Geological and geotechnical surveys in tectonically active rock masses

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ABSTRACT: The excavation of tunnels in rock masses in which are identified characteristics of tectonic activity requires the adoption of special geological and geotechnical survey procedures. This particular attention arises from the need to prevent any risk of undesirable behaviors during to the excavation works and its following impact on costs and deadlines. Under these conditions is the design of some tunnels in India, interfering with the rock masses assigned to the wrinkling of the Himalayas, where they will be excavated. In this article are discussed the major problems that were put to the geological reconnaissance and the guidelines followed on the geotechnical characterization and surveys, as well as the presentation of the geological and geomechanical models followed in the design studies.

1 INTRODUCTION

The Kiratpur – Ner Chowk section of NH-21 Project comprises 5 two lane tunnels, being this paper referring to Tunnel 1. This tunnel will have 1.836 m in length and, in order to respond to the international recommendations for emergency situations, it is also planned an escape tunnel, which will develop in parallel to the main tunnel gallery and connects to it by several linking galleries.

The Study Area is Located in the district of Bilaspur, state of Himachal Pradesh in the Northwest of India. The implementation site of Tunnel 1 is given by Figure 1.

The aim of this paper is to present the criteria that led to the definition of the Geotechnical Investigations to carry out, analyse its results and establish a correlation between the results of the geotechnical investigations and the geomechanical parameters, considered in the calculations, which allowed the design of the tunnel.

2 GEOTECHNICAL INVESTIGATIONS

Ground conditions including geological, geotechnical, and hydrological conditions, have a major impact on the planning, design, construction and cost of a road tunnel, and often determine its feasibility and final route. Subsurface investigations are the most important type of investigation to obtain ground conditions, as it´s the main technic for:

- Defining the subsurface profile (i.e. stratigraphy, structure, and principal soil and rock types);
- Determining soil and rock material properties and mass characteristics;
- Identify geological anomalies, fault zones and other hazards (squeezing soils, methane gas, etc.);
- Defining hydrogeological conditions (groundwater levels, aquifers, hydrostatic pressures, etc.); and
- Identifying potential construction risks (boulders, etc.).

Subsurface investigations typically consist of borings, sampling, in situ testing, geophysical investigations, and laboratory material testing. The main purpose of these investigation techniques are summarized below:

- Borings are used to identify the subsurface stratigraphy, and to obtain disturbed and undisturbed samples for visual classification and laboratory testing;

