Alternating sandstone-siltstone-claystone		favorable	19.5	708.0	223.5	397.5	151.5	0.0	0.0	
8628	9162.94	17	535	310÷500	Alternating sandstone-siltstone-claystone		favorable	57.8	215.1	115.0	142.3	4.8	0.0	0.0	
9163	9212.94	18	50	310÷355	Shear zone		favorable	0.0	0.0	4.4	3.0	14.6	28.1	0.0	
9213	9728.94	19	516	200÷355	Alternating sandstone-siltstone-claystone		irrelevant	88.2	160.5	118.7	147.6	1.0	0.0	0.0	
9729	10010.9	20	282	150÷200	Alternating sandstone-siltstone-claystone		irrelevant	60.3	74.2	69.9	77.6	0.0	0.0	0.0	
10011	10148.9	21	138	80÷150	Alternating sandstone-siltstone-claystone		irrelevant	0.0	0.0	18.1	113.4	5.4	0.0	1.1	
10149	10262.2	22	113	0÷80	Alternating sandstone-siltstone-claystone		irrelevant	0.0	0.0	0.0	104.8	0.0	0.0	8.5	
								506	3428	1411	2504	673	435	25.5	8982.6
								6%	38%	16%	28%	7%	5%	0%	100%



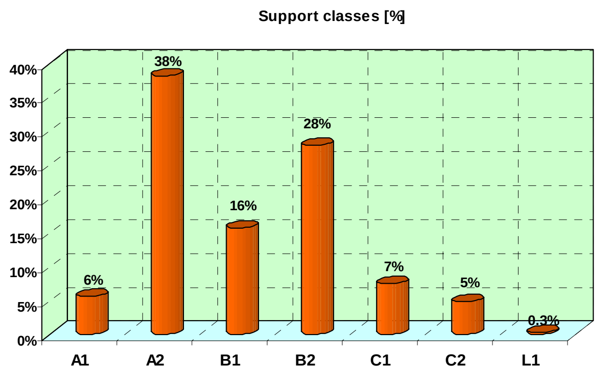


Fig. 9 Behaviour rock classes (hazards) and support sections definition for the Chenani-Nashri Tunnel.

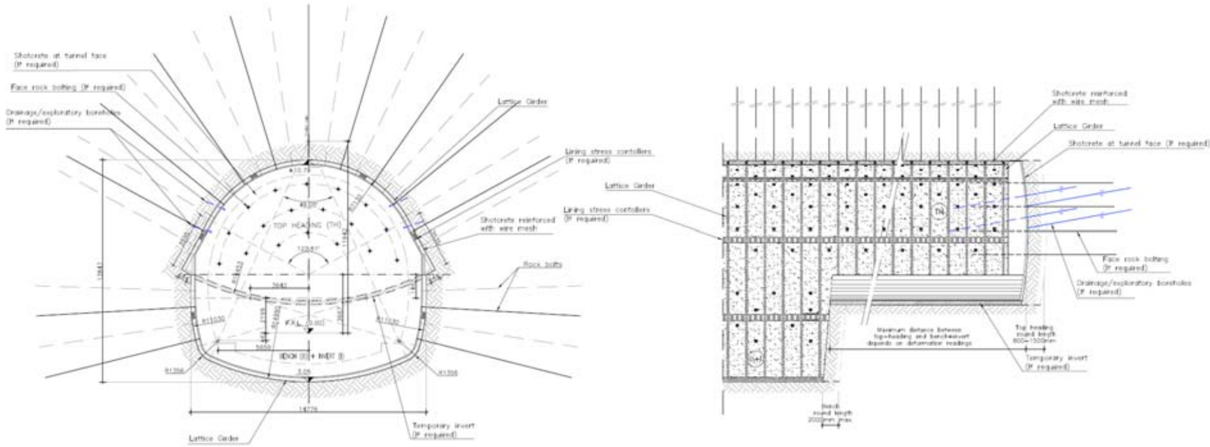


Fig. 10 Support class C2 applied in severe squeezing conditions (heavily pressure exerting).

4.3 Primary Support and Final Lining Dimensioning

Empirical, analytical (ground reaction curves) and numerical methods (FEM analyses) have been used to verify the adequacy of the supports. In particular, for the evaluation of the confinement effect (silo effect) in correspondence of the fault zones axisymmetrical FEM analyses have been performed. This approach allowed calibration of the plain strain numerical model in the fault zone (Figure 11).

