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Stability assessment for proposed Hemja–Patichaur Road Tunnel using Empirical and analytical methods, western Nepal

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ABSTRACT

Stability assessment is an important aspect for the safety and proper design of underground structures. This paper aims to analyze the stability issues for the proposed 13 km long road tunnel alignment along the Pokhara-Baglung Highway which stretches from the southeast Hemja (Kaski District) to the northwest Patichaur (Parbat District), Gandaki Province, Nepal. For this purpose, initially, an engineering geological mapping in the field was conducted followed by comprehensive analysis of stability. The study has shown that rock mass class within the proposed tunnel stretch ranges from good to extremely poor as per Q and RMR systems of rock mass classification. In this study, stability assessment was carried out by empirical methods and analytical methods. The sections along tunnel alignment were selected for stability study according to the overburden, rock class and presence of weakness zones. The study concluded that empirical methods has shown squeezing conditions at four sections and non-squeezing conditions at four sections as per Q-values. Similarly, analytical methods have shown the rock bursting condition as stable at the roof at most of the sections except minor spalling at some sections. However, the tunnel wall may suffer severe rock bursting with increased in-situ stress conditions of the rock mass. Further, the instantaneous and final deformation of the tunnel was obtained by Panthi and Shrestha's approach as less than one percent which is negligible.

Keywords: RMR, Q-system, rock mass quality, tunnel and stability assessment

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INTRODUCTION

Increasing urbanization and population growth are challenges for the socio-economic growth of mountainous country like Nepal. For an accelerated growth, Nepal needs to develop efficient transportation infrastructures including good road networks (Panthi, 1998). This is because, an efficient and good quality road network gives the possibility of transportation faster which increases cost-effectiveness and it also increases safety standards. The existing roads are challenging to provide efficient transportation service due to the conditions of many roads similar as shown in Figure 1.

Various geological and topographic issues (Fig. 2) influence on the cut-slope of road passing along the valleys that run through thrust zones, shear zones, fragile ground conditions, ingress of water and seismic vulnerability (Panthi, 1998).

During the last 50 years, many hydropower tunnels have been constructed in Nepal. This experience should be exploited to improve road quality by introducing road tunnels in Nepal.

Various studies (Panthi, 2012; Panthi and Nilsen, 2005; Nilsen, 2010; Khadka and Maskey, 2017; Karki et al., 2020; Basnet, 2013) highlighted challenging instability issues in the construction of tunnels consisting namely squeezing, bursting ground conditions as well as tunnel collapses associated to encountering faults and fracture zones. These instability challenges indicate that there is a need to improve prediction capacity through the use of proper and thorough geological investigations during the planning and design phase (Panthi, 2006). The need for analysis of stability conditions based on the rock mass behavior is much more important while planning and designing a tunnel project.

This manuscript is focused on the assessment of potential stability challenges that may be met along the proposed Hemja-Patichaur road tunnel. The manuscript evaluates the rock mass quality along the tunnel alignment which passes through the Lesser Himalayan rock formation of Nepal. The manuscript also evaluates the potential instability conditions using empirical and analytical approaches. It is highlighted here that if the proposed road tunnel is implemented, it will contribute significantly to the socio-economic growth of central Nepal through reduced travel distance and time, and transportation costs. Hence, this road tunnel is seen as a lifeline to Pokhara since it will cross the transboundary trade between India, Tibet, and China.

THE PROJECT DESCRIPTION

Project location

The proposed road tunnel project is expected to reduce the length of the road from 38 km to about 13 km. The entry portal and exit portal of the tunnel are considered to be located at 28°17.434' N, 083°52.568' E and 28°16.489' N, 083°44.607' E respectively (Fig. 3).

Fig. 1: Surface road conditions of Nepal (a) deep rock cutting, (b) winding road (Shakya, 2021).

Fig. 2: Difficulties faced on roads of Nepal (a) landslide on Pokhara Baglung Highway (Nepal Press, 2021), (b) landslide on Siddhartha Highway (Kathmandu Post, 2017).

