Tunnelling Asia 2025

International Conference on

Revolutionizing Underground Infrastructure through Advanced Tunneling Techniques

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Contents

AIR PURIFICATION SYSTEMS FOR ROAD TUNNELS

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ABSTRACT

Tunnel ventilation systems are essential to maintain air quality, ensure visibility, and provide safety in case of fires in the tunnel. Traditionally, exhaust stacks or chimneys are used in road tunnels when contaminated air cannot be released directly at the portals or when the tunnels are too long.

The implementation of an Air Purification System (APS) for filtering exhaust air from a road tunnel provides clear advantages when compared with the traditional solution of using an exhaust stack. While chimneys only displace pollutants to higher altitudes, air purification systems actively remove harmful substances such as nitrogen oxides (NOx), particulate matter (PM), and unburned hydrocarbons before the air is discharged into the surroundings. This results in a significant reduction of the environmental and visual impact, improving compliance with increas-ingly strict air quality standards.

The paper will focus on describing the main characteristics, operational principles, and design considerations of air purification systems applied to road tunnels, highlighting their role in improving air quality and tunnel sustain-ability. In addition, a comparison will be made between traditional ventilation solutions based on exhaust chimneys and the use of advanced air purification systems.

1 INTRODUCTION

Road tunnels are critical components of modern transportation infrastructure, providing efficient routes through urban and mountainous regions while reducing congestion and travel time. However, these enclosed environments present significant challenges in maintaining air quality due to the accumulation of vehicular emissions such as carbon monoxide (CO), nitrogen oxides (NO_x), and particulate matter (PM).

Tunnel ventilation systems dilute and remove contaminated air from the tunnel, ensuring a safe environment for users and maintenance workers, avoiding dangerous concentrations that exceed health limits and reduced visibility inside the tunnel caused by smoke or dust. Furthermore, they are essential to extract smoke in the event of an in-tunnel fire. They can be classified according to airflow direction and mechanical arrangement. The main options include:

- Longitudinal ventilation, where jet fans move air along the tunnel. This system is suitable for short to medium-length tunnels.
- Transverse ventilation, which uses ducts to supply fresh air and extract polluted air across the entire tunnel. It provides an effective smoke control in both normal and emergency conditions but requires large ducts and space.
- Semi-transverse ventilation, a combination that injects or extracts air at specific points. This system is suitable for long tunnels but more complex than a purely longitudinal system.

Traditionally, tunnel ventilation has relied on exhaust stacks to disperse polluted air into the atmosphere, diluting contaminants through high-volume airflow. While effective in maintaining in-tunnel air quality, this approach does not eliminate pollutants—it merely relocates them, contributing to localised air quality degradation around tunnel outlets and increasing overall environmental impact. However, in recent years, Air Purification Systems have emerged as a promising alternative or complement to conventional exhaust ventilation. These systems consist of electrostatic precipitation for particles, adsorption at activated carbon for gaseous pollutants, and optionally other techniques such as catalytic or photocatalytic oxidation to actively remove harmful pollutants from the tunnel air before discharge. Such technologies offer the potential for significant reductions in pollutant emissions, improved urban air quality, and alignment with increasingly stringent environmental regulations.

This paper provides a comprehensive description of Air Purification Systems for road tunnels and compares the performance, environmental impact, and operational feasibility of air purification systems versus traditional exhaust stack ventilation in road tunnels. By examining their respective advantages, limitations, and implementation challenges, the study aims to identify conditions under which advanced purification technologies can provide sustainable and cost-effective solutions for tunnel air quality management.

2 STRUCTURE OF AIR PURIFICATION SYSTEM

An Air Purification System for tunnel ventilation typically comprises two key components: an electrostatic precipitator for particulate matter removal and an activated carbon filter for gaseous pollutant control, particularly nitrogen dioxide (NO_2). These components are arranged in series to achieve complementary pollutant reduction, ensuring both particulate matter and gaseous contaminants are treated before air discharge.

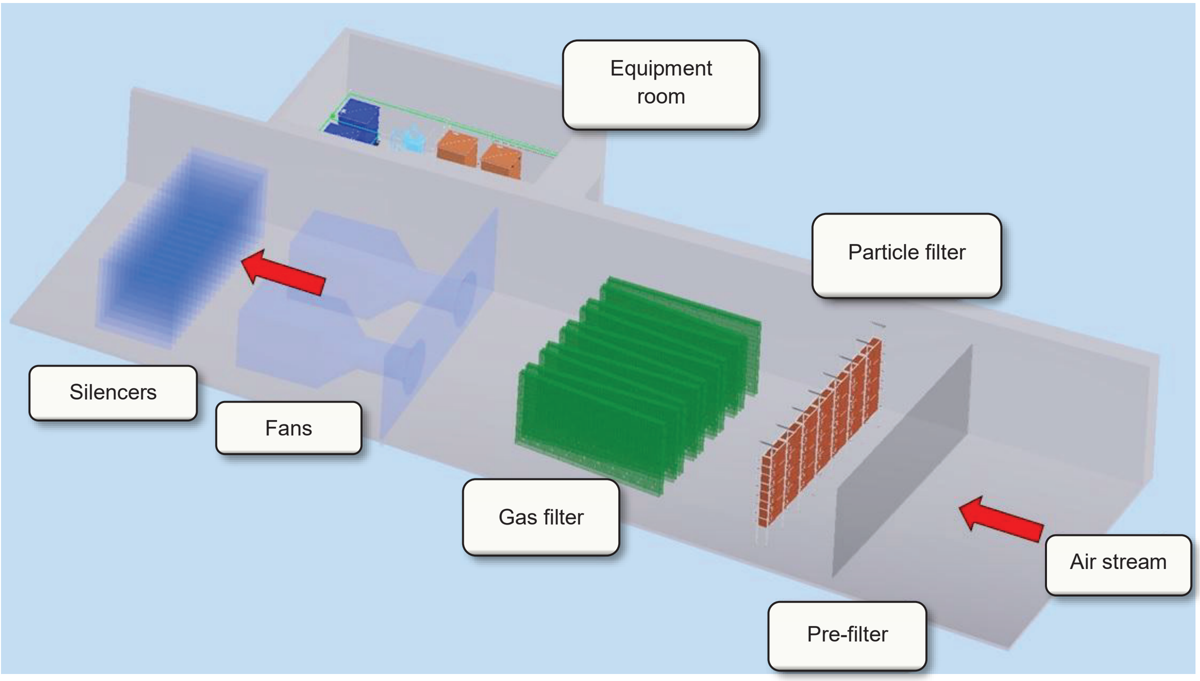

Figure 1. Air Purification System general layout

A general layout showing all parts is given in Figure 1. The main filter components are installed in an air plenum in front of the main ventilation fans. The auxiliary equipment as for voltage supply, wash-down and control is installed in the vicinity in a separate auxiliary equipment room. The location of this room is flexible and an optimum solution is to be found for each project.

2.1. Electrostatic Precipitator (ESP)

The electrostatic precipitator effectively removes very small particles, which are particularly hazardous to human health as they can penetrate deep into the respiratory system.

The integrated components for the particle filter are as follows:

- Roughing filter
- Electrostatic precipitator
- HV transformer rectifier
- Wash-down equipment

- Water recycling plant
- Control system and electrical components

The first part of the filtration system is a roughing filter, equipped with a protective grid to prevent large debris – such as paper, plastic bags, and leaves – from reaching the electrostatic precipitator (ESP). Air then passes through the ESP, the core of the system, which consists of two-stage modules arranged in a rack. The first stage functions as an ioniser, electrically charging particles, while the second stage, the collector, removes them from the airflow, achieving over 80% efficiency for particles between 0.1 µm and 10 µm. The modules are made of high-grade stainless steel (AISI 316). The whole ESP system is equipped with rinsing nozzles, water supply lines, and electrical connections to form a filter wall that is operated fully automatically.

A three-phase transformer-rectifier unit supplies high voltage to the modules, with independent control for the ioniser and collector. A high-voltage regulator ensures consistent optimal performance, while the compact design reduces installation and operational costs.

Collector plates are periodically washed with water via integrated spray nozzles. During cleaning, the modules are electrically disconnected, optionally compressed air pre-dries the plates to minimise downtime. A controlled voltage ramp-up after the wash-down prevents localised flashovers, maintaining high separation efficiency even with residual moisture. The cleaning cycle is typically scheduled during low-traffic periods and lasts around 20 minutes. Wash water is collected, treated for reuse, and stored, while concentrated sludge is safely disposed of through the tunnel sewage system.

The entire system is automated and PLC-controlled with multiple safety features including door contacts to prevent unauthorised access and strategically placed emergency stop buttons for rapid shutdown.

2.2. Gas filter – Activated Carbon Filter

Following particle removal, the exhaust air passes through an activated carbon filter designed to reduce in particular nitrogen dioxide concentrations. Activated carbon possesses a large surface area with numerous micropores capable of adsorbing large numbers of gaseous molecules. In tunnel applications, the activated carbon may be impregnated with chemical agents to enhance the adsorption and conversion of NO_2 into less harmful compounds such as nitrates or nitrogen.

The gas filter designed by FILTRONtec has a modular design like the particle filter. The carbon consisting of pellets or granulate is filled into the filter walls, which are made of high-quality stainless-steel perforated plates. The exhaust air flows through the filter wall. In order to achieve the necessary residency time for the adsorption process, the filter walls in the air channel are arranged in a "W" shape. The gas filter is a static structure with no moving parts that requires no energy to operate, other than the electricity needed to run the ventilation fans for moving the air.

The effectiveness of activated carbon filters depends on contact time, air temperature, humidity, and pollutant concentration. Typically, these filters can achieve up to 90% removal efficiency for NO_2 when properly maintained with higher adsorption rates at the beginning. Regeneration or replacement of the carbon media is required periodically, but typically after many years of operation, depending on loading rates and tunnel traffic volumes. Combined with ESP technology, these systems can achieve comprehensive purification of tunnel exhaust air, substantially reducing environmental emissions.

2.3. Behaviour of the air purification system in case of a fire

The particle filter is designed to withstand gas temperatures up to 400°C without losing integrity and without contributing to the thermal load. If there is a fire, it is therefore also possible to feed the fire gases from the tunnel through the filter. When the fire starts, the smoke particles with any attached toxic substances are deposited on the filter and consequently kept out of the ambient air. If the filter is arranged in the bypass, when a fire starts, the visibility in the traffic tubes can be enhanced by the filter in addition to fume extraction giving tunnel users and rescue teams more time for safe evacuation. This feature was proven by fire tests in a tunnel. After cleaning, the filter is ready for use again without time consuming overhauling.

Activated carbon can ignite at high temperatures (>250°C). Thus, the passage of fire gases through the filter must be avoided. Otherwise, activated carbon pores would get saturated by the smoke generated in the fire and the activated carbon had to be replaced completely. Isolation of the activated carbon filter is usually done by dampers and a bypass.

3 COMPARASION BETWEEN EXHAUST STACK SOLUTIONS AND AIR PURIFICATION SYSTEMS

3.1 Efficiency Against Pollutants

Traditional exhaust stack systems rely on a relatively simple concept; the dilution and dispersion of pollutants to manage air quality. They can be effective at reducing local concentrations near tunnel portals by releasing pollutants at higher altitudes so they dilute before reaching street level. However, this approach shifts rather than removes pollutants: it relies on meteorology to dilute and transport contaminants, and therefore can cause wider regional impacts and episodic downwind exposure. Technical assessments of tunnel stacks emphasise their dependence on atmospheric mixing and site-specific dispersion conditions, and warn that stacks do not reduce total emitted mass [1].

Furthermore, studies indicate that conventional exhaust stacks can reduce gas concentrations to safe levels within minutes during peak traffic but have limited capacity to address ultrafine particulate matter ($PM_{2.5}$ and PM_{10}) [2].

In contrast, Air Purification Systems actively remove both gaseous and particulate pollutants from the airflow before release to the atmosphere ensuring that the outlet concentrations are well below hazardous concentration limits. The extensive experience accumulated by FILTRONtec over the years in several international projects indicates that overall removal efficiencies of over 80% for PM and up to 90% for NO_2 can be achieved when combining ESPs with activated carbon filtration. This active purification significantly reduces the total pollutant load entering the surrounding environment, improving local and regional air quality as well as exposure of residents living near tunnel portals.

