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Geotechnical Design and Rock Support Evaluation of Dimapur–Kohima Railway Tunnel, Nagaland, India

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This study evaluates the rock support requirements for a specific tunnel design, aligning with guidelines from the *Indian Railways Schedule of Dimensions* for 1676 mm Broad Gauge (BG), Revised (2004), and Addendum and Corrigendum Slip (ACS) No. 26. By examining geological, geotechnical, and geo-engineering parameters along the tunnel alignment—including rock mass characteristics and in situ stress conditions—the study aims to identify optimal design and support systems. Using RocLab software, key rock mass properties like deformation modulus and shear strength were evaluated, while RS2 (Finite Element Method software) assessed primary tunnel support needs. For full-face excavation and tunnel cross sections without invert, the recommended support classes III and IV require bolts of 3–3.5 m in length and 10–15 cm shotcrete. For cross sections with invert, classes IV, V, and VI support systems are advised. The maximum observed deformation for Class VI tunnels ranged from 80–150 mm, with a yielding radius of approximately 8 m, indicating poor rock quality in this section. To mitigate deformation risks, a high-strength support system with yielding capabilities is suggested, along with pipe roofing. However, further evaluation through 3D simulations is recommended to assess the pipe roof's effects on stress and deformation. Finally, Class VII supports are proposed for their robust design, especially in areas with shallow overburden and low-stress environments, capable of managing substantial loads. This real-time study offers valuable insights for academicians and consultants focused on tunnel support assessment and related geotechnical applications.

Keywords: Rock class, Rock support, RocLab, RS2 FEM, Tunnelling

Introduction

The north–eastern region of India is undergoing a revolution in infrastructure development. A total of Rs 3,84,426 lakh crore has been earmarked towards many infrastructure development projects with the aim of enhancing connectivity, primarily roadways and railways connectivity.[1] As a part of the aforesaid projects, the Dhansiri (Dimapur) – Zubza (Kohima), a new 90.5 kms long broad–gauge railway line has been planned. This project's alignment traverses a difficult, mountainous, and steep landscape with adverse geological conditions. Therefore, it requires construction of tunnels in certain sections to bypass the climatic and terrain factors. However, regardless of the fact that construction of tunnels is one of the best option, it still has proved to be very high hazard structures due to a number of risks involved in its construction.[2] The majority of the risks are associated with unstable terrain. Tunnels alter the natural environment, which leads to ground instability and, eventually, failure.[3] In addition, tunnelling in the adverse geological conditions, particularly weak geological strata, poses its own challenges. Excavation through weak strata has its own demerits, both during the excavation itself and also during the assessment and application of support systems. In case of railway transportation tunnels, such as the one being studied here, the main aim of the engineers is to make the ceiling and face stable during the excavation process, through application of suitable support systems. In selection of most suitable support systems for aforesaid type of tunnel excavations, ground control engineers face significant challenges, owing to various factors including rock mass parameters, stress conditions, and geological factors.[4]

Additionally, the stability of the otherwise stable slopes or strata is negatively impacted by the tunnel excavation on the other side of the mountains. Excavation of tunnel effectively causes redistribution

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of stresses acting on the slopes or strata in and around the excavation, eventually leading to failure. This makes it very prudent to make an assessment of a suitable support system, with proper structural components that would ultimately resist the failure of the excavation.

In view the above, this paper attempts to address the issue of finding the principal rock support needed for one of the main tunnels along the entire project, for various rock classes throughout the tunnel alignment. RS2 (Finite Element Method software) has been used to assess the main rock support for the tunnel in the present study.

Relevant Features of the Proposed Tunnel

The area with the proposed main tunnel is located in Dimapur district, Nagaland. The tunnel is proposed to be a single–track rail tunnel that will be built using New Austrian Tunnelling Method (NATM) construction techniques amid inconsistent and challenging ground conditions. Length of the tunnel has been planned to be about 4940 m with a ruling gradient of 1 in 80. The portal altitudes have been estimated to be about 480.35 m and 561.68 m. The area falls within seismic zone V.

Methodology

The methodology for the present study was designed based on literature on several case studies on tunnel support design in the Himalayan region characterized by presence numerous geological structures, with variable disposition. These earlier works involved assessment of geological, geo–engineering and geotechnical characteristics of rock masses subject to tunnelling and application of subsequent support systems and techniques. Based on these aforesaid works, following methodology was adopted in this study (Fig. 1).

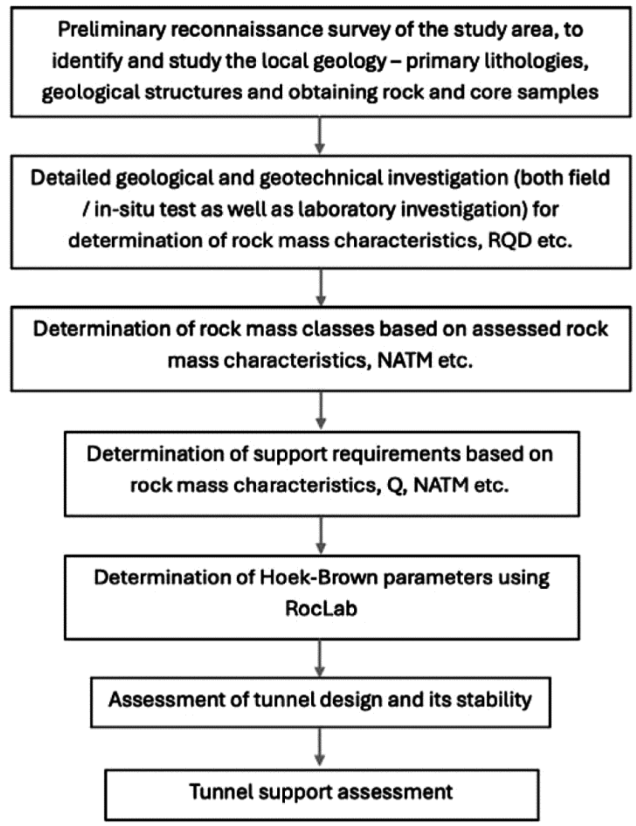

Fig. 1 — Flow chart showing the methodology adopted in the present work

Geology along the Tunnel Alignment

The main Tunnel is expected to be constructed through the Surma and Barail group of rocks. These tertiary sediments present in the study area is geologically similar to those present in Upper Assam region. Stratigraphically, the Barail Group of rocks are overlain by the Surma Group of rocks, represented by alternating units of sandstone and shale and medium to coarse grained sandstone pertaining to Oligocene epoch respectively.[5]

These formations also contain siltstone and minor conglomerate with current bedding, flute cast, flaser bedding, ripple markings, and convolute laminations.[6] The entire tunnel alignment follows the ridges and spurs of the terrain, which is rough and steep.

