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Stability Assessment and Support Design of a Tunnel Excavation in Different RockMass Conditions

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Abstract: Presently, India is at the cusp of tunneling revolution as it can be witnessed through several ongoing and upcoming tunneling projects, especially in the Himalayan region. It is a critical phase of any project to carry out stability assessment of a tunnel excavation prior to the actual excavation work considering different rock mass conditions pre-existing in hilly terrains. For this, a detailed numerical analysis has been performed based on the often observed common features of tunneling projects previously and presently being carried out in the Himalayas. The purpose of this study is to emphasize the need of a parametric analysis for tunnel excavations and their support design in Phyllite rock mass which is commonly found in the North-Western Himalayas having three different geological surface conditions based on the Geological Strength Index (GSI) values. A D-shaped tunnel with two different overburden heights have been modelled and analyzed using the two dimensional finite element method. The deformation at the tunnel opening and global Factor of Safety values have been computed. Further, the most vulnerable cases have been re-analyzed with a bolting and shotcrete support scheme to check the enhancement in its stability as well as to restrict the crown displacement within the permissible limits.

Keywords: North-Western Himalayas; Tunnel excavation; Parametric analysis; FEM; FoS; Support measures.

1 Introduction

The Himalayan mountain ranges exhibit heterogeneous lithological rock formations and anisotropic rock mass behaviour due to presence of numerous unconformities, including faults, fractures, and joints. Difficult topography, challenging geology, unfavorable tunnel orientation and sometimes non-availability of a suitable excavation technology often cause instability issues during tunnel constructions in mountainous environments. The various ground conditions experienced during tunnel excavations are namely squeezing, rock burst and swelling etc., while tunnel roof collapse, cavity formations and water infiltration are some of the major concerns (Goel et al., 1995) [1].

Till date, many research works have been carried out to assess the stability of the tunnel sections, tunnel portals, and tunnel with support measures. For stability assessment of tunnels and slopes, the rock mass classification systems are often used in-prior. Both empirical and numerical approaches are frequently used to judge tunnel stability

(Zhang et al., 2022) [2]. The design and modeling of tunneling and underground excavation can benefit from techniques like geotechnical indices and rock mass classification (Bieniawski 1993[3]; Palmström and Stille 2007[4]; Barton 2007[5], 2012[6]). Hoek and Brown (1997) [7] established the Geological Strength Index (GSI) system, which is an easy, rapid, and reliable approach based on the visual evaluation of geological conditions. Using the Hoek–Brown failure criterion, Suchowerska et al. (2012) [8] presented the stability chart for rectangular cavities and predicted the displacement of the tunnel roof. Azad et al. (2022) [9] applied empirical methods such as Rock Mass Rating (RMR), Tunneling Quality Index (Q-system), and NATM rock mass for determining ground conditions and providing proper support measures during the tunnel construction stage. Kaya et al. (2011 [10], 2017 [11]) carried out geotechnical investigations and provided a remediation design for the failure of the tunnel portal section. Aygar and Gokceoglu (2020) [12] studied the potential failures of portal slopes while excavating tunnels and revealed their impact on the tunnel support system. Based on the Convergence–Confinement Principle, Liu et al. (2018) [13] performed the stability analysis of two parallel closely spaced tunnels and used the NATM method for the support design. Kockar and Akgun (2003) [14] proposed a design methodology in mixed schist, phyllite, and limestone conditions for tunnels and portals. Qiu et al. (2020) [15] discussed the mechanism of rock deformation around a shallow tunnel using in-situ monitoring data and made recommendations for actions that may be adopted to limit tunnel collapse risk and control rock deformation. In Indian Himalayas, studies have been published mostly on road-cut slopes and hydro-power project tunnels. Naithani et al. (2009) [16] presented a geological and geotechnical investigations of Loharinag–Pala hydroelectric project located in the Garhwal Himalaya, Uttarakhand. The research works of Sarkar et al. (2021) [17] and Siddique et al. (2020) [18] focused on the stability assessment of cut slopes along the National Highway-108 connecting Uttarkashi town to Gangotri temple and the National Highway-94 from Rishikesh to New Tehri, Garhwal Himalaya, Uttarakhand, respectively. The rock type along these highways are consisting of limestone, quartzite, schist, phyllite and metabasic rocks. Along the highway between Rudraprayag to Kedarnath, Singh et al. (2014) [19] performed stability analysis of phyllite and quartzite rock slopes by using two-dimensional limit equilibrium and finite element methods.