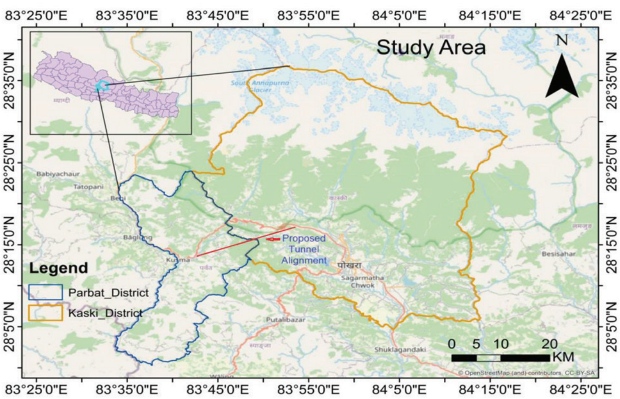


Fig. 3: Location of the study area.

The entry portal to the east is proposed at an altitude of 1167 m in Hemja of Kaski District (15.9 km northwest of Pokhara) near Ghatte Khola. On the other hand, the exit portal is proposed to be located at an altitude of 912 m in Dimuwa, Patichaur of Parbat District (47.3 km northwest of Pokhara).

The proposed road tunnel alignment passes through the hills of considerable slopes having topography variation from 900 to 2050 m amsl. The recommended project area covers the Baglung- Pokhara Highway in the north at the bank of Modi Khola and Rate Khola towards the south side of the hill that connects Baglung, Parbat, Myagdi and Mustang districts to Pokhara (Nepal). The detail alignment of the road tunnel is shown in Figure 4.

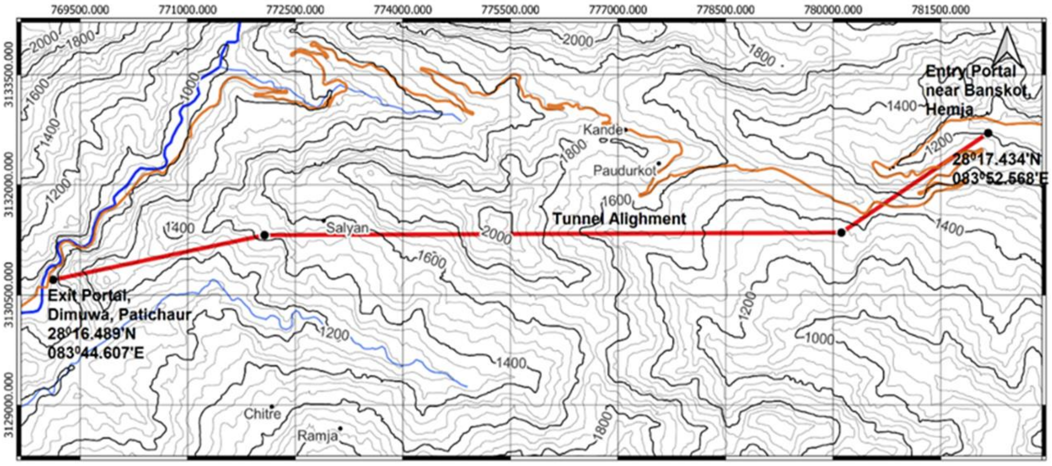
The geometry of the proposed tunnel

Traffic volume is a prime factor in the design of the roads for the required number of lanes, and width of the road. It is normally decided by the records of Annual Average Daily Traffic (AADT) volume. The recorded AADT on Pokhara-Baglung road exceeds 3000 according to the data of the Department of Roads (DoR). Analyzing the traffic volume trend of the past and considering the possible trend of future traffic volume in view of an extension of the Pokhara-Baglung Highway up to the border of China, it is assumed that AADT may reach to about 10000 within 10 to 15 year-time. Considering this possibility, the required size of the road tunnel is fixed using the manual for Norwegian Road Tunnel (MNPRT, 2004) as indicated in Figure 5.