The sandstone that is exposed (Fig. 2a and b) along the Surma formation is grey to dark grey in colour, medium to coarse–grained, thin to moderately thick bedded. These aforesaid beds are characterized with sedimentary structures and prevalent ripple marks. There are also tiny bends and kink bands in these rocks.

Shales are primarily carbonaceous in nature[7], and they range in colour from grey to dark grey (Fig. 2c and d). They are weak to very weakly laminated and have thin walls. These shales are frequently characterized by folding. The shale in the region has been observed to be occurring bounded by sandstone layers (Fig. 2e and f), showing rhythmic characteristics. The region is also characterized by the presence of both colluvial and alluvial deposits (Fig. 2g and h) pertaining to quaternary age. The colluvial deposits are characterized by the presence of slide debris material constituted of angular to sub–angular rock fragments of sandstone mixed together

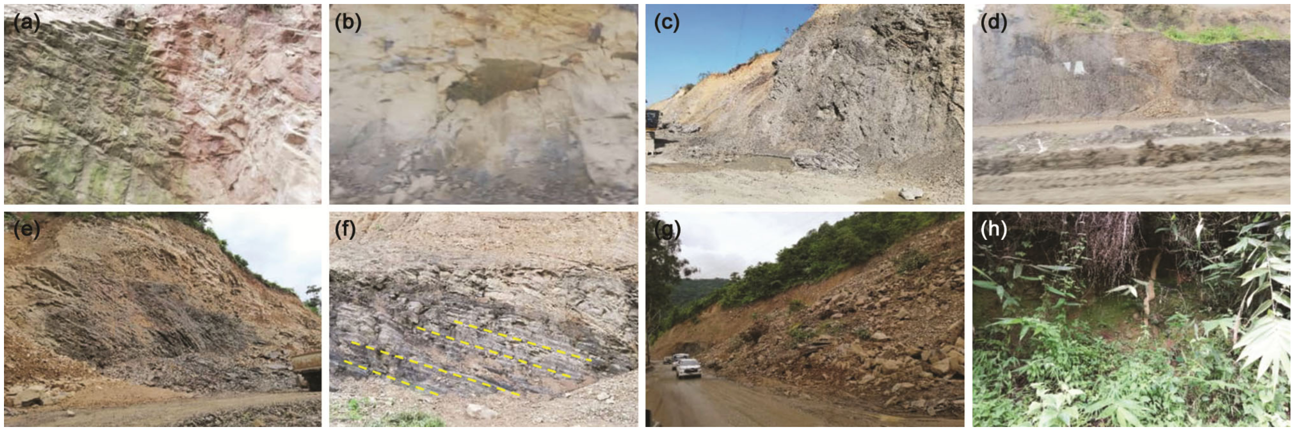

Fig. 2 — Geological formations and conditions along the tunnel alignment: (a) and (b) Medium grained sandstone pertaining to Surma Group with repetitive occurrence of current bedding structure exposed along the tunnel alignment, (c) and (d) outcrop photographs of dark grey laminated shale, (e) and (f) interlayering between sandstone and shale, whereby shale is sandwiched between sandstone layers (shown by yellow broken lines), (g) and (h) slide debris material constituting the colluvial deposits and dense vegetation covering the alluvial deposits of the area

with shale, thereby forming a heterogenous mixture, with the matrix constituted of soil, silt and clay.

Geotechnical Analysis

Based on the geology and geotechnical data, the area's rock mass was divided into seven distinct ground types in order to evaluate the tunnel's design.

Rock mass classification – the rock mass of the area was characterized based on the Q system i.e., Norwegian Q system–Tunneling Quality Index.[8] The assessment incorporated all geological parameters, including the volume of intact rock, characteristics of discontinuities (spacing, roughness, wall strength, aperture, filling material), Rock Quality Designation (RQD), and rock mass structure. It also evaluated block size, joint wall roughness and friction, infilling materials, and active stress conditions. These factors collectively provided a comprehensive geomechanical characterization.

In rock mass characterization, the Geological Strength Index (GSI) is a widely used tool paired with the Hoek–Brown failure criterion to evaluate rock mass properties. GSI evaluations were conducted based on specific site conditions to assess strength reduction in the rock masses, followed by a detailed geomechanical assessment of the study area. A probabilistic approach further processed the classification, considering variations in initial parameters.[9] This assessment led to categorizing the rock masses into four types: (i) heavily fragmented rock masses with a GSI below 25, (ii) moderately weathered rock masses with intersecting joints and GSI values of 25–45, (iii) moderately fragmented rock masses forming angular blocks due to multiple joint sets with GSI values of 45–65, and (iv) slightly weathered, undisturbed rock masses with a GSI above 65. This classification provides insight into the geomechanical behavior of each rock type.

The rock mass parameters viz., deformation modulus and shear strength parameters were assessed using RocLab software developed by RocScience.[10] The shear strength parameters i.e., cohesion and angle of internal friction was assessed using Mohr–Coloumb material model in case of soil strata and rock stiffness using Hoek–Brown model in case of rock strata.[11]

Design parameters for the proposed tunnel, obtained considering all the above–mentioned parameters and assessments is shown in the Table 1 below.

Tunnel Design

The design of the tunnel cross section (Figs 3 & 4) and kinematic envelope for the proposed main tunnel was developed (as per the INDIAN RAILWAYS SCHEDULE OF DIMENSIONS for 1676 mm Gauge (BG), REVISED 2004) and Addendum and Corrigendum Slip (ACS) No. 26.[(12)]

The minimum horizontal distance from the centre of track to any structure in tunnel was considered as per the INDIAN RAILWAYS SCHEDULE OF DIMENSIONS item 13 (ii) as listed in Table 2.

