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Research Article

Blow-out analysis of head race tunnel: A case study of Solu Khola (Dudh Koshi) hydroelectric project, Nepal

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Abstract

This paper examines the analysis, rectification, and performance of the blowout that occurred at chainage 1+154.12 meters (critical section) in the Head Race Tunnel (HRT) of the Solu Khola Dudh Koshi Hydroelectric Project in Eastern Nepal. The Tunnelling Quality Index (Q) for the critical and adjacent to the collapsed section was derived using probe drilling results and face mapping. The rock support system was evaluated using the Q-system support chart and is validated with the finite element-based program Rocscience (Phase^2). The parametric study was conducted to evaluate the support required for varying the overburden depth from 100m to 650m. The tunnel convergence at the critical section was observed almost for two years before application of final rock supports and was found to be a maximum of 4.277 mm at 27 days, which was quite low and within acceptable limits. The performance study has also been carried out for nearly two years and has been found to be in stable condition since its commissioning. This study presents practical rectification procedure for blowout analysis in Himalayan tunnels, and the findings of the present case study will be helpful to geologists, tunnel engineers, and designers in future tunnelling projects with similar site conditions and constraints.

1. Introduction

According to current estimations, Nepal has a technical potential of 83,000 MW of hydropower plants, with around 42,000 MW being economically viable [1]. Tunnelling requirements for hydropower development in Nepal is simply enormous, as are those for the other sectors of infrastructure development such as highways, irrigation, water supply, mining and storage facilities [2,3]. Despite this, the country is lagging behind in tunnelling projects due to a lack of funding, technical know-how, and a vision for the infrastructure development, as well as a key parameter that contributes significantly to tunnelling instabilities: a complex geological setting with numerous shear zones, folds, faults, and fragile and weathered rock masses, as well as constant tectonic uplifting and downcutting effects by several river systems. Certain specific issues have been reported, like tunnel squeezing [4-6], shearing [6,7] and rock bursts [8,9]. The past tunnelling experience in Nepal shows that the variation between anticipated geology during the planning phase and the actual rock mass found during execution can create significant issues and delays [10]. The geological features of these regions, which includes overstressed rock and the possibility of severe deformation, provide difficult engineering problems. High rates of strain accumulation in the thrust blocks in the area worsen these conditions, which frequently result in seismic events and make tunnelling projects even more difficult [11-13]. The geological complexities of the region make this a constant obstacle to successful tunnelling.

In this context, multiple tunnelling projects are increasingly being planned and carried out in almost all infrastructure sectors across the country, posing numerous uncertainties and challenges due to the the Himalaya's unique geological structure. The excavation of the HRT in the recently

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constructed and commissioned Solu Khola Dudhkoshi Hydroelectric Project, a run-of river (ROR) scheme with a gross head of 614.70 m and a design discharge of 17.05 m^3/s, is one example of such tunnelling. During the excavation of the HRT, a significant geological instability occurred in the form of a blowout at chainage 1+154.12 m. An extensive search of the published data about the similar problems in the region was conducted, such as a case study of the Neelum-Jhelum Hydroelectric Project, Divakar K.C. et al. [14] analysed three hydropower tunnel projects from Nepal and six neighbouring countries to identify similar geological characteristics and engineering issues encountered in tunnelling around the Himalayas. According to [15], in the case of shear zones and running ground conditions, additional remedial measures such as core drilling at the face to determine the precise location of the shear zone, reducing pull length, face advancement with concurrent rib support with backfill concrete, and installation of grouted self-drilling anchors have proven very successful. The researchers determined that the complex interaction between a wide range of geological and geotechnical factors play a vital role in Himalayan tunnelling.

Case studies such as [16], at the Ranganadi Hydro Electric Project (HEP), Subansiri, India, demonstrate the difficulties faced during the construction of the HRT owing to unforeseen geological conditions, as well as the methods adopted to solve the difficult situations. Alam and Sharma [6] at Kameng HEP, Arunachal Pradesh, India, recognized that tunnelling through Himalayan rock formations is a challenging task, but unforeseen problems are unavoidable in any underground excavation. If not addressed effectively, these issues can lead to serious consequences, including project failure. [17] demonstrated that developers and tunnelling professionals must exercise extra caution and preparedness when confronted with severe geological scenarios in the challenging Himalayan terrain. [18] Clearly reveals that, it would be advantageous for engineering projects to be properly conceptualised and planned systematically to ensure smooth implementation, as tunnels are typically located in difficult environments in rocks of various types, hard or soft media, and the alignment may traverse through zones of varying complexities. Rehman et al. [19] suggested using micro seismic prediction technologies for advanced ground investigation and over coring for in situ stress estimation to promptly determine the precise location and time of rock burst occurrences. [20] Concluded that it is difficult to accurately estimate the overbreak risk at the time of tender due to the uncertainty associated with the geology and construction technique and the interaction of these two parameters. The associated construction claims are minimized if decisions are made on time. [21] uses the Tunnelling quality index (Q) to access the rock quality and recommended the optimal support design for adit tunnel constructed in the North Western Himalaya. The tunnel support validation was also carried out using Rocscience software.[22] uses the numerical methods to analyse the excavation sequence and second primary lining support for large section tunnels using the double side heading method.[23] Carried out the integrated approach of numerical and theoretical analysis for investigating the blowout analysis of shield tunnel in sandy soil, and found that the analytical approach shows more accurate findings than the numerical model.

Currently, the majority of the studies in this region are devoted to the geological exploration of construction sites. There are also few case studies of specific projects which focusses on specific challenges or problems [14]. However detailed blowout documentation in Himalayan tunnels is scarce, this may be beneficial to analyze /discover general geological characteristics, as well as probable causes and solutions. In addition, many tunnels, shafts, galleries and underground caverns are likely to be developed in Nepal in the future; however, tunnelling may provide several challenges because of the high degree of uncertainty and risk posed by the complex geological setup of the Himalaya. Therefore, an important aspect of this study is to describe the geological conditions, tunnel instability and the solution to the Himalaya region's blowout (present case study).The major geological uncertainties and challenges encountered during the blowout incident occurred at chainage 1+154.12 m (critical section) in the HRT of the Solu Khola Dudh Koshi Hydroelectric Project, Eastern Nepal are summarised below, along with its rectification and the relevant performance study, which lasted nearly two years. This is one of the few detailed blowout analyses documented in Himalayan tunnels with combined Q-system and numerical validation.

This study presents practical rectification procedures for blowout analysis in Himalayan tunnels, which can be applied to future hydropower projects in the Himalaya with similar site conditions and constraints.

2. Description of the Case study

Solu Khola Dudhkoshi HEP is a ROR scheme with a gross head of 614.70 m and the design discharge of 17.05 m^3/s. This is Nepal's first largest privately owned and operated project. The project has an installed capacity of 86 MW and is capable of generating 520.20 gigawatt hours of electricity per year. The project is located 130 kilometres (aerial) east of Kathmandu, Nepal. Figure 1 shows the geologic map of Nepal with the project location indicated on it. Figures 2 and 3 show the project's layout plan and longitudinal profile, which consists of a simple diversion weir built across the Solu-Khola to divert the water.

Fig. 1. Project location in geographic map of Nepal

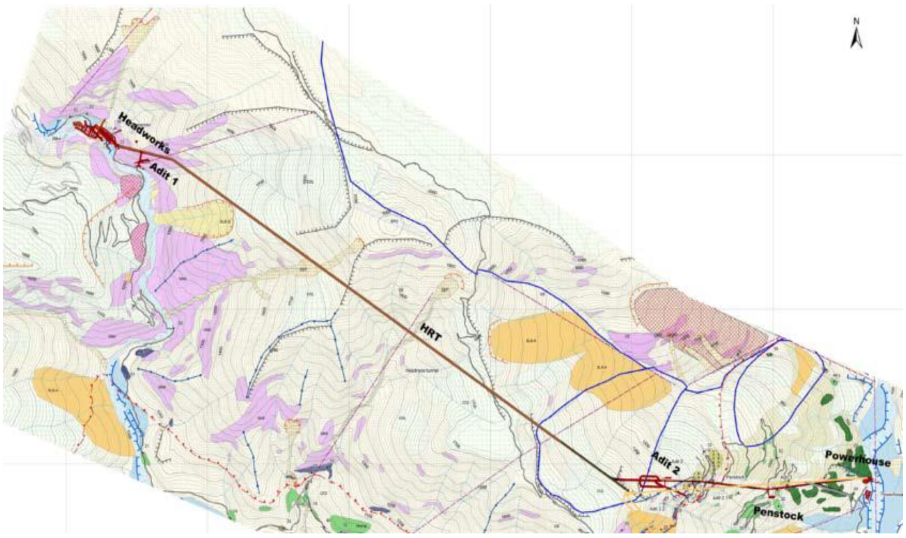


Fig. 2. Project layout

Three intake orifices on the left bank deliver the design discharge into the buried settling basin, which is divided into three bays. A 4469.61 m long headrace tunnel (inverted D shape with a size of 4.0 m (W) x 4.25 m (H), finished) and a 1955.45 m long underground penstock pipe, including vertical shafts and horizontal underground sections up to bifurcation that convey water to the 3-

pelton runner (vertical axis) in the surface powerhouse located on the right bank of the Dudhkoshi River at Maikhu-Besi, in the foothills of the world's highest peak, Mount Everest.

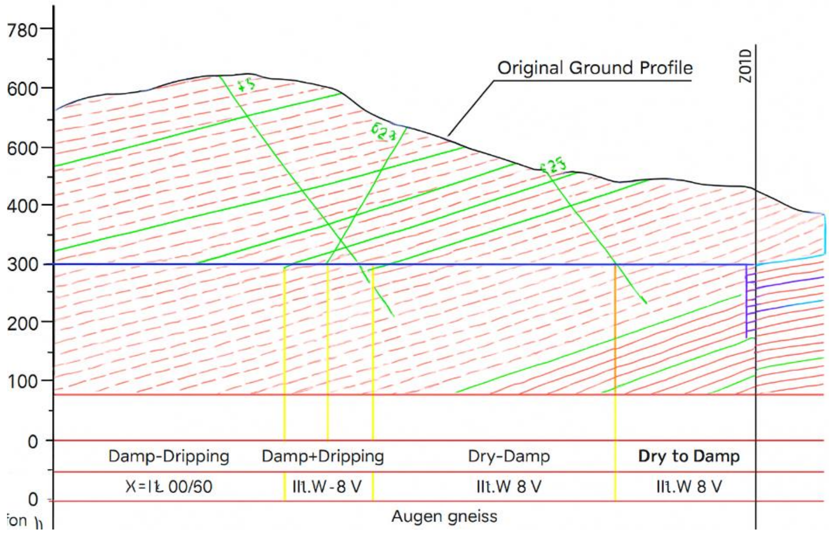


Fig. 3. Longitudinal profile of the project

2.1. Project Geology

The Solu Khola (Dudhkoshi) HEP is located in the Lesser Himalaya zone and is bounded to the north by the Main central Thrust (MCT) and to the south by the Main Boundary Thrust (MBT), as shown in Figure 4, which forms a tectonic window known as the Okhaldhunga Window based on the Nepal Geological Map compiled by the Department of Mines & Geology (1996).