3.2 Visual and Environmental Impact

The visual and environmental impacts of these systems also differ substantially. Traditional exhaust stacks are typically tall and prominent structures that can disrupt urban aesthetics and skyline views, generating public concern in densely populated areas. Beyond visual impact, the expelled air contributes to localised atmospheric pollution, dispersing NO_x, CO, and particulate matter into surrounding environments. This may exacerbate urban air quality issues, particularly near tunnel exits, and can have cumulative effects on nearby vegetation and human health [3].

Furthermore, the installation of chimneys presents other environmental disadvantages related to the extensive earthworks required in particular with high overlying formation in tunnels through mountains. Constructing these large vertical structures often demands deep excavation and significant alteration of the surrounding terrain. Moreover, the extraction, transport, and disposal of large volumes of soil generate additional emissions from heavy machinery, contributing further to air pollution and carbon footprint. The construction phase can also produce noise and dust pollution, affecting nearby communities and wildlife.

Air Purification Systems, on the other hand, are often integrated within the tunnel infrastructure close to the portals, minimising above-ground visual intrusion. Since the exhaust air is treated before release, the outlet structures can be smaller and located closer to ground level without adverse effects on local air quality (see Figure 3). This results in a lower visual impact and greater flexibility in tunnel design. Environmentally, APS reduce secondary pollution and contribute to compliance with emission reduction targets in line with international climate and air quality directives.

Figure 2. Air outlet of an Air Purification System installed in Madrid, (right side of the picture)

3.3 Implementation and Operational Costs

Cost is a critical determinant in choosing between purification and exhaust-only strategies. Costs can be divided into implementation costs, also known as initial capital expenditure (CAPEX), and operational costs, also known as ongoing operating expense (OPEX). Implementation costs include equipment, structural works (ducts, supports, rooms), integration with existing ventilation, and installation. Operational costs include energy for fans and treatment devices, maintenance, spare parts (filters, catalyst replacement), monitoring and control, and periodic major overhauls.

From a cost perspective, traditional exhaust stack systems are well-established. Initial capital costs mainly include the construction of ducts, fans, and the stack itself, as well as electrical systems for fan operation. However, the land use and construction costs for large stacks – especially in urban areas – can be substantial. Furthermore, operational costs are significant due to high energy consumption required to move large volumes of air continuously and with high velocities, particularly during peak traffic periods.

Air Purification Systems involve higher initial capital investment due to specialised equipment such as ESP modules, activated carbon filters, and control systems. Operational costs arise from energy consumption for fans and ESP power supplies, as well as periodic cleaning oh the electrostatic precipitator. Considering that the air velocity in the air purification system designed by FILTRONtec is around 5 m/s, while air velocity at exhaust stacks can reach up to 20 m/s, the energy consumption of the fans is considerably lower in the case of the air purification system. The electrostatic precipitator (ESP) operates at a voltage of 12-16 kV and is powered by direct current (DC). As it is an electrostatic system, currents are low – usually in the range of microamperes (µA) to a few milliamperes (mA) – resulting in a low overall electrical power consumption despite its high operating voltage. The auxiliary equipment required for cleaning of the ESP only operates for a few minutes per day making its power consumption negligible.

Lifecycle analyses indicate that air purification systems can offer competitive total costs when environmental benefits, reduced land requirements, and potential avoidance of air quality penalties are considered [4]. Furthermore, technological advancements and modular designs are steadily reducing costs and improving scalability.

4 EFFICIENCY OF AIR PURIFICATION SYSTEMS

FILTRONtec has conducted numerous tests to determinate the efficiency of their filtration technology, both in the laboratory and on the air purifications systems installed in its most recent projects.

For example, an efficiency test was carried out over several days for the air purification system installed in the Central Wan Chai Bypass road tunnel in Hong Kong. FILTRONtec successfully passed the tests, demonstrating that the filtration system meets the efficiency requirements set out in the project specifications, which are as follows:

- For particle, when inlet concentration equal to or greater than 0.5 mg/m^3, not less than 85% of PM_{10} shall be removed after the air is treated by the APS. For inlet concentration lower than 0.5mg/m^3, the outlet concentration shall not be greater than 0.1 mg/m^3.
- For NO_2, when inlet concentration equal to or greater than 0.25 ppm, not less than 85% of NO_2 shall be removed after the air is treated by the APS. For inlet concentration less than 0.25 ppm, the outlet concentration shall not be greater than 0.05 ppm.

In addition, each air purification system for the Central Wan Chai Bypass road tunnel is equipped with its own Air Monitoring System to monitor the performance continuously. The systems measure the following parameters:

- PM_1, $PM_{2.5}$, PM_{10}
- NO_2
- Temperature, Humidity and Pressure

Every day, the programmable controller (PLC) of the Air Purification System creates one log file containing the measurements from each Air Monitoring System. The data are recorded every five seconds. FILTRONtec analysed the data recorded since tunnel opening on 24th February 2019 for longer than two years.

As an example, the separation rates both on a daily basis (Figure 3) as well as on a monthly basis (Figure 4) are shown below for the Air Purification System installed in the West Ventilation Building of the Central Wan-Chai Bypass road tunnel.

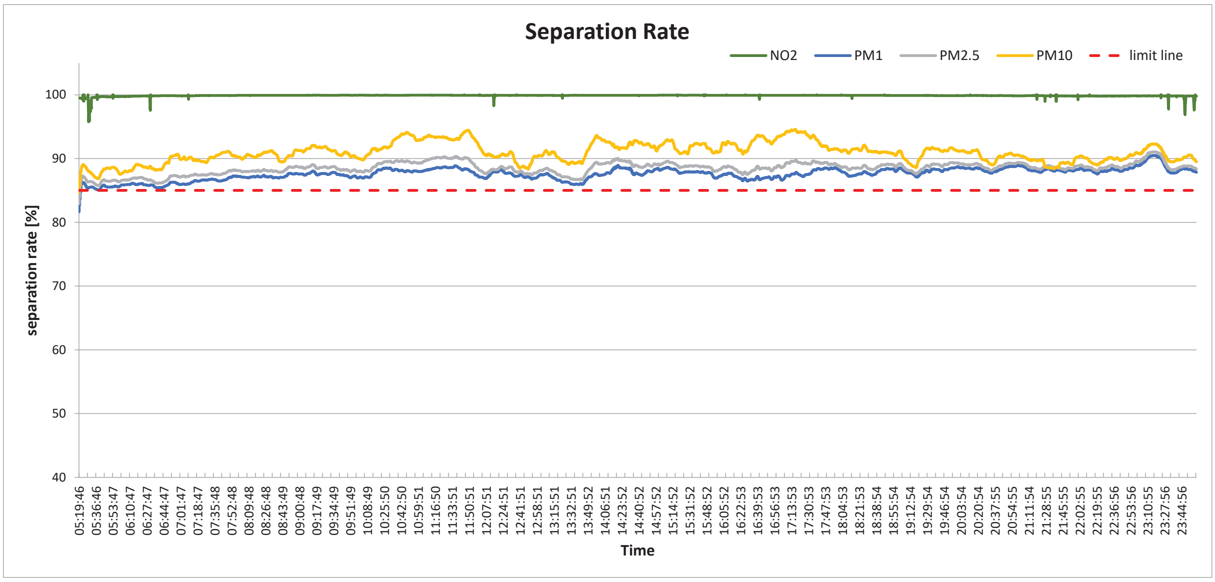

Figure 3. NO_2 and particle separation rates vs time. Central Wan Chai Bypass road tunnel. 24 February 2019.

As shown, the separation rate for NO_2 was even close to 100% for the newly installed filter which usually drops over time. In any case, the performance for particulate matter and gaseous pollutants was well below the specified criteria over a complete day under changing traffic conditions.

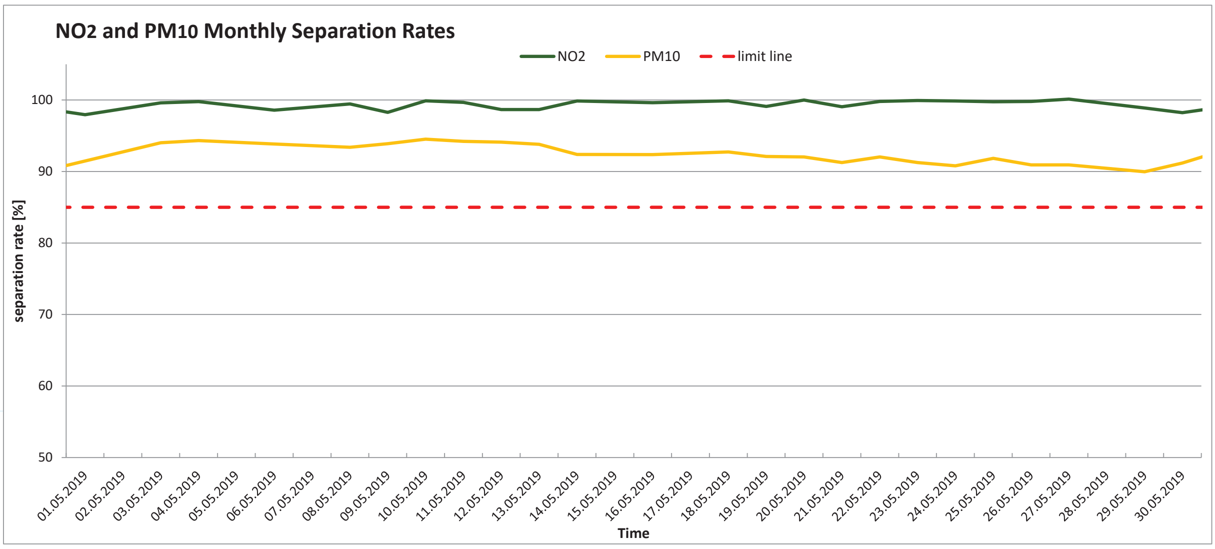

Figure 4. NO_2 and PM_{10} separation rates vs time. Central Wan Chai Bypass road tunnel. May 2019.

In the most recent installation of the Nuevo Mahou-Calderón tunnel in the M-30 ring road in Madrid, the project specifications did not require the installation of permanent air monitoring system. However, an independent laboratory performed the corresponding efficiency tests for two days as a final step in the commissioning process. For this purpose, portable particle measurement equipment was installed before and after the particle filter. The project does not include a gas filter, therefore, no NO_2 data are available. The diagram in Figure 5 shows the calculated separation rates in percentage for PM_1, $PM_{2.5}$, and PM_{10} for the data measured on 26 April 2023. The fulfilment of the specification requirement could also be proven for this filter.