Furthermore, extra clearance has been suggested to be given on the inside of a curve in order to account

Table 1 — Summary of the design parameters

Rock type	Intact UCS (MPa)	GSI	Mi	Hoek– Brown Criterion			Rock Mass Parameters (MPa)			Mohr's Coulomb fit		Modulus ratio	Intact modulus Ei (MPa)	Deformation modulus (Mpa)	Q ranges	Rock class and Ground type (GT)
				Mb	S	a	Sigt	Sigc	sigcm	Cohesion	Friction angle					
Thickly bedded sandstone	40	60	17	4.074	0.0117	0.503	0.115	4.28	11.105	1.446	46.26	275	11000	5720	4–10	III / Fair GT 1
Thinly bedded sandstone	30	45	17	2.384	0.0022	0.508	0.028	1.345	6.08	0.996	39.73	275	8250	1845	1–4	IV / Poor GT 2
Thinly laminated shale	15	25	6	0.412	0.0002	0.531	0.009	0.179	1.094	0.329	20.89	200	3000	179.57	0.01–0.1	VI / Extremely poor GT 3
Thinly bedded sandstone with shale parting	20	35	17	1.668	0.0007	0.516	0.009	0.482	3.254	0.708	33.81	275	5500	623.74	0.5–1	V / Very poor GT 4
Thinly laminated shale with minor sandstone	15	30	6	0.493	0.0004	0.522	0.013	0.258	1.266	0.295	25	200	3000	244.15	0.1–0.5	V / Very poor GT 5
Crushed and sheared / faulted rock mass	12	20	6	0.345	0.0001	0.544	0.005	0.096	0.74	0.26	18.07	200	2400	109.61	0.01–0.1	VI / Extremely poor GT 6
Colluvium or slide material										0.05–0.1	20–30				0.001–0.004	VII / Exceptionally poor GT 7

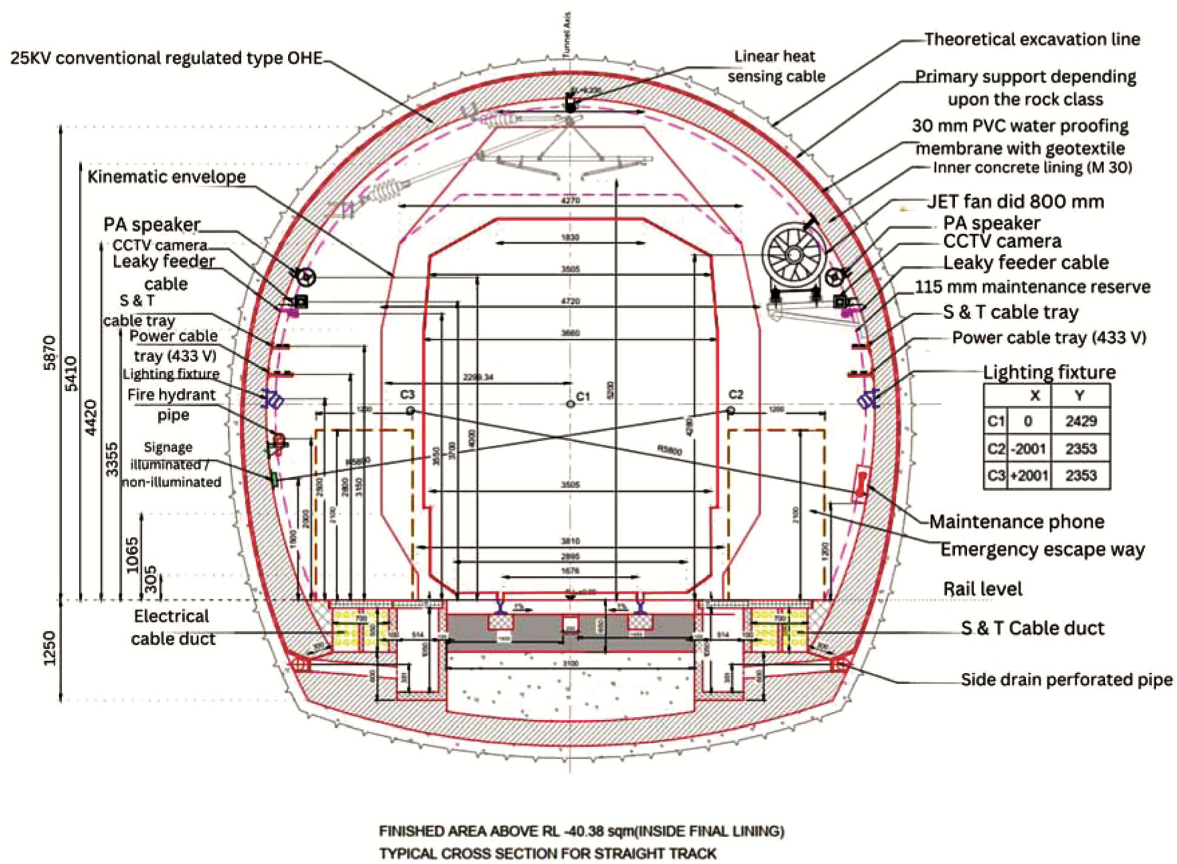

Fig. 3 — Typical cross–section of the proposed main tunnel for straight track (without invert)