Figure 6 shows the typical cross-section of road tunnel with different elements. As indicated in Figure 5, the tunnel cross-section profile hence is selected as T 9.5 for this 13 km long road tunnel with the provision of the two-lane single tube after the AADT crosses the 10000 vehicle limit. Therefore, the geometry of the proposed road tunnel will be as given in Table 1.

Geology of the project area

The project site is situated in the Lesser Himalaya Zone which lies between the Siwalik in the south and the Higher Himalaya in the north. Structurally, the Lesser Himalayan Zone is demarcated by the Main Central Thrust (MCT) to the north and the Main Boundary Thrust (MBT) to the south. Both these faults were very active in the past and the MBT is still active which caused rock mass deformed, faulted, folded, sheared, jointed and weathered. Typically, the Lesser Himalayan Zone comprises of low- to medium-grade metamorphic rocks, with overriding crystalline nappes and klippen (Upreti, 1999). The project site covers both the Kushma Formation (Ku) and Kuncha Formation (Kn). Both these two formations are



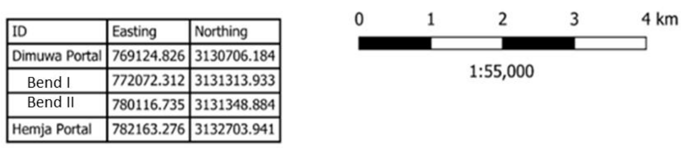


Fig. 4: Tunnel alignment map of the project area.

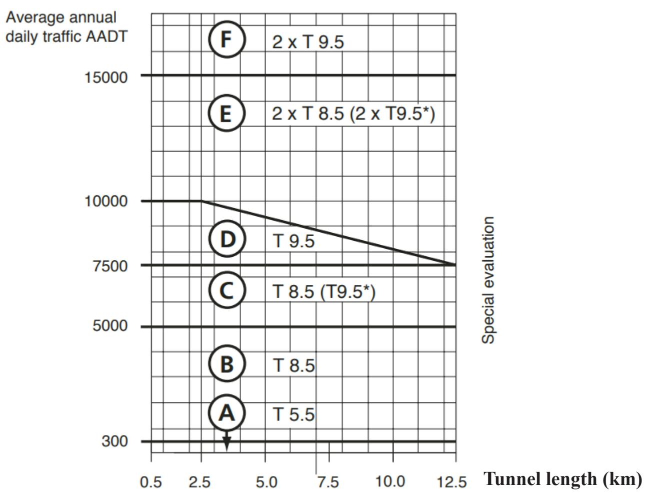


Fig. 5: Tunnel cross-section in relation with AADT and tunnel length (MNPRT, 2004)

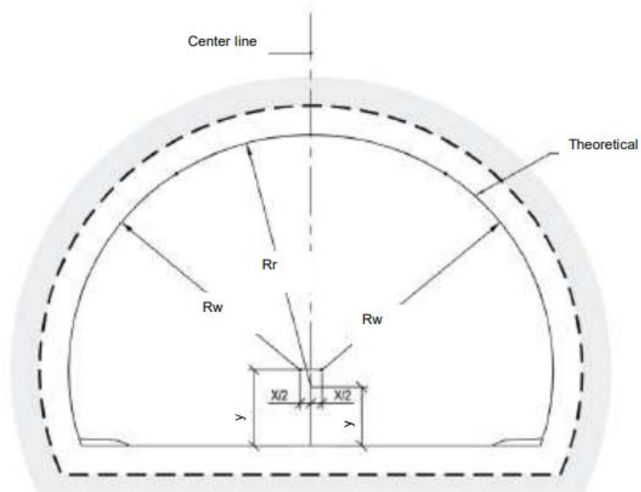


Fig. 6: Typical road tunnel cross-section (MNPRT, 2004).

Table 1: The geometry (profile) details of the road tunnel.