Tunnel failures may arise from nearby rockfalls, landslides, liner cracking, and sinking of roadways that often occur in the hilly areas (Fig. 1). As a result, there are significant losses in terms of life, structures, and economy, Wang et al. (2009) [20]. Several other studies have emphasized the significance of on-site monitoring to analyze the deformation of nearby rock mass and suggested actions to ensure safe construction during tunnel excavation.

(a) (b)

Fig. 1. (a) Collapsed tunnel portal at Tapovan Vishnugad hydroelectric plant, Uttarakhand, India (2021); (b) Under-construction tunnel collapse on the Jammu-Srinagar National Highway near Khooni Nallah, in Ramban, India (2022)

Therefore, a detailed pre-excavation study of the tunnel and its portals, particularly in hilly regions, is necessary for the stability evaluation and damage control measures. In the present study, a thorough numerical analysis has been performed based on the frequently observed common characteristics of previously executed tunneling projects and presently being carried out in the Indian Himalayas. The study aims to highlight the necessity of a parametric analysis for tunnel excavations and their support design in the phyllite rock mass, which is commonly found in the North-Western Himalayas, spanning across Uttarakhand, Himachal Pradesh, and Jammu & Kashmir. The methodology adopted and results observed are discussed in the following sections.

2 Methodology

For the present study, the rock mass has been selected as phyllite since it is one of the commonly observed rock masses in the North-Western Himalayas. Accordingly, a database has been prepared based on the information of the tunneling sites in the **S**tates of Uttarakhand, Himachal Pradesh, and Jammu & Kashmir. From the database, it has been observed that the "D" shaped tunnel is a commonly used tunnel shape with overburden heights mostly varying from 50 to 100 m. The numerical models were developed in a two-dimensional finite element software (RS2) with their other dimensions chosen based on mesh sensitivity analysis and by varying model dimensions so that the global safety factor (FoS) of the models do not change significantly based on model dimensions and mesh density chosen**.** The FoS has been computed by using the Strength Reduction Factor (SRF) method. In this study, rock mass strength is selected in accordance with the Geological Strength Index (GSI) values, which represent from blocky with good, rough surface quality to disintegrated rock mass with poor surface quality. For this, GSI values of 25, 50, and 75 have been selected in the model studies. From these GSI values, the Generalized Hoek-Brown criterion parameters ("m_b", "s" and "a") were calculated (Eq. 1 to 4). The obtained results were examined for the displacement at tunnel crown and the global FoS. In addition, the most susceptible cases were selected for further analysis with addition of adequate support measures.

The geological strength index (GSI) is a system of rock-mass characterization that has been developed in engineering rock mechanics to meet the need for reliable input data, particularly those related to rock-mass properties required as inputs into numerical analysis or closed form solutions for designing tunnels, slopes or foundations in rocks. This index is based upon an assessment of the lithology, structure and condition of discontinuity surfaces in the rock mass and it is estimated from visual examination of the rock mass exposed in outcrops, in surface excavations such as road cuts and in tunnel face and borehole cores.

The expression for the generalized Hoek-Brown criteria (Hoek et al. 2002) [21] is expressed as follows:

$$\sigma_1 = \sigma_3 + \sigma_{ci}\left(m_b \frac{\sigma_3}{\sigma_{ci}} + s\right)^a \tag{1}$$

Where, ‘m_b’ is an empirical constant, which depends upon the rock type; and ‘s’ is an empirical constant, which varies between 0 (for crushed rock) to 1 (for intact rock).

$$m_b = m_i \, exp\left(\frac{GSI-100}{28-14D}\right) \tag{2}$$

$$s = exp\left(\frac{GSI-100}{9-3D}\right) \tag{3}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}\right) \tag{4}$$

Where, D is a disturbance factor.