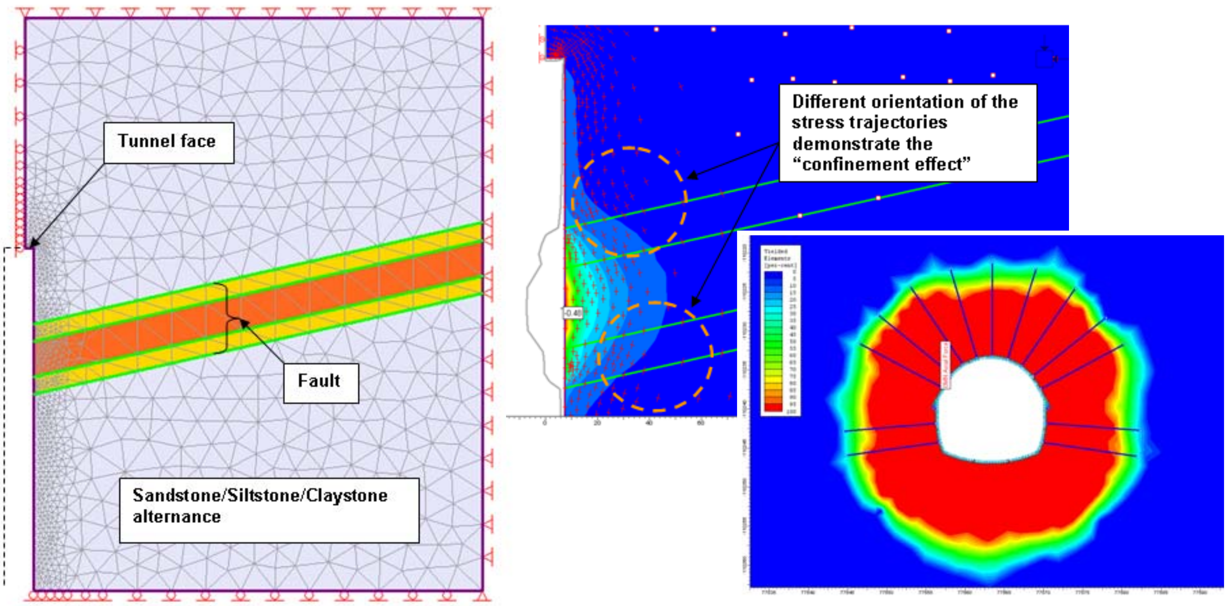


Fig. 11 FEM analyses for evaluation of the "silo effect" in the fault zones.

Final lining verification has been conducted without considering the contribution of the primary support, due to the presence of the waterproofing membrane that constitutes a structural separation between the temporary and inner lining and prevents shear stress to be transmitted across. Different criteria have been applied to evaluate the acting load on the final lining depending on elastic or plastic behavior of the rock mass and according to the overburden conditions: shallow (portal zones) or deep. Structural verification has been done using the hyper-static reactions method (Bedded Beam Mode) to simulate the ground-structure interaction. Seismic verifications have been done (using the availing effect method) only for section applied at portals zones, where the seismic effect is not negligible (project location lies within seismic zones IV and V of the seismic zoning map of India).

5 PORTALS AND SPOIL DUMP AREAS DESIGN

The design of portals and spoil dump areas has been probably the most critical aspect due to the environmental constrains (severe weather phenomena during Monsoon Seasons) and critical geological conditions. The portals' geometry has been defined in such a way to provide enough space to accommodate all buildings and facilities required for the operation of the tunnel, in compliance with the ROW limits. As the portals areas (both South and North) are on slopes bounded by "nallahs" on the downhill side, significant excavations have to be formed uphill to obtain the required space.

The South Portal area location, in particular, is characterized by the presence of colluvial deposits with a thickness locally in excess of 20m. For this superficial layer of eluvial-colluvial material an apparent cohesion can be appreciated due seasonal variation of the degree of saturation of soil : in reason of its spatial variability in terms of composition and its lack of structure the apparent cohesion is considered to vary between 0 and 40kPa, decreasing and disappearing in the superficial portion. When full saturation reduces this suction, slip surface may form. A back analysis of a limited land slip occurred during the construction of a temporary road further to intensive rain events in February 2011 was carried out for a lower bound estimation of the geotechnical parameters. The values were than adopted in the design.