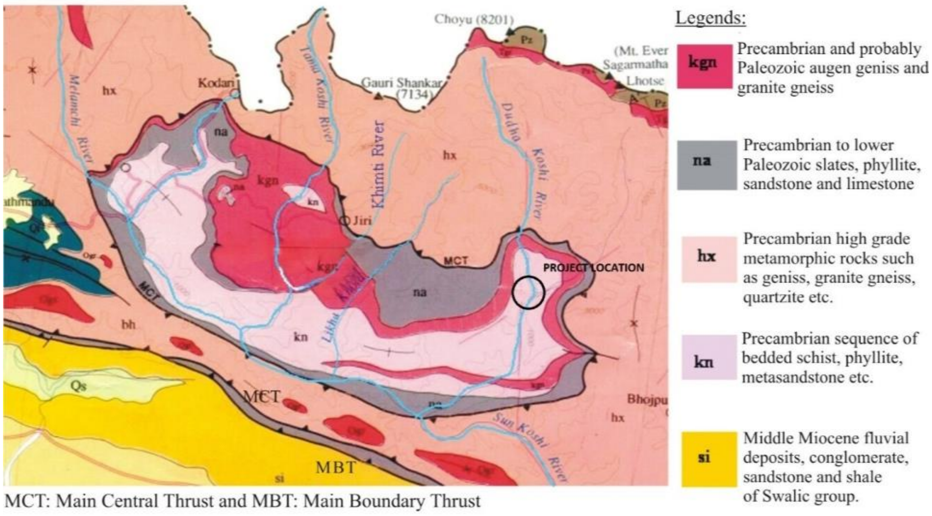


Fig. 4. Regional geological map (compiled by department of Mines & Geology, 1996) showing project site

The project area is situated in the Kuncha formation, which consists of green phyllite of Precambrian age, metasandstone, and quartzite. The area also contains Palaeozoic-aged Augen gneiss. Along the MCT, the tectonic window is separated from the overlying higher Himalaya rock. It is a large dome-shaped anticline deeply cut by the major river, exposing a deep section of the Lesser Himalaya Sequence (LHS). Based on lithology, three stratigraphic sequences of the LHS have been identified in the Okhaldhunga Window, from bottom to top: Lower Nawakot Group, Upper Nawakot Group and Higher Himalaya. The geology along the headrace tunnel is also influenced by three vertical fault zones, 30-35m thick, formed by fractured, brecciated rocks with fine grained fault gouges at around Ch. 0+350 m, Ch. 2+450 m, and Ch. 3+600 m and could be encountered locally for the presence of low angle weak schist bands (shear planes) parallel to foliation, see Figure 5.

2.2. Geological and Geotechnical Investigation

Geological and geotechnical investigations were conducted in the project area to identify geological settings, engineering conditions, and civil construction foundations. The investigation includes surface geological mapping, geomorphology, main fractures and shear zones measurement, identification of foundation conditions for the various structures, overburden conditions, geotechnical properties and support types for underground structures. According to field mapping, the project area primarily comprises of Augen gneiss and green phyllite.

2.2.1 Augen Gneiss

The Augen gneiss extends from the Tingla, Headworks to the Panchan, Surge tunnel area. The Headrace and Surge tunnels were constructed in gneiss. The gneiss is composed of feldspar (55-60%), quartz (20-25%), mica (15-20%), and other minerals (5%). Similarly, the tectonized schist bands made up of muscovite (25%), chlorite (30%), biotite (10%), quartz (10%) and talc & clay (25%) with minor accessories.

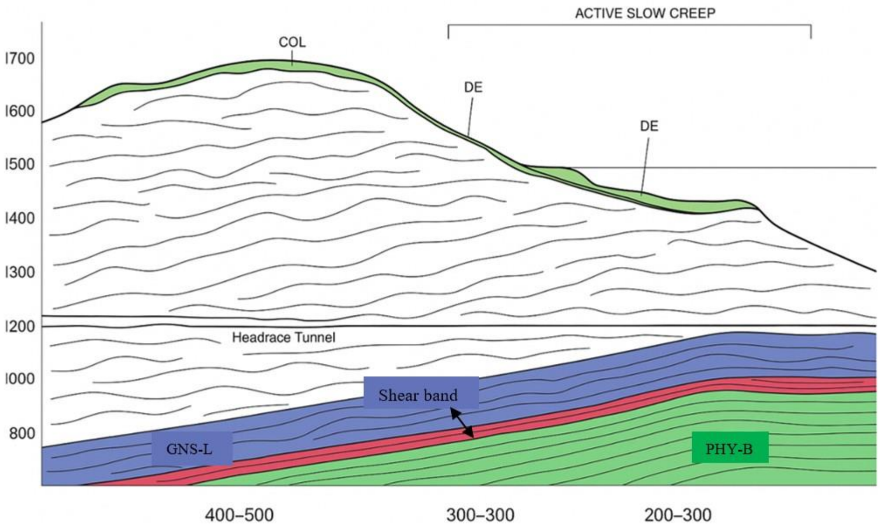


Fig. 5. Geological profile along the HRT

2.2.2 Green Phyllite

The green phyllite known as Kuncha phyllite is grey to green, fine to coarse-grained, slightly to moderately weathered, closely to widely foliated, weak to medium strong, crenulated with strong green quartzite intercalation and has bands 1m to 20m thick. Gritty phyllite, thick quartzite bands with conglomeritic layer and basic intrusions were also observed but not recorded in the Project area. The phyllite extends from Panchan, vertical shaft to the powerhouse area. The vertical shafts, penstock tunnel and powerhouse lie in the phyllite zone.

The rock mass consists of light to dark grey, medium to coarse grained, moderately foliated, moderately to highly weathered, medium to low strength, Augen Gneiss with quartz veins. In some specific sections, augen gneiss is intercalated with phyllite and schist bands. The rock mass consists of three main joint sets and a few random joints. The joint surfaces vary from rough to smooth, irregular, undulating, and planar. The joint spacing ranges from close to moderate (60-600 mm), with short to medium persistence (1-10 m). Joint aperture varies from moderately open to tight. The joint alteration is medium to strongly over consolidated, with clay and silt minerals filling in. The values of dip direction and dip amount of the joints are shown in Table 1.

Specific section contains multiple shear joints (1-20 cm), fracture zones, and shear zones comprising clay, silt minerals, and disintegrated, loose rocks. The overburden in the shear zone varies from 100 to 500 m, and the shear/weak zones are nearly oriented parallel to the foliation plane. The ground water condition of the headrace tunnel is predominantly dry, with some areas being damp and experiencing minor dripping and others having significant inflow. The project area is covered with loose, coarse-grained colluvial and alluvial soils. The flat ridges and terraces also contained residual soil.

Table 1. Orientations of the main joint sets in headrace tunnel

Foliation (J1)	Joint (J2)	Joint (J3)	Joint (J4)
020°-045°/10°-25°,310°-350°/10°-40°	100°-190°/30°-85°	200°-290°/30°-85°	300°-355°/50°-90°,020°-090°/30°-85°

2.2.3 Alluvial Soil

Alluvial soil consists of sub-rounded to well-rounded boulders, cobbles, pebbles, and gravel in a sandy clayey silt matrix. The soil is light grey, with 30-40% fine materials and 60-70% coarse materials. Alluvial soil is a heterogeneous Bimsoil that is described as clean sandy gravel/boulder. The thickness of the alluvium deposit ranges from 5 to 30 m. A diversion weir, settling basin, and tailrace canal are constructed in alluvial soil.

2.2.4 Colluvial Soil

They comprise angular large boulders (<4 m dia.) and gravels in a clayey sandy silt matrix. Colluvial are brown in color and contain about 60-70% coarse materials and 30-40% fine materials. The fine material contains low plastic sandy silt. The deposits also contain lenses of solid clay. Colluvium deposits are considered as a heterogeneous Bimsoil classified as silty sandy gravel/boulder. The thickness of the colluvium deposit ranges from 10 to 50 m. Soil and rock properties and the distribution of different soil/rock types in the project area are mentioned in Table 2. The in-situ stress is not based on actual site testing but rather on measurement carried out in similar rock types in Khimti I Project, about 52 km west.