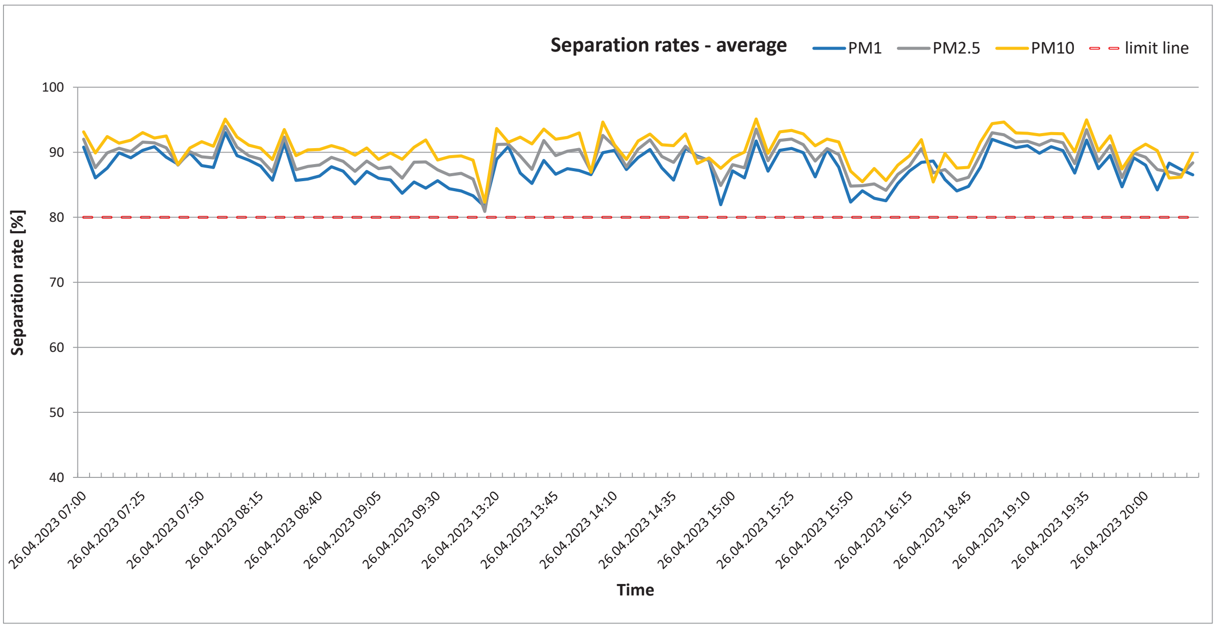

Figure 5. Particles separation rates vs time. Nuevo Mahou Calderon, M-30 ring road. 26 April 2023

5 CONCLUSION

Air Purification Systems represent a transformative advancement in road tunnel ventilation technology. By integrating electrostatic precipitation for particulate removal and activated carbon filtration for NO_2 adsorption, these systems provide a comprehensive approach to air quality management that extends beyond mere dilution. Compared with traditional exhaust stacks, Air Purification Systems offer superior pollutant removal efficiency, reduced environmental impact, and improved architectural flexibility. Ongoing technological improvements and environmental regulations are making these systems increasingly viable for future tunnel projects.

From an environmental perspective, chimneys may simply transfer the problem of pollution rather than eliminate it. By releasing pollutants at higher altitudes, they can spread contaminants over wider areas, potentially affecting air quality far from the tunnel itself. In addition, the visual impact of tall chimneys can degrade the landscape and reduce the aesthetic value of natural or urban environments. Overall, while chimneys may improve local air conditions near tunnel exits, their broader environmental implications make them a controversial solution.

As transportation infrastructure continues to evolve toward sustainability, incorporating air purification technologies into tunnel design and retrofitting projects will be essential for minimising environmental footprints and ensuring compliance with stringent air quality objectives.

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CHALLENGES IN ROAD DEVELOPMENT IN THE HIMALAYAS: A CASE FOR HIGHWAY TUNNELS AS A SUSTAINABLE ALTERNATIVE

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ABSTRACT

The Himalayan region poses unique and formidable challenges to road infrastructure development due to its complex geology, fragile ecosystems, extreme weather conditions, and high seismic vulnerability. Conventional road construction methods primarily involving cutting through mountainous terrain have led to frequent landslides, road subsidence, erosion, and significant ecological degradation. These issues not only escalate maintenance costs but also compromise the reliability and safety of the transport network.

This paper critically examines the long-term technical, environmental, and economic limitations of traditional road-building techniques in the Himalayas. It argues for the strategic shift towards the construction of highway tunnels as a sustainable and cost-effective alternative. Drawing upon case studies from India and other mountainous countries, the study highlights how tunnels reduce surface disruption, mitigate landslide risks, offer all-weather connectivity, and minimize environmental impact.

The paper further provides a comparative analysis of life-cycle costs, construction timelines, and socio-economic benefits of tunnels versus surface roads in hilly terrains. The findings suggest that while tunnel construction involves higher initial investment, it proves more economical and reliable over time. By evaluating case studies, cost-benefit analyses, and environmental impacts, the paper advocates for a policy and engineering shift towards tunnelling as a long-term solution for resilient Himalayan connectivity.

***Keywords**: Himalayan Road, Tunnel Construction, All Weather Connectivity, Environmental Sustainability*

1. INTRODUCTION

The Himalayas, stretching across Northern and Northeastern India and neighbouring countries, are among the youngest and most geologically active mountain ranges in the world. They are home to critical transport routes linking remote areas with major economic centres. However, road development in this region has been fraught with challenges, primarily due to:

- Steep gradients and rugged topography
- Frequent landslides and slope instability
- High rainfall and snow accumulation
- Earthquake-prone zones
- Environmental sensitivity and biodiversity concerns

Despite heavy investment, many Himalayan roads suffer from poor durability, seasonal closures, and dangerous driving conditions. This paper explores these challenges and introduces highway tunnels as a technically viable and economically prudent alternative.

2. CHALLENGES IN TRADITIONAL ROAS CONSTRUCTION IN THE HIMALYAS

2.1 Geological and Geotechnical Constraints

- Highly heterogeneous rock mass: The Himalayas comprise young, tectonically active formations such as phyllites, schists, quartzites, and crushed fault-zone materials, which are weak and highly jointed.
- Frequent shear zones and thrust faults: Major tectonic discontinuities (e.g., Main Boundary Thrust, Main Central Thrust) cause abrupt changes in rock conditions, leading to instability of cut slopes and foundations.

- Weathered and colluvial soils: Shallow slopes are often covered with loose debris and colluvium, which have low shear strength and are prone to failure when disturbed by excavation.

2.2 Slope Instability and Landslides

- Excavation-induced failures: Conventional bench-cutting methods steepen natural slopes, often triggering translational or rotational failures, particularly during monsoon seasons.
- Reactivation of dormant slides: Old and partially stabilized landslides are frequently reactivated during widening or construction, causing recurring maintenance issues.
- Over-steepened hill slopes: Limited right-of-way and terrain constraints lead to over-steepening of slopes, increasing risk of slope collapse.

2.3 Hydrological and Drainage Challenges

- Intense rainfall and poor drainage: The Himalayas experience high-intensity rainfall leading to erosion, surface runoff, and infiltration-induced slope failures.
- Inadequate subsurface drainage: Traditional Road designs often lack subsoil drainage provisions, resulting in pore pressure buildup and weakening of slope materials.
- Water-induced erosion: Poor control of surface water contributes to gully erosion and undercutting of road shoulders and retaining walls.

2.4 Seismic Vulnerability

- Active tectonic environment: The Himalayas lie within one of the most seismically active regions globally. Earthquakes frequently induce ground shaking, differential settlements, and slope failures along cut slopes.
- Inadequate seismic design: Traditional construction methods often overlook seismic slope stability analysis and earthquake-resistant retaining structures.

2.5 Climatic and Environmental Constraints

- Extreme weather conditions: Heavy snowfall, freeze–thaw cycles, and prolonged monsoon periods lead to rapid degradation of pavement and slope stability.
- Short construction seasons: Harsh winters and heavy rainfall limit effective working periods to only 6–8 months annually.
- Environmental sensitivity: Road construction often disturbs fragile ecosystems, leading to deforestation, habitat loss, and increased erosion.

2.6 Maintenance and Lifecycle Issues

- High maintenance demand: Frequent landslides, erosion, and pavement failures require continuous maintenance and periodic rehabilitation.
- Increased cost and downtime: Recurrent slope repairs, debris clearance, and traffic disruptions significantly escalate the cost and reduce road reliability.
- Limited sustainability: Conventional hill-cutting methods often lead to unsustainable long-term performance due to repeated geotechnical failures.

2.7 Socio-Technical and Logistical Constraints

- Difficult access and transportation of materials: Remote terrain and limited approach roads complicate logistics for construction materials and heavy machinery.

- Restricted working space: Narrow valleys and steep hillsides restrict construction zones, making mechanized operation and slope stabilization difficult.
- Community and land acquisition issues: Widening existing hill roads often requires additional land, leading to socio-environmental conflicts.

3. JUSTIFICATION FOR ADOPTING TUNNEL ALLIGNMENTR IN THE HIMALAYAS

Conventional cut-and-fill road construction methods in the Himalayan terrain have long been constrained by the region's complex geological, topographical, and climatic conditions. The young and tectonically active mountains comprise highly deformed and weathered rock masses, frequent thrusts, and steep slopes that render traditional surface alignments both geotechnically unstable and environmentally unsustainable. Continuous occurrences of landslides, slope failures, and erosion not only increase construction and maintenance costs but also pose serious safety risks and traffic disruptions.

In contrast, tunnel-based alignments offer a technically superior and sustainable alternative by bypassing unstable hill slopes and minimizing environmental disturbance. With the advance tunnelling techniques such as the Controlled Blasting and New Austrian Tunnelling Method (NATM), it is now feasible to construct long road tunnels safely through variable Himalayan geology. Tunnel construction enables improved operational reliability, reduced maintenance, and all-weather connectivity, which are critical for strategic and economic development in mountainous regions.

The following table (Table 1) presents a comparative assessment between traditional hill road construction and tunnel-based construction methods, highlighting their relative performance under Himalayan conditions.

Table 1: Comparison between Traditional Road Construction and Tunnel Construction in Himalayan Conditions

Parameter	Traditional Hill Road Construction (Cut-and-Fill Method)	Tunnel Construction (NATM / Modern Methods)
Geological Interaction	Direct exposure of cut slopes to heterogeneous and unstable rock masses; high risk of slope failures and erosion.	Minimal surface disturbance; tunnel passes through confined rock mass with controlled stress redistribution.
Topographical Constraints	Requires extensive hill cutting, leading to large-scale slope modification.	Follows natural contours beneath surface, avoiding steep terrain and unstable slopes.
Stability and Safety	Frequent landslides, rockfalls, and slope failures; high maintenance and safety risks.	Stable underground environment when properly supported; negligible geotechnical failures.
Environmental Impact	Significant vegetation loss, deforestation, and soil erosion; major impact on hydrology.	Reduced surface disturbance and minimal ecological footprint.
Weather and Climatic Influence	Adversely affected by rainfall, snow, and freeze–thaw cycles causing pavement and slope damage.	Protected from weather extremes; all-weather operability ensured.
Drainage and Water Management	Poor surface and subsurface drainage; high erosion and slope weakening.	Controlled drainage through invert systems and lining provisions.
Seismic Response	Exposed slopes more vulnerable to seismic-induced failures.	Underground alignment better resists seismic activity due to confinement effect.

Parameter	Traditional Hill Road Construction (Cut-and-Fill Method)	Tunnel Construction (NATM / Modern Methods)
Construction Time and Accessibility	Slow progress due to difficult terrain, limited work fronts, and monsoon interruptions.	Faster progress with multiple faces and parallel excavation; less affected by external conditions.
Maintenance Requirements	High, due to recurring slope stabilization, erosion control, and pavement restoration.	Low, with periodic inspection and localized lining maintenance.
Operational Efficiency	Frequent traffic disruptions due to landslides and blockages.	Uninterrupted traffic flow; improved travel time and safety.
Long-Term Sustainability	Poor; repeated slope failures and degradation increase lifecycle cost.	High; durable infrastructure with reduced environmental and maintenance costs.

The following illustrative photographs vividly demonstrate the severe geotechnical instability along the national highways in Himalayan region, where recurring landslides and slope failures result in prolonged closures of vital highways. These disruptions not only impede the safe and reliable movement of traffic but also adversely affect tourism inflow, supply-chain connectivity, emergency response operations, and the socioeconomic well-being of communities dependent on these transport links.

4. EXAMPLES OF ROAD TUNNEL IN HIMALAYAS

Over the past years, numerous road tunnels have been successfully constructed across the Himalayan region, with many more currently under execution. These vital infrastructure works have significantly enhanced all-weather connectivity, ensuring safer, faster, and more reliable transport throughout some of the nation's most challenging terrain. Such projects are not merely engineering achievements—they represent a substantial contribution to the country's development, regional integration, and the well-being of society at large.

4.1 Operational Tunnels

- Dr. Syama Prasad Mookerjee (Chenani Nashri) Tunnel - 9.28 km in Jammu and Kashmir
- Atal (Rohtang) Tunnel - 9.02 km in Himachal Pradesh
- Banihal–Qazigund - 8.45 km in Jammu and Kashmir
- Sonamarg Z-Morh Tunnel - 6.5 km in Jammu and Kashmir
- Sela Tunnel - 2.53 km in Arunachal Pradesh
- Churia Tunnel (historic; non-operational) in Nepal
- Nagdhunga Tunnel in Nepal

4.2 Under Construction / Proposed

- Zoji La Tunnel - 14 km in Jammu and Kashmir
- Chattergala Tunnel - 6.8 km in Jammu and Kashmir
- Sinthan Top - 10.3 km in Jammu and Kashmir
- Sudhmahadev–Dharanga - 8 km in Jammu and Kashmir
- Bhaderwah Tunnel - 3.5 km in Jammu and Kashmir
- Sinkula Tunnel – 4.1km in connecting Himachal Pradesh and Ladakh
- Various strategic BRO tunnels in Jammu and Kashmir

5. COST BENEFIT ANALYSIS

Cost–Benefit Analysis (CBA) for comparing a tunnel alignment for road versus traditional cut-and-fill road construction in an Indian Himalayan scenario is important to understand and has been explained in the following paragraphs.