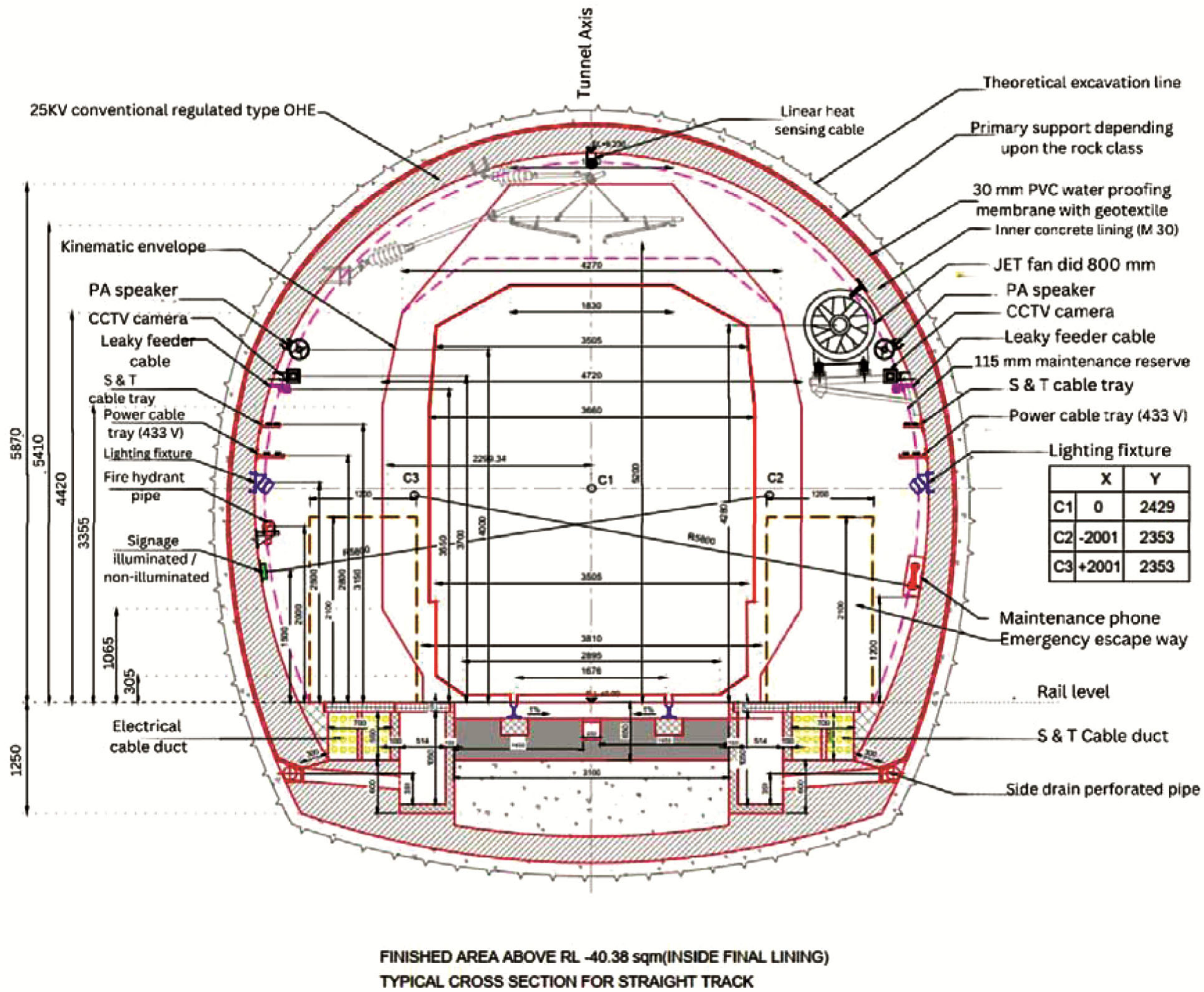

Fig. 4 — Typical cross – section of the proposed main tunnel for straight track (with invert)

Table 2 — Horizontal distance from track centre (for straight track)

Height above rail level (mm)	Horizontal distance from centre of track (mm)
0 – 305	1905
305 – 1065	1905 – 2360
1065 – 3355	2360
3355 – 4420	2360 – 2135
4420 – 5870	2135 – 915

for the effect of curvature, the lean brought on by increased elevation, and an allowance for any excess wobble of the railway car above the already supplied space on straight lines.

Allowance for curvature: The allowance for curvature for a vehicle 21340 mm long, 14785 mm between bogie centre shall be calculated as under (INDIAN RAILWAYS SCHEDULE OF DIMENSIONS 1676 mm Gauge (BG), REVISED 2022): –

At the centre of vehicle:

$$V = \frac{14.785 \times 14.785 \times 1000}{8R} = \frac{27330}{900} = 30.36 \text{ mm}$$

At the end of vehicle:

$V_o = \frac{21.340 \times 21.340 \times 1000}{8R} = 32.88$ mm, where, R is the radius of the curve in metres

Additional clearance for superelevation: Lean due to super elevation at any location at height h above rail level has been taken into account by using the following formula:

$L = \frac{h}{g} \times S$, where, S = superelevation, g = track gauge

Additional clearance for sway: One fourth of the lean (L/4) owing to super elevation was established as the allowance for additional lurch and sway on the inside of a curve.

Additional total clearance inside the curve; was provided as;

$C_{inside} = V + \frac{5}{4} L - 51$ mm, where, L = lean (mm)

Since the inward lean of the vehicle due to extra elevation can be considered as entirely counteracting the sway or lurch caused by a curve, only the effect of curvature was taken into account on the outside of a curve. Additional sway resulting from a curve that bends in an outward direction has not been taken into account.

i.e., $C_{outside} = V_o$

Calculation of Superelevation and Extra Clearances at Curve for the Main Tunnel

Calculation of degree of curve:

Radius of curvature; R = 440 m

Degree of curve; D = $\frac{5729.578\,(feet)}{R\,(feet)} = \frac{1746.37\,(m)}{R\,(m)} = \frac{1746.37}{440} = 3.9^{o}$

Calculation of superelevation / Cant

Maximum speed of passenger train; V_m = 100 kmph

Assuming booked speed of goods train; V_g = 65 kmph

Now, $C_a + C_d = \frac{(13.76 \times V_m 2)}{R} = 312.73$ mm, where, C_a = superelevation, C_d = cant deficiency

Now, when taking C_d = 75 mm; then C_a = 237.73 mm

However, since as per INDIAN RAILWAYS SCHEDULE OF DIMENSIONS, maximum cant on curved track should be under / upto 140 mm

Therefore, C_a = 140 mm.

Now, C_{eq} for good trains = $\frac{(13.76 \times V_g 2)}{R}$ = 132.13 mm

Hence, Cant Excess; C_{ex}= 7.87 which is less than 75 mm

Therefore, allowable speed for cant of 140 mm = 82.9116 KMPH.