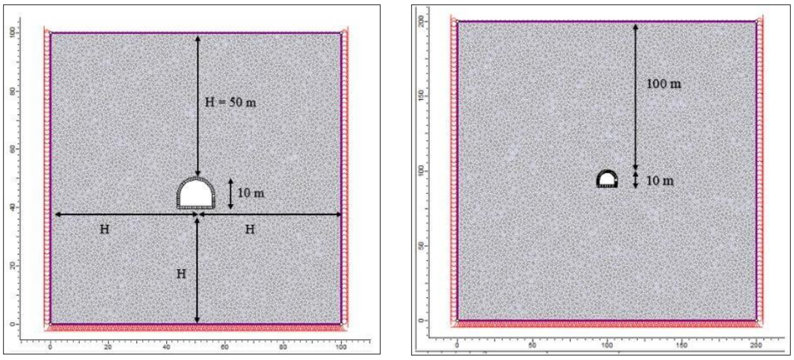
For computation of the deformation modulus of rock mass (E_{rm}), a Simplified Hoek and Diederichs (2006) [22] equation is used. It is expressed as:

$$E_{rm}(\text{MPa}) = 100{,}000\left(\frac{1-D/2}{1+e^{\left(\frac{75+25D-GSI}{11}\right)}}\right) \tag{5}$$

The equation for Strength Reduction Factor method (SRF) in relation to Generalized Hoek-Brown Criterion proposed by Hammah et al. (2005) [23] is expressed as:

$$\tau^{red} = \frac{\tau^{orig}}{SRF} = (\sigma_1 - \sigma_3)\,\frac{\sqrt{1 + a\, m_b\left(m_b \frac{\sigma_3}{\sigma_{ci}} + s\right)^{a-1}}}{2 + a\, m_b\left(m_b \frac{\sigma_3}{\sigma_{ci}} + s\right)^{a-1}} \cdot \frac{1}{SRF} \tag{6}$$

The geometry of the FEM models has been shown in Fig. 2 in which the model widths have been set as equal to the overburden height on either side of the tunnel's crown and bottom. A D-shaped tunnel having dimensions of 12 m as width (B) and 10 m as the opening height (D) is used for the parametric studies.



(a) (b)

Fig. 2. Geometry of the numerical models with overburden thickness, (H) of (a) 50 m; (b) 100 m

Phyllite rock mass properties were assigned to the models for three cases of GSI (25, 50, and 75) having two different overburden thicknesses (50 m and 100 m). The details of the properties are presented in Table 1.

Table 1. Properties of Phyllite rock mass

S. No.	**Properties**	**Values**		
		GSI = 25	GSI = 50	GSI = 75
1	Unit weight (γ), kN/m^3	26.0	26.0	26.0
2	Poisson’s ratio (μ)	0.25	0.25	0.25
3	Intact UCS, MPa	75.0	75.0	75.0
4	m_i	7.0	7.0	7.0
5	Disturbance factor (D)	0.5	0.5	0.5
6	Modulus ratio, MR	550.0	550.0	550.0
7	m_b	0.197	0.647	2.129
8	s	4.54 E-05	1.0 E-03	36 E-03
9	a	0.531	0.506	0.501
10	Modulus of deformation (MPa)	1461.07	6061.36	21371.97

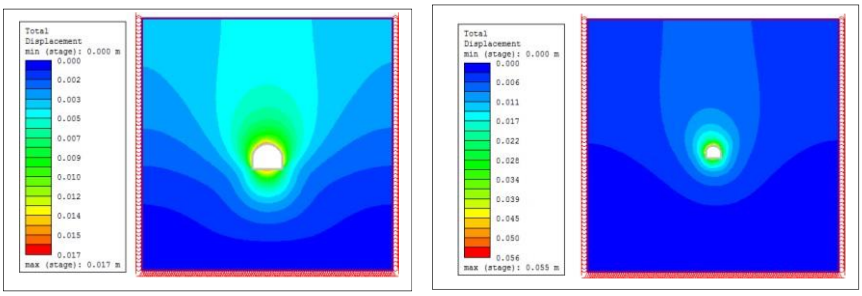
In order to perform the finite element analysis, a uniform mesh size with six-nodded triangular mesh elements were created in the model. The applied boundary conditions for the models were: (i) fixed boundary at the model bottom, (ii) top free, and (iii) roller supports at the sides of the models. The obtained results and discussions are presented in the following sections.