5.1. Key Cost Items

Cost Item	Tunnel Alignment	Cut-and-Fill Road Construction
Construction Cost (CapEx)	• Excavation and support (including pre support measures, shotcrete, rock bolts, lattice girders / steel ribs) contributes towards high cost due to complex heterogenous geology. • Waterproofing, ventilation, lighting, portals, emergency systems. • Longer construction period due to geological challenges • Specialised labour and equipment.	• Earthwork (cutting and filling) costs: excavation, disposal of cut material, transport of fill/borrow. • Slope stabilisation (retaining walls, rockfall protection) often required on hillside alignments. • Land acquisition, environmental mitigation, drainage works.
Operation & Maintenance (OpEx)	• Tunnel maintenance: lining upkeep, ventilation/lighting systems, monitoring. • Lower exposure to weather and landslides, so fewer disruptions.	• Road maintenance: frequent slope repairs, landslide clearances, drainage management, pavement repairs. • Higher recurrence of maintenance due to geotechnical instability in mountainous terrain.
Risk, Safety & External Costs	• No risk of landslides, rock-falls and weather-induced closure. • Improved all-weather connectivity, fewer disruptions.	• High probability of slope failures, landslides, traffic disruptions, possibly longer closures. • Environmental/land-use impacts may be higher (deforestation, erosion).
Travel/Operational Benefits	• Often shorter alignment, higher speed, fewer grade changes, less distance/time travelled.	• Follow longer surface path, more curves/grades resulting in higher vehicle operating costs, longer travel time.
	• Higher reliability (fewer closures, safer) resulting in higher user benefit.	• Subject to frequent interruptions due to slope failures/closures.

5.2. Illustrative Quantitative Example

Let's assume a hypothetical 5 km section in a Himalayan terrain (steep, unstable slopes) and compare major cost elements (all values in Rs. crores).

Parameter	Tunnel (5 km)	Cut & Fill Road (5 km)
Construction cost per km	Rs 150 crore/km, total ≈ Rs 750 crore	Earthwork + slope stabilisation say Rs 40 crore/km, total ≈ Rs 200 crore
Annual maintenance & disruption cost	Lower: mean Rs 1 crore/year maintenance + minimal closure cost	Higher: mean Rs 5 crore/year (slope/repair + closure cost)
Travel time saving & reliability benefit	Suppose annual user benefit Rs 10 crore (time saving, safety, reduced vehicle costs)	Suppose benefit Rs 4 crore/year
Lifespan considered	30 years	30 years
Discount rate (for simplicity)	8 %	8 %

Tunnel cost estimate is hypothetical; actual cost can vary widely depending on site-specific factors (geology, overburden, length, traffic, etc.). According to some international standards, cost of tunnels may be up to ~10 times the equivalent open-air structure in favourable conditions.

In following paragraphs an attempt has been made to calculate rough estimate for Net Present Value (NPV) based on the above example:

1. NPV for Tunnel alignment

Present Value of Uniform Annual Benefits

$$PV_{\text{benefits}} = A \times \frac{1-(1+r)^{-n}}{r}$$

Where:

- A= Annual benefit (Rs/year) = Rs. (10+1) Crore = Rs. 11Crore
- r= discount rate = 8%
- n= project life (years) = 30 years

Therefore

$$PV_{\text{benefits}} = 11 \times \frac{1-(1+8)^{-30}}{8}$$

= 11 x 10.6

= Rs. 116.60 Crore

Now, Net Present Value (NPV)

$NPV = PV_{\text{benefits}} - C_0$, where C_0= initial capital investment (CapEx)

= 116.60 – 750

= Rs. (-) 633.40 Crore

Which shows that even though Annual benefits are positive, due to high Capex involved, the NPV is deeply negative.

2. NPV for Cut and Fill Road Construction in Himalayas

Applying the same formula as above, the NPV comes to Rs. (-) 104.60 Crore

From the above discussion, on raw cost/benefit alone, both options may show negative NPV under these assumptions, but the differential cost is much larger for tunnel. However, when factoring in reliability, social/economic development, avoided risk of closure (which may be much higher than ₹ 5 crore/year), and long-term maintenance, the tunnel may become more favourable, especially in very unstable terrain.

6. CONCLUSION

While tunnel alignments typically demand higher upfront investment than conventional cut-and-fill approaches, a broader evaluation reveals their long-term value—especially in geologically fragile Himalayan terrain. When factors such as slope instability, high overburden, active thrust zones, recurring maintenance, traffic disruptions, and the economic impact of frequent closures are fully accounted for, tunnels often emerge as the more reliable and sustainable option. Decisions on Himalayan transportation should be decided not by initial construction cost alone, but by a comprehensive life-cycle cost assessment that incorporates construction, maintenance, user benefits, risk mitigation, and environmental and social impacts. This paper aims to highlight the need to seriously consider tunnel alignments, recognizing their potential to deliver lasting economic, strategic, and all-weather connectivity benefits for the region.

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OVERCOMING GEOTECHNICAL CHALLENGES IN HIMALAYAN TUNNELING USING NATM : CASE STUDY OF SHIMLA BYPASS TUNNEL – PACKAGE 1

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ABSTRACT

The construction of highway tunnels in the geologically complex and seismically active Himalayan region poses significant engineering challenges due to variable overburden, weak and sheared rock masses, high in-situ stresses, water ingress, and frequent face collapses. This paper presents a comprehensive study on the implementation of the New Austrian Tunnelling Method (NATM) for the Shimla Bypass Tunnel – Package 1, a critical infrastructure project aimed at decongesting urban traffic and improving regional connectivity.

The NATM approach, with its emphasis on observational methods, flexible support systems, and staged excavation, proved instrumental in negotiating the highly variable geological conditions encountered during the execution of the project. Through detailed geotechnical investigations, real-time monitoring, and adaptive tunnel design, the project team successfully addressed major challenges such as ground behaviour, shear zones, water inflows, and overbreak control.

Key aspects covered in this paper include geological mapping, classification of rock mass, support system design (including lattice girders, shotcrete, rock bolts, and pipe roofing), instrumentation and monitoring protocols, and ground response strategies. Special focus is given to the role of face advance control, pre-support measures in poor ground conditions, and the coordination between design and construction teams to ensure safety and stability.

Lessons learned from the Shimla Bypass Tunnel provide valuable insights into the practical application of NATM in Himalayan tunnelling projects and underline the importance of geotechnical adaptability, proactive risk management, and continuous feedback between monitoring and design.

***Keywords**: New Austrian Tunnelling Method (NATM), Himalayan Geology, Geotechnical Monitoring, Tunnel Support Systems, Face Advance Control*

2. PROJECT OVERVIEW

2.1 Location and Significance

The Shimla Bypass Tunnel – Package 1 is located in Himachal Pradesh, India, and forms part of a broader road infrastructure upgrade intended to bypass the congested urban core of Shimla. The tunnel passes through highly folded and faulted geological strata with an overburden ranging from 15 m to over 200 m.

The Shimla Bypass Tunnel Project (SBTP) is an ongoing road infrastructure initiative being implemented in the state of Himachal Pradesh, under the aegis of the National Highways Authority of India (NHAI) and forms a broader road infrastructure upgrade intended to bypass the congested urban core of Shimla. The work package forms part of the four-laning of National Highway-5 (NH-5) from Kaithlighat to Shakral Village, designated as Shimla Bypass Package–I (Km 128.835 to Km 146.300), with a design length of 17.465 km, executed on a Hybrid Annuity Mode (HAM) framework.

The tunnel construction under this package is being undertaken by Sammon Infracorp Pvt. Ltd. on an Engineering, Procurement and Construction (EPC) basis for SP Singla Construction Pvt. Ltd., the appointed concessionaire. The project primarily comprises two twin-tube tunnels, initiating near Kaithlighat Village in Solan District and terminating near Shakral Village in Shimla District, thereby facilitating efficient traffic movement and reducing congestion along the existing highway alignment. The key plan of the Shimla Bypass Tunnel Project (SBTP), illustrating the alignment and tunnel locations, is presented in Figure 1.

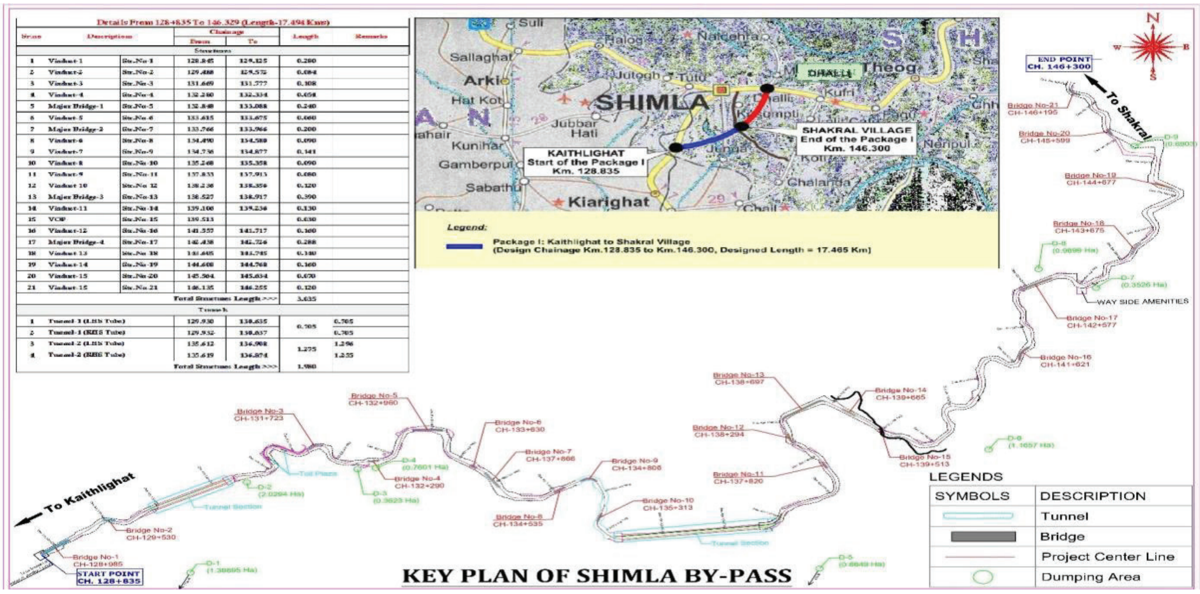


Figure1: Key Plan – Package 1, Shimla Bypass Project

2.2 Tunnel Specifications

The specifications of Tunnel 1 and Tunnel 2 under Shimla Bypass Project is given in Table 1 and Table 2 below:

S.No	Location	Start Design Chainage	End Design Chainage	Length of tunnel (m)	Carriageway width including Foot path & walkaway (m)
1	Tunnel-1 (LHS & RHS)	LHS, CH: Km 129+940	LHS, CH: Km 130+620	705	10.5
		RHS, CH: Km 129+930	RHS, CH: Km 130+620	705	11.95
2	Tunnel-2 (LHS & RHS)	LHS, CH: Km 135+595	LHS, CH: Km 136+800	1205	11.95
		RHS, CH: Km 135+622	RHS, CH: Km 136+800	1178	11.95

Table 1: Details of Tunnel-1 & Tunnel-2

S.No	Chainage (Km)	Type of Cross Passage	Width of CP (m)	Length of CP (m)
1	130+290	Vehicular	10.50	20.1
2	135+970	Vehicular	10.50	18.3
3	136+470	Vehicular	10.50	16.9

Table 2: Details of Cross Passages

General Arrangement Drawing (GAD) for Rock Class P4 (Poor) and P5 (Very Poor) is given in Figure 2.

