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Monitoring of a tunnel through mixed geology in the Himalaya

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ABSTRACT: National Highway Authority of India (NHAI) has taken up the project of four laning of National Highway 1A (NH-1A) between Udhampur and Banihal. The work of 9.0km long Chenani-Nashri highway tunnel enroute was entrusted to ITNL on BOT (annuity) basis. The project area lies in western Himalayan region. The rock masses along the Chenani-Nashri tunnels, belong to the lower Murree formation that includes a sequence of inter-bedded sandstone, siltstone and claystone layers of varying thickness. As expected the experience of tunnelling, so far, in mixed or non-uniform geology having bands of sandstone, siltstone, claystone, intermixed siltstone and claystone and sheared siltstone and claystone is not very encouraging. The convergence from Barton's approach is estimated for various rock masses. Displacements of roof and walls have also been monitored regularly and compared with the convergence values estimated from Barton's approach. The measured values of displacements have also been analysed to study the effect of non-uniform geology and the two tunnels in parallel. The benefits of displacements monitoring have been highlighted to understand the behaviour of non-uniform geology.

1 Introduction

There is a significant commercial boom in India during last one and a half decade. This also brings higher demands for traffic infrastructure of the country. As a result many rail and road projects, which involve the construction of tunnels are also under different phases of construction. The new four-lane highway link project between Udhampur and Banihal on National Highway 1A (NH-1A) in J&K state of India is one of the recent, most important Indian projects planned to connect the Kashmir valley with the rest of the Indian transportation network. The highway is passing through the tough and fragile terrain of Himalaya having steep slopes, areas of frequent landslides, and passes through an altitude of about 2030m near a tourist place called Patnitop. In winter season, the road remains closed for hours because of heavy snow fall around Patnitop range. Hence, it was decided to by-pass Patnitop by 9km long Chenani-Nashri tunnel. The distance between Chenani and Nashri by present road is about 41km (from km 89 to km 130.0). The tunnel on completion, therefore, will not only reduce the journey time but also provide trouble-free road journey in winters.

The Chenani-Nashri road tunnel project comprises of two parallel tunnels, a bi-directional traffic main tunnel and a escape tunnel. National Highway Authority of India (NHAI) has entrusted the responsibility of the construction of Chenani-Nashri road tunnels to ITNL on BOT (annuity) basis. For tunnel excavation works, ITNL engaged M/s Leighton Contractors (India) Pvt Ltd. (LIN) on engineering, procurement and construction (EPC) basis who have enagaged M/s Geodata Engineering, Italy (GEODATA) as their design and supervision consultant. Both tunnels, the main and the escape, have been planned to excavate from the two ends, south end towards Chenani village and the north end near Nashri village. The tunnel construction was started in August 2011 and shall be completed by June 2016. The tunnels are being constructed through the sedimentary rocks of Muree formations of the Himalaya which are influenced by regional and local faults and shear zones.

Design and construction of tunnels through such a complex and non-uniform geological conditions with a rock cover of more than 1050m is a difficult and challenging task. The tunnel design methodology and a couple of construction problems have been briefly discussed by Goel et al. (2012). The present paper highlights the tunnel-displacement-monitoring results in different rock masses exposed at one section around tunnel periphery.

2 Details of tunnels

The main tunnel (MT) has typical cross-section as shown in Figure 1. This has been worked out based on the required clearance dimensions and on the M&E requirements (in particular ventilation ducts shape, and dimensions for exhaust air extraction and fresh air inlet) are – horizontal (H):9.35m x vertical (V):5.00m, plus 1.2m wide footpath each side (Facibeni et al. 2011). The excavated dimensions varies with the rock class. Typical excavation dimensions for rock class B1 are - height 11.68m; bottom width 12.9m, diameter 14.02m; perimeter 45.12m and area 139.91m^2. The road elevation level at north and south portals are approximately 1209m and 1230.5m respectively.