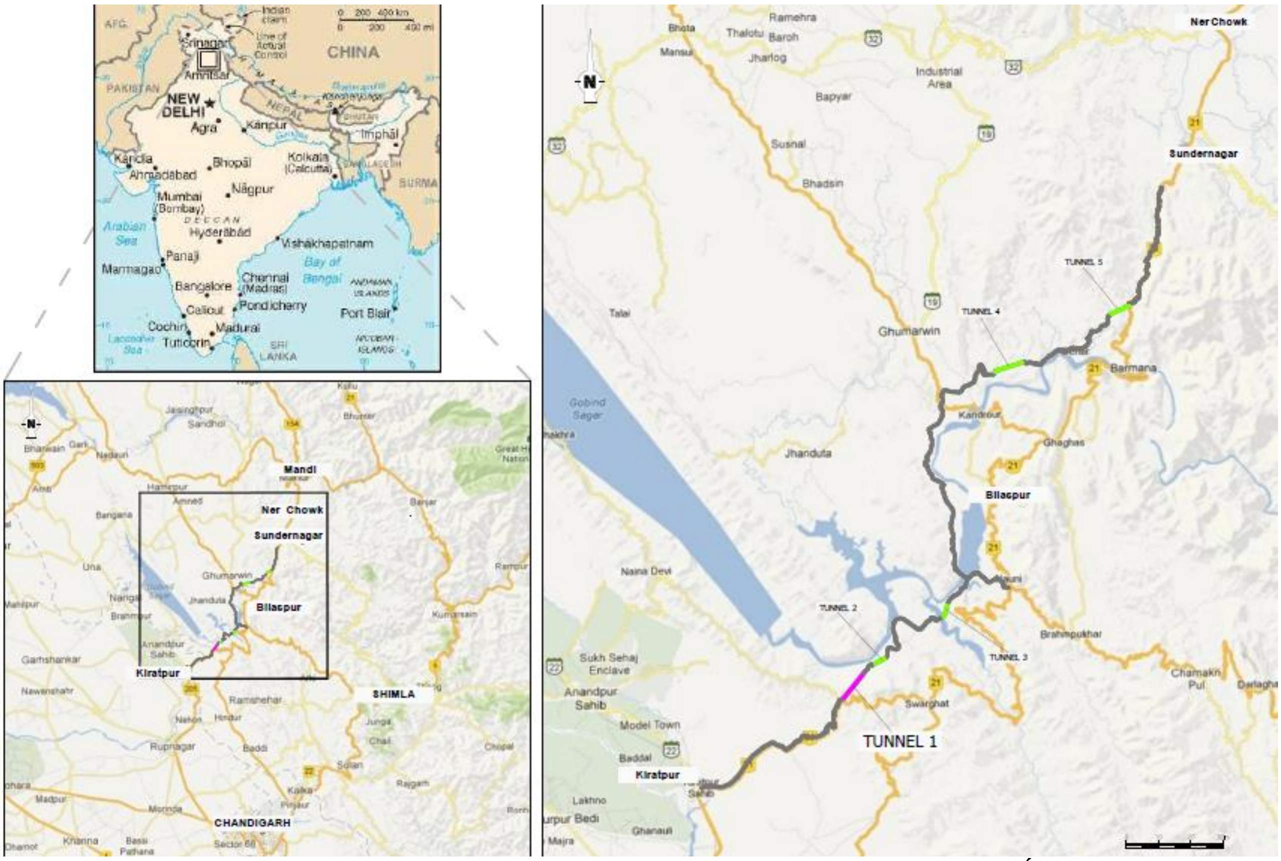

Figure 1.Location of Tunnel 1, district of Bilaspur, state of Himachal Pradesh, Índia.

- In situ tests are commonly used to obtain useful engineering and index properties by testing the material in place, in order to avoid the disturbance inevitably caused by sampling, transportation and handling of samples retrieved from boreholes; in situ tests can also aid in defining stratigraphy;
- Geophysical tests quickly and economically obtain subsurface information (stratigraphy and general engineering characteristics) over a large area to help define stratigraphy and to identify appropriate locations for performing borings; and
- Laboratory testing provides a wide variety of engineering properties and index properties from representative soil samples and rock core retrieved from the borings.

Field testing for subsurface investigations includes two general categories of tests:

- In situ tests;
- Geophysical testing.

In situ tests are used to directly obtain field measurements of useful soil and rock engineering properties. Geophysical tests, the second general category of field tests, are indirect methods of investigation in which changes in certain physical characteristics such as magnetism, density, electrical resistivity, elasticity, or a combination of these are used as an aid in developing subsurface information. In Tunnel 1 were performed 5 boreholes and 10 Seismic Refraction Tests, Table 1 and Table 2 presents these works.

Table 1. Location of Boreholes - Tunnel 1.

Designation	Depth of Bo-rehole (m)	Chainage
BH1 T1	32	≈ 12+750
BH2 T1	103	≈ 12+930
BH3 T1	116	≈ 14+300
BH4 T1	85	≈ 14+380
BH5 T1	33	≈ 14+480

In Figure 2, is present the location of BH4 T1 with a general view of the area.

Figure 2.Tunnel 1, portal 1 – Borehole 4 Location.

Table 2. Location of Seismic Refraction Tests - Tunnel 1 and Escape Tunnel.