Again, extra clearance for curve, was calculated as per "INDIAN RAILWAYS SCHEDULE OF DIMENSIONS, 1676 mm Gauge (BG), 2004", considering three components:

i. Overthrow and Endthrow
ii. Lean and,
iii. Sway

As per paragraph 8 & 9 on Appendix to Schedule of Dimensions, extra clearance on inside of curve = (Overthrow + lean + sway) Where,

Overthrow = $\frac{27751}{R}$

Lean at various heights = $\frac{h}{g \times C_a}$

and Sway = $\frac{Lean}{4}$

Finally, Endthrow = $\frac{29600}{R}$, where R = radius of curvature, h = height of the point under consideration above rail level in mm, g = gauge of the track = 1676 mm for broad gauge and C_a = superelevation/ actual cant in mm.

Accordingly, the calculated extra clearance required at various heights are tabulated below (Table 3)

The cross section for the curved alignment of the main tunnel has been designed based on the principle that the cross–section of the tunnel should be straight in case of completely straight tunnels and cross–section of the tunnels should be curved in case of tunnels that are a combination of both straight and curved. Accordingly, based on the aforementioned concept and above calculations the typical cross–section for the proposed main tunnel with the maximum cant of 140 mm with and without invert have been shown the figures below (Fig. 5 and 6)

Tunnel Support Design

The tunnel support design assessment has been carried out by considering the rock mass type, rock mass quality and in–situ stress conditions along the tunnel alignment based on the geotechnical data.

Further, the minimum support required as well as calculation of actual deformation and stress built up around the tunnel under different ground type and stress situations has been done by empirical and numerical analysis respectively. Finite element analysis was carried out using software Phase 2 (Rocscience) Version 8. The analysis was performed in plane deformation conditions with $\sigma_z = 0$, (σ_z being outside the plain deformation), primarily following plane–strain transverse model in a vertical section of tunnel.

Table 3 — Required extra clearance at various heights

Height above rail level (in mm)	Extra clearance inside of curve (in mm)	Extra clearance on outside of curve (in mm)
305	95	67
1065	174	67
3355	413	67
4420	525	67
5410	628	67
5870	676	67

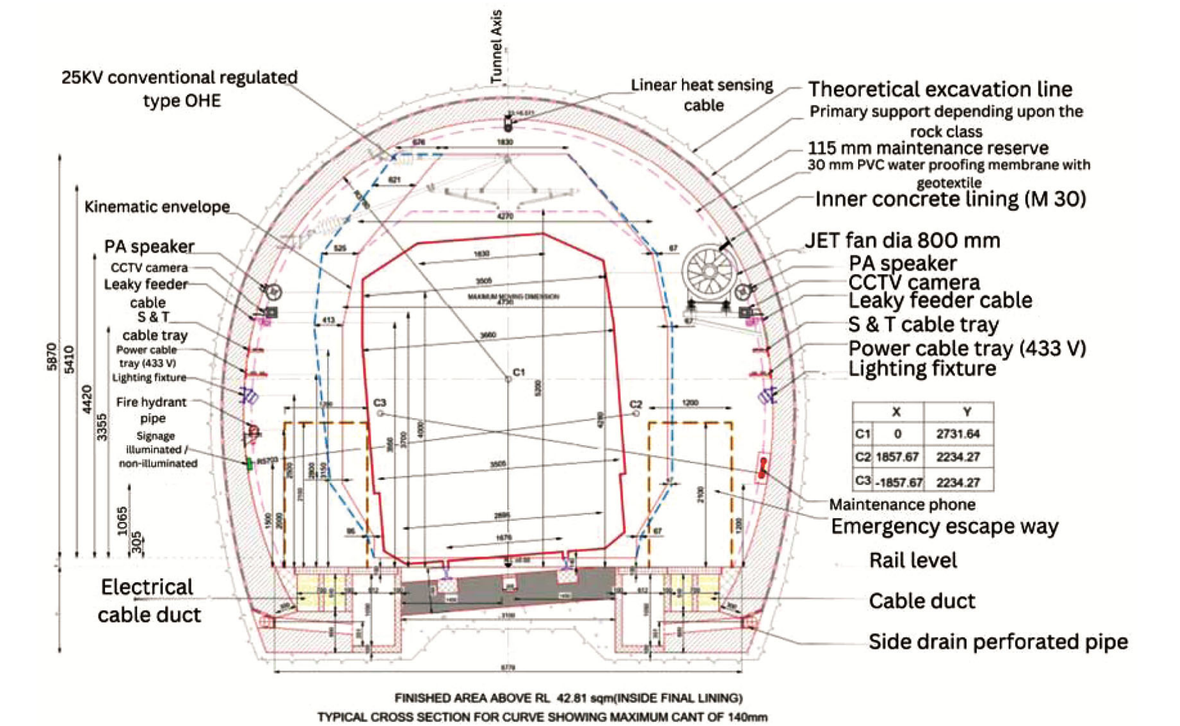

Fig. 5 — Tunnel Cross Section for curve showing maximum cant of 140 mm (without invert)