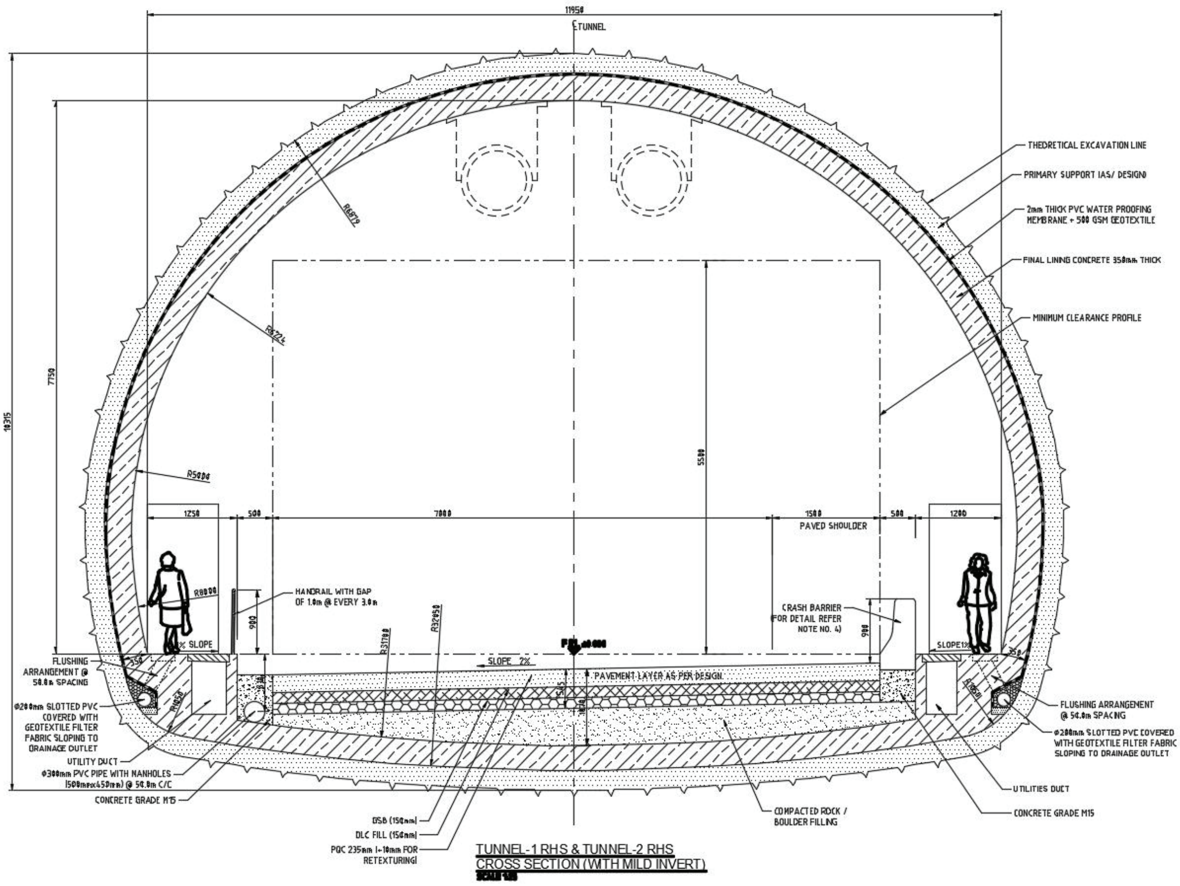
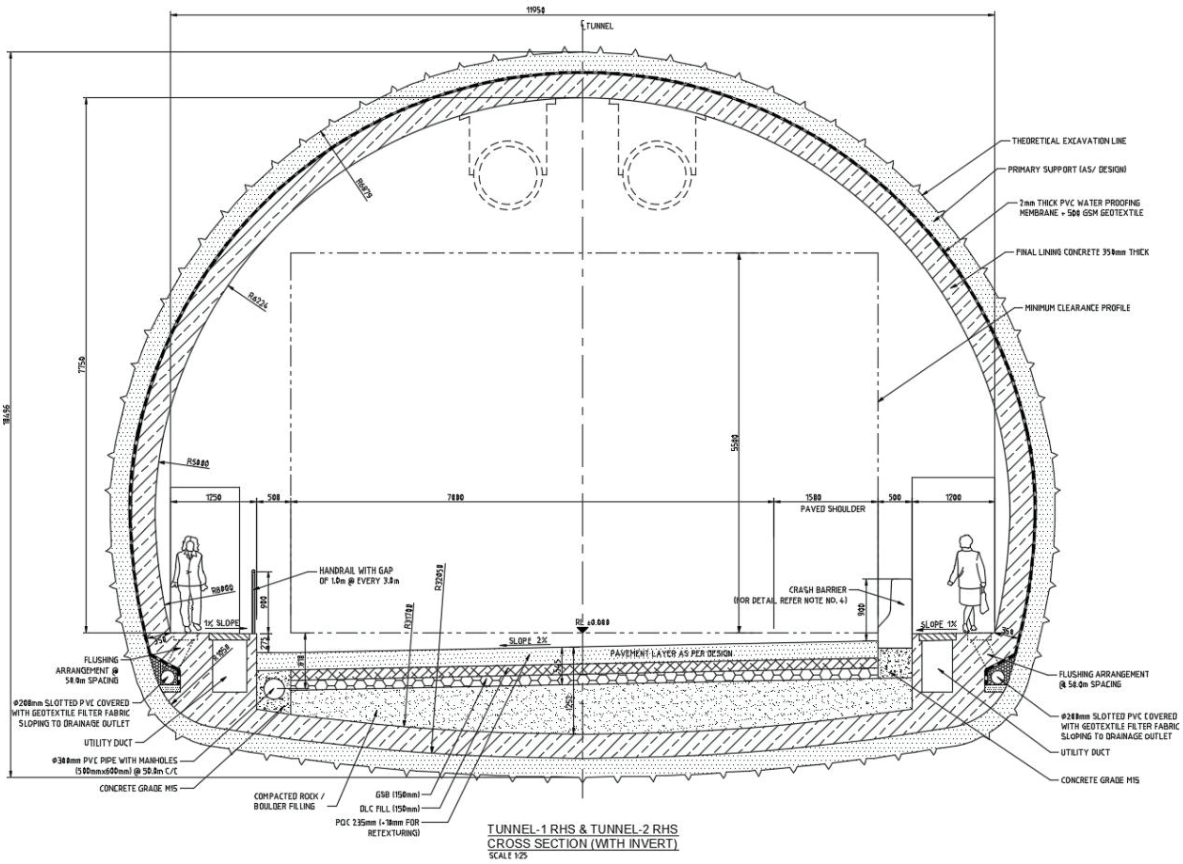


Figure 2: GAD for Rock Mass Class P4 (above) and P5 (below)



3. GEOLOGICAL AND GEOTECHNICAL CHALLENGES

3.1 Geological and Geotechnical Conditions

Geologically, the tunnel faces under Package 1 are predominantly excavated through dark grey, fine-grained phyllites and phyllitic quartzites belonging to the Manal Formation of the Jutogh Group, as well as through lithounits of the undifferentiated Jaunsar Group. The alignment lies within a tectonically active domain influenced by the Jutogh Thrust and Jaunsar Thrust, which exert a significant structural control on the prevailing rock mass conditions. The interaction of these regional thrust systems has resulted in the development of multiple shear zones characterized by highly jointed, crushed, and intensely sheared rock masses. These zones frequently exhibit tight folding, slickensided surfaces, and clayey gouge material, reflecting intense deformation and low rock mass strength. Consequently, the presence and orientation of these thrust-related shear zones play a critical role in governing the geotechnical behaviour and stability of the rock mass along the tunnel alignment, necessitating adaptive excavation and support strategies.

Rock mass characterization along the tunnel alignment was carried out in accordance with the Rock Mass Rating (RMR) system (Bieniawski, 1989), as specified in the project contract requirements. Field assessments were conducted through systematic geological face mapping, incorporating parameters such as uniaxial compressive strength (UCS) of intact rock, RQD (Rock Quality Designation), spacing and condition of discontinuities, groundwater conditions, and joint orientation adjustments.

Based on these evaluations, the calculated RMR values ranged from 5 to 35, indicating predominantly poor to very poor rock mass conditions prevailing along significant stretches of the tunnel alignment. Such low RMR values are reflective of highly weathered, sheared, and jointed rock masses, frequently associated with thrust-influenced zones and fault gouge materials. These conditions have resulted in extremely low stand-up time, pronounced deformation behaviour, and increased susceptibility to face instability during excavation.

Representative geological face maps and photographs of the exposed tunnel faces are presented in Figure 3, effectively illustrating the adverse geological and geotechnical conditions encountered during the tunnelling operations.

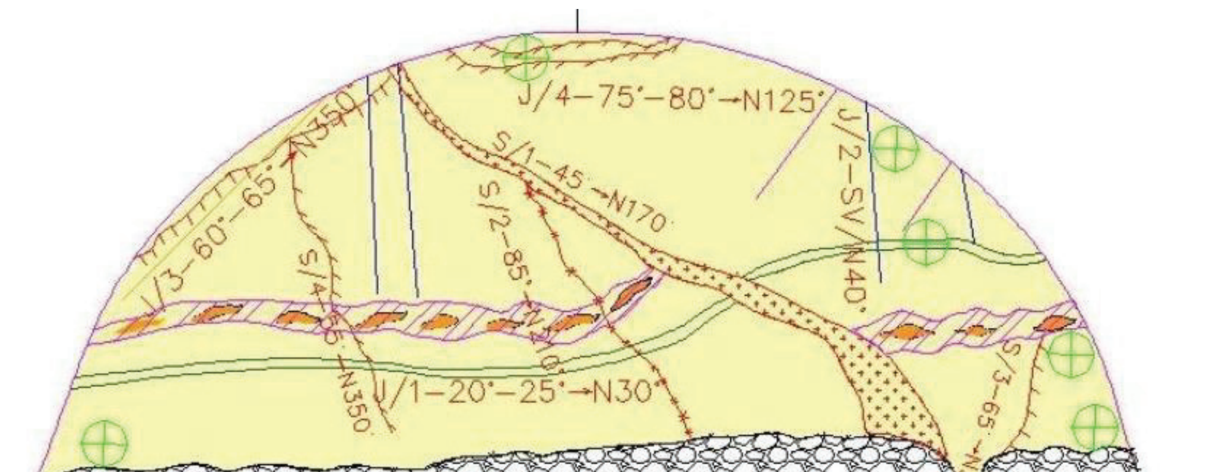
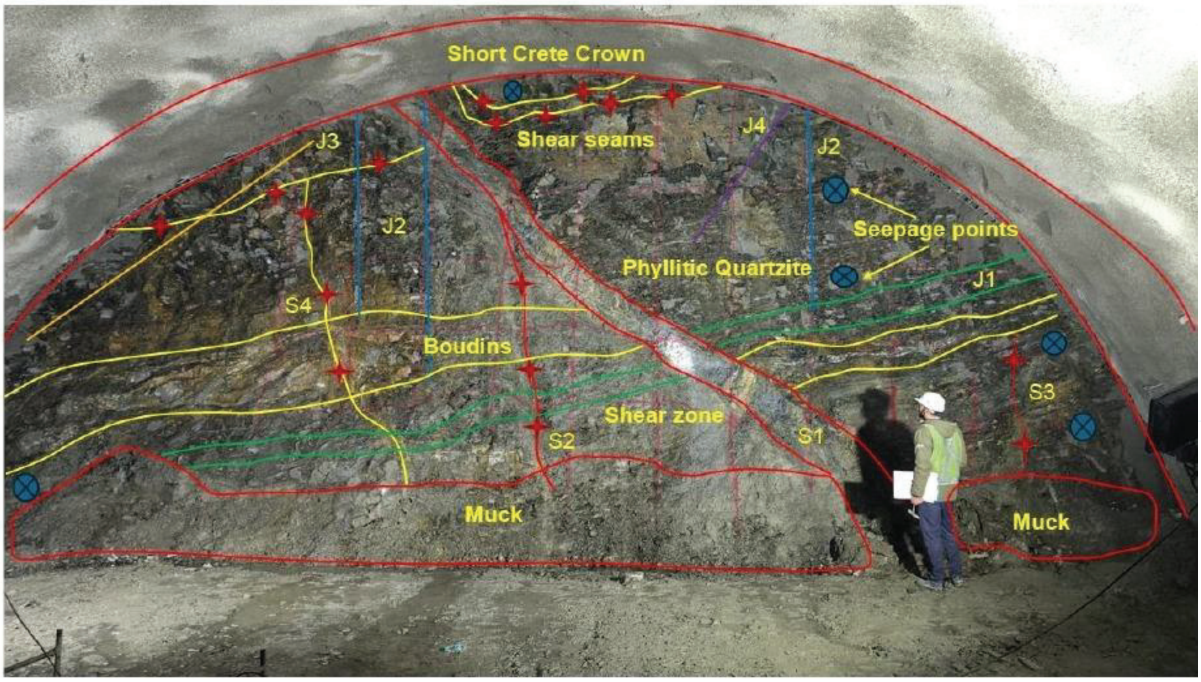