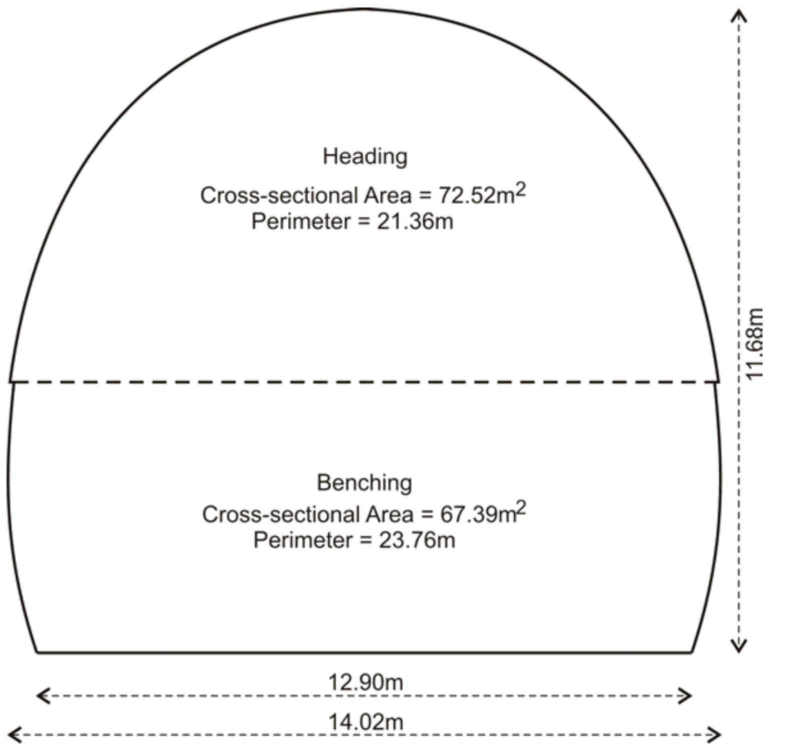

Figure 1. Typical cross-section of main tunnel

The escape tunnel is fully in compliance with the required clearance dimensions for the operative stage (H:5.00m x V:2.50m) with footpath on each side (Facibeni et al. 2011). For escape tunnel also the excavated dimensions varies with the rock class. The excavated dimensions for rock class B1 are - height 6.15m; bottom width 5.86m, diameter 6.55m, perimeter 16.03m and area 34.89m^2.

Facing Chenani end portal, the escape tunnel lies on the left of main tunnel. The two tunnels, having pillar width of 33.5m in between, are interconnected at regular intervals to provide cross passages. Two types of cross passages have been provided, viz. pedestrian and vehicular emergency exits at a distance of 300m and 1200m center to center respectively. The typical section of pedestrian cross passage has been designed considering the size to escape tunnel; vehicular cross passages are larger, with the clearance requirements of (H):7.50m x (V):4.50m.

The tunnels have a up gradient of 0.5% from Chenani and downwards gradient of 1.0% towards Nashri from tunnel centre. Maximum road level is 1252.40m at Ch. 4445m from south end. Tunnel excavation is being carried out using drill and blast methods by top heading, bench and invert for the main tunnel and vehicular cross passage while full face excavation will be adopted for the escape tunnel and pedestrian cross passages. The tunnel is supported by a primary lining (shotcrete, steel ribs, lattice girders, rock bolts in combination as per the rock class) 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 tunnels.

3 Geology along the tunnel alignment

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. These main thrusts as well as most of the belts and units of this NW region of Himalaya orogen show a regional strike of NW-SE to WNW-ESE with moderate to steep dips either towards north or the south (Facibeni et al. 2011).

The rock masses along the project of the Chenani-Nashri tunnel belong to the Lower Murree formation. This sedimentary succession is classified as the 'Lower Tertiary Sediments' of the 'Murree Structural Belt' and bounded on the south by the Main Frontal Thrust and on the north by a complex of thrusts regionally referred as the Main Boundary Thrust (MBT) that delimited from the metamorphic complex.The Muree Formation is represented by a sequence of argillaceous and arenaceous rocks that includes a sequence of interbedded sandstone, siltstone/claystone beds with thickness ranging from a few metres up to 10m.

The strike of bedding plane is almost parallel to the tunnel axis at Chenani end, whereas it is perpendicular to tunnel axis and dipping in the tunnel direction (favourable) from Nashri end. The bands of sandstone, siltstone, claystone of varying thickness are frequently encountered during tunnel excavation. There is no fixed pattern of the bands of these rocks (Figure 2). Infact the bands of mixed rocks, for example, intermix siltstone & sandstone and intermix siltstone & claystone are also encountered frequently. The uniaxial compressive strengths of freshly obtained rock samples of sandstone, siltstone and claystone are 70-120MPa, 25-40MPa and 8-15 MPa respectively.

Figure 2. Photo showing exposure of different rocks at one location in main tunnel, south end

The claystone rock specimen, if left exposed to atmosphere, degrades and crumbles to small pieces in about a week or so. The joints in siltstone have clay fillings, which are erodible. The freshly excavated claystone on tunnel face sometimes give deceptive appearance of massine rock or one or two joints, but after a day, it starts giving way.