3 Results and Discussions

To assess the stability of the tunnel portal, displacement at its crown and global FoS were chosen as the governing criteria. There were total six cases each for these criteria. According to strength reduction analysis, if the factor of safety does not satisfy the targeted minimum value which was kept at 1.5 for static gravity loading condition, the simulation case was re-analyzed but with provision of a design support scheme.

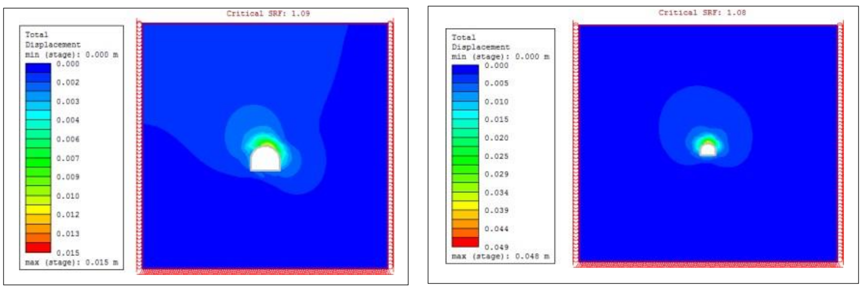
3.1 Cases: without any support measure

The outcomes of the parametric study (Table 2) demonstrate that the displacement of the GSI = 50 and GSI = 75 cases are within acceptable bounds (less than 0.1% of the minimum of the tunnel opening sizes which is 10 m here, leading to a maximum acceptable crown displacement of 10 mm), and the Global FoS of these cases satisfied the desired FoS values. For GSI = 25 case, with overburden thicknesses of 50 and 100 m, the crown displacement values obtained were 16 mm and 52 mm and the global FoS values were obtained as 1.09 &1.08, respectively. So, the results of GSI = 25 case show that this condition may be a susceptible case for any impending tunnel failure which requires an appropriate design support strategy. Accordingly, support measures have been chosen and the simulation results described in following section.



(a) (b)

Fig. 3. Displacement contour for GSI = 25: (a) H = 50 m, obtained displacement at crown = 16 mm; (b) H= 100 m, obtained displacement at crown = 52 mm



(a) (b)

Fig. 4. Factor of safety contour for GSI = 25: (a) H = 50 m, obtained FoS = 1.09; (b) H= 100 m, obtained FoS = 1.08

Table 2. Tunnel displacement and global Factor of Safety (FoS) values computed from the parametric analyses

Case No.	GSI	Overburden Height from tunnel crown (m)	Crown Displacement (mm)	FoS
1	25	50	17.0	1.09
2		100	51.0	1.08
3	50	50	2.0	2.98
4		100	4.0	2.80
5	75	50	1.1	9.48
6		100	2.0	7.55

3.2 Case: with support measure

For the vulnerable cases, support methods like shotcrete and rock bolting are used in accordance with the design criteria (support chart of Q-system) to limit crown displacement and improve stability. Barton et al. (1974) [24] suggested that the value of the Excavation Support Ratio (ESR) is 1.0 for the excavation categories of power plants, significant road and railroad tunnels, and portal intersections. The chart provided by Grimstad and Barton (1993) [25] for the design support system is used in the present study (Fig. 5). Table 3 shows the characteristics of support units used in numerical analyses.