Figure 3: Geological Face Map (above) and Photograph of the exposed tunnel face (below)



3.2 Key Challenges Encountered

The close proximity of the Jutogh and Jaunsar Thrusts, along with their associated splays intersecting the tunnel alignment, has significantly influenced the geological complexity and presented substantial challenges to tunnelling operations. The intense tectonic deformation associated with these thrust zones has resulted in heterogeneous and anisotropic ground conditions, requiring constant adaptation of excavation and support strategies. The major geotechnical challenges encountered during excavation are summarized below:

- **Frequent occurrence of shear zones and fault gouge**: The alignment intersects several shear and fault zones composed of crushed, clayey gouge material with low cohesion and high deformability, resulting in poor stand-up time and frequent face instabilities.
- **Unpredictable transitions between rock classes**: Rapid and abrupt changes in rock mass quality—from fair phyllitic quartzite to very poor, sheared phyllite—posed difficulties in maintaining consistent support systems and excavation methodologies.
- **Groundwater ingress**: Significant water inflows were observed in fractured and permeable zones, leading to softening and deterioration of the surrounding rock mass, reduction in shear strength, and challenges in shotcrete application and adhesion.
- **Face instability**: Localized collapses occurred in weathered and highly sheared zones where the rock mass exhibited extremely low strength and cohesion, necessitating immediate face stabilization and pre-support interventions.
- **Overbreak Control** in heterogenous rock mass.
- **Variable overburden conditions (12–300 m)**: The wide range of overburden resulted in variable in-situ stress distributions, influencing ground response and requiring continuous modification of support design.
- **High in-situ stresses and squeezing ground behaviour**: Sections under high overburden experienced stress-induced deformation, including inward movement of sidewalls and roof convergence, leading to distortion of lattice girders and overstressing of the shotcrete lining.

Figure 4 below illustrates some of the key challenges encountered while tunnelling:

Face Instability

Overstressing of Shotcrete and distortion in LG

Ground Water Ingress

Face Collapse due to heavy ingress of water

Figure 4: Key Challenges Encountered during tunneling

4. IMPLEMENTATION OF NATM

The New Austrian Tunnelling Method (NATM) was adopted due to its flexibility and reliance on ground–support interaction principles. The method emphasizes:

- Observational construction with adaptive design modifications.
- Immediate support installation to mobilize ground self-stability.
- Systematic monitoring of displacements and loads to guide adjustments.

4.1 Excavation Strategy

- Excavation was carried out in stages (top heading, bench, and invert) – essential for ground control and ensuring tunnel stability (Figure 5)
- Typical face advance: 0.8–1.2 m/day, reduced in weak zones.
- Controlled blasting and mechanical excavation were alternated depending on rock conditions.

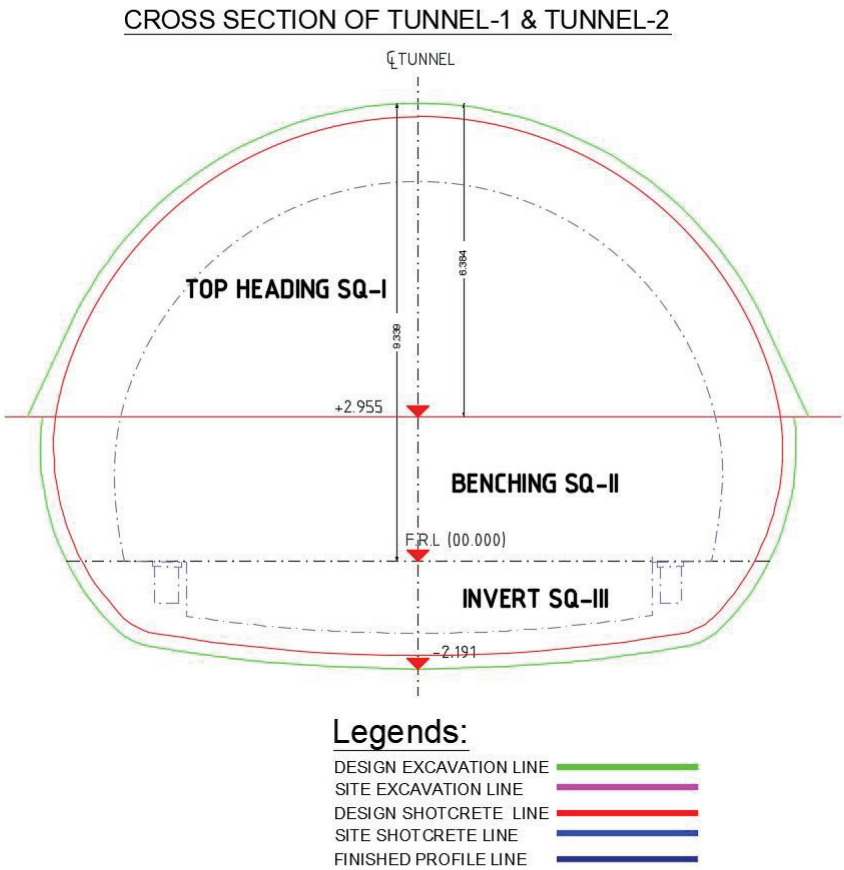


Figure 5: Excavation Sequence

4.2 Pre-Support Measures

- Drainage holes to relieve water pressure ahead of the face
- Pipe Roofing – 12m long 114mm dia (in very poor rock mass) and 89mm (in poor rock mass) with 4m overlap in either case (Figure 6)
- Installation of fiberglass dowels on the face in very poor rock mass condition as and when required

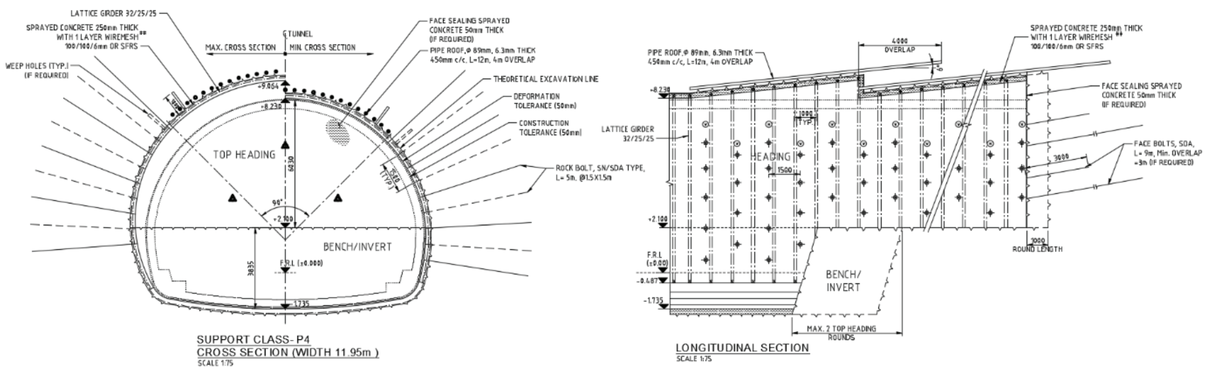
Figure 6: Pipe Roofing in Progress

4.3 Primary Support System

Depending on the rock mass condition, the following support systems have been adopted:

- **Sealing Shotcrete** (50 to 100mm thick)
- **Lattice Girders**: Spaced at 0.5–1.0 m
- **Wiremesh** – Single / double layer
- **Shotcrete**: 250–275 mm thick
- **Rock Bolts**: 5–6 m length, staggered 1-1.5m c/c
- **Temporary invert** – Closed in very poor rock mass

The primary support system implemented in Poor (Class IV) to Very Poor (Class V) rock mass conditions has been designed in accordance with the NATM design philosophy and site-specific geotechnical requirements. The adopted support configuration is illustrated in Figure 7 for a clearer understanding of the applied support methodology and its field execution in weak ground conditions.



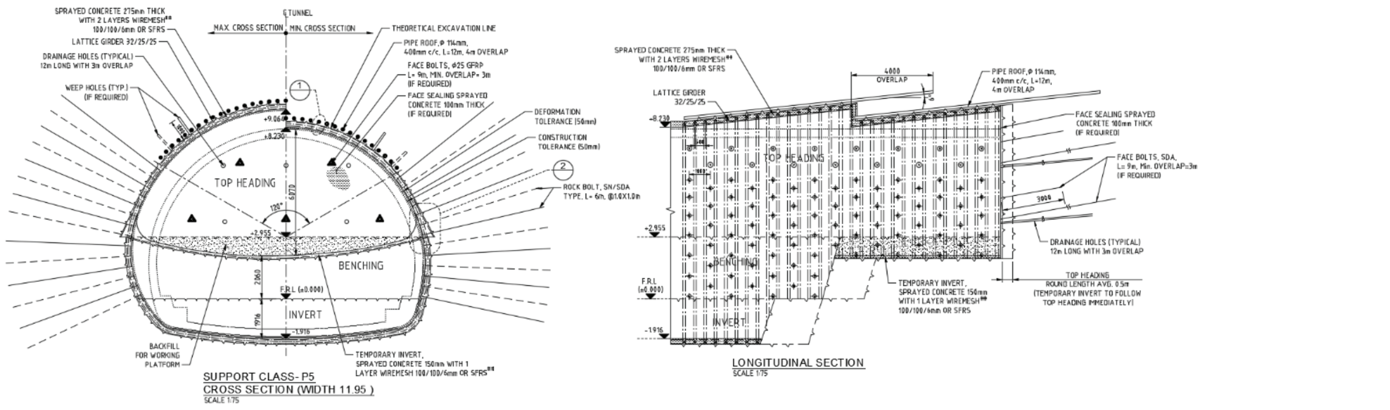


Figure 7: Primary Support System

5. TUNNEL INSTRUMENTATION AND MONITORING

Instrumentation and monitoring were implemented in accordance with the observational principles of the New Austrian Tunnelling Method (NATM) to assess ground response and support performance during excavation. The monitoring system primarily included Bi-Reflex Targets (BRTs) installed at specified intervals based on the prevailing rock mass conditions. These optical targets facilitated precise measurement of surface deformations at the tunnel crown and walls, providing reliable data on convergence and ground behaviour.

Based on BRT observations and site-specific geological conditions, real-time monitoring sensors were deployed in unstable or critical reaches to enhance risk assessment and response. The Real-Time Monitoring System (RTMS) comprised an integrated network of load cells, multipoint borehole extensometers (MPBX), pressure cells, and strain meters, connected through sensor nodes and gateways to ensure continuous data transmission and analysis (refer Figure 8). These instruments recorded critical parameters such as load distribution, rock deformation, stress, strain, and angular displacement, enabling real-time evaluation of tunnel stability.

A structured “Triple A” protocol (Alert–Action–Alarm) was followed for monitoring response, ensuring timely intervention based on threshold exceedances. Recorded data were reviewed and interpreted daily to facilitate adaptive modification of the support system in accordance with observed ground performance.