Table 2. Soil/Rock properties and distribution of different soil/rock types in the project area

Lithology	Characteristic	Percentage based on core logs	Maximum availability
Alluvial Deposit	Sub-rounded to well-rounded boulders (<1 m in diameter), cobbles, pebbles and gravels in a sandy clayey silt matrix. The soil is light grey and contains about 30-40% fine materials and 60-70% coarse materials.	~27%	Headworks and Powerhouse areas
Debris Deposit/Residual Soil	Red and grey in color and mostly developed over alluvial deposits and weathered rock, mostly in gentle slopes dipping towards south. Grey to brown color residual soil observed in Kagel and Panchan ridges. The soil is formed of cohesive clay mix with sand and silt and occasionally contains angular gravels of parent rock.	~10%	Headworks and Powerhouse areas
Augen Gneiss	Light grey colored, thinly to thickly foliated, medium grained, medium strong, parallel laminated, slightly to moderately weathered	~29%	Headworks, Headrace tunnel, Surge tunnel, Penstock tunnel 1
Phyllites B Type	Light greenish grey colored, thinly foliated, medium strong, moderately to highly weathered.	~55%	Powerhouse area. Penstock shaft, Penstock tunnel and surface Powerhouse

Phyllites A Type	Light greenish grey colored, thinly foliated, medium strong, moderately to highly weathered. Texture of Phyllite-A show high abundance of Quartz (48-50%) and minor micas (Muscovite 29% and Biotite 10%) alternating in bands	~6%	Drop shaft 1 and Penstock tunnel 2

2.3. Blowout Mechanism

The HRT was excavated using a conventional drill and blast technique. Several obstacles were encountered during the excavation because to weak zones and joints orientation, with one of the major geological challenges faced and addressed at Ch. 1+154.12m. A sudden and unpredictable emergence of water caused a small collapse on the left crown following a face blast at Ch. 1+151.39 m. This collapse then turns into a huge blowout, with the ingress of a large volume of debris flow (sludge with boulders and cobbles mixed with shear materials) with 4050 m^3 of muck rolled down in the tunnel, forming a huge cavity in the crown approximately 4.5 to 5.0m thick (The failure mechanism is observed as wedge-shaped aided by oblique shear bands and transient hydraulic action) and significantly impeding overall work progress. Figure 6 shows the conceptual sketch of cavity formation at Ch. 1+154.12 m and debris volume calculation. This outflow was accumulated over almost 300m of tunnel section. During the debris flow, water inflow was measured at 3-5 litre/sec.

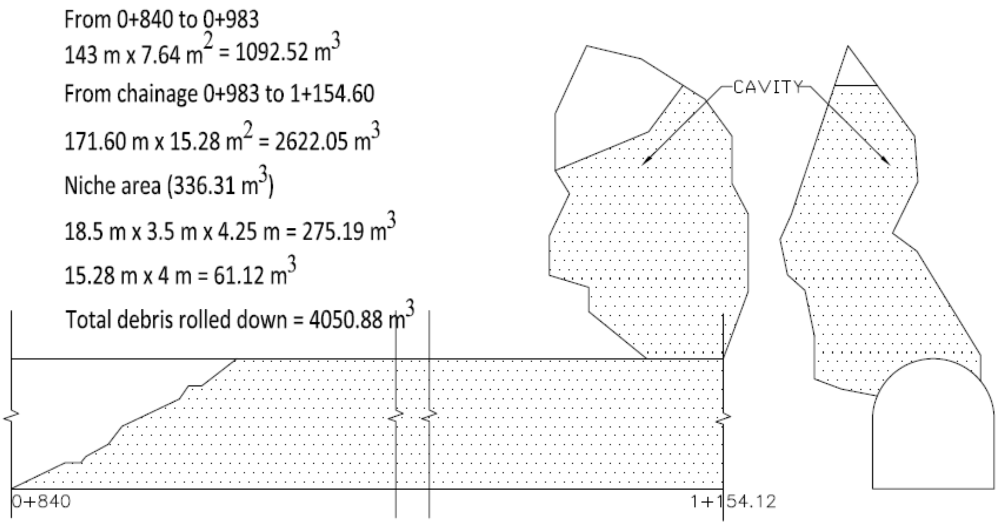


Fig. 6. Conceptual sketch of cavity formation at Ch. 1+154.12 m

Table 1. Brief summary of the blowout at Ch. 1+154.12 m

S.No.	Descriptions	Date	Remarks
1	Excavation at Ch. 1+148.94 m	16-Jan-20	RQD=50, Q=0.5, dry to damp, pull length=2.45 m
2	Face blast at Ch. 1+151.39 m	17-Jan-20	8.55 am
3	Face mapping at Ch. 1+154.12 m		Approx. 11:00 am, RQD=50, Q=0.41, refer calculation in the Table 4, dry to damp (Figure 5a), pull length=2.73 m
4	Overbreak recorded		Approx. 6:40 pm, spring line to crown left HRT face
5	Back fill & seal at face	20-Jan-20	
6	Concreting at overbreak face	Jan 21-25, 2020	But without success

7	OPC grouting via SDR	26-Jan-20	SPL to left crown, 106 bag consumption
8	Steel rib Installation	27-Jan-20	2 numbers
9	Removal of sealed face	28-Jan-20	trimmed the face, collapsed occurred
10	1st and 2nd debris Flow	28-Jan-20	mini excavator partly buried; water seepage 4 Liter/sec
11	3rd debris flow	31-Jan-20	debris reached back to Ch. 0+840 m
12	Start removal of debris	4-Feb-20	

The face at Ch. 1+154.12 m was found to be almost dry to damp during face mapping in the morning of the same blast (Figure 7a). A significant collapse occurred at nearby Niche, which extends from Ch. 1+118 m to Ch. 1+142 m, and all of the steel supports at the face were twisted and uprooted, as were all of the utility's services. Table 3 provides a brief summary of the blowout and Figure 7 (a-d) shows the photographs at Ch. 1+154.12 m before and after the blowout, which resulted in three debris flows inside HRT. The material accumulated during three successive debris flows within the tunnel consist several big blocks of rock mass, along with the fine materials of the sheared zone. These blocks were most possibly considered the potential wedges of hanging walls and crowns next to the sheared band. The volume of debris accumulated in the tunnel indicates the formation of a very big cavity at the centre, extending from the left corner of the crown inwards to the left side slopes.

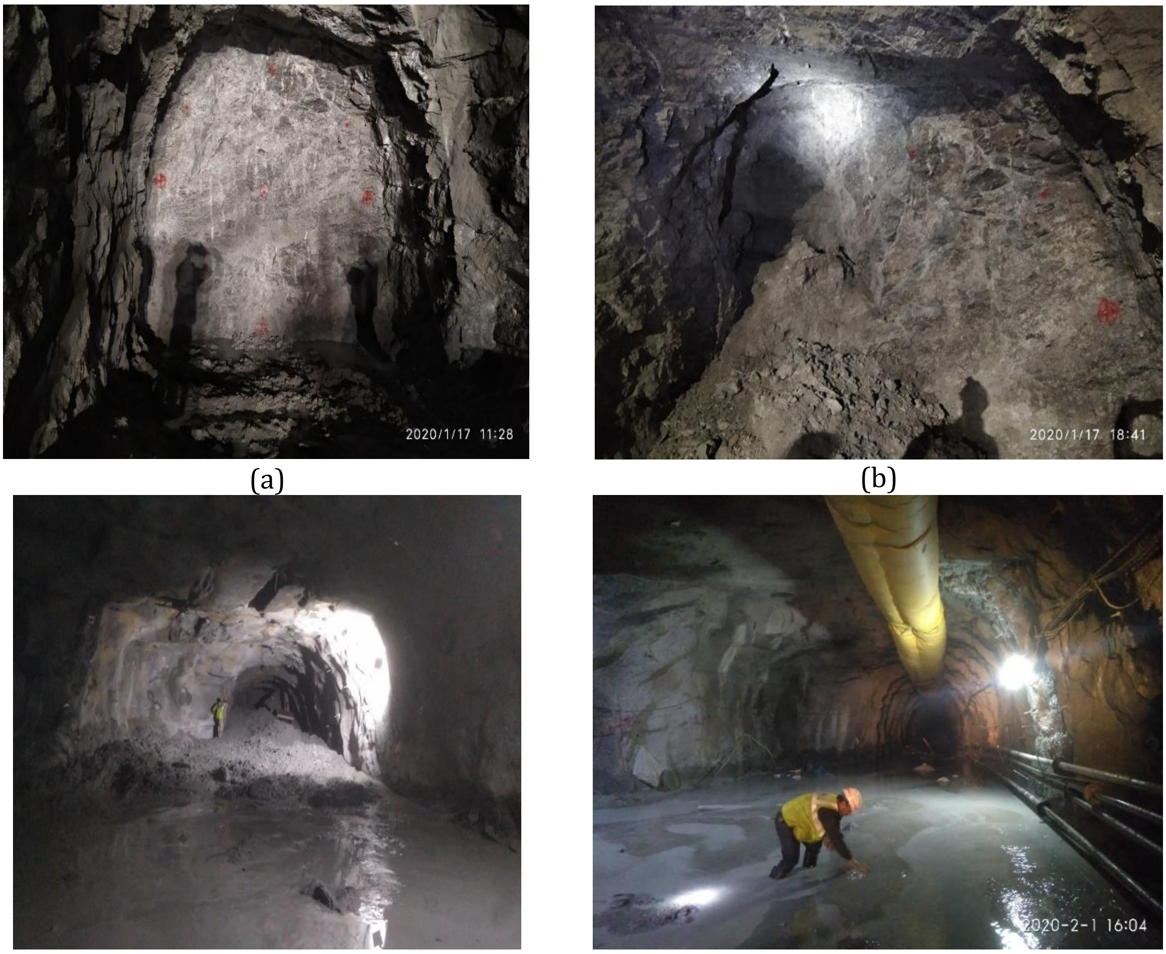


Fig. 7. Photographs of HRT before and after the blowout (a) Before collapse, (b) Small over break on the left crown, (c) First debris burst and(d) Final debris flow

2.4. Blowout Analysis and Estimation of Support

The purpose of blowout analysis is to ensure the tunnel's stability and safety during the excavation and operation. In the present case study, the large volume of outwash debris must have created a large cavity at and adjacent areas of HRT chainage 1+154.12 m, offering a significant risk of instability to HRT and the adjoining niche (at chainage 1+118 m to 1+142 m). The probe drilling findings through the niche walls revealed that the sound rock mass occurs up to the depth of 20 m. It was indicated that the cavity was not significantly migrated towards the back of the left walls. The rock mass quality at Ch. 1+154.12 m prior to the collapse was evaluated using the Barton's Q-system [24]. In the present study, Q values are estimated, and Rock Mass Rating (RMR) values are computed by applying the following equations proposed by Barton (1995).

$$RMR = 15 \log Q + 50 \tag{1}$$

During construction, the predicted Q-value and computed RMR value along the tunnel alignment are further evaluated. The Q-method is a numerical description of the rock mass quality with respect to rock stability, and it includes more support categories with a wide range of flexibility for varied ground surface. Therefore, in this work, Q-method is used to estimate the required rock support in tunnels and caverns. Table 2 shows the rock mass quality assessment at Ch. 1+154.12 m prior to the collapse.