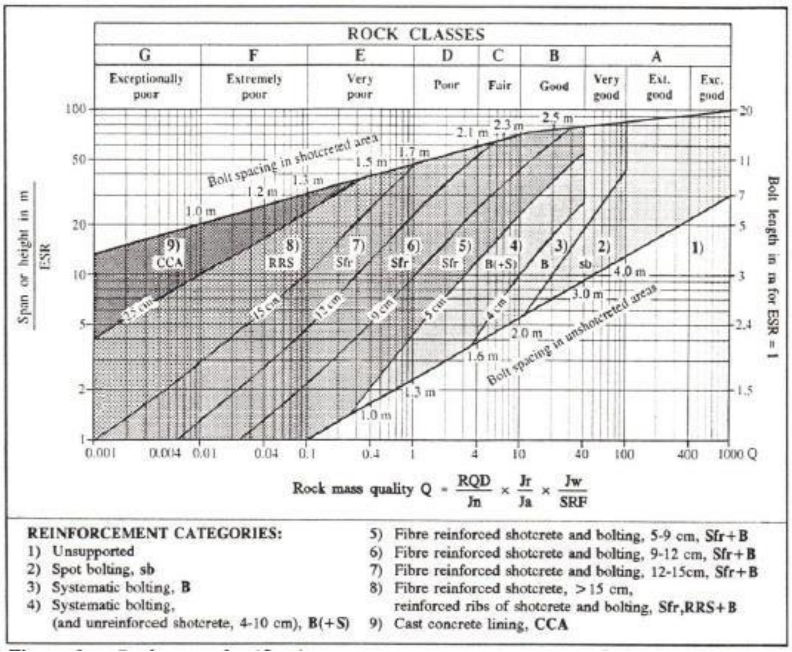
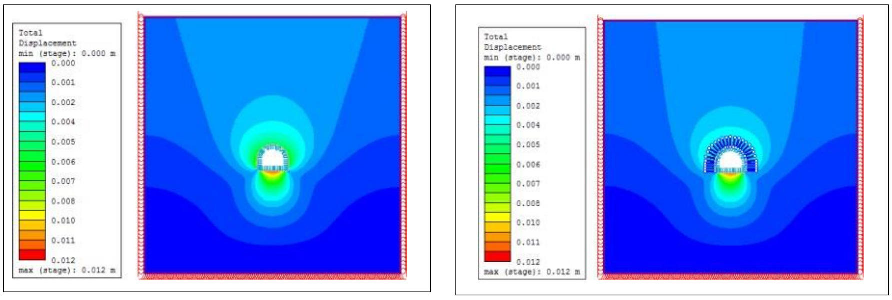


Fig. 5. Estimated support categories based on the tunneling quality index Q (Grimstad and Barton, 1993)

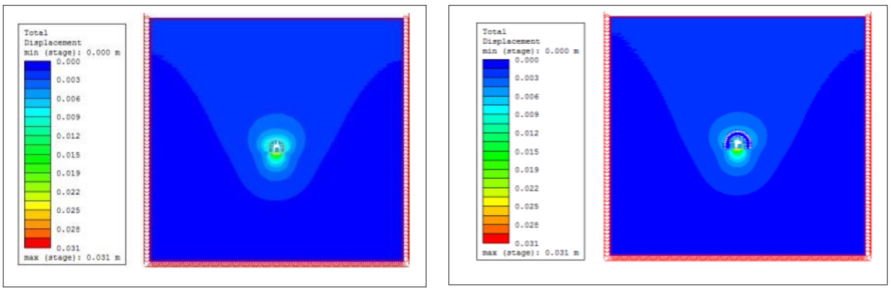
Table 3. Properties of the selected support systems

S.No.	Properties	Shotcrete	Wire-mesh	Rock bolt
1.	Young's modulus (E, GPa)	27.386	200	200
2.	Poisson's Ratio (μ)	0.2	0.25	-
3.	Compressive strength (σ_c, MPa)	30	400	-
4.	Tensile strength (σ_t, MPa)	3.834	400	485
5.	Tensile capacity (MN)	-	-	0.298
6.	Descriptions	Thickness = 15 cm	Ø 8.0 mm Mesh spacing 150*150 mm	Ø 28 mm, fully bonded, length: 4.0 m, Mesh spac-ing:2.0*2.0 mm