Barton's rock mass quality Q (Barton et al. 1974) and Bieniawski's rock mass rating RMR (Bieniawski 1989) have wide range for different rock masses being encountered in the tunnel (Table 1). The variation in the values of Q in a particular rock mass is mainly beacuse of the variation in RQD, J_a and SRF, whereas variation in RMR is beacuse of variation in RQD, UCS and joint condition. In most of the cases there are three joints plus random including the bedding plane. In case of mixed rocks, the values are also influenced by the per cent of different rocks. In these cases the minimum percentage of weaker rock was about 30-35 per cent. The Q and RMR values reported in Table 1 are for the tunnel depth upto 500m. With increasing depth, these values may again vary depending upon the rating of various parameters of Q and RMR.

It is understood from the values of Q and RMR in Table 1 that the roof supports designed for sandstone will not be safe for claystone or for the mixture of siltstone and claystone. Since the tunnel axis is almost parallel to the bedding strike from the Chenani (south) end and the bedding thickness is quite varying, the roof of tunnel has exposure of claystone and other rocks also. In such conditions, it was thought to classify the rocks separately for the supports and accordingly design the supports as per the weaker rock mass.

Table 1. Q, RMR and N values for different rocks of Chenani-Nashri tunnels

S.No.	Rock(s)	Range of RMR	Range of Q	Range of N
1	Sandstone	50-64	3.5-8.0 (5.30)	8.75-20.0 (13.2)
2	Siltstone	49-54	2.0- 4.58 (3.02)	5.0-11.45 (7.5)
3	Claystone	22-26	0.08-0.14 (0.10)	0.4-0.7 (0.53)
4	Mixture of sandstone and siltstone	44-48	1.3-1.85 (1.55)	3.25-4.62 (3.87)
5	Mixture of siltstone and claystone	32-43	0.3-1.0 (0.54)	1.5-5.0 (2.74)

Note: RMR – Bieniawski's rock mass rating; Q – Barton's rock mass quality; N – rock mass number (Q with SRF=1) and values in () are the log average values

Failure in an inhomogeneous geological material is generally progressive, whereas a homogeneous rock fails suddenly. Hence, the advantage of inhomogeneous materials offered by nature is that they give advance warning of the failure process starting slowly from the weakest zone (Singh and Goel 2011). About 1.4km of main tunnel has been excavated so far from Chenani (south) end. In these tunnels also it has been experienced that the failure process is starting slowly from the weakest rock or zone in non-uniform or mixed geology or inhomogeneous rocks.

4 Estimation of convergence using rock mass quality Q

On the basis of a large number of case histories, Barton (2008) found the following approximate correlations for estimating convergence of roof and and centre of wall in nearly homogeneous rock mass, away from shear zone/weak zones (for B/Q = 0.5 to 250),

$$\delta_V = \frac{B}{100\,Q}\sqrt{\frac{\sigma_V}{q_c}} \qquad (1)$$

$$\delta_h = \frac{H_t}{100\,Q}\sqrt{\frac{\sigma_h}{q_c}} \qquad (2)$$

Where δ_v and δ_h are roof and wall convergence values respectively; σ_v and σ_h are in situ vertical and horizontal stresses normal to the wall of tunnel respectively; B is span or width of the tunnel; H_t is total height of the tunnel; Q is average Barton's rock mass quality; and q_c is uniaxial compressive strength of intact rock material.

As per Equations 1 and 2 the convergence of tunnel roof and wall depends upon the in situ stresses, the uniaxial compressive strength of intact rock material, the span or height of tunnel and rock mass quality. Accordingly, the convergence for different rock masses assuming $\sigma_h = \sigma_v = \gamma H$ (H is tunnel depth; unit weight of the rock mass γ = 2.5g/cc) are computed and given in Table 2. The tunnel span in Table 2 is considered for support class B1 as given in section 2. Similarly, wall convergence using Equation 2 can also be estimated.

Aydan (2011) opined that UCS of rock material for squeezing condition shall be less than 30MPa. Considering the approach of Singh et al. (1992) for estimating the squeezing ground condition for tunneling ($H>350Q^{1/3}$; H=tunnel depth in metres) and UCS condition of Aydan (2011), it is found that claystone and intermixed siltstone and claystone are likely to get the squeezing ground condition when the tunnel depth is more than 160m and 285m respectively.