Description	Details
Length	13.5 km
Total effective width (B_t)	9.5 m
Driving width (B_d)	7 m
Total construction width	12m
Theoretical excavation area	66.62 m^2
Theoretical required area	53.61 m^2
Roof radius (R_r)	5.2 m
Wall radius (R_w)	4.8 m
Centre distance to roof radius (Y)	1.2 m
Centre distance to wall radius (X)	0.45 m
Shape	Inverted D-shape
Construction method	Drill and blast
Strike of tunnel	N 75° E

associated with the Lower Nawakot Group consisting of rocks such as phyllite, quartzite, and meta-sandstones (Stocklin and Bhattarai, 1977).

METHODOLOGY

For this study, thirty-six locations were assigned for the observations and engineering geological field mapping depending on the rock type, outcrops extension and topography. During the field mapping, the rock mass was characterized by using both Q and RMR systems of rock mass classification. Figure 7 describes the methodology used for this research work.

Engineering geological mapping along the alignment

While carrying out engineering geological mapping, the assessment on the presence of different joint sets and orientations was carried out, joint condition (joint spacing, aperture, surface roughness, infilling thickness and materials), groundwater condition, the occurrence of weakness zones, weathering condition, estimation of rock quality designation (RQD) and uniaxial compressive strength (UCS) etc. were conducted over the surface outcrops and existing road cuts along the road tunnel alignment.

Rock mass classification

Rock mass rating (RMR) system developed by Bieniawski (1989) was used for assessing the quality of rock mass based on the evaluation of six parameters consisting: uniaxial compressive strength of rock material, rock quality designation (RQD), spacing of discontinuities, condition of discontinuities, groundwater conditions and orientation of discontinuities. The ratings for each of the six parameters were summed to assess the overall value of RMR at each location. Similarly, Q-system of rock mass classification proposed by Barton et al. (1974) was also used to assess the quality of the rock mass along the tunnel alignment. The Q-system also consists of six parameters such as rock quality designation (RQD), joint set number (J_n), joint roughness number (J_r), joint alteration number (J_a), joint water reduction factor (J_w) and stress reduction factor (SRF). In Q-system, RQD/J_n defines the block size, J_r/J_a defines the inter-block shear strength and J_w/SRF defines the active stress (J_w/SRF).

Estimation of input parameters for stability assessment

It is essential to estimate rock mass parameters for the design of underground openings. Input parameters such as unit weight, elasticity modulus, Poisson's ratio, uniaxial compressive strength, tensile strength etc. are required for the stability assessment. These parameters are either mapped or estimated during the field mapping or by the use of literatures within the rock engineering field or with verbal instructions.

Stability assessment methods

Stability assessment and support design are key aspects for the planning and design of the tunnel projects which will be the base for the estimation of rock support need, quantity and cost calculations. In this study, methods such as empirical methods by Singh et al. (1992) and Goel (1994) and the analytical methods by Hoek and Brown (1980) and Panthi and Shrestha (2018) approach have been used for stability assessment.

For detailed assessment, eight sections as illustrated were selected representing chainages (m); 0+285, 3+717, 4+683, 7+240, 10+033, 10+672, 11+530 and 13+220 respectively covering high overburden, presence of weakness zones and rock mass class (Fig. 8).

RESULTS AND DISCUSSION

Engineering geological conditions

In the north-west exit portal at Patichaur, the rock type belonging to meta-sandstone to quartzite of greenish-white color, fine to medium-grained crystalline texture, and medium to the thick-bedded structure was observed. At the outcrop. The persistence of most of the joints ranged between 3-10 m. The joints have apertures varying between 1-5 mm and these joints have silt/sand infilling material <5 mm. Most of the rock outcrops were slight to moderately weathered and medium-grade metamorphosed. The south-eastern portal on the other hand consists of deformed, fine- to medium-grained light grey color and fractured phyllite intercalated with meta-sandstone.