Table 2. Rock mass Quality assessment of the face at 1+154.12 before the collapse

Q=RQD/Jn*Jr/Ja*Jw/SRF		
RQD	Rock Quality Designation	50
Jn	Joint set number	12
Jr	Joint roughness number	2
Ja	Joint alternation number	4
Jw	Joint water reduction factor	1
SRF	Stress Reduction Factor	5
Q	Tunneling Quality Index	0.41(Very poor)

The Q value came out to be 0.41, indicating very poor rock mass. The support system for the face at Ch.1+154.12m was predicted using the Q-chart (see Figure 8). The Excavation Support Ratio (ESR) of 1.6 was used to estimate the support class and its type. This is because, in addition to the Q-value, two other criteria, such as safety requirements and opening dimension, play a crucial part in support design. A road tunnel or underground power house requires higher level of safety than a water tunnel. Furthermore, the Q-System (Barton & Grimstad, 1994) [25] recommends an ESR value of between 1.6 and 2.0 for predicting support requirements for hydropower and water tunnels.

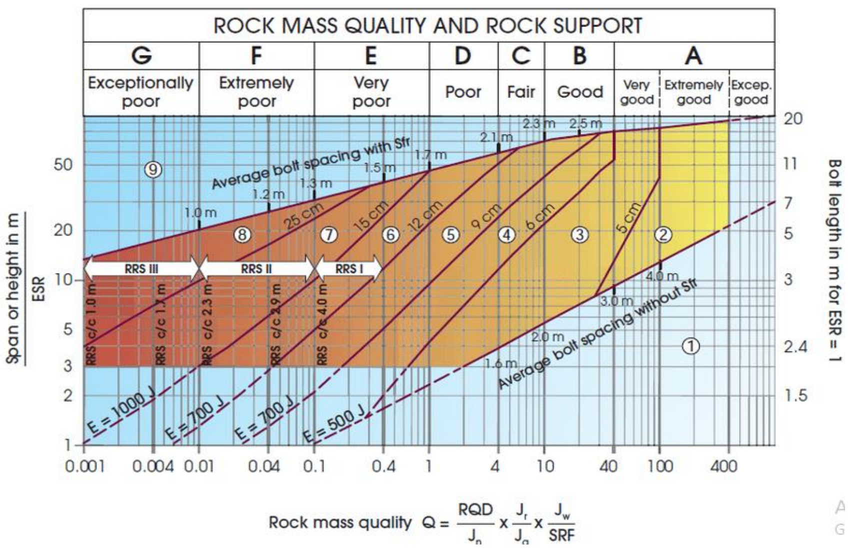


Fig. 8. Q-system Support Chart (After Grimstad and Barton, 1994) [25]

Figure 9 shows the features of support applied for very poor rock mass at chainage 1+154.12 m; after rectification this support system was reinforced again from Ch. 1+142 m. The geological condition of and adjacent chainage to the collapsed section (Ch. 1+154.12m) was assessed using the tunnel face photographs during excavation, face maps with Q estimation, and the probe drilling findings. As the excavation advanced, the rock mass continued to degrade, attaining a low Q value of 0.0042 from Ch. 1+155.23 to Ch. 1+165.60m (Figure 10) which is almost constant. After Ch. 1+165.60m, it gradually increased after reaching a low point when a major collapse occurred, indicating that the rock mass was beginning to stabilize, which was achieved through pre-grouting.

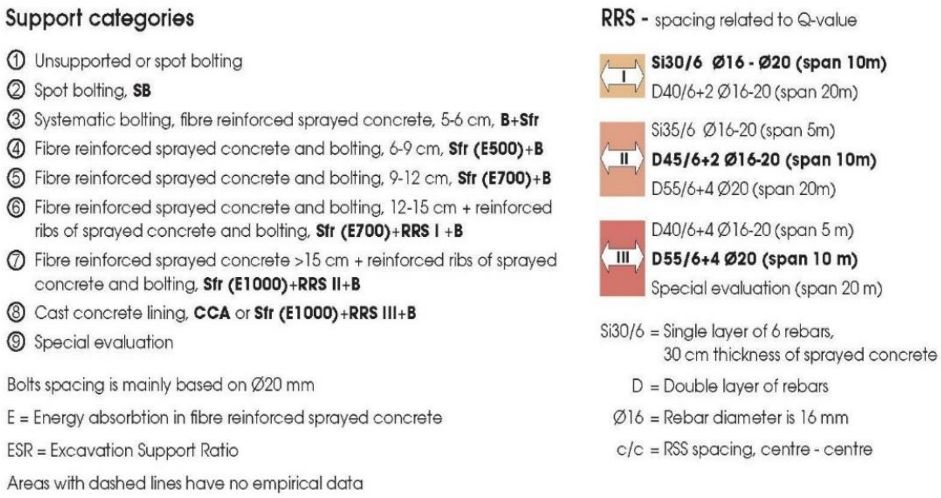


Fig. 9. Prediction of rock support at chainage 1+154.12 m

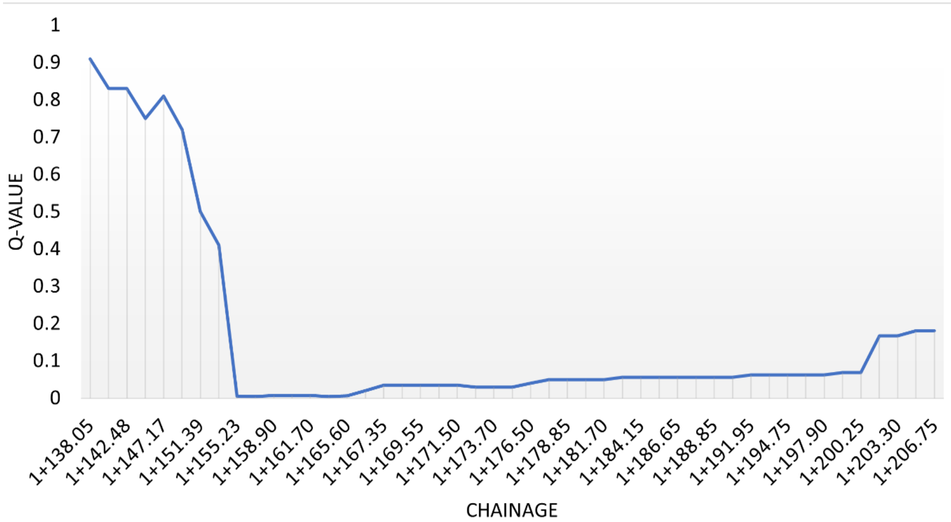


Fig. 10. Variation of Q-value in the problematic stretch

Post-collapse, the geology of the problematic section was investigated using the probe drilling technique. Figure 11 shows the delineation of the location and orientation of probe drilling. Crushed shear bands crossing oblique to the tunnel axis was observed, whereas Figure 12 shows additional probe hole locations.

The probe drilling results indicated that the total thickness of the weak mass at and adjacent to Ch.1+154.12 m must have been roughly 5m, but it was reported to be considerably higher at some sections due to swelling and pinching occurring laterally; see Figure 11. While the thickness of crushed material can be approximately estimated to be no more than a meter at face Ch. 1+156.94m after rectification, as illustrated in Figure 13.

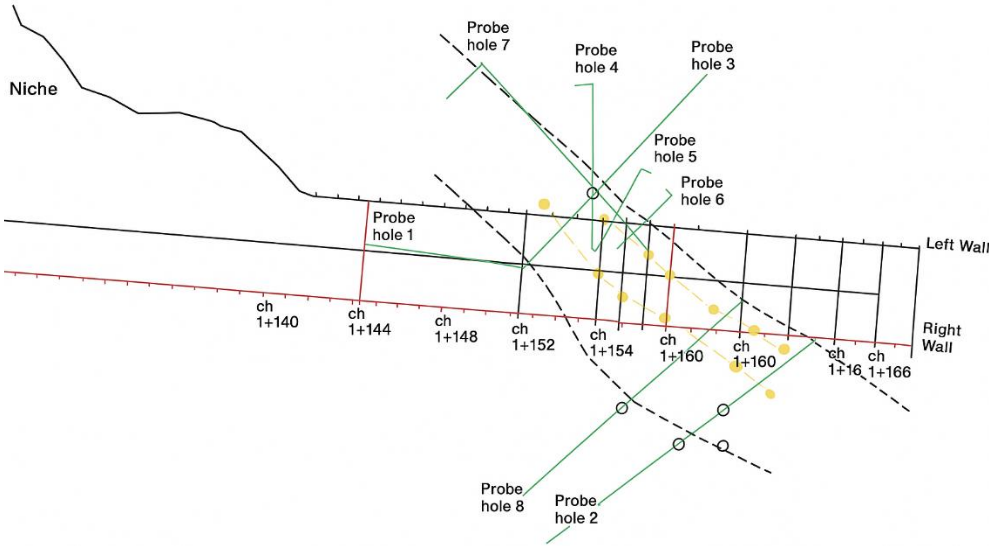


Fig. 11. Delineation of the location & orientation of probe drilling shown on plan of HRT

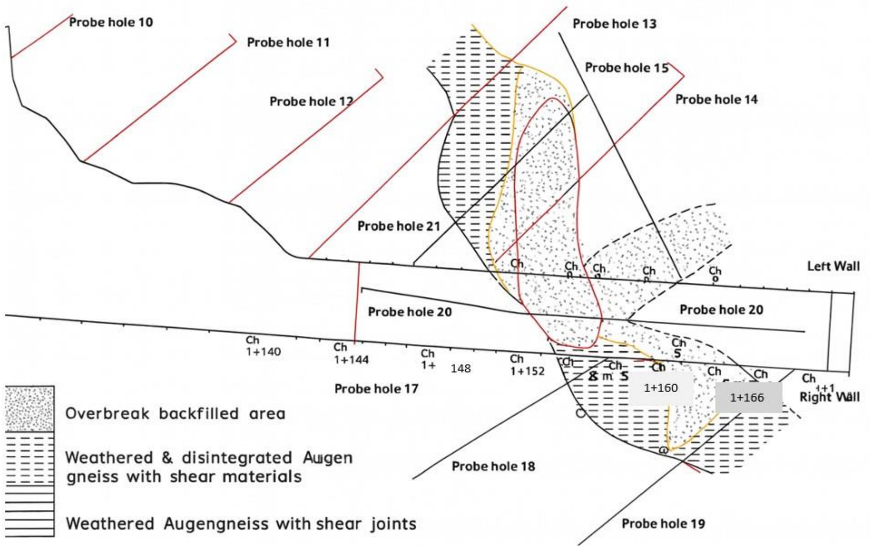


Fig. 12. Geology of the collapsed section (HRT plan view after probe drilling)

Fig. 13. Face Ch. 1+156.94 m after rectification of collapse section on June 21, 2020

The rock support class for "poor" rock mass obtained in the niche section (1+118 m to 1+142 m) was reviewed, with the Joint Set Number (J_n) adjusted as recommended by the Q-system for the intersections or junctions. As a result, the revised Q-value for the same was estimated to be "very poor" rock mass, and the corresponding rock supports were applied as shown in Figure 14, along with an additional 20 cm of fibre reinforced shotcrete, as briefly described in Table 8 below.