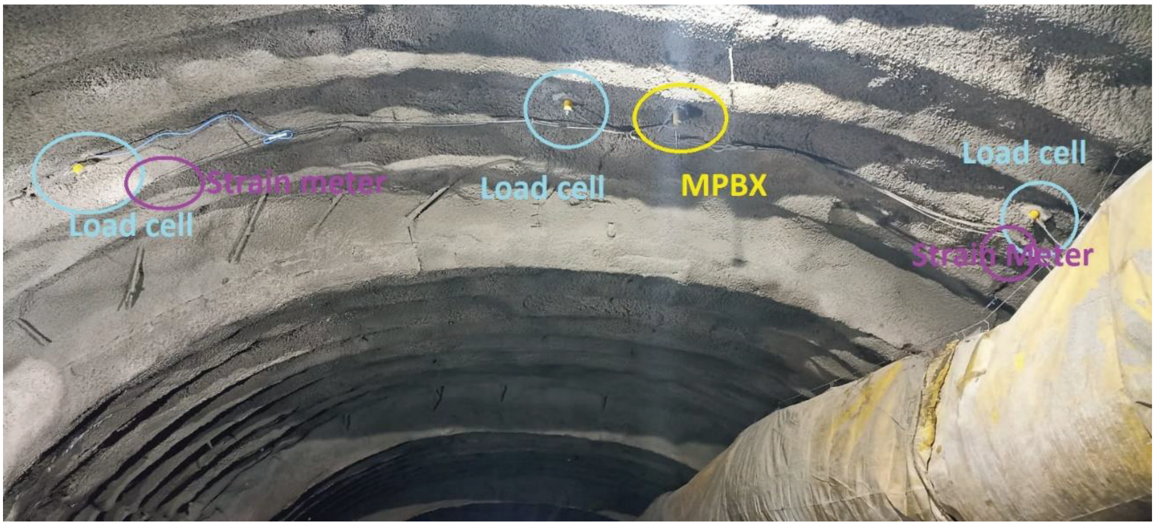


Figure 8: Real Time Monitoring Sensor

6. GROUND BEHAVIOUR AND MITIGATION MEASURES

Ground response varied considerably across tunnel reaches. To address the aforementioned geotechnical challenges, a combination of preventive, adaptive, and observational measures consistent with the principles of the New Austrian Tunnelling Method (NATM) was implemented throughout the excavation process. Depending on the rock mass condition pre support measures as described earlier were carried out in form of systematic pipe roofing and fibreglass dowels to ensure face stability and minimize overbreak. Zones affected by excessive groundwater ingress were treated with pre-excavation drainage holes, effectively reducing water pressure and preventing deterioration of the shotcrete lining.

To manage rapid lithological transitions, real-time geological face mapping and classification updates were conducted, allowing immediate adjustments to the support category and excavation sequence. The deformability of support elements, including shotcrete and Lattice Girders, was closely monitored through instrumentation and convergence measurements, enabling timely strengthening or relaxation of support as required.

Overall, the integration of geotechnical monitoring, observational design feedback, and close coordination between design and site teams ensured effective mitigation of adverse ground conditions, maintaining both construction safety and tunnel stability under the complex geological regime of the Himalayan terrain.

7. COORDINATION AND RISK MANAGEMENT

A strong feedback loop between design and field teams was crucial. A daily reporting system ensured that face conditions, support requirements, and deformation trends were continuously reviewed. Design changes were made proactively to respond to observed behaviour.

A dedicated risk management protocol addressed:

- Emergency Face Collapse Response
- Unforeseen Water Inflows
- Instrumentation Failure

8. LESSONS LEARNED

- Geotechnical adaptability is essential in Himalayan tunnelling; pre-defined designs often need on-site modifications.
- Enhanced Support density and advance rate reduction helped a lot in tackling problematic reaches
- Real-time monitoring and quick interpretation ensure stability and prevent failures.
- Face control using pipe roofing, face bolting and slow advance rates significantly reduces collapse risks.
- Continuous coordination between engineers, geologists, and contractors enables timely decision-making.

9. CONCLUSION

The successful implementation of NATM in the Shimla Bypass Tunnel – Package 1 highlights its effectiveness in managing complex and variable Himalayan geology. Through an integrated approach involving detailed ground investigation, flexible support systems, and continuous monitoring, the project team navigated major geotechnical hazards. This case study serves as a valuable reference for future tunnelling projects in similar geo-tectonic settings and underscores the importance of observational methods, risk awareness, and field-driven design evolution.

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RELIABILITY-BASED DESIGN OF TUNNELS : FUNDAMENTALS TO EMERGING METHODOLOGIES

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ABSTRACT

The design and analysis of structures, especially tunnels, necessitates detailed geotechnical investigations. Geotechnical parameters, such as cohesion, friction angle, and unit weight, are always associated with a certain degree of inherent uncertainty. These uncertainties are often ignored in the deterministic analysis of tunnels, which remains standard practice, leading to either uneconomical or unsafe designs. Probabilistic frameworks such as first order reliability method, second order reliability method, and inverse reliability methods address these challenges and allow engineers to quantify the reliability index. However, these methods do not explicitly model the spatial variation of soil properties. To overcome this limitation, random field-based approaches are used, providing a more realistic assessment of tunnel performance. This paper presents a comprehensive overview of the significance of reliability- based analysis and design of tunnels, along with major tools adopted for the reliability analysis of tunnels with and without spatial variation of soil properties. The paper also discusses the available codal provisions and recommendations guiding the reliability analysis of tunnels and the scope for future work are discussed, emphasising the need for a risk-informed approach to tunnel design and analysis.

Keywords: Tunnel, Geotechnical Uncertainties, Ground Heterogeneity, Reliability-Based Design

1.0 INTRODUCTION

Rapid urbanisation and space constraints have prompted increased utilisation of underground space through tunnelling for subways, roads, railways, and utilities. Ensuring the stability of such tunnels is of critical concern in geotechnical engineering. The inherent uncertainty associated with soil or rock properties poses significant challenges concerning stability and requires careful assessment of their behaviour. Traditionally, tunnel safety and performance have relied on deterministic methods [1-5] to ensure stability, primarily using safety factors. These methods involve assigning single, fixed values to parameters, which are often based on judgment or conservative assumptions and thus do not explicitly account for uncertainty. To provide a more realistic assessment of tunnel performance and reliability, probabilistic methods are used [6-8].

This paper provides a comprehensive overview of the different probabilistic methodologies that govern the reliability-based analysis and design of tunnels. Two primary approaches are highlighted. The first method is the random variable method (RVM), which considers parameters as random variables, each described by a specific probability distribution, while disregarding spatial variability. It quantifies the probability of failure using most probable point-based methods and sampling-based methods. Sampling-based approaches include studies that integrate Monte Carlo simulations or Latin hypercube sampling [7, 9]. Most probable point methods include the first-order reliability method (FORM), second-order reliability method (SORM), first order second moment method (FOSM), inverse FORM and others. Sampling-based methods demand numerous simulations, while point-based methods require gradient evaluations of numerical performance functions, making both computationally intensive. The Response surface method (RSM) helps to reduce this burden by approximating the performance function and enables efficient reliability analysis with fewer full model evaluations [8, 10-16]. The second approach, advancing upon the RVM, explicitly handles spatial variability through random field modelling (RFM) [17]. It describes the property distribution throughout the soil mass by a spatial correlation structure [18-22]. This approach is commonly implemented in frameworks such as random finite element method (RFEM) [23-24], random finite difference method (RFDM) [7, 25] and random finite element limit analysis (RFELA) [18, 26].

This paper presents relevant international standards and codal provisions, emphasising the need to achieve an appropriate balance between human safety, economic cost, and acceptable risk [27-29]. A sample tunnel stability problem is also shown to highlight the difference between the RVM and RFM methods. This paper lays the foundation for practising engineers and researchers, highlighting the significance of reliability-based analysis and design of tunnels. It provides a thorough overview of the fundamental knowledge in the area, highlights the scope for future work and the challenges in the implementation.

2.0 FUNDAMENTALS OF RELIABILITY-BASED DESIGN

The process of soil formation is associated with several physicochemical and environmental processes, such as weathering [30]. The combined action of these processes makes the soil non-homogeneous in nature, which in turn causes uncertainties in geotechnical parameters, namely cohesion (c), friction angle (ϕ) unit weight (γ) etc. Moreover, three major causes of uncertainty in soil parameters are inherent variability (due to natural environmental, geological, and physiochemical parameters), measurement errors (caused by errors in measuring instruments), transformation uncertainty (error due to the use of empirical correlations) [31]. Due to the combined action of all three causes, no soil parameters are free from uncertainty. In conventional deterministic analysis, this associated non-homogeneity is not considered, which may lead to over-predicting the stability of the tunnel [15]. Therefore, for the correct quantification of tunnel stability, it is necessary to incorporate the associated uncertainty with the soil. Uncertainties are quantified using the mean (μ) and the coefficient of variation (COV) of the geotechnical parameter. COV is expressed as the ratio of the standard deviation to the mean. The effect of uncertainty in the analysis is mathematically quantified by determining the failure probability (P_f) and reliability index (β) of the geotechnical structure. P_f represents the probability that the structure will not perform its indented function for its design period.

Conventional methods adopted for the determination of P_f and β include [32]:

- First order reliability method (FORM)
- Second-order reliability method (SORM)
- Monte Carlo simulation method.

One of the widely used expressions to evaluate the β based on FORM was given by Hasofer and Lind [33]:

$$\beta = \min\sqrt{(x_i - \mu_i)' C^{-1} (x_i - \mu_i)} \tag{1}$$

$$P_f = \phi(-\beta) = 1 - \phi(-\beta) \tag{2}$$

x_i is considered a random variable, μ_i is the mean of the corresponding i^{th} random variable, C is the covariance matrix and ϕ(.) is the standard normal distribution function. The probabilistic analysis conducted considering μ and COV is referred to as the random variable method of analysis.

RVMs provide a quick estimation of P_f and β, dealing with uncertainty. One of the major drawbacks of such methods is, they do not incorporate the effect of spatial variability of soils. The effect of spatial variability is incorporated using μ, COV and an autocorrelation function, characterized by correlation length (CL). The correlation length represents the distance over which soil parameters are correlated. It is expressed in both horizontal and vertical directions (CL_x and CL_y). If CL_x and CL_y are equal, it is known as an isotropic correlation length; otherwise, it is an anisotropic correlation length. Reliability studies conducted to consider the spatial variability of geotechnical properties are referred to as the random field method of analysis. Such studies involve generating the realisations of random fields of geotechnical parameters. As geotechnical parameters have non-negative values, therefore majorly log-normal distribution is adopted to model the spatial variation of parameter [32]. Covariance matrix method [34], Spectral representation method [35], KL expansion method [36], Modified linear estimation method [37] etc, are widely adopted methods for generating random field.

3.0 CODAL PROVISIONS AND GUIDELINES

Uncertainties in material properties, geometry, environmental actions, and modelling assumptions are inherent in geotechnical problems and provide risk-based, reliability-based, and semi-probabilistic (partial factor) approaches to ensure consistent safety levels across different types of structures [38]. The European Commission [38] emphasizes reliability verification of structures in three systematic steps: (i) defining the limit state function and identifying the uncertain parameters in the ground model, (ii) assessing the uncertainties linked to geotechnical properties, loads, and pore pressures, (iii) choosing an appropriate reliability method and interpreting results to calculate system reliability, β and compare it against target reliability index (β_t).

In reliability-based design the engineer first decides on an acceptable β_t value. Eurocode 2 Commentary [39] recommended β_t values for ultimate limit states as 3.3, 3.8 and 4.3 for a 50-year design life and 4.2, 4.7 and 5.2 for 1-year design life corresponding to three reliability class (RC1 to RC3). The classes roughly correspond to low, moderate and serious consequence cases (CC1, CC2, CC3). Bjureland [40] highlights that rock tunnels typically fall under RC3 and CC3 category. In the partial factor approach, partial safety factors are applied to both resistances and actions to ensure that the limit-state equations are satisfied. These factors are calibrated so that the resulting design decisions for different structures achieve the desired target reliability. Hence, β_t is implicitly determined through the chosen set of partial factors or the corresponding consequence classes CC1 to CC3. For example, the partial factors in EN 1990 [41] are calibrated so that they will lead the structures to fall in RC2 class, which roughly achieves β_t greater than 3.8 for a 50-year reference period. For serviceability limit states (SLS), there is no formal minimum reliability specification. The target probability of failure ($P_{f,t}$) for SLS typically ranges between 1–5% [38].