The field mapping was carried out in the dry season, and the presence of lichens and vegetation on the joints showed possible water seepage during the monsoon period indicating a damp condition. The average joint frequency in rock mass varies from three to more than twenty-seven numbers in one cubic meter block. The difference in dip direction indicated the

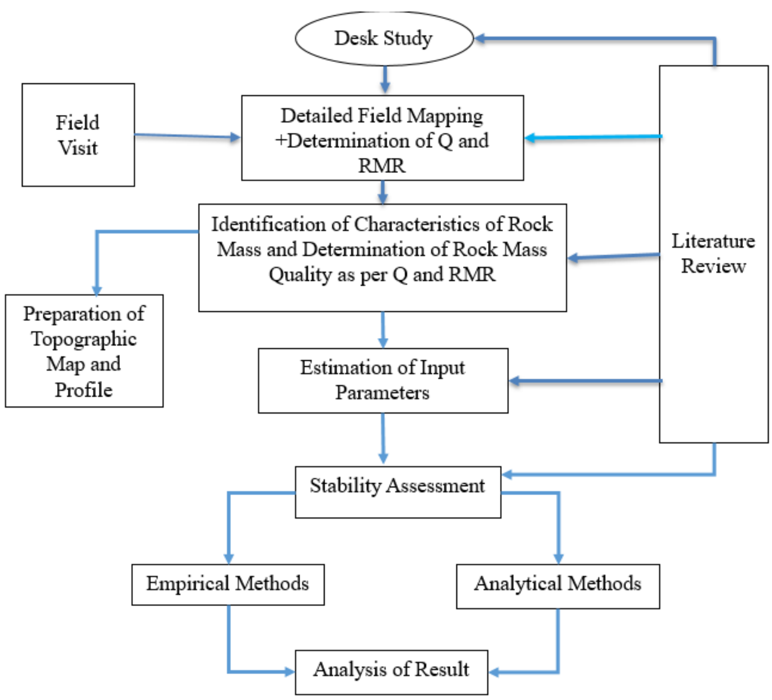


Fig. 7: Adopted research methodology.

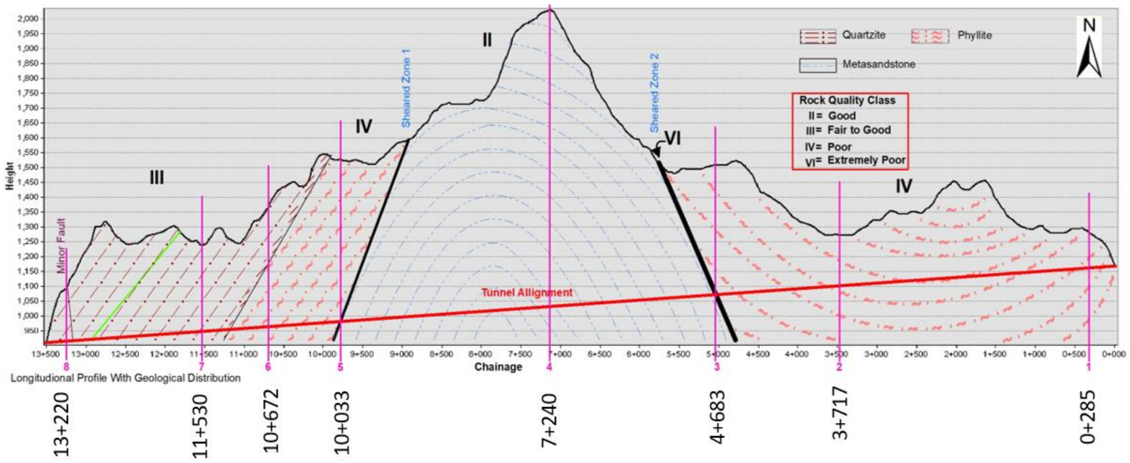


Fig. 8: Selected sections for stability assessment.

formation of the anticline and syncline folds. Similarly, minor other types of folds like recumbent folds were also observed (Fig. 9a,b). During field mapping, a crushed rock (shear/weakness zone) of meta-sandstone was found at 28°17'03"N, 83°48'44"E on the way to Bhadaure. This shear/weakness zone meets the tunnel at an approximate chainage of 5+000 m (Fig. 8). The compressive strength of rock varies from 35 to 45 MPa for phyllite and 135 to 250 MPa for meta-sandstone and quartzite respectively.