As demonstrated in Figure 10, the rock mass Q-value gradually decreases from 0.833 to 0.0042 (from Ch.1+42 m to 1+156 m), indicating exceptionally poor rock. This is due to the deterioration of rock mass characteristics. The rock mass quality assessment was carried out using geological face mapping with visual inspection and each discontinuity parameters are observed for the Ch.1+42 m to 1+156 m. Table 5 shows the rock mass quality assessment Ch.1+42 m to 1+156 m after post collapse.

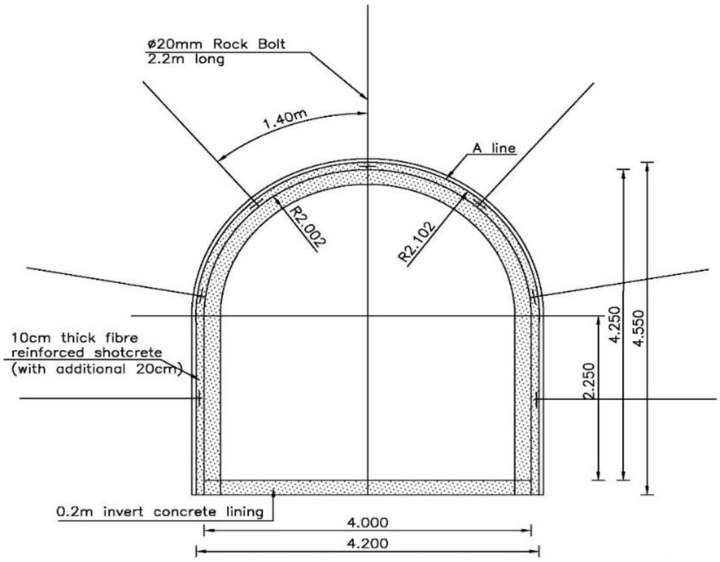


Fig. 14. Final Concrete Lining for Chainage 1+118 m to 1+142 m

Table 5. Rock mass quality assessment after post collapse from Ch.1+42 m to 1+156 m

Rock Quality Designation (RQD)	Joint set number (Jn)	Joint roughness number (Jr)	Joint alternation number (Ja)	Joint water reduction factor (Jw)	Stress Reduction Factor (SRF)	Tunneling Quality Index (Q)
50	12	3	3	1	5	0.8333
60	12	1.5	2	1	5	0.7500
65	12	1.5	2	1	5	0.8125
65	12	2	3	1	5	0.7222
50	12	2	3	1	5	0.5556
50	12	2	4	1	5	0.4167
10	20	1	10	1	10	0.0050
10	20	1	12	1	10	0.0042

The RQD decreased from 50 to 10, while the Jn increased from 12 to 20, indicating an increase in rock mass instability. Furthermore, the increase in Ja from 3 to 12 and decrease in Jr from 3 to 1 indicate a significant decrease in the shear strength of rock mass. According to the Q-system Support Chart [25], the recommended rock support (based on the Q-value) is "exceptionally poor rock" mass, which is classified as support category 7 is as follows;

- Rock support as per Q-System: Support Categories 7
- Fiber reinforced sprayed concrete> 15cm+reinforced ribs of sprayed concrete and bolting.
- Sfr (E1000) + RRS II @ c/c 1.7 m + B;

- RRS II= Si35/6 Ø16-20 (Span 5m) (single layer 6 rebars), 35cm thickness of sprayed concrete.

A similar analysis was carried out for the construction of HRT of Kameng Hydroelectric project, India, located in North East Himalaya region. The rock support system was designed based of Q system. During the excavation of HRT, formation of cavity and squeezing effect was observed at faces-II and III. It was analysed that the squeezing problem was predominant in the areas with overburden ranging from 800m to 1000m. In the present case study, the parametric study was carried out to determine the rock support system for overburden variation ranging from 100 to 650 m.

3. Numerical Modelling Using Rocscience (Phase^2)

The rock support designed according to the Q system is validated by numerical simulation using the finite element -based software, Rocscience (Phase^2). The main rock type throughout the headrace tunnel is augen gneiss, with weak tectonised schist bands (weak/shear zones) of varying thickness along the tunnel section. The Generalized Hoek-Brown Failure criterion parameters are used in the analysis, Table 6 shows the rock mass strength parameters (mb,s,a), GSI and Disturbance coefficient (DF) for the are gneiss and shear/weak zones.

The Geological Strength Index (GSI) assesses the quality of rock mass. Its values range from 10 (for exceptionally poor rock) to 100 (for unjointed rock mass). GSI values can also be correlated with RMR, as shown by the following relation:

$$GSI = RMR - 5 \tag{2}$$

Table 6. Geotechnical parameters of the rock mass (Gneiss) and shear/weak zones

Rock mass properties	Gneiss		Shear/weak zones	
	Peak	Residual	Peak	Residual
Intact UCS (σ_{ci} , MPa)	75	75	25	25
GSI	40	40	20	20
Intact Rock Constant (m_i)	28	28	12	12
DF	0	0.25	0	0.51
m_b	3.285	2.419	0.6892	0.2625
S	0.127	0.069	0.0138	2.282E-05
a	0.5114	0.5114	0.5437	0.5437
Modulus Ratio	525	525	250	250
Rock mass Modulus (E_{rm}, MPa)	6286.3	4352.9	770.7	500.6

In the present study, the GSI value used for exceptionally poor rock mass with overburden less than or equal to 350 m is 20, however during modelling, a GSI value of 25 was used because the Q value was corrected above for the Strength Reduction Factor (SRF). The RMR was derived for the observed Q value and then used the preceding equation (2) to obtain the GSI.The total displacement contours (Figure 15) and support capacity plots of final rock supports (Figures 16 and 17) for exceptionally poor rock mass with an overburden of 650 m (higher rock cover) are shown as follows.

The maximum total displacement at tunnel face near the crown and side wall is 0.097 and 0.114m, indicating that the preinstalled support at crown will move less than invert/side wall. Displacement adjacent to the lining larger, ranging from 0.06 to 0.15m, indicating that the ground movement occurred near the lining by nearly 0.1m and decreased almost zero further away. This could be due to load redistribution at the lining, which will create a shear and bending.

Support capacity plots are used to check the stability of the applied supports, which includes envelopes of the lining support for different factors of safety and each support finite elements as points. Figure 16 indicate the support capacity plot of ISMB200 at 1m c/c spacing, whereas Figure 17 represent the plot for final concrete lining of 25cm thickness. The MN curve shows the relationship between applied bending moment and associated tensile capacity of the section.