Table 2 shows that the roof convergence in claystone (Q=0.1) is very high even at a depth of 100m. Hence, claystone shall pose severe squeezing problems. At tunnel depth of 200m the convergence in both the tunnels have exceed 6% of tunnel size, which indicates failure of the rock mass (Singh and Goel 2011). Hence, the supports in the claystone shall have high stiffness to control the convergence. It may noted here that the convergence estimated in Table 2 is without providing the tunnel supports, whereas the displacements given in Table 3 are with the supports.

Table 2. Estimated roof convergence from Equation 1

S.No.	Average Q	Average UCS, MPa	Tunnels	Roof convergence for different tunnel depths, mm					
				100m	200m	300m	400m	500m	600m
1	5.30	95	Main	3.9	5.6	6.9	7.9	8.9	9.7
			Escape	1.8	2.5	3.1	3.6	4.0	4.4
2	3.02	32.5	Main	11.9	16.9	20.7	23.9	26.7	29.2
			Escape	5.4	7.6	9.3	10.7	12.0	13.2
3	0.10	11.5	Main	606.1	857.2	1049.8	1212.2	1355.3	1484.7
			Escape	273.2	386.4	473.2	546.4	610.9	669.2
4	1.55	60	Main	17.1	24.2	29.6	34.2	38.3	41.9
			Escape	7.7	10.9	13.3	15.4	17.2	18.9
5	0.54	21	Main	83.06	117.5	143.9	166.1	185.7	203.4
			Escape	37.4	52.9	64.8	74.9	83.7	91.7

Note: Q = Barton's rock mass quality; UCS=uniaxial compressive strength

Convergence values in case of mixture of siltstone and claystone exceed the limit of 1% of tunnel size (condition for squeezing) around the tunnel depth of 300m (Table 2), which is surprisingly matching with the depth of 285m obtained from the approach of Singh et al. (1992). Almost same tunnel depths are obtained for squeezing condition from the approach of Goel et al. (1995) using rock mass number N (Q with SRF=1).

As mentioned earlier, the tunnels in this project have exposure of different layers (Figure 2), non-uniform geology. In case of bedded rock masses having the layers of different thickness of various rocks, the convergence of roof and wall may vary depending upon the location of rocks exposed in the roof and wall and the in situ stresses. The displacements have been measured in the tunnels and it has been tried to study the measured displacements and the estimated values of convergence from south end in main and escape tunnels.

5 Displacement monitoring in tunnels

Systematic tunnel monitoring by fixing bireflex targets for tunnel displacement is being carried out in the tunnels by GEODATA/LIN for understanding the rock mass-tunnel support interaction. The bireflex targets are fixed at regular interval of 50m or as and when required on the basis of the ground condition/geology.

At one location five bireflex target points (T1, T2,….,T5) are fixed to measure tunnel displacement (Figure 3). X,Y & Z co-ordinates of each target point are recorded (Figure 4). The readings are then analysed to get the displacement of individual target points and the chord convergence between various target points (Figure 3). In this study only the displacements of targets (radial displacement) have been studied, which are taken as half of convergence values.

Displacement of various target points in main and escape tunnels from south end are given in Table 3. Some representative locations only have been shown in Table 3. These value of convergence is after application of supports of class B1, B1* and B2, decided as per the expected rock behavior (methodology of support design is not discussed because of page limitations). With the supports, the displacements at some locations are more than 1% of tunnel size, e.g. ch. 385m, 621m, 837m and 948m in main tunnel and all the chainages, except ch. 1014m, in escape tunnel. Considering the exposure of various rock masses, Q value is estimated and given in Table 3.

Main tunnel has been excavated using heading and benching method. The escape tunnel is more than 500m ahead of main tunnel. The displacements of targets T2, T3 and T4 in main tunnel in Table 3 are given before and after bench blasting. Monitoring of T1 and T5 in main tunnel was started after the bench excavation. Therefore one value of displacement for targets T1 and T5 is given in Table 3. Rock exposures near the target position at various locations (chainages) in the tunnels are given in Table 4.