Rock mass quality assessment

All parameters associated with Q and RMR systems of rock mass classification were evaluated by carrying out detailed field mapping. The discontinuity conditions and characterization were assessed to identify overall rock mass quality along the proposed road tunnel alignment. The results of the rock mass quality distributions are presented in Table 2.

As per the RMR system, the rock mass quality was observed to vary from extremely poor to good. The highly sheared, folded, and weathered phyllite belongs to extremely poor-quality rock mass and massive meta-sandstone and quartzite belong to the fair to good-quality rock mass. The values of RMR ranged from 31 to 77. Similarly, the Q-values vary between 0.02 and 11.56.

Stability assessment

Empirical methods

Empirical approaches such as Singh et al. (1992) and Goel (1994) were used to predict potential squeezing. The results of the analysis for the selected sections are shown in Table 3 and Figure 10.

Analytical methods

The stability assessment was also carried out using analytical methods such as; Panthi and Shrestha (2018) approach and Hoek and Brown (1980) approach. The achieved results are presented in tables 4, 5 as well as in Figure 11, respectively.

Fig. 9: (a) Recumbent fold on the way to Bhadaure, Kaski, (b) normal fault at Thamarjung, Parbat.

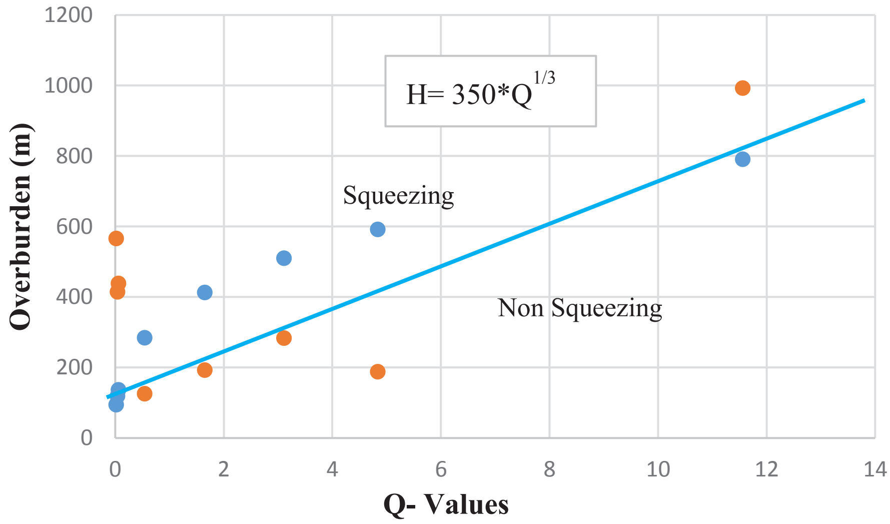


Fig. 10: Squeezing and Non-squeezing conditions of tunnel (Singh et al., 1992).