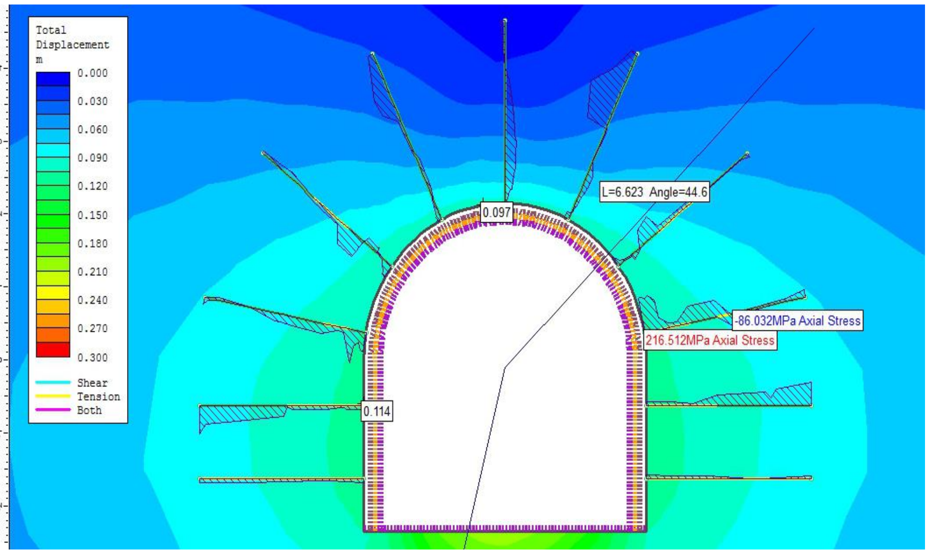


Fig. 15. Total displacement contours with axial stress on rock bolts

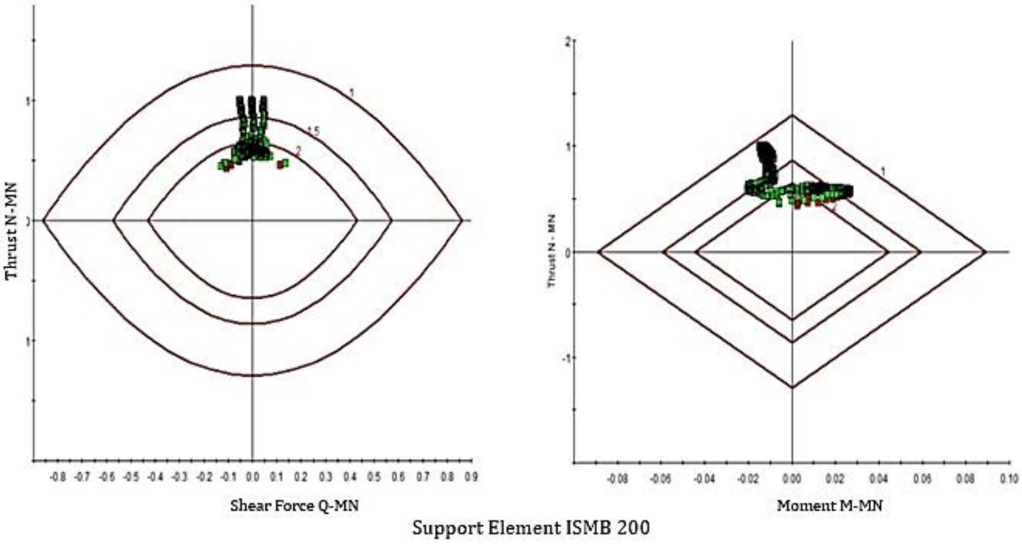


Fig. 16. Support capacity plot of ISMB 200 at 1m c/c spacing

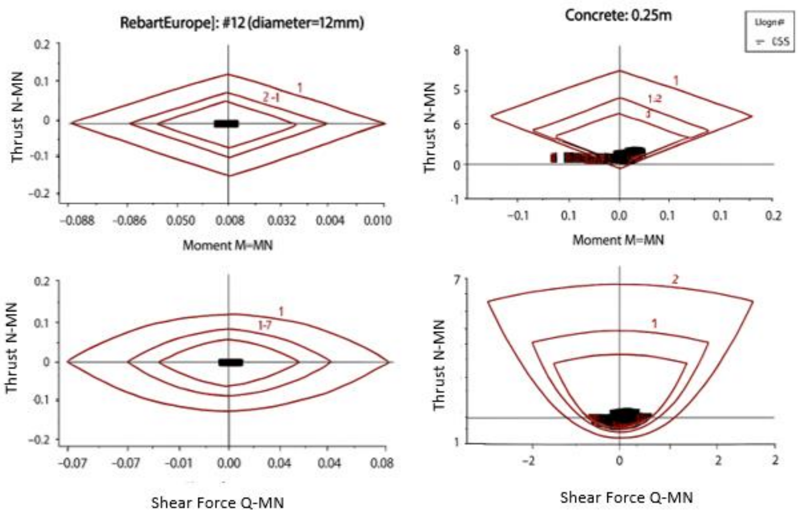


Fig. 17. Support capacity plot of final concrete lining of 25 cm thickness

The aforementioned figures shows that the support element of Rib fall inside the capacity plot of 1.0. It can be concluded that the use of ISMB 200 provided at 1m c/c meets the necessary safety factor in all conditions. Because the applied load does not follow the path where stiffness decreases, it remains within the elastic zone, where deformation does not increase the stresses and the risk of progressive deformation, such as development of shear zone or stiffness degradation, is low. The support capacity plot of concrete lining for overburden of 650 m falls slightly outside of the Factor of Safety 1.5 envelope, as shown in Figure 17, but this is only the case for the corner of the invert; this is due to stress concentration (idealized sharp corners), which may naturally put the structure back in to the stable loading zone. Although such sharp corners are not formed in the rock mass during actual construction, it is anticipated that this level of stress concentration will not occur. Overall, the displacement and support capacity curve revels that the rock support system is stable and free from any progressive deformation. Some modifications to the rock supports were proposed for the "exceptionally poor rock mass" for overburden depths >350 m to 650 m, as shown below.

- Fiber reinforced sprayed concrete =20 cm and Rock Bolts= 2.5 m long (Ø 25 mm) @ 1.0 m spacing and Spilling bolt= 6m long (Ø25 mm) @ 0.4 m c/c spacing
- Steel Rib= ISMB 200 @ 1.0 m c/c spacing and Concrete Lining= 25 cm, Grade C25.

When analyzing ground behavior with weak zones, 3D models provide more precise findings than 2D models due of few of limitations such as;

- The 2D model cannot account for the sequential nature of excavation and support placement. 2D models cannot exactly depict the tunnel alignment stresses. These models are less capable of handling varied geological environments.
- Despite these limitations, 2D analysis is the primary tool for tunnel behavior, support analysis, and design. Furthermore, they are frequently less expensive and quicker to develop.

In 2D modelling, the influence of soil structure interaction can be considered using a structural interface, while in 3D modelling, lining compositions are considered, which is more adaptable. However, the 2D modelling programs are integrated, making it simple to convert 2D models into 3D models. Compared to 3D models, 2D models are more suited for sensitivity analysis. The sensitivity analysis can be used to verify the uncertainty of parameters. This process can identify the parameter that have the greatest influence on model output or probabilistic technique can be used to identify the impact of uncertainty on model output.

3.1. Parametric Study

The numerical modelling was carried out for exceptionally poor rock mass with variation overburden depth and the support required for the variation of overburden depth from less than 100m to 650m was obtained and presented in Table 7.

Table 7. Support required for exceptionally poor rock mass for different overburden depth

Overburden depth	Rock Bolts	Shotcrete	Spilling Bolt	Steel Rib	Concrete Lining	As built support
<=100 m	2.5 m long (ø25 mm) @ 1.0 m spacing	15 cm	6 m long (ø25 mm) @ 0.4 m c/c spacing	Not Required	Not Required	25mm ø bolt of 2.5m long @ 1.m spacing; 15cm sprayed concrete with spiling bolt of 25mm ø &6m long. At weak area temporary steel rib
>100 m - 350 m	2.5 m long (ø25 mm) @ 1.0 m spacing	20 cm	6 m long (ø25 mm) @ 0.4 m c/c spacing	Not Required	20 cm, Grade C25	25mm ø bolt of 2.5m long @ 1.m spacing; 20cm sprayed concrete with spiling bolt of 25mm ø &6m long with 20 cm concrete lining of C25 grade and weep holes

>350 m – 650 m	2.5 m long (ø25 mm) @ 1.0 m spacing	20 cm	6 m long (ø25 mm) @ 0.4 m c/c spacing	ISMB 200 @ 1.0 m c/c spacing	25 cm, Grade C25	25mm ø bolt of 2.5m long @ 1.m spacing; 20cm sprayed concrete with spiling bolt of 25mm ø &6m long with an additional ISMB 200 steel rib with 1.0m c/c spacing and a final lining thickness of 25 cm of C25 grade

According to the analysis, lower overburden depths (<100 m) do not require concrete lining and steel ribs, as 15 cm of shotcrete can serve as the final lining. However, for overburden depths more than 100 m and up to 350 m, a 20 cm layer of shotcrete with a final concrete lining of 20 cm is required. For overburden depths greater than 350 m, an additional ISMB 200 steel rib with 1.0m c/c spacing and a final lining thickness of 25 cm is needed. This is due to higher stress, which can be stabilized by allowing the stress to relax by controlled deformation and then simply resisting the remaining force. This leads to improved ground conditions and financial assistance. Consequently, the tunnel was stabilized using a sliding steel set (ISMB 200) with 6.3662% strain before locking for higher overburden stress. It is recommended to apply the sliding steel set immediately after the excavation with safety shotcrete if overburden/rock cover exceeds 400 m. Four number of sliding joints should be provided. The sliding joints should have 25 cm gap to allow for 6.3662% strain before locking.

4. Case Study-Based Solution

The methodology adopted aim to address specific requirements, which included identifying the source, cause, location and extent of the cavity. The cavity that had formed remained unidentified at the time of blowout and was targeted with probing as suggested in [26], stabilization, and continuous monitoring while advancing the rectification procedure between February 4th and February 20th, 2020, muck and debris were cleared from Ch. 0+0840 m to Ch. 1+142 m, over 15 m behind the collapsed face. The face was sealed at Ch. 1+142 m to improve rock support prior to further muck cleaning and advancement. To resume further excavation work, as briefly described in in Table 8, additional 12 mm metal sheet lagging with voids between the applied rock supports and the steel liner filled with contact grouting. This was supported by additional lattice girders with concrete at the wall up to half of the spring line, as shown in Figure 18.

The tunnel width is reduced from 4.30 m/4.50 m to 3.55 m and was evaluated to have no significant hydraulic issues, assuming a 20 cm final lining at a later stage in addition to the initial rock support provided during the restoration operations. The hydraulic capacity for the reduced cross section was checked by comparing the flow velocity of the finished section to the permissible velocity suggested by P.J. Bier's [27] equation and was found to be within the permissible limit.

$$v = 0.125\sqrt{2xgxH} \tag{3}$$

$$v = 0.125\sqrt{2x9.81x25}$$

$$v = 2.76\ m/s$$

Where; v is permissible velocity in m/s; H is head of water in m and g is 9.81 m/s^2

The actual velocity of the finished tunnel section for design discharge of 17.05 m^3/s and cross-sectional area of 11.28 m^2 is come out to be 1.51m/s, which is less than the permissible velocity of 2.276m/s.

Though the supported section was stable when empty, the final rock support along this stretch should be designed to withstand the maximum internal water pressure during water filling and plant operation. Because of the constant water head exerted by the hydraulic gradient line, which is approximately 25 m above the crown at this stretch (Ch. 1+142 m to Ch. 1+162 m). Following the completion of the entire excavation, initial rock supports in the portion were strengthened with self-compactable reinforced concrete lining of grade C35 together with steel lining (12 mm), as indicated in Figure 19. Ribs were closed at the base with struts encased with concrete, forming a ring closure. The collapse occurred during the HRT excavation, which delayed the inlet excavation by 6 months due to the necessary rectification work on that section, and the first blast was only possible on July 21st, 2020. The rectification work sequence at the HRT inlet from Ch. 1+142 m to Ch. 1+154 m and onwards is presented below.