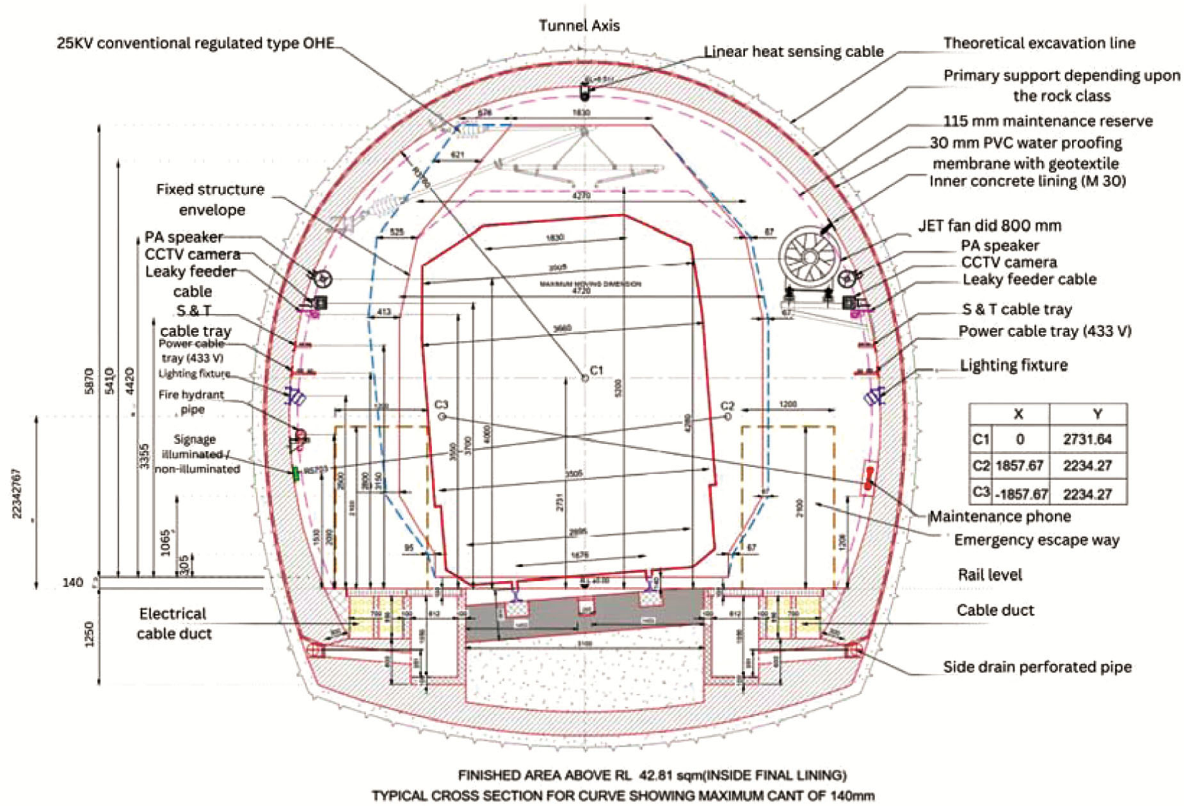

Fig. 6 — Tunnel Cross Section for curve showing maximum cant of 140 mm (with invert)

The assumptions considered in the above–mentioned modelling were as follows:

- Same geometry of the entire tunnel along its entire length, thereby allowing modelling of a three – dimensional problem in two dimensions.
- The adjoining rock mass around the tunnel is homogenous, and isotropic in all directions.
- Lithology represented as Mohr – Coulomb strength criterion in case of soil strata and Hoek–Brown criterion in case of rock layers in the model.
- The boundary conditions for the model were fixed to simulate deep seated tunnel conditions. However, the boundary conditions were modified depending on the section and site excavation conditions.
- The primary tunnel lining was modelled as elastic beam elements in 2D plane strain.

Numerical analysis was carried out considering both the rock / ground type as well as associated stress conditions for anticipated variable overburden stress along the entire tunnel length and alignment. Accordingly, the support types were deciphered for the aforesaid conditions and rock / ground type.

The various parameters for numerical analyses cases are tabulated below (Table 4).

The different structural elements considered in the analysis are summarized herewith:

For shortcrete, concrete grade was considered to M25 (according to IS 456:2000), with a concrete cover of 20 mm. the minimum 28–day concrete strength taken into account was 25 N/mm^2. The rock bolts of steel grade Fe 415 with a minimum diameter of 25 mm and anchor plates with dimensions of 200 × 200 × 12 (according to IS 1786:1985) were considered with a yield load of Fy ≥ 230 kN.

Steel wire mesh Φ6, 150 × 50 according to IS 1786:1985 of grade Fe 415 with a yield strength of fy = 415 N/mm^2 and steel ribs with a minimum yield strength of 240 N/mm^2 were considered respectively. In case of steel bars, a grade of Fe 415 with a diameter of 32 mm (according to IS 1786:1985) and a yield strength of Fy ≥ 230 kN was considered.

Tunnel support evaluation

In tunnel design, for Class III support by means of empirical method (Q system), the different input parameters included were Q–(4–10), ESR (excavation support ratio)–1.0; for major road and railway tunnels as per *Barton et al*, 1974[8], and span of 8.5m. Now, assessed categories of supports based on the Fig. 7 below[13] are, systematic bolting being SN bolts, L_{min}= 3m @ 2.1 m c/c and shotcrete of 4–5 cm.

Table 4 — Parameters for numerical analyses

Support class	Q-value	Overburden criteria	Ground type
III	4–10	0–300	GT-1
IVA	1–4	0–300	GT-2
IVB	0.5–1	0–200	GT-4
V	0.1–0.5	200–250	GT-4
VI	0.01–0.1	0–100	GT-3
VIA	0.01–0.1	100–300	GT-3
VII	0.001–0.004	0–100	GT-7

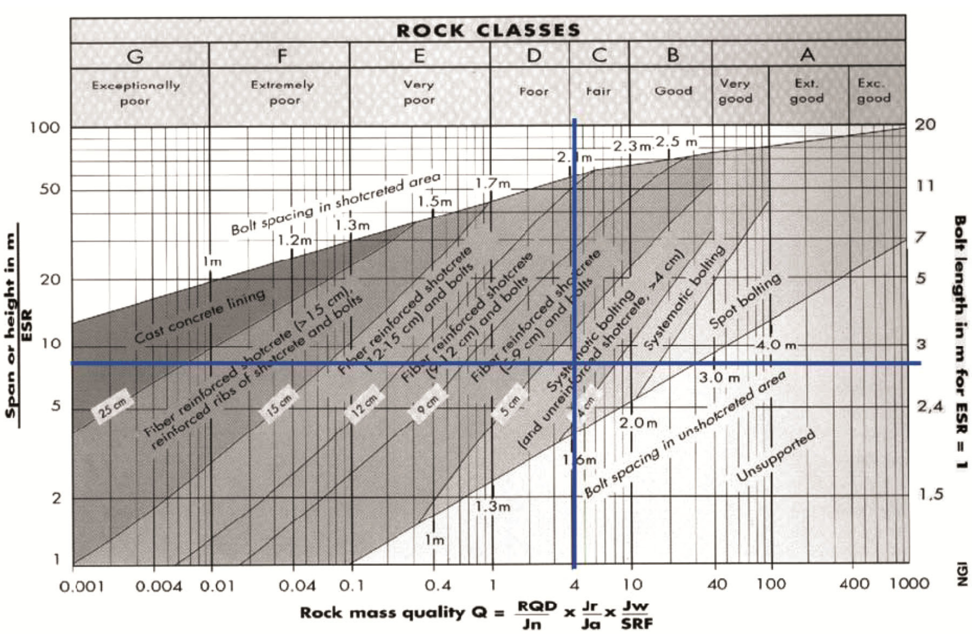

Fig. 7 — Assessed categories of supports based on the tunneling quality index Q (After Grimstad & Barton 1993) – Rock Class III