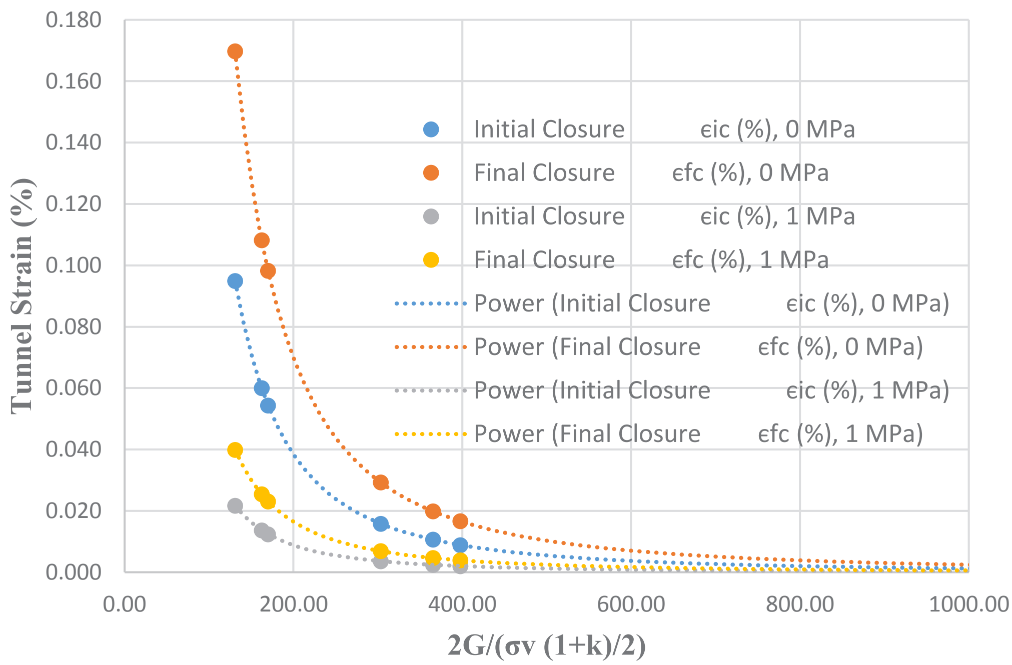


Fig. 11: Rock mass shear modulus, in situ-stress vs tunnel strain for different magnitude of support pressure (Panthi and Shrestha, 2018).

Table 2: Rock mass quality distribution along tunnel length.

Chainage of tunnel (m)	Value		Rock class	Quality
	RMR	Q		
0+000 to 4+940	53-56	1-4	IV	Poor
4+940 to 5+000	32	0.06	VI	Extremely Poor
5+000 to 9+750	65-77	10.56-11.56	II	Good
9+750 to 9+850	31-35	0.02-0.04	VI	Extremely Poor
9+850 to 11+200	52-55	1.21-3.11	IV	Poor
11+200 to 13+500	60-65	4.17-10	III	Fair to good

Table 3: Squeezing prediction as per Singh et al. (1992) and Goel (1994).

Chainage (m)	Overburden (H) (m)	Singh et al. (1992)			Goel (1994)		
		Q-value	H'	Squeezing prediction	Rock mass number (N)	H''	Squeezing prediction
0+285	126	0.54	285.014	No	1.35	238.014	No
3+717	193	1.65	413.583	No	0.75	196.048	No
4+683	439	0.06	137.02	Yes	0.45	165.635	Yes
7+240	993	11.56	791.384	Yes	11.56	483.468	Yes
10+033	566	0.02	95.005	Yes	0.2	126.745	Yes
10+672	415	0.04	119.698	Yes	0.3	144.891	Yes
11+530	284	3.11	510.883	No	7.775	424.153	No
13+220	188	4.84	592.038	No	4.84	362.736	No

Table 4: Estimation of tunnel squeezing using Panthi and Shrestha (2018) approach.