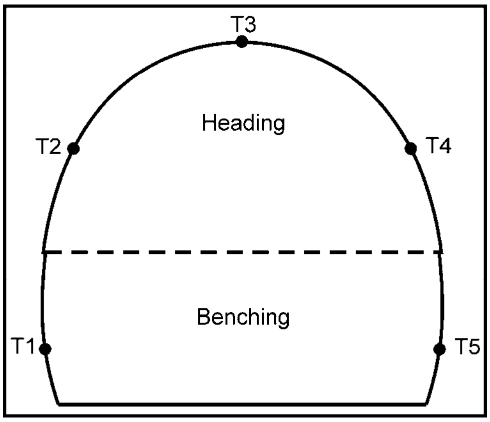

Figure 3. Array of five target points at one location to measure tunnel displacement

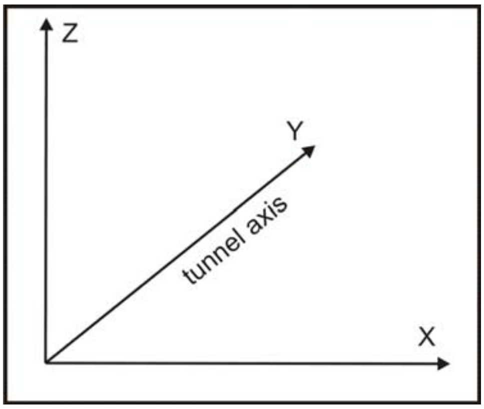

Figure 4. Directions of X, Y and Z co-ordinates of readings

Table 3. Displacement of various target points in main and escape tunnels from south end

S.No.	Chainage (ch.), m	Q/Support class	Tunnel depth, m	Displacement of targets, mm				
				T1	T2	T3	T4	T5
				Main tunnel				
1	269	0.86/B1*	140	33	15 (10)	15 (10)	20 (20)	5
2	385	0.58/B1*	200	37	10 (10)	20 (15)	83 (50)	20
3	465	0.58/B2	210	20	30 (20)	45 (35)	55 (25)	30
4	527	0.83/B2	230	10	41 (35)	10 (5)	45 (25)	30
5	621	1.03/B1*	265	0	10 (5)	25 (15)	78 (50)	30
6	837	1.37/B1*	345	0	45(35)	45(40)	130(105)	12
7	948	2.72B1*	385	10	30 (20)	30 (25)	95 (55)	10
				Escape tunnel				
8	535	2.04/B1*	240	64	8	5	8	18
9	742	0.65/B1	315	57	42	15	20	70
10	850	0.7/B1*	360	72	20	5	35	47
11	905	0.75/B1*	375	37	15	10	60	90
12	1014	0.75/B1*	415	5	11	9	8	4
13	1196	0.58/B2	455	12	12	18	55	70
14	1804	2.81/B1*	520	50	25	15	32	28

Note: B1 = Rock bolt and 150mm thick SFRS; B1* = rock bolt, 200mm thick SFRS and lattice girder; B2 = rock bolt, 200mm thick SFRS and lattice girder displacement value in ( ) is the displacement due to heading excavation; shotcrete thickness in escape tunnel is 50mm less for various support classes

Displacements in escape tunnel are affected when the heading and benching of main tunnel has reached to escape tunnel location (chainage). In sandstone there is practically no effect of main tunnel excavation on the displacement in escape tunnel (ch. 535m), except at T1 where siltstone and claystone is exposed (Table 4). In siltstone and claystone the displacement in escape tunnel increases when the main tunnel is excavated at that location. At few locations (e.g., ch. 850m) target T3 shifted towards left side, i.e. away from the main tunnel. Maximum displacement is recorded in target T5 (upto 90mm at ch. 905m).

In general, the displacement of target T4 is more than any other target in main tunnel (Table 3). Maximum displacement of 130mm (about 2% of tunnel size) is recorded in target T4 at ch. 837m where siltstone is exposed. Vertical displacement in target T3 as reported in Table 3 is less in comparison to T4, but the lateral displacement (X-direction) towards the escape tunnel has also been recorded at some locations in target T3. It is highlighted here that escape tunnel is excavated before the main tunnel. Hence, it may be said that the displacement of targets in main tunnel are also influenced by escape tunnel. The in situ stresses first disturbed and re-distributed during the escape tunnel excavation. These in situ stresses are again disturbed due to main tunnel excavation. The displacement of target T4 may be because of maximum stress in NE-SW direction, i.e. the direction of the displacement of target T4, which is almost perpendicular to the bedding planes, and because of anisotropic deformation modulus in layered rocks.