Designation	Description of profile	Profile length (m)
SRT1 T1	semi- transverse profile at entry portal of T1	115
SRT2 T1	longitudinal profile at entry portal of T1	115
SRT3 T1	transverse profile at central part of T1	115
SRT4 T1	longitudinal profile at central part of T1	115
SRT5 T1	transverse profile at exit portal of T1	115
SRT6 T1	longitudinal profile at exit portal of T1	115
SRT7 T1	semi- transverse profile at entry portal of Escape Tunnel	115
SRT8 T1	longitudinal profile at entry portal of Escape Tunnel	115
SRT9 T1	semi-transverse profile at exit portal of Escape Tunnel	115
SRT10 T1	longitudinal profile at exit portal of Escape Tunnel	115

As it was mentioned previous for the final design of Tunnel 1, a geotechnical investigation programme was carried out with the purpose of deciphering the subsurface strata of the tunnel. This programme included 5 boreholes, 10 seismic refraction profiles, Lugeon permeability tests and several laboratory tests.

The main lithologies in the boreholes are sandstones and siltstones ordered in an alternate way. Along the tunnel alignment the rock formation consists mainly of those two lithologies, which are in continuous alternation and in stratigraphic conformity.

In the portal P1, on Kiratpur side, the layer of top soil is predominant above the crown, consisting on cobbles, pebbles and boulders. Under the top soil occurs an interbedded stratum between sandstone and siltstone. The P2 portal, on Ner Chowk side, is widely covered by top soil, consisting on cobbles, pebbles and boulders.

2.1 *"Seismic Refraction Tests"*

In all alignment of Tunnel 1, Seismic Refraction shooting was carried out along 10 profiles by measuring the P-wave Refraction velocity. The maximum value measured for P-wave (Compression Wave Refraction) velocity was 2558m/s.

In the portal P1 of Tunnel 1, four seismic profiles were processed (SRT-1, SRT-2, SRT-7 and SRT-8), in which the P-wave velocity achieved values in a range of 300 to 850m/s in the top soil. In the highly to completely weathered sandstone/siltstone layer the values of P-wave velocity are in a range of 1450 to 1550m/s. At last, in the moderately to highly weathered sandstone/siltstone layer the values of P-wave velocity are around 2400m/s (Figure 3).

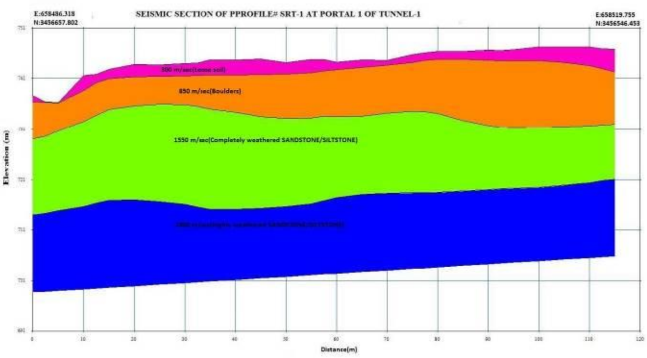

Figure 3.Tunnel 1, portal 1 – seismic section of profile SRT1.

In the tunnel portal P2, also four seismic profiles were processed (SRT-5, SRT-6, SRT-9 and SRT-10). The top soil layer revealed P-wave velocity values in a range of 321 to 1678 m/s, increasing with depth. In the highly to completely weathered sandstone/siltstone layer the range of values is of 1835 to 2558m/s.

In the middle of tunnel alignment, two seismic profiles were processed (SRT-3 and SRT-4), in which the P-wave velocity achieved values in a range of 300 to 329m/s in the top soil. In the highly to completely weathered sandstone/siltstone layer the values of P-wave velocity are in a range of 1608 to 1683m/s. At last, in the highly weathered sandstone/siltstone layer the values of P-wave velocity are around 2425m/s.