Chainage (m)	Over-burden (H), m	Q-value	σ_v (MPa)	σ_h (MPa)	k	σ'_{cm} (MPa)	E_{rm} (MPa)	G (MPa)	Initial closure ϵ_{ic} (%), 0 MPa	Final closure ϵ_{fc} (%), 0 MPa	Initial closure ϵ_{ic} (%), 1 MPa	Final closure ϵ_{fc} (%), 1 MPa
0+285	126	0.54	3.604	3.52	0.977	4.06	1457.16	708.32	0.009	0.017	0.0020	0.0039
3+717	193	1.65	5.520	3.82	0.692	4.06	1457.16	708.32	0.016	0.029	0.0036	0.0069
4+683	439	0.06	12.555	4.9	0.390	4.06	1457.16	708.32	0.060	0.108	0.0137	0.0254
7+240	993	11.56	26.315	8.63	0.328	15.97	6550.40	3190.6	0.011	0.020	0.0024	0.0047
10+033	566	0.02	16.188	5.46	0.337	4.06	1457.16	708.32	0.095	0.170	0.0217	0.0399
10+672	415	0.04	11.869	4.8	0.404	4.06	1457.16	708.32	0.054	0.098	0.0124	0.0231
11+530	284	4.00	7.4408	5.26	0.706	93.95	20564.7	10019.	0.000	0.000	0.000	0.0001
13+220	188	4.84	4.9256	4.48	0.909	93.95	20564.7	10019.	0.000	0.000	0.000	0.000

Table 5: Rock burst condition (Hoek and Brown, 1980).

Chainage (m)	σ_v (MPa)	k	σ_{ci} (MPa)	Tangential stress at roof ($\sigma_{\theta r}$) (MPa)	Tangential stress at wall ($\sigma_{\theta w}$) (MPa)	For roof		For wall	
						$\sigma_{ci}/\sigma_{\theta r}$	Prediction	$\sigma_{ci}/\sigma_{\theta w}$	Prediction
0+285	3.6036	0.9768	39	7.66	4.77	5.09	Minor spalling	8.18	Stable
3+717	5.5198	0.6921	39	6.70	8.88	5.82	Minor spalling	4.39	Minor spalling
4+683	12.555	0.3903	39	3.12	23.98	12.48	Stable	1.63	Severe rock bursting
7+240	26.315	0.328	73	1.30	51.89	56.09	Stable	1.41	Severe rock bursting
10+033	16.188	0.3373	39	1.28	31.77	30.36	Stable	1.23	Severe rock bursting
10+672	11.869	0.4044	39	3.49	22.50	11.17	Stable	1.73	Heavy support required
11+530	7.4408	0.7069	221	9.39	11.85	23.53	Stable	18.64	Stable
13+220	4.9256	0.9095	221	9.41	6.85	23.48	Stable	32.27	Stable

Discussion

Both empirical and analytical methods of stability analysis were considered for the proposed road tunnel. The selected tunnel sections showed varying stability conditions depending on the varying geological parameters. The empirical methods of both Singh et al. (1992) and Goel (1994) showed squeezing conditions at Chainage (m) 4+683, 7+240, 10+03 and 10+672 and non-squeezing conditions at Chainage (m) 0+285, 3+713, 11+530 and 13+220 as per Q-values.

According to Hoek and Brown (1980), most of the sections except near the entry portal were found to be stable at the roof but the tunnel wall may suffer from rock bursting at the high overburden area due to an increase in the in-situ stress magnitude. Tunnel strain of less than one percent gives no such stability challenges as per Panthi and Shrestha (2018), which means tunnel rock support is optimized for safety requirements and potential block fall situations. It is envisaged that the empirical methods provide no clear guidelines regarding the unstable situation in comparison to the analytical methods.

CONCLUSIONS

The required engineering geological and mechanical properties of rock mass were estimated on the basis of geological mapping conducted in the study area, and by literature review. The stability assessment carried out using empirical methods such as Singh et al. (1992), Goel (1994), and analytical methods such as Hoek and Brown (1980), Panthi and Shrestha (2018) gave fruitful results. According to empirical methods, squeezing and non-squeezing conditions were predicted based on Q values. This method only provided an initial feeling about squeezing possibility. Hoek and Brown approach showed rock bursting condition as stable on the roof at selected sections except for minor spalling and severe rock bursting along the tunnel wall in the high rock cover area. Further, Panthi and Shrestha (2018) approach gave the possibility to investigate both instantaneous and time-dependent deformation (tunnel strain) having a magnitude less than one percent. The results presented here may be further improved by carrying out further engineering geological investigations consisting of extensive field mapping and by the use of geophysical investigation methods.

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