The lower portion of the slope in bedrock has been designed with steeper angles (75°), while the upper portion in eluvial-colluvial deposits is cut back at slope varying from 45° to 60° locally with 4m wide berms provided every 15m of height. Slope protection at South Portal includes a shotcrete layer 150mm thick reinforced with wire mesh, self drilling bolts (IBO type) and drainage pipes as shown in Figure 12 below.

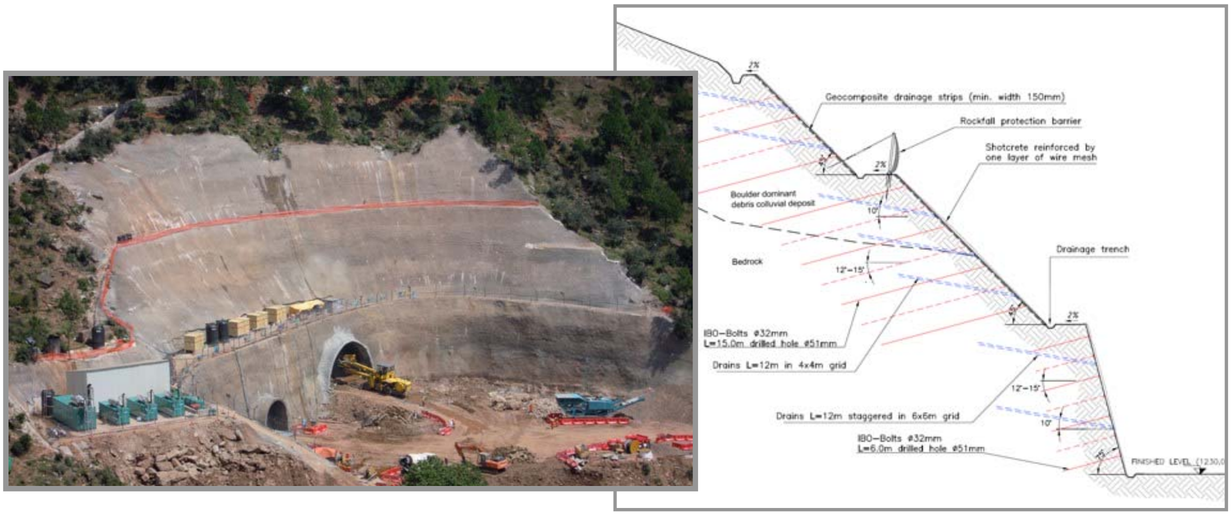


Fig. 12 South Portal construction works and slope protection detail.

Location of the North Portal excavation is composed by shallow layer of eluvial-colluvial deposit and boulder debris above the bedrock. Considering this geometry, the lower portion of the slope in bedrock is formed with steeper angles (75°) and application of systematic bolting (SN-bolts) and rock net, while the upper portion in soil is formed at 60° angles. In soil the slope protection is the same as per South Portal.

For the design of Spoil Dump Areas the guiding criterion in defining the position of the retaining structures (gabion walls or earth retaining walls) has been to remain beyond the hydraulic flood limits determined for the 100-year return storm event. Both Spoil Dump Areas are located in narrow and steep-sided valleys with nallahs characterized by high values of hydraulic gradients above all upstream of the existing NH-A1 bridge and the valleys excavated by the rivers are quite.

The South Dump Area (SD-02) is located approximately 3 km south of the South Portal, at km 86 of the NH-1A. The SD-02 design has been developed taking into account an approximate planned capacity of 695.000 m^3 of spoil material, and is able to cater for the disposal of at approximately 30% of the entire volume of material resulting from the excavation of portals and tunnel. Final configuration of the dump site is shown in Figure 13. Gabion walls provide flexible retaining structures to hold back the disposed muck material. As a further toe protection against erosion and scour, a cemented boulder apron was included in front line of the lower portion of the gabion wall.

The North Dump Area (ND-01) is located between the Tunnel North Portal and the end of the project chainage, at 130km and in developed with the same guiding criterion of remaining outside the 100-year return flood limit. The Dump Area will be have a flat ground on top providing space for Toll Plaza, other technical and ancillary buildings and parking areas for the new NH-1A. This main constrain calls for very high retaining structures, locally up to 40m in order to obtain the required flat area for the installation of all facilities. This dump area should cater for about 50% of the muck extracted during the tunnel excavation.