2.2 *"In Situ Permeability Tests"*

The results of the core drilling campaign revealed that, in general and at tunnel depth, the rock mass is permeable, in which the hydraulic conductivity is clearly superior by the tunnel portals with values superior than 10 Lugeons. The obtained results in the remaining tunnel alignment were around 3 to 6 Lugeons, which reveals lower permeability than in the tunnel portals.

Table 3.In Situ Permeability Tests performed in Tunnel 1.

Designation of Borehole	Chainage (Km)	Depth (m)		Lugeon Value	
		From	To	Max	Min
BH1 T1	12+750	15,00	32,00	13,76	10,62
BH2 T1	12+930	75,00	88,00	4,34	3,50
BH3 T1	14+300	101,00	116,00	4,24	3,61
BH4 T1	14+380	70,00	85,00	6,06	4,37
BH5 T1	14+480	24,00	33,00	13,55	11,18

Nevertheless, given the weathering and fracturing degrees identified in the removed rock cores of the drilling campaign it was expected to achieve superior Lugeon values, as those parameters describe a high permeable rock mass with greater water absorption capacity through its joints. The summary of results is given by table 3.

2.3 *Laboratory Tests*

Thus, the mentioned laboratory testing programme includes seven laboratory tests types, namely:

- Bulk Density;
- Water Content at Saturation;
- Slake Durability Index;
- Uniaxial Compressive Strength (with Young's Modulus and Poisson´s Ratio);
- Joint Shear Strength Test;
- Point Load Strength Index Test on rock cores;
- Petrography Analysis.

2.3.1 *Bulk Density*

This test method is used to determine the density and specific weight of compacted materials, such as the rock mass occurring in the Tunnel 1. Hence, 32 Bulk Density Tests were conducted and its obtained results are summarized next.

The analysis of the results allowed to identify the range of values for both parameters, which are of 2,50 to 2,69 g/cm^3 for density and of 2,63 to 2,78g/cm^3 for unit weight. With emphasis on the lithology the corresponding maximum and minimum values of density and specific weight are presented in table 4.

Table 4. Maximum and Minimum values of Bulk Density.

Lithology	Density (g/cm3)		Specific Gravity (g/cm3)	
	Min	Max	Min	Max
Sandstone	2,50	2,69	2,63	2,78
Siltstone	2,58	2,69	2,66	2,77

The obtained range of values is quite realistic as it is typical of the rock materials that were detected (sandstone and siltstone), besides the small deviation in the results confirms the existence of a rather homogeneous rock material, implying constant lithology with an interbedded stratum between sandstone and siltstone.

2.3.2 *Water Content at Saturation*

In order to determine the water content at saturation point of the rock materials occurring in the Tunnel 1 there were conducted 32 Water Absorption tests.

It is verified that the values obtained in the Water Absorption Tests are in a range of 0,29 to 3,83%. It is possible to notice that the water absorption of the samples is not linear with depth, that is to say that this parameter of this rock mass doesn't depend on depth, as it doesn't decrease with depth.

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Table 5. Maximum and Minimum values of Water Absorption.

Lithology	Water Absoprtion (%)	
	Min	Max
Sandstone	0,49	2,14
Siltstone	0,29	3,83

The values deviation may be due to the presence or absence of micro-cracks and differences in the porosity between samples. The minerals disintegration and decomposition leads to the growing volume of voids and presence of micro-cracks in the rock mass and as a result the weathering of the rock mass. In this way the values of water absorption will increase with weathering degree and thus the mineralogical structure of rock mass will become even more water sensitive.

In conclusion the mineralogical composition, texture and microstructure of rock mass may have influenced the obtained water absorption results.

2.3.3 *Slake Durability Index*

This test method covers the determination of the Slake Durability Index used to estimate qualitatively the probable amount of deterioration of weak rocks, over a period of time, after simulated exposure to natural wetting and drying cycles. There were conducted 25 Slake Durability Tests with the rock materials occurring in the Tunnel 1.

According to the Gamble's Classification System the rock material specimens are in between medium and very high durability classification.