In numerical method (RS2) used for tunnel analysis, the field stress; (K) was considered constant owing to high overburden almost all along the tunnel length.

The different steps incorporated in numerical modelling are mentioned herewith in Fig. 8.

Using the Phase 2 (Rocscience) Version 8 software, a finite element analysis of tunnel excavation was performed. Multi–stage sequential modelling was used to simulate the proposed tunnel's excavation process. In the numerical modelling, the assessed supports were further verified using the support capacity curve option available with RS2.

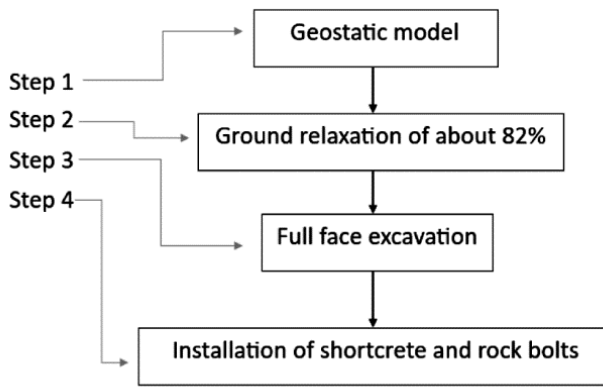

Fig. 8 — Considered steps for numerical modelling

M – N verification: In M – N verification, for primary and final lining supports, all the points were observed to be lying within the envelope (Fig. 9 and 10), thereby suggesting that the calculated concrete thickness is enough.

Similar calculations were done both by empirical as well as by numerical method for Class IVA, Class IVB, Class V, Class VI, Class VIA, Class VIA with ground improvement along with M–N verification., and similar results in all the cases were obtained.

Accordingly, based on all the above assessments, geology, overburden volume, different support classes identified have been tabulated below (Table 5 and 6).

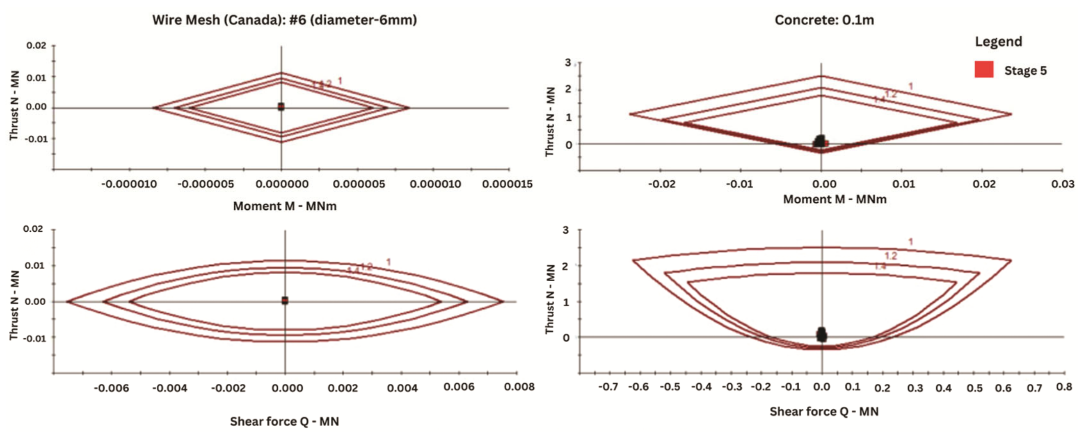

Fig. 9 — Support capacity curve for primary shortcrete lining (Support element: 100 mm shortcrete with single layer wire mesh)

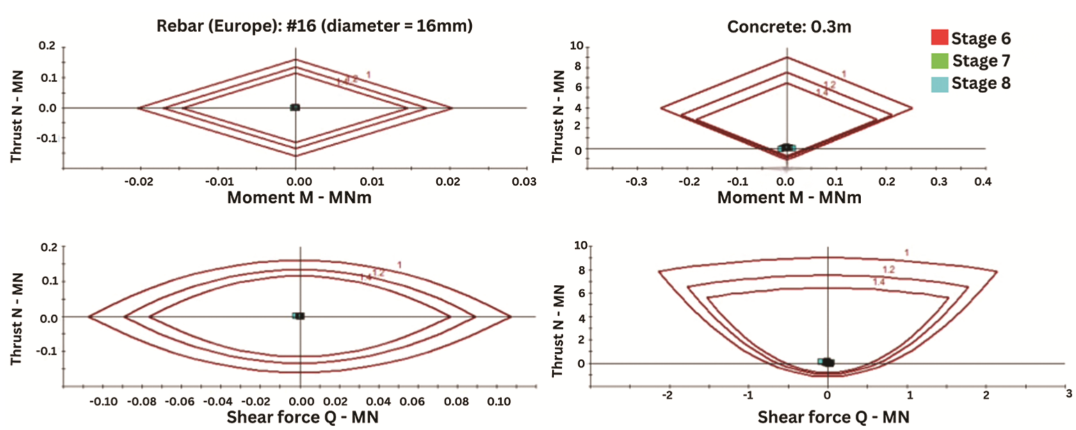

Fig. 10 — Support capacity curve for final lining (Support element: RC lining @ 16 dia 200 mm spacing)