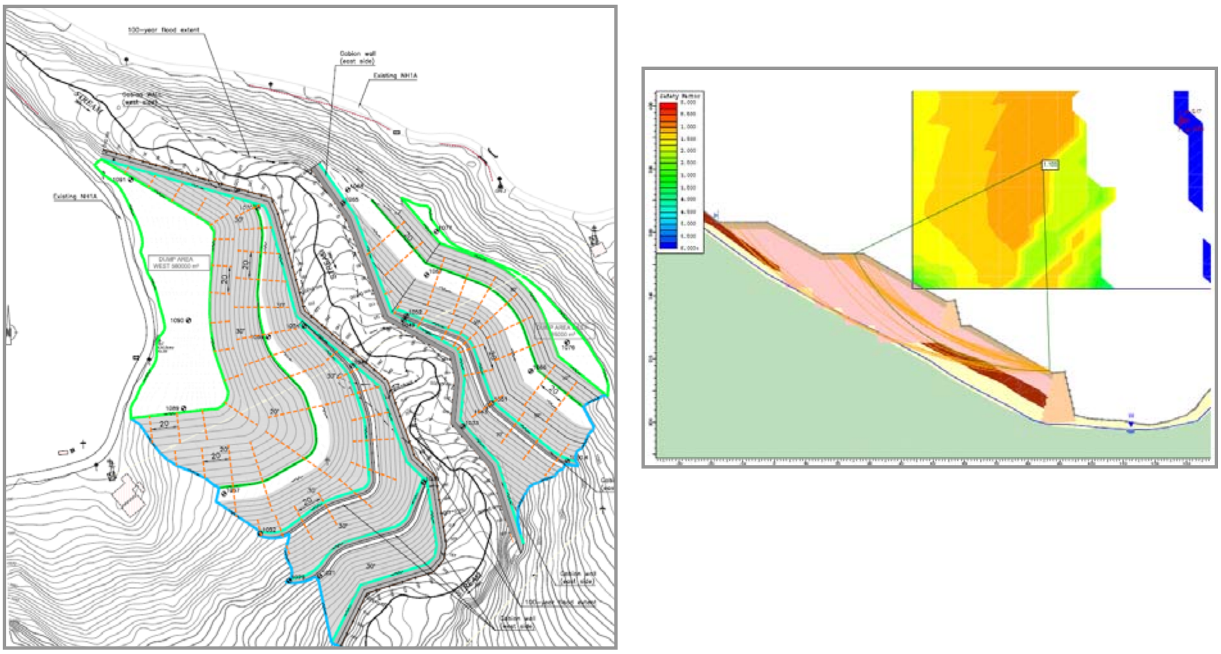


Fig. 13 Detailed design of the South Dump Area.

6 CONCLUSIONS

The design and construction of the Chenani-Nashri Tunnel is challenging experience due to the many aspects characterizing the essence of an "integrated project". Particular attention has been devoted to considerable critical issues in the design, including:

- choice of technical feasible solutions which are compatible with the local geographic context, available material and equipment;
- detailed probabilistic risk analysis for the evaluation of potential hazards which may occur during the excavation of the tunnel in a challenging geological context;
- evaluation of functional and safety aspects for definition of the project lay-out;
- design of surface works structures with respect to project requirements as well as environmental constrains.

REFERENCES

[1] Bieniawski, Z.T. (1989). Engineering Rock Mass Classification, *John Wiley & Sons.* New York. 251 pp.

[2] Hoek, E and Marinos, P. (2000). Predicting tunnel squeezing problems in weak heterogeneous rock masses, *Tunnels and Tunnelling International*, Part 1, 32(11), 45-51; Part 2, 32(12), 33-36.

[3] Pacher, L., Rabcewicz, L.V. and Golser, J. (1974). *Zum derzeitigen der Gebirgsklassifizierung in Stollen und Tunnelbau*, Reference 1, S1-8.

[4] Rabcewicz, L.V. (1964-65). *The New Austrian Tunnelling method*, Water Power, London, Vol. 16, m. 11-12 and Vol. 17 n. 1.

[5] Russo, G. and Grasso, P. (2007). On the classification of rock mass excavation behaviour in tunnelling, *11th Congress of International Society of Rock Mechanics ISRM*, Lisbon.

[6] Russo, G. (2009). A new rational method for calculating the GSI, *Tunnel. Underground Space Technol.* 24:103-111.