Fig. 18. Support at collapse section prior excavation resumes

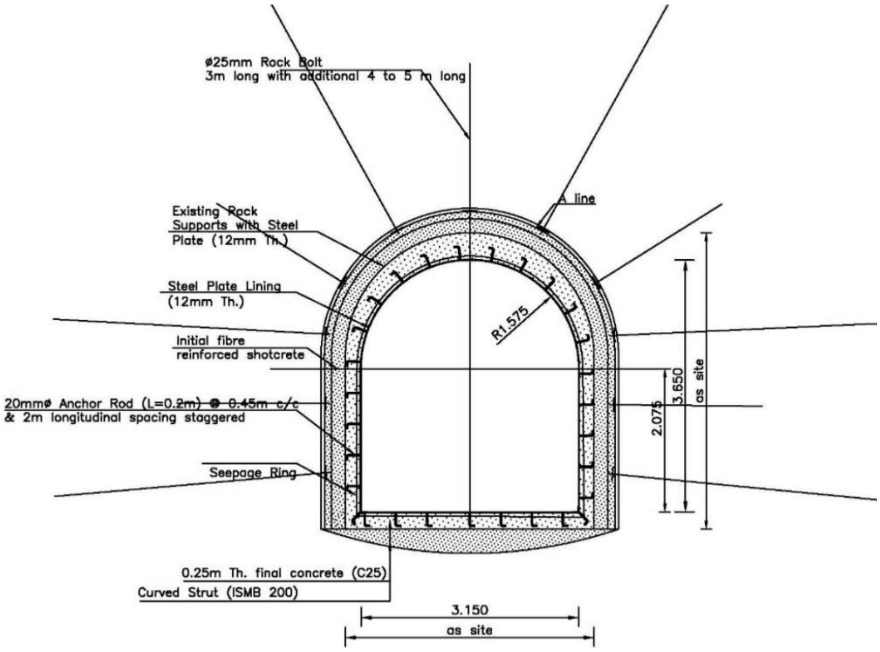
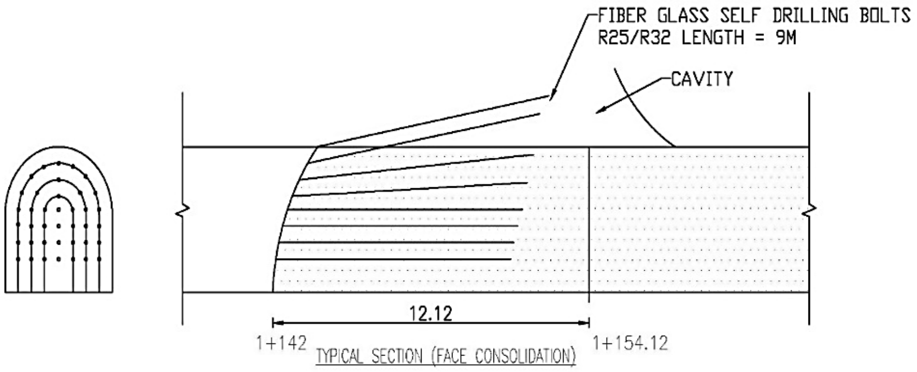


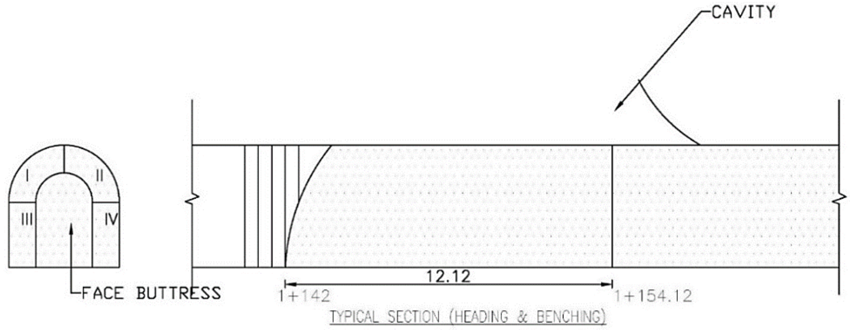
Fig. 19. Final support for chainage 1+142 m to 1+162 m

4.1. Work sequence for HRT inlet rectification from Ch. 1+142 m to Ch. 1+154 m and onwards

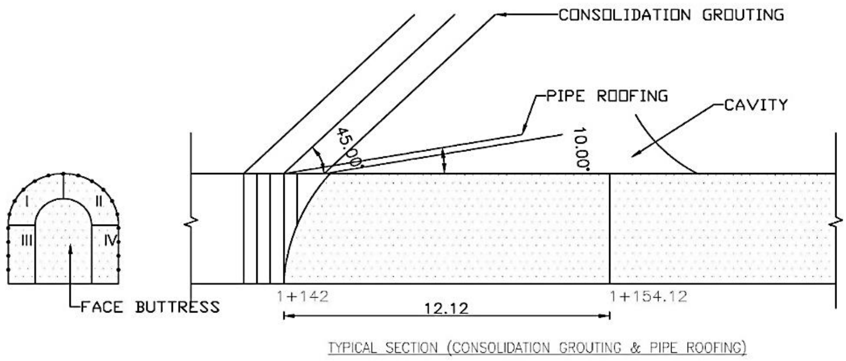
The methodology adopted to rectify the HRT from Ch. 1+142 m to Ch. 1+154 m and onwards is presented below. Prior to the rectification, the back chainage (Ch. 1+118.00 m to 1+142.00 m) also known as niche section was strengthened. This process includes installing a safety plug at Ch. 1+142.00 m and face stabilizing the muck using fibre glass SDRs. To increase stability, an additional 200mm of Fibre Shotcrete was applied. Longer rock bolts (4 to 5 m long, 25mm diameter) were installed in a staggered pattern, 2m apart. Finally, consolidation grouting was carried out. The work sequence for rectification from Ch. 1+142.00 m to Ch. 1+154.12 m and onwards is listed below in Table 8 and Figure 20 (a-e), shows the key action and the support elements/outcomes.



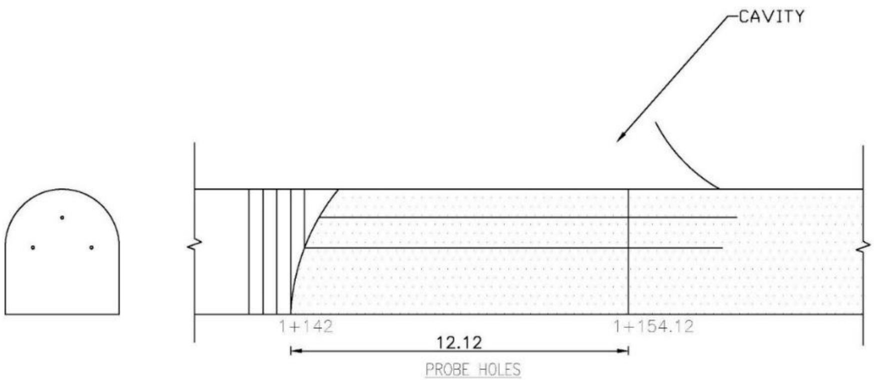
(a) Face stabilization



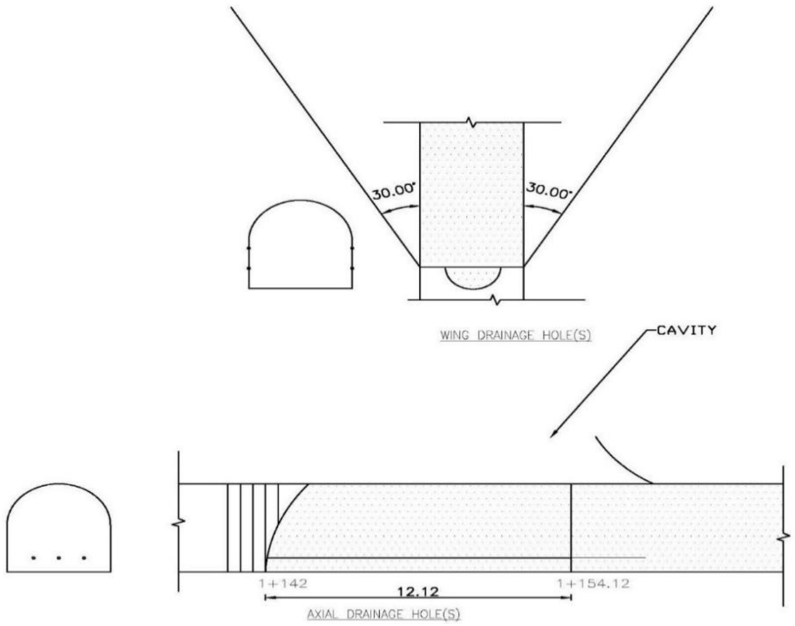
(b) Heading and benching



(c) Consolidation grouting in a fan pattern



(d) Probe holes process



(e) Drilling drainage holes

Fig. 20. (a-e). Work sequence for rectification of critical section (Ch.1+42 m to 1+156 m)

Table 8. Rectification methodology for critical section (Ch.1+42 m to 1+156 m)

Sr. No.	Key action	Support element
1	Face stabilization	wire mesh/fiber shotcrete/fiber glass SDRs
2	Heading and benching	support with Rib/Lattice Girder for the excavated upper part, and extension of the Rib/Lattice Girder
3	Support	Full support for the installed Rib/Lattice Girders with wire mesh/fiber shotcrete
4	Consolidation grouting in a fan pattern	Consolidation grouting in a fan pattern with installation of pipe canopy
5	Probe hole process	For cavity treatment
6	Drilling of drainage holes	Drilling for grouting and drainage was used for continuous probing and assessing the tunnel face and crown conditions

The above process repeated until the normal face excavation started. During the rectification process, the following precautionary steps were implemented:

- A face Buttress was sufficiently kept at the centre of the face to prevent releasing thrust during heading and benching.
- Drilling for grouting and drainage was used for continuous probing and assessing the tunnel face and crown conditions.
- Water inflow was continuously monitored to prevent pressure buildup near and above the face. In addition to regular inspection and cleaning, any clogged or blocked holes had to be cleared to allow free passage of water and prevent pressure buildup.

Installed and conducted convergence monitoring system to ensure long-term stability of the headrace tunnel. The major materials used in the treatment process included injection (pre- or post-consolidation) grouting with OPC, grouting with MFC, MS Pipe 75/50 mm diameter for pipe canopy, SDR (Fiber glass) R25, SDR (Metal) R38, Lattice Girder in standard section, Steel ribs (ISMB 200 or other sizes), Steel Fiber-reinforced shotcrete, wire mesh, rock bolts, concrete of grade

C15/C35, and MS plates. The collapsed section was rectified; further excavation resumed on June 21, 2020. However, during the tunnel advancement prior to the placement of the final lining, it was reported that the ground water was only dripping in random locations.