The lowest values of slake durability percentage correspond to the samples from borehole BH1 T1 revealing medium to medium high durability (between 96,0% and 98,0%). The samples from the remaining boreholes achieved values above 98,0% thus between high and very high durability.

Table 6. Maximum and Minimum values of Slake Durability Tests.

Lithology	Slake Durability Index (%)		Specific Durability Classification	
	Min	Max	Min	Max
Sandstone	97,50	99,25	Medium High	Very High
Siltstone	96,00	99,25	Medium	

2.3.4 *Uniaxial Compressive Strength (UCS)*

In order to determine the Uniaxial Compressive Strength (UCS), Young's Modulus (E) and Poisson's Ratio (μ) of rock materials occurring in the Tunnel 1, 17 UCS tests were carried out in which 13 of those the Young's Modulus and the Poisson's Ratio were obtained.

The Tables 7, 8 and 9 summarize the results obtained, regarding lithology.

Table 7. Maximum and Minimum values of UCS.

Lithology	Uniaxial Compressive Strength (MPa)	
	Min	Max
Sandstone	24,43	79,97
Siltstone	30,74	88,37

Table 8. Maximum and Minimum values of Young´s Modulus.

Lithology	Young´s Modulus (GPa)	
	Min	Max
Sandstone	3,95	21,42
Siltstone	8,25	22,95

Table 9. Maximum and Minimum values of Poisson´s Ratio.

Lithology	Poisson´s	
	Min	Max
Sandstone	0,16	0,33
Siltstone	0,08	0,33

The obtained values for UCS are in a range of 24,43 to 88,37 MPa which correspond to a medium to medium low strength. These results match with the expected ones taking into account the type of rock mass in study.

Referring to the Young's Modulus and the Poisson's Ratio results it was obtained values in a range of 3,95 to 22,95 GPa and of 0,08 to 0,33, respectively. The presented values for Young's Modulus are considered rather high in relation to the prevailing lithologies detected in the tunnel alignment. In the same way, the values obtained for Poisson's Ratio revealed a deviation unlikely to the type of lithologies under study due to showing a larger range of values than first expected.

In relation to the Young's Modulus, it is clear that siltstone reached the highest rate of the minimum values (8,25GPa) while sandstone kept the lowest minimum value (3,96GPa). Also, siltstone achieved the upper limit of maximum values with 22,95GPa.

The minimum values for Poisson's Ratio are in a range of 0,08 (siltstone) to 0,17 (sandstone in the top soil). The corresponding maximum values are in a range of 0,29 for sandstone with boulders/cobbles to 0,33 for both sandstone and siltstone.

2.3.5 *Joint Shear Strength Test*

In order to determine the cohesion and the friction angle of the rock materials occurring in the Tunnel 1, a Joint Shear Strength test was conducted using a sample containing joint planes. The small number of tests is due to the reduced quality of the samples

The results of the Joint Shear Strength test are presented below.

Table 10. Results of Joint Shear Strengh performed in Tunnel 1.

Designation of Borehole	Core Piece Number	Joint Shear Strength Test	
		C (Kg/cm2)	ϕ (º)
BH4 T1	44A-44B	0,00	17,30

2.3.6 *Point Load Strength Index*

This test method is performed to determine the Point Load Strength Index of rock specimens and it is used as an index test for strength classification of rock materials. There was carried out 22 Point Load Strength Tests with uniaxial compression using the rock materials occurring in the Tunnel 1.

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Table 11. Maximum and Minimum values of Point Load Strength Index.

Lithology	Point Load Strength Index (MN/m^2)	
	Min	Max
Sandstone	2,56	5,35
Siltstone	2,96	6,82

The obtained results exhibit some deviation as they range from 2,56 to 6,82MN/m², this reveals a variation in strength index of the rock specimens that may be explained by their lithology diversity and therefore by their variation in strength classification with regards to geomechanical terms. By calculating the mean value of tests results it is obtained 4,23MN/m² which corresponds to a rock hardness of medium strength.