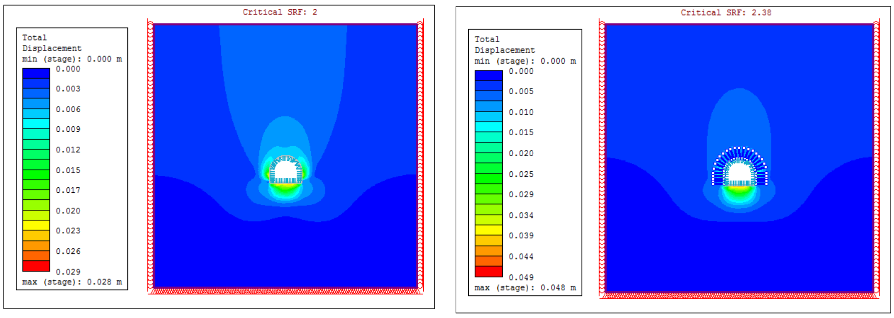
(a) (b)

Fig. 6. Displacement contours for GSI 25 & H = 50 m case with: (a) shotcrete with wire mesh, obtained displacement at crown = 5.0 mm; and (b) shotcrete with wire mesh along with rock bolting, obtained displacement at crown = 5.0 mm



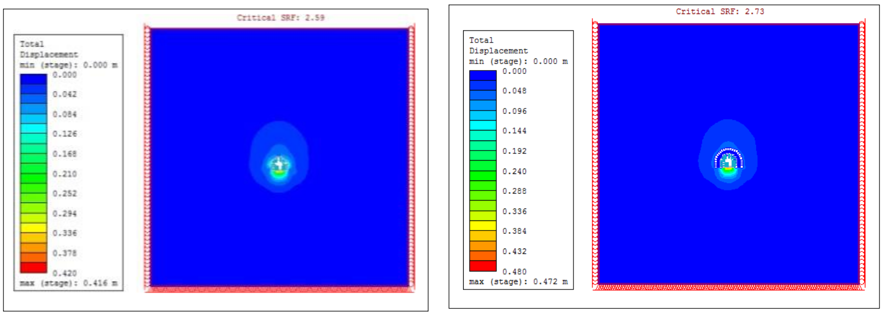
(a) (b)

Fig. 7. Displacement contours for GSI = 25 & H = 100 m case with: (a) shotcrete with wire mesh, obtained displacement at crown = 10.0 mm; and (b) shotcrete with wire mesh along with rock bolting, obtained displacement at crown = 9.0 mm



(a) (b)

Fig. 8. FoS contours of GSI = 25 & H = 50 m case with: (a) shotcrete with wire mesh, obtained FoS = 2.0; and (b) shotcrete with wire mesh along with rock bolting, obtained FoS = 2.38



(a) (b)

Fig. 9. FoS contours of GSI = 25 & H = 100 m case with: (a) shotcrete with wire mesh, obtained FoS = 2.59; and (b) shotcrete with wire mesh along with rock bolting, obtained FoS = 2.73

The results of support system show that the percent reduction of displacement for shotcrete (with wire mesh) and shotcrete with rock bolting were found to be 69% for both the cases having overburden thickness of 50 m; and 81% and 83% respectively for overburden thickness of 100 m. When compared with the FoS results of without support cases, for overburden thickness of 50 m, the FoS value increased from 1.09 to 2.0 and to 2.38 for shotcrete and shotcrete with rock bolting, respectively. In addition, for overburden thickness of 100 m, there the FoS increased from 1.08 to 2.59 and 2.73 for shotcrete and shotcrete with rock bolting, respectively. These obtained FoS values satisfied the targeted value (i.e., FoS >1.5). The results are summarized in Table 4.