Table 4. Exposure of rocks near the target at various chainages

S.No.	Chainage (ch.),m	Exposure of rocks				
		T1	T2	T3	T4	T5
				Main tunnel		
1	269	Siltstone and claystone	Siltstone	Siltstone and claystone	Claystone	Siltstone
2	385	Claystone	Sandy siltstone and claystone	Claystone	Claystone and siltstone	Siltstone
3	465	Siltstone	Siltstone and claystone	Claystone	Claystone	Clayey siltstone
4	527	Siltstone	Siltstone and claystone	Siltstone and sandstone	Siltstone	Siltstone
5	621	Siltstone	Siltstone and sandstone	Siltstone	Siltstone	Sandy siltstone
6	837	Siltstone	Siltstone	Siltstone	Siltstone	Siltstone
7	948	Claystone and siltstone	Siltstone	Sandy siltstone	Sandy siltstone	Claystone and siltstone
				Escape tunnel		
8	535	Claystone and siltstone	Sandstone	Sandstone	Sandstone	Sandstone
9	742	Siltstone	Claystone	Siltstone and claystone	Siltstone and claystone	Siltstone and claystone
10	850	Siltstone	Siltstone	Siltstone	Siltstone	Siltstone
11	905	Claystone and sandy siltstone	Clayey siltstone	Siltstone	Siltstone	Clayey siltstone
12	1014	Sandstone	Sandstone	Siltstone	Siltstone	Siltstone and sandstone
13	1196	Sandstone	Siltstone and sandstone	Siltstone	Claystone	Claystone and siltstone
14	1804	Claystone and siltstone	Claystone and siltstone	Siltstone and claystone	Siltstone and claystone	Siltstone and claystone

Wherever there is exposure of claystone (Table 4), the displacement is found to be more and vise-a-versa. Displacements in targets T2, T3 and T4 in main tunnel are also influenced by the bench excavation. The increment in displacement of T2 and T4 is more than in T3. Because of benching, maximum increment in displacement is recorded, as expected, in claystone and mixture of claystone and siltstone (ch. 465m and 948m). Hence it is not only the width or span of the tunnel, which affects the displacement in arch above spring level, but the bench excavation can also affect the displacement of arch above soring level in weaker rock formations.

Displacements of targets at one location are not uniform, even if the rock mass is same (ch. 837m, main tunnel). At ch. 837m the displacement ranges from 0 to 130mm. Hence in addition to the non-uniform geology, in situ stresses, anisotropy of deformation modulus in bedded rocks and two tunnels in parallel affects the displacement values in both the tunnels.

To arrest the displacement around target T4, at many locations, the supports have been strengthened in terms of installing longer rock bolts and spraying thicker layer of shotcrete. The displacement of targets T1 and T5 in escape tunnel, in general, is more than the displacement in the main tunnel (Table 3). Displacement of target T4 in main tunnel, on the other hand, is maximum. Therefore the supports shall be installed as per the trend of the rock mass behavior, i.e. more supports (longer rock bolts spaced at 1.75m centre to centre; thicker layer of SFRS 250mm) shall be applied to contain the displacements and loosening on the rock mass around the weaker rocks exposed in the tunnels in non-uniform geology. The above explanation shows that claystone and mixture of claystone and siltstone have high order of displacement in Chenani-Nashri tunnels and therefore shall be tackled with care while tunnelling.

The convergence values obtained from Barton (2008) (Table 2) cannot be compared with the displacement values given in Table 3 because of the non-uniform geology and the displacement monitored in tunnel changes with supports, whereas Table 2 shows displacements without supports. In a few cases the support stiffness values have also been used to compare the displacement values. But still there is scope of improvements. Hence, in such non-uniform geology the Q value shall be obtained for all the rock masses exposed to evaluate Equations 1 and 2 and if required to develop a new approach to estimate the convergence in non-uniform geology. It is expected that the heterogeneous rock mass shall behave better than the worst rock conditions locally.

6 Conclusions

Systematic tunnel monitoring by fixing bireflex targets for tunnel displacement has helped in drawing the following conclusions.

The displacements at one tunnel section are not uniform. This is because of the inhomogeneity in the rock mass or non-uniform geology, the in situ stress regime, anisotropic deformation modulus in bedded rocks and two tunnels in parallel.

It is not only the width or span of the tunnel, which affects the displacement in arch above spring level in tunnels, but the bench excavation also affects the displacement of arch above spring level in weaker rock formations. As such, observed displacements are much less than those predicted from the correlation of Barton (2008).

The claystone being the weakest has the maximum displacement. Hence, special attention in terms of extra supports is required for claystone or such weak rock exposed in non-uniform geology.

The rock masses in such conditions need to be classified differently.

7 Acknowledgements

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