With emphasis on the lithology of the rock mass, it is clear that sandstone obtained the lowest values for both minimum (with 2,56MN/m^2) and maximum (with 5,35MN/m^2) limits of the Point Load Strength Tests. The minimum values of siltstone (2,96MN/m^2) reveal close proximity though siltstone exhibits the highest maximum value of the Point Load Strength Tests with 6,82MN/m^2.

It is noteworthy to mention that the correlation between the results of the Point Load Strength Index and the ones of the Uniaxial Compressive Strength was verified with the Indian Standard formula represented by the relation UCS = 22*IS_{50}. The values of UCS obtained from the Indian Standard formula were considerably higher than the ones obtained in the performed laboratory tests. The values obtained in the laboratory tests are clearly considered more realistic. In this case the correlation between the results of both laboratory tests has a multiplication factor lower to the one presented in the formula (22). The multiplication factor needed to apply the Indian Standard formula would take values in a range of 8,3 to 18,8.

2.3.7 *Petrographic Examination*

In order to study the petrology and mineralogy of the rock materials occurring in the Tunnel 1 it was performed 10 examines of petrographic thin sections from the rock mass.

The obtained data in the petrological and mineralogical analyses reveal that sandstone is the predominant rock type and so its main minerals prevail, such as quartz, feldspar, calcite and occasionally mica.

The sandstone matrix content shows variations between iron/silica content and carbon content. Relating to the texture it has shown a fine to medium grained sandstone. It should be mentioned that the two samples from BH5 T1 borehole identified as top soil sandstones revealed a carbon matrix content in the microscopic analysis.

3 GEOTECHNICAL ZONING

The Rock Mass Geotechnical Zoning here submitted is based on the "Austrian Guideline for the Geotechnical Design of Underground Structures (Austrian Society of Geomechanics, 2010)".

As per it is mentioned in the Austrian Guideline for the Geotechnical Design of Underground Structures with Conventional Excavation (Austrian Society of Geomechanics, 2010) the rock mass behaviour is determined for each rock mass type by evaluating the effect of the influencing factors on the response of the rock mass with the full excavation geometry.

Associated to the definition of the different rock mass behaviour types it was applied the Geomechanics Classification or the Rock Mass Rating (RMR). This system was developed by Bieniawski in 1973. For this study the classification was based upon the version (Bieniawski, 1989) of the classification system.

It was carried out the classification of the rock mass, based in the Geotechnical Investigations held. The RMR divides the rock mass in 5 classes as presented in table 12.

Table 12. Rock mass classes determined from total ratings.

Rating	100-81	80-61	60-41	40-21	<21
Class Number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor Rock	Very poor rock

This classification, RMR, was performed on all boreholes in 5 meter lengths; consequently the results of the classification of the rock mass are presented in the table below.

Table 13. Determined RMR classes in the Drilling Phase.

Depth	Rock Mass Classes Driling Phase (December 2012 to April 2013)				
	BH-01	BH-02	BH-03	BH-04	BH-05
0-5	V	V	V	V	IV
5-10	V	V	III	V	IV
10-15	IV	IV	III	IV	IV
15-20	IV	III	IV	IV	V
20-25	IV	V	III	IV	V
25-30	II	IV	III	III	V
30-35	II	IV	IV	IV	V
35-40		IV	III	IV	
40-45		III	IV	III	
45-50		V	IV	IV	
50-55		V	III	IV	
55-60		IV	III	IV	
60-65		IV	IV	III	
65-70		IV	IV	II	
70-75		IV	IV	III	
75-80		IV	III	II	
80-85		IV	IV	III	
85-90		IV	IV		
90-95		IV	II		
95-100		IV	IV		
100-105		IV	IV		
105-110			III		
110-115			IV		
115-116			IV		

3.1 *NATM Rock Mass Classes*

Based in the "Rock Mass Behaviour Types" three Geotechnical Zones were defined, which will allow estimating the type of support to install and to define the method according to which the tunnel is going to be excavated. The three Geotechnical Zones are the following:

- ZG1 – RMBT 3/1;
- ZG2 – RMBT 3/2;
- ZG3 – RMBT 5/11 and 8/11.