Table 4. Displacement and global FoS for the studied tunnel portal faces with support measures

<table>
<tr><th rowspan="3">S. No.</th><th rowspan="3">GSI</th><th rowspan="3">Over-burden thick-ness (m)</th><th colspan="3">Displacement at tunnel crown(mm)</th><th colspan="3">Global Factor of safety (FoS)</th></tr>
<tr><th rowspan="2">Without support</th><th colspan="2">With support</th><th rowspan="2">Without support</th><th colspan="2">With support</th></tr>
<tr><th>Liner</th><th>Liner + Bolting</th><th>Liner</th><th>Liner + Bolting</th></tr>
<tr><td>1.</td><td rowspan="2">25</td><td>50</td><td>17</td><td>5</td><td>5</td><td>1.09</td><td>2.0</td><td>2.38</td></tr>
<tr><td>2.</td><td>100</td><td>52</td><td>10</td><td>9</td><td>1.08</td><td>2.59</td><td>2.73</td></tr>
<tr><td>3.</td><td rowspan="2">50</td><td>50</td><td>2</td><td colspan="2" rowspan="4">As per design criteria, cases are within the permissible limits</td><td>2.98</td><td colspan="2" rowspan="4">As per design criteria, cases satisfies the designated value i.e. (FoS>1.5)</td></tr>
<tr><td>4.</td><td>100</td><td>4</td><td>2.80</td></tr>
<tr><td>5.</td><td rowspan="2">75</td><td>50</td><td>1</td><td>11.09</td></tr>
<tr><td>6.</td><td>100</td><td>1</td><td>8.69</td></tr>
</table>

4 Conclusion

In the present study, the geotechnical properties were selected from the prepared database consisting data of published research works and project reports where the study areas have been in the North-West Himalayas. The GSI system was used to categorize the rock mass strength parameters which were then used in the parametric analysis of the tunnel portals. The conclusion of the study are as follows:

1. The study covers a range of GSI values for phyllite rockmass, from poorly interlocking, severely broken rock masses with low surface quality to well-interlocked, undisturbed rock masses with good surface quality.

2. GSI of 25 was found to be the most susceptible case for overburden thicknesses of 50 and 100 m. Shotcrete with wiremesh was observed to have enough strength to mitigate the tunnel portal collapse risk. However, use of rock bolting along with shotcrete with wiremesh is suggested for better stability improvement.

Following are the limitations of the present study: (i) the work done in the present study is solely focused on the phyllite rock mass, hence the outcomes could alter if the rock mass changes. Also, (ii) as the tunnel dimensions are fixed in this study, it might have an impact on the stability analysis when done with other dimensions.

5 References

1. Goel R.K., Jethwa J.L., Paithankar A.G.: Tunnelling through the young Himalayas - A case history of the Maneri-Uttarkashi power tunnel. Engineering Geology 39, 31-44 (1995).

2. Zhang Z., Chen F. He M. et al.: An analytical calculation method for displacement criterion on tunnel surrounding rock mass stability. Advances in Civil Engineering (2022).

3. Bieniawski ZT: Classification of rock masses for engineering: the RMR system and future trends. In: Hudson JA (ed) Comprehensive rock engineering: principles, practice, and projects. Pergamon, Oxford, pp 553–574 (1993).

4. Palmstrom A., Stille H.: Ground behaviour and rock engineering tools for underground excavations. Tunnelling and Underground Space Technology 22, 363–376 (2007).

5. Barton N.: Rock mass characterization for excavations in mining and civil engineering. In: Mark C, Pakalni R, Tuchman RJ (eds) Proceedings of the international workshop on rock mass classification in underground mining. NIOSH, Pittsburgh, pp 3–13 (2007).

6. Barton N.: From empiricism, through theory, to problem solving in rock engineering. ISRM Cong. 12th ISRM International Congress on Rock Mechanics, Volume 1, pp. 3-14 (2012).

7. Hoek E., Brown E.T.: Practical estimates of rock mass strength. International Journal of Rock Mechanics and Mining Sciences 34 (8), 1165-1186 (1997).

8. Suchowerska A.M., Merifield R. S., Carter J. P. et al.: Prediction of underground cavity roof collapse using the Hoek-Brown failure criterion. Computers and Geotechnics 44, 93–103 (2012).