4.2. Convergence Measurement at Chainage 1+154.26 m

Tunnel convergence measurement is crucial in any type of tunnel excavation because it helps to ensure its long-term stability. For safety reasons, it is not recommended to carry out lining work in rocks where the loads and deformations do not attain stable values. If the final lining is installed after the tunnel has been stabilized by initial support, it will undergo very little additional loadings. Thus, in the present case study, the final deformation was closely monitored using a total station with one- second accuracy during the excavation process. The observation was monitored on daily for 44 days, then monthly for two years. Figure 21 shows the 5-point convergence monitoring system (A to E) at chainage 1+154.26 m, whereas Figure 22 show the variation of convergence over a period of days. The convergence was observed almost for two years before application of final rock supports (final lining). Figure 22 shows that the DC line prior to the final lining, which had a maximum convergence of magnitude 4.277 mm at 27 days, when the tunnel face was 144.79 m ahead of the chainage 1+154.26 m.

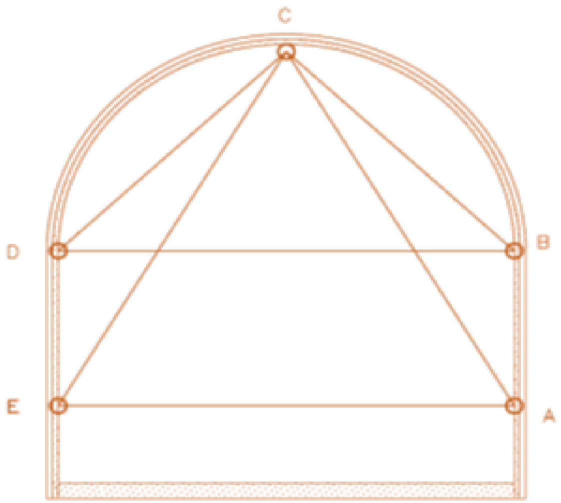
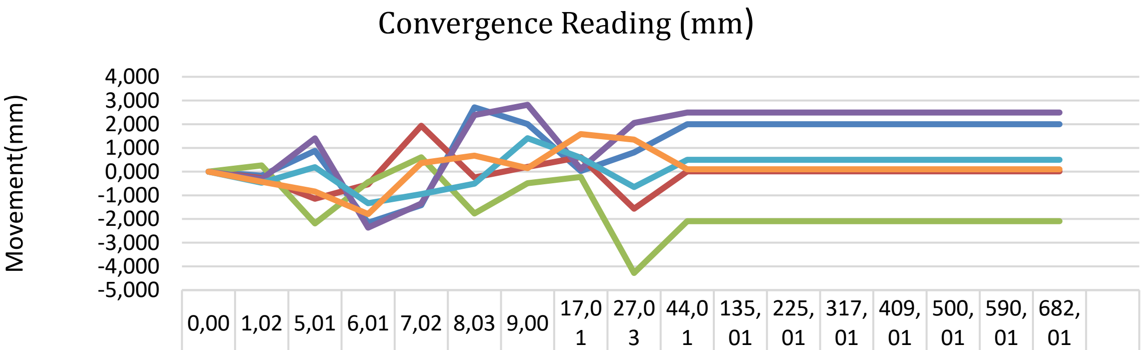


Fig. 21. Location of 5-points convergence monitoring system at Ch. 1+154.26 m



	0,00	1,02	5,01	6,01	7,02	8,03	9,00	17,01	27,03	44,01	135,01	225,01	317,01	409,01	500,01	590,01	682,01
(AC)(mm)	0,00	-0,1	0,87	-2,1	-1,4	2,71	2,00	0,02	0,81	1,99	1,99	1,99	1,99	1,99	1,99	1,99	1,99
(EC)(mm)	0,00	-0,2	-1,1	-0,5	1,93	-0,2	0,20	0,61	-1,5	0,00	0,00	0,00	0,00	0,00	0,00	0,00	0,00
(DC)(mm)	0,00	0,26	-2,1	-0,4	0,60	-1,7	-0,4	-0,2	-4,2	-2,0	-2,0	-2,0	-2,0	-2,0	-2,0	-2,0	-2,0
(BC)(mm)	0,00	-0,3	1,40	-2,3	-1,3	2,38	2,81	0,11	2,05	2,49	2,49	2,49	2,49	2,49	2,49	2,49	2,49
(DB)(mm)	0,00	-0,4	0,18	-1,3	-0,9	-0,5	1,40	0,58	-0,6	0,49	0,49	0,49	0,49	0,49	0,49	0,49	0,49
(EA)(mm)	0,00	-0,4	-0,8	-1,7	0,36	0,67	0,14	1,58	1,35	0,09	0,09	0,09	0,09	0,09	0,09	0,09	0,09

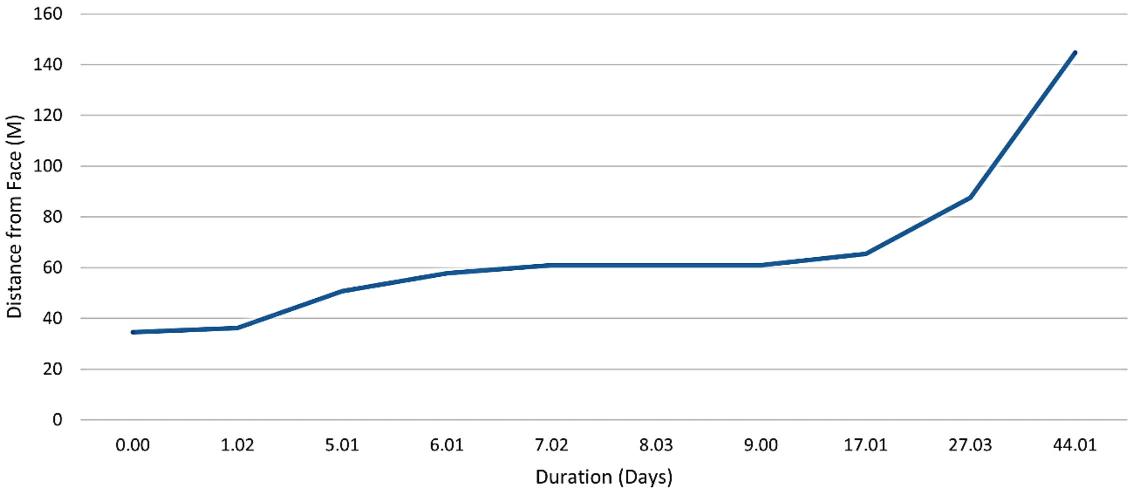


Fig. 22. Convergence reading at Ch. 1+154.26 when the tunnel face was 144.79 m front

At 44 days, the convergence was found to be 2.07 mm, which is almost 50% lower than at 27 days (At 27 days it is 4.277mm reducing to 2.07mm by day 44). It indicates that after 27 days, the stabilization phase begins and convergence decreases, however after 44 days, it is virtually an exponential decay, or implying that by 44 days, almost 95% stabilization has completed. This indicates that the surrounding geology almost completed the time -dependent deformation, ensuring that the final lining will be responsible for taking just the designed load and ensuring safety. Other convergence points, with the exception of the EA line, demonstrated a similar tendency. The observed convergence was quite low, being within an acceptable range for the stretch overburden of 495 m. The time-dependent convergence showed that the overall tunnel movement had stabilized, and no substantial movement would occur subsequently. The monitoring continued for nearly two years until final lining.

4.3. Performance Measure

After rectification, the rectified stretch was monitored for over two years before the final lining was applied, and the performance of the HRT collapsed section at Ch. 1+154.12 m was observed almost for two years after commissioning. According to the site data analysis and the 2023 project commissioning state of the HRT collapsed section at Ch. 1+154.12 m, neither deformation on the applied steel or concrete supports, nor buckling of metal sheets, was reported. The structural integrity of applied supports is observed. Figure 23 shows the photograph of a nondeformed 12mm thick steel liner. The proper usage of rock support in those regions resulted in stability along this length. The project has been running successfully for the last two years, and the stretch is trouble-free.

Fig. 23. Photograph of 12mm Thick Steel Liner (non-deformed)

5. Conclusion and Recommendations

5.1. Conclusion

Based on the above findings, few conclusions are drawn;

- The rock support for the critical section was evaluated using the Q-system support chart and also validated using the finite element-based program Rocscience (Phase^2). Modifications were made to accommodate increasing overburden depth.
- Parametric study indicate that lower overburden depths (<100 m) can be lined with 15 cm of shotcrete instead of concrete lining and steel ribs. However, for overburden depths more than 100 m and up to 350 m, a 20 cm layer of shotcrete with a final concrete lining of 20 cm is required. For overburden depth greater than 350 m, an additional ISMB 200 steel rib with 1.0m c/c spacing and a final lining thickness of 25 cm is required.
- The tunnel convergence at the critical section was observed almost for two years before application of final rock supports and was found to be a maximum of 4.277 mm at 27 days, which was quite low and within acceptable limits.
- At 44 days, the convergence was found to be 2.07 mm, which is almost 50% lower than at 27 days. It indicates that after 27 days, the stabilization phase begins and convergence decreases, while after 44 days, it is practically constant, indicating that the surrounding geology has almost completed the time -dependent deformation.
- The hydraulic capacity for the reduced cross section was checked by comparing the flow velocity of the finished section to the permissible velocity suggested by P.J. Bier's equation and was found to be within the permissible limit.
- This study presents practical rectification procedures for blowout analysis in Himalayan tunnels, which can be applied to future hydropower projects in the Himalaya.
- Failure to make the right judgement resulted in the collapse of HRT, which was not economically repairable. There were two options: pick the bypass tunnel, which would cross the same shear zone again (possibly resulting in more surprises), or open the same alignment. It was the correct judgement by project management to select option 2.

5.2. Recommendations

- For extremely weak rock with few or no joints, it is essential to combine the Q-system with deformation measurements and numerical simulations.
- Before using sprayed concrete during underground excavation, it is crucial to visually examine the rock surface around the tunnel. Altered rock may have the same geological structures as the original fresh and un weathered rock, which may go unnoticed when observed from the invert, but will be observed with a closer inspection.
- When excavating tunnels especially in poor rock mass (V) and exceptionally poor rock mass (VI) geology, predrilling should be carried out and lithology mapping can help to assess the location and length of weak areas.
- In squeezing areas, convergence measurement should be carried out until stabilization is proven, that is, prior to final lining.
- A water injection test is recommended to determine the permeability of the rock mass sheared band and adjacent strata for effective grouting.
- If the section's final lining dimensions meet the tolerance limit, avoid re-profiling. If necessary, a risk assessment and an approved correct approach should always be used.
- It is imperative to ensure that all safety guidelines and procedures are implemented throughout the rectification process of such complexity.

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