After this definition it was established a correlation between the three Geotechnical Zones and the Support Classes, based in ONORM B 2203. In table 14 it is presented the support classes considered in the design and its characteristics.

It were added the following support sub-classes, according to the overburden and the geotechnical characteristics of the rock mass:

Main Tunnel

C1-A – Support class C1 with overburden less than or equal to 200 meters;
C1-B – Support class C1 with overburden more than 200 meters;
C2-A – Support class C2 with overburden less than or equal to 200 meters;
C2-B – Support class C2 with overburden more than 200 meters.

Escape Tunnel

C2-A – Support class C2 with overburden less than or equal to 200 meters;
C2-B – Support class C2 with overburden more than 200 meters.

Table 14. Correlation between Support Classes and Geotechnical Zones.

Geotechnical Zone (ZG)	Rock Mass Behaviour Type (RMBT)	Support Class (ONORM B 2203)
ZG1	RMBT 3/1	A1 Stable
		A2 Slightly Overbreaking
ZG2	RMBT 3/2	B1 Friable
		B2 Very Friable
ZG3	RMBT 5/11	C1 Squeezing
		C2 Heavily Squeezing
	RMBT 8/11	L Loose Ground

4 CONCLUSIONS

The estimated Geomechanical Parameters of Rock Mass, used in the design calculations, were based on the following:

- Geotechnical investigations;
- Laboratory tests;
- Rock Mass Rating.

The calibration of these parameters was made using **RocLab,** a software program for determining rock mass strength parameters, based on the generalized Hoek-Brown failure criterion. This calibration was not performed for the support class L because this support class is not considered in the rock mass, is in the field of the soil mass.

The performed calibration leads to the estimated Geomechanical Parameters, which are summarized in Table 15.

Table 15. Geomechanical Parameters.

Geotechnical zone	Support class	RMR_{89}	RMR (Rock Class)	GSI	Natural density (kN/m^3)	ϕ (º)	c (kPa)	ν	Ed (GPa)
ZG1	A1	≥ 61	I-II	≥ 56	25-27	> 70	400-450	0,2 - 0,3	12-20
	A2	60-51	III	55-46	25-27	50-70	350-400	0,2 - 0,3	5-12
ZG2	B1	50-41	III	45-36	22-25	35-49	300-350	0,2 - 0,3	3-5
	B2	40-31	IV	35-26	22-25	20-35	250-300	0,2 - 0,3	1-3
ZG3	C1	30-20	IV	25-15	20-22	20-10	100-250	0,2 - 0,3	1-0,5
	C2	<20	V	< 15	20-22	< 10	100-250	0,2 - 0,3	1-0,5
	L	N/A	N/A	N/A	19-21	N/A	< 100	0,2 - 0,4	< 0,5

These parameters will be continuously calibrated with the advancement of the excavation of the tunnel. The values presented are the result of the interpretation of laboratory tests, in consonance with the RMR classification, taking always in to account the performed geotechnical investigations. The Hoek-Brown failure criterion calibrated the obtained values.

The area where the tunnel will be executed, being tectonically active, results in particular geotechnical behavior of the rock mass.

Actually, the compressive tectonic, typical of the sub-Himalayas area, is controlled by a mesh of reverse faults. The influence of the compressive movements, during the construction of the tunnels is felt primarily on the increase of the expected deformations, which may lead to occurrences such as squeezing and rock burst.

These deformations should be controlled through the instrumentation to install inside the tunnel, during the construction phase.

Additional preventive measures could also be adopted to control the tunnel deformations, such as Lining Stress Controllers (LSC) or relief holes.

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