Table 5 — Support classes for different ground types

Tunnel support application matrix

Overburden range	Ground type					
	GT1	GT2	GT3	GT4	GT5	GT6
0 – 100	III	IVA	VI	IVB	V	VI
100 – 200	III	IVA	VIA	IVB	V	VIA
200 – 250	III	IVA	VIA	V	VIA	VIA
250 – 310	III	IVA	VIA	VI	VIA	VIA

Table 6 — Summary of assessed support system

Support class	Excavation	Round length	Invert (m)	Temporary invert	Fore poling / pipe roofing			Rock bolt	
					Type	Length (m)	Placing/ɵ	Length (m)	Pattern (m^2)
III	Full face	2.5	NA	NA	NA	NA	NA	3	2.5 × 2.5
IVA	Full face	2	NA	NA	NA	NA	NA	3.5	2.0 × 2.0
IVB	Heading, benching invert	1.5	3	NA	NA	NA	NA	4	1.5 × 1.5
V	Heading, benching invert	1	2	200 mm thick shotcrete with single layer of wire mesh (if necessary)	Fore poling φ32 SDR	5	300 c/c @20°, θ =120°	6	1.0 × 1.5
VI	Heading, benching invert	0.8	2	200 mm thick shotcrete with single layer of wire mesh (if necessary)	Fore poling φ32 SDR	5	301 c/c @20°, θ =120°	6–9	0.8 × 1.5
VIA	Heading, benching invert	0.8	1.6	200 mm thick shotcrete with single layer of wire mesh (if necessary)	Fore poling	5	302 c/c @20°, θ =120°	6–9	0.8 × 1.5
VII	Heading, benching invert	0.8	1.6	200 mm thick shotcrete with single layer of wire mesh (if necessary)	Pipe roofing φ114 mm, 6.3 mm thick steel pipe	12	300 c/c @ 5°, θ =120°	4	0.8 × 1.0

Conclusions

The study analyzed the support requirements for different support classes in a proposed tunnel, considering geotechnical and geological characteristics. For support classes III and IV, full-face excavation methods and tunnel cross-sections without an invert are proposed. Support measures include bolts with lengths between 3 and 3.5 meters and a layer of shotcrete (sprayed concrete) about 10–15 cm thick is suggested. For support classes IV, V, and VI, tunnel cross-sections with an invert are recommended. Yielding supports are suggested in Class VI to absorb extra deformation and provide flexibility to the ground. This study also highlights the need for stronger and more flexible support systems in areas with poorer rock conditions and higher stresses, particularly in Support Class VI. The maximum deformation in Class VI is estimated to be in the range of 80–150 mm and maximum yielding radius of about about 8 meters. For support Class VII, rigid supports for shallow tunnels are proposed as most of the stresses would be absorbed by the support itself due to the low stress environment. As additional measure for tunnel support in different classes include the use of pipe roof umbrellas and rigid steel ribs embedded in a thick layer of shotcrete have been assessed to be used for tunnel portals to ensure structural integrity.

References

1 Mondal A, Infra boon in North–East! Here's how much the government spent in developing the infrastructure of NE states, *Financial Express*, 14th March, (2023).

2 Kumar A & Singh M S, Back analysis of rock support and final–lining recommendation for rock class C1 of a tunnel, *Int J Eng Trends Technol*, **71(1)** (2023) 79–93, https://doi.org/10.14445/22315381/IJETT–V71I1P208.

3 Claudio O, Pierpaolo O & Giovanni S, Creep behaviour of two–component grout and interaction with segmental lining in tunnelling, tunnelling and underground space technology, 104216, **119** (2021) 1–13, ISSN 0886-7798, https://doi.org/10.1016/j.tust.2021.104216.

4 Ghorbani M, Shahriar K, Sharifzadeh M & Masoudi R, A critical review on the developments of rock support systems in high stress ground conditions, *Int J Mining Sci Technol*, **30(5)** (2020) 555–572, https://doi.org/10.1016/j.ijmst.2020.06.002.

5 Mandal J, Palynofossils from the Tertiary (Barail Group) of Nagaland: Palaeoecological interpretation and age,

Palaeobotanist, **45** (1996) 98–108, https://doi.org/10.54991/jop.1996.1223.

6 Shijoh C, Peter V S & Singh L S, Detailed corridor mapping along Dimapur–Kohima, Merapani–Wokha and Mariani–Changtongya across frontal fold thrust belt of naga Arakan Yoma Suture of Nagaland, Item No. RP / NER / MNG / 2015 / 006, Final Report for F. S 2015 –17, Government of India, Geological Survey of India, North–Eastern Region, (2017) 33–36.

7 Moiya J N, *Tectono–geomorphic evolution of the intermontane basins and quaternary deformation in the schuppen belt in parts of the Dimapur and Peren Districts, Nagaland*, Ph D Thesis, Department of Geology, Nagaland University, 2019.

8 Barton N, Lien R & Lunde J, Engineering classification of rock masses for the design of tunnel support, *Rock Mech Felsmechanik Mecanique des Roches*, **6** (1974) 189–236, https://doi.org/10.1007/BF01239496 .

9 Heidarzadeh S, Saeidi A, Lavoie C & Rouleau A, Geomechanical characterization of a heterogenous rock mass using geological and laboratory test results: a case study of the Niobec Mine, Quebec (Canada), *SN Appl Sci*, **3** (2021) 640, https://doi.org/10.1007/s42452-021-04617-1.

10 Pradhan S P & Siddique T, Stability assessment of landslide–prone road cut rock slopes in Himalayan terrain: A finite element method–based approach, *J Rock Mech Geotech Eng*, **12(1)** (2020) 59–73, https://doi.org/10.1016/ j.jrmge.2018.12.018.

11 Ramamurthy T, Shear strength response of some geological materials in triaxial compression, *Int J Rock Mech Mining Sci*, **38(5)** (2001) 683–697, https://doi.org/10.1016/S1365–1609(01)00035–1.

12 Indian Railways, Schedule of Dimensions 1676 mm Gauge (BG), REVISED 2022.

13 NGI, Handbook on Using the Q System, Rock Mass Classification and Support Design, Revised and new edition, (Oslo) 2022, 32–41.