9. Azad M.A, Singh S. K., Alok Apurva et al.: Geotechnical and geological studies of Adit-6 of the railway tunnel between Rishikesh and Karnprayag in India focusing on the excavation methods and design of support analysis: a case study. Arabian Journal of Geosciences 129, 1-15 (2022).

10. Kaya A., Fikri B., Selçuk A. et al.: Analysis of support requirements for a tunnel portal in weak rock: A case study from Turkey. Scientific Research and Essays 6 (31), 6566-6583 (2011).

11. Kaya A., Karaman K., Bulut F.: Geotechnical investigations and remediation design for failure of tunnel portal section: a case study in northern Turkey. Journal of Mountain Science 14(6), 1140-1160 (2017).

12. Aygar E. B., Gokceoglu C.: Effects of Portal Failure on Tunnel Support Systems in a Highway Tunnel. Geotechnical and Geological Engineering 39(2), (2020).

13. Liu S., Feng J., Zhu P. et al.: Stability Analysis of Two Parallel Closely Spaced Tunnels Based on Convergence-Confinement Principle. Journal of Construction Engineering and Management. 144(6), (2018).

14. Kockar M.K., Akgun H.: Methodology for tunnel and portal support design in mixed limestone, schist and phyllite conditions: a case study in Turkey. International Journal of Rock Mechanics & Mining Sciences 40, 173–196 (2003).

15. Qiu H.Z., Chen X.Q., Wu Q.H. et al.: Deformation mechanism and collapse treatment of the rock surrounding a shallow tunnel based on on-site monitoring. Journal of Mountain Science 17(12), 2897-2914 (2020).

16. Naithani Ajay K., Bhatt A. K., Murthy K. S. Krishna: Geological and geotechnical investigations of Loharinag–Pala Hydroelectric Project, Garhwal Himalaya, Uttarakhand. Journal Geological Society of India 73, 821-836 (2009).

17. Sarkar, S., Pandit, K., Dahiya, N. *et al.*: Quantified landslide hazard assessment based on finite element slope stability analysis for Uttarkashi–Gangnani Highway in Indian Himalayas. Nat Hazards 106, 1895–1914 (2021). https://doi.org/10.1007/s11069-021-04518-x

18. Siddique T., Mondal M.E.A., Pradhan S.P. et al.: Geotechnical assessment of cut slopes in the landslide-prone Himalayas: rock mass characterization and simulation approach. Nat Hazards 104, 413–435 (2020).

19. Singh, R., Umrao, R.K., Singh, T.N.: Stability evaluation of road-cut slopes in the Lesser Himalaya of Uttarakhand, India: conventional and numerical approaches. Bulletin of Engineering Geology and the Environment 73, 845–85 (2014).

20. Wang Z., Gao Bo, Jiang Y., Yuan S.: Investigation and assessment on mountain tunnels and geotechnical damage after the Wenchuan earthquake. Science in China Series E: Technological Sciences 52(2), 546-558 (2009).

21. Hoek E., Carranza-Torres C., Corkum B.: Hoek-Brown Failure Criterion – 2002 Edition. Proc. NARMS-TAC Conference, Toronto, vol. 1, pp. 267-273 (2002).

22. Hoek E., Diederichs M.S.: Empirical estimation of rock mass modulus. International Journal of Rock Mechanics & Mining Sciences 43, 203–215 (2006).

23. Hammah, R.E., Yacoub, T.E., Corkum, B.C et al.: The shear strength reduction method for the Generalized Hoek-Brown Criterion, The 40th U.S. Symposium on Rock Mechanics (USRMS). American Rock Mechanics Association (2005).

24. Barton N., Lien R., Lunde J.: Engineering classification of rock masses for the design of tunnel support. Rock Mechanics 6, 189-236 (1974).

25. Grimstad E., Barton N. R.: Updating the Q-system for NMT. Proc. Int. Symp. on Sprayed Concrete, Fagernes, Norway, (1993).