In risk-based analysis however the β_t is not prescribed a priori but effectively set by the overall risk tolerance based on individual risk, cost–benefit analyses, incidents with large-scale fatalities, and the life quality index rather than a single code value [38, 42].

Beyond the Eurocode framework, several other international standards provide similar safety philosophies. The Joint Committee on Structural Safety (JCSS) [43] recommends β_t values that reflect balance between the relative cost of implementing safety measures and the expected failure consequences (Table 2). The Committee also provides a cost ratio parameter (C_r), defined as total construction and direct failure costs relative to construction costs to classify consequences. C_r evaluates failure severity based on risk to life and economic impact, increasing from small to high with failure consequence level. The committee also provides guidance on irreversible serviceability limit states as given in Table 2, allowing a variation of approximately 0.3.

\Table 1 β_t for ultimate limit states with a one-year reference period [43].

Relative Cost of Safety Measures	Minor failure consequences $C_r \leq 2$	Moderate failure consequences $2 < C_r \leq 5$	Large failure consequences $5 < C_r \leq 10$
Large	3.1	3.3	3.7
Normal	3.7	4.2	4.4
Small	4.2	4.4	4.7

*For $C_r \geq 10$, consequences are extreme; a full cost-benefit analysis is essential and may not warrant construction at all.

Table 2 Tentative β_t and associated target failure probability $P_{f,t}$ corresponding to one year reference period and irreversible serviceability limit state [43].

Relative cost of safety measure	β	P_f (approximately)
High	1.3	10^{-1}
Normal	1.7	5×10^{-2}
Low	2.3	10^{-2}

USACE [6] guidelines recommend β_t and $P_{f,t}$ specifying performance levels as given in Fig.1. In this context, for example, if tunnel face collapse is treated as the limit state and $P_{f,t}$ is 0.006 (corresponding to β_t of 2.5 in Fig.1), it means that about 6 collapses can be expected in 1,000 analysed scenarios and the tunnel will be expected to have a below average performance level.

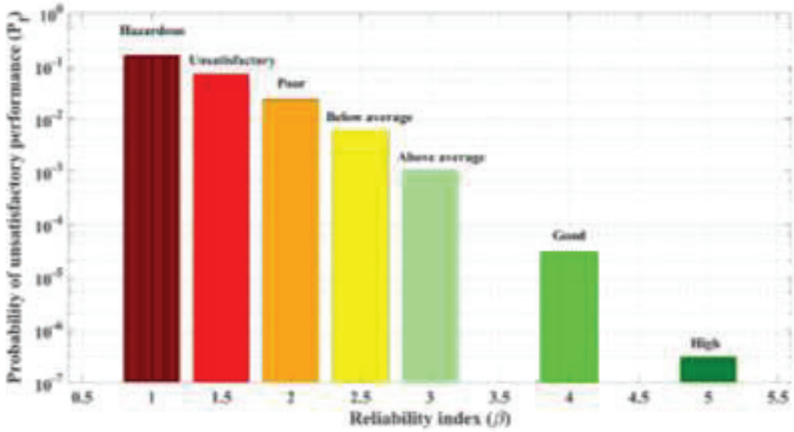

Fig. 1 Expected performance levels based on reliability index and failure probability [6].

ISO 2394 [44] advises that $P_{f,t}$ should consider consequences, economic impacts, and the costs associated with reducing failure risk. Societal effects and environmental concerns shall also play a key role in decision-making. If no lives are at risk, economic factors can guide the decision. However, if human life is involved, decision is influenced by life-saving costs. Acceptable failure probabilities must be based on proven, reliable cases from experience.

ASCE [45] specified lifetime β_t values for structures in the range of 2.5 to 4.5 depends on consequence classes.

It must be noted that the existing provisions in various standards and reports may not all explicitly address tunnels, however the underlying principles of reliability, safety, and economic considerations presented can be effectively applied to tunnel design.

4.0 REPRESENTATIVE TUNNEL STABILITY EXAMPLE: RANDOM VARIABLE VS. RANDOM FIELD METHODS

This paper presents a comparison between the random variable method and the random field method to highlight the influence of spatial variability on tunnel stability. A 6 m diameter tunnel at a depth of 6 m is analysed as a representative case. Geotechnical parameters effective cohesion (c'), effective friction angle (ϕ') and γ are considered as random parameters with mean 10 kPa, 22° and 16 kN/m^3 respectively and $COV_{(c')}$ is considered as 10% and 40% and $COV_{(\phi', \gamma)}$ is considered 10% and 40%. β using RVM is calculated using the expression in equation (1), and the corresponding P_f is then computed using equation (2) in Microsoft Excel. For the RFM analysis, along with mean and COV values adopted in RVM, CL_x and CLy is considered as 1m for isotropic case and, and CL_y/CL_x is kept as 20 for anisotropic case of correlation in a geotechnical software OPTUM G2. Thereafter, the probability of failure considering the spatial variation of soil parameters is calculated by generating random fields and adopting Monte Carlo simulations. The mathematical expression used to evaluate the P_f is shown in equation (3).

$$P_f = P\{(p/c')_{RFM} < (p/c')_{det}\} \qquad (3)$$

Here, $(p/c')_{RFM}$ and $(p/c')_{det}$ are the normalised tunnel support pressure obtained from random field method using Monte carlo simulation and deterministic analysis, respectively. Fig. 2 shows a realisation of random field of c' used in the analysis.

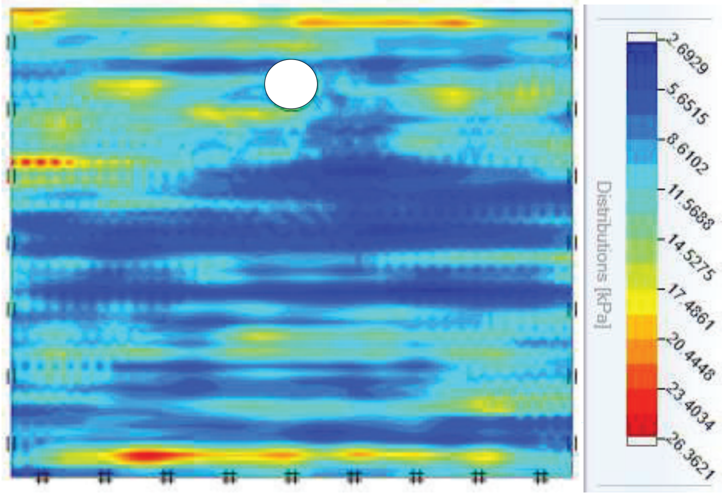

Fig. 2: A typical realisation of random field of c' for COV(c') = 40%, CL_x = 2m and CL_y = 40m.

Table 3 shows the comparison between P_f calculated using RVM and RFM. From the table, it is inferred that P_f calculated from RVM for the majority of cases is higher than RFM. These observations are in good agreement with the study of Cheng et al. [21]. However, P_f for RFM with isotropic CL and COV (c', ϕ', γ) = (40%, 10%, 10%) is higher as compared to RVM. A detailed parametric study is required to further ascertain the influence of correlation length and COV on this deviation and is proposed for future research.

A comparison of computational time required reveals RVM, executed in MS Excel, completes in seconds, while for 1000 Monte Carlo runs using RFELA, about 7 days are required. Therefore, the computational cost of RFM is very high compared to RVM.

Table 3: Comparison between P_f from RVM and RFM.

COV (c', ϕ', γ)	**Correlation length**	***P_f with spatial variation (RFM)**	**P_f without spatial variation (RVM)**
(10%, 10%, 10%)	CL_x = 1 m CL_y = 1 m	0.648	0.846
	CL_x = 2 m CL_y = 40 m	0.563	
(40%, 10%, 10%)	CL_x = 1 m CL_y = 1 m	0.871	0.754
	CL_x = 2 m CL_y = 40 m	0.659	

**P_f is the mean of the upper and lower bound probabilities obtained from RFELA.*

5.0 CHALLENGES AND GAPS IN PRACTICES

Reliability-based design of tunnels suffers from several challenges that limit its widespread application in geotechnical engineering. The initial difficulty lies in the geotechnical investigation phase. Inherently sparse and discrete in-situ data make it difficult to accurately characterise the spatial variability and statistical properties of soil parameters [46, 47]. It is a common scenario in geotechnical projects, where time, budget, and logistical constraints often limit the extent of site investigations. Additionally, the implementation of probabilistic methods, particularly RFM-based approaches such as RFELA, RFEM, and RFDM, demands a large number of simulations. This significantly increases computational cost [8, 48], making it less practical for routine engineering projects. Moreover, limited knowledge and expertise in defining appropriate limit states, selecting suitable probabilistic models and interpreting reliability results for different geological and structural conditions in alignment with codal provisions create a hindrance for practitioners. Also, the availability of user-friendly software tools that support reliability-based geotechnical design is limited. Overall, these challenges contribute to a general reluctance among engineers to adopt reliability-based methods in tunnel design, despite their advantages in risk quantification and decision-making.

6.0 CONCLUSIONS AND FUTURE SCOPE

1. Reliability-based design and analysis of tunnels provide a rational framework to consider the inherent uncertainties in geotechnical parameters and/or loading conditions and helps to quantify the reliability index.

2. MCS, FORM, SORM, and inverse FORM are methods available for the reliability analysis of tunnels that enables the quantification of the reliability index with minimal computational cost and simplicity. However, they typically ignore the spatial variability of soil properties.
3. Advanced probabilistic approaches based on the RFM, including RFEM, RFDM, and RFELA are superior methods for reliability analysis of tunnels that explicitly model spatial variability. However, they remain computationally demanding and lack user-friendly software support.
4. The paper highlights different provisions given by the USACE [6], European Commission [38], Eurocode [39], JCSS [43], ISO 2394 [44], and and ASCE [45] that provides a rational basis for verifying reliability in geotechnical and structural engineering practice. However, guidelines specifically detailed for tunnel reliability analysis remain limited.
5. From the comparison analysis results, it has been found that P_f computed from RVM is higher as compared to RFM. However, computational cost of RVM is much lower than that of RFM.

Future research should include detailed parametric studies to better understand the differences between RVM and RFM methods in tunnel reliability design and analysis. It should also focus on developing efficient and practical tools for reliability analysis that considers spatial variability while reducing computational effort and upgrading the current methods. Bayesian and machine learning approaches should be used to integrate even limited site-specific data to better characterize soil spatial variability. Additionally, design codes should be updated with clearer guidelines and case studies related to tunnels. This will help tunnel engineers to assess and communicate risk more effectively and objectively.

Acknowledgements

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EXCAVATION OF TWIN HYDROPOWER TUNNELS THROUGH TUNNEL BORING MACHINE (TBM) AT HIGHER HIMALAYAS - A CASE STUDY

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ABSTRACT

Tunneling in Higher Himalayas for Hydropower project imparts innumerous extraordinary geological occurrence (EGO) like poor geological media, ingress of ground water, rock burst and squeezing ground condition, rock failures and geothermal conditions. To mitigate such challenges, the twin tunnels of Head Race Tunnel (HRT) separated by the distance of 50m of Pakal Dul Hydro Electric Power Project (1000MW) located in Kishtwar District of J&K, UT of India is being excavated through Double Shield Gripper type TBMs manufactured by M/s Herrenknecht (HK).

The rockmass negotiated through the tunnel is Kibar and Dul formation. Sequence of gneissic granite, gneissic quartzite with bands of schist, quartzite-phyllite with schists, phyllites and slates along with Buzensheru fault. The rockover all along the TBM sections the rock cover varies between 400m to 2000m.

Double Shield Gripper type TBM is suitable for use in all kinds of rock and achieving very high tunneling performances by installing the concrete segmental lining parallel to mining/ excavation are designed together with all required accessories and auxiliary equipment with diameter 8.33m for the boring in the Himalayas to overcome the problems in underground space. At present, approximately 65% of tunneling of HRT of TBM portion has been completed.

This paper outlines the selection of TBM Tunneling in Himalayan region with special features/ facilities by installing the concrete segmental lining parallel to mining/excavation by overcoming various extraordinary geological occurrence (EGO) and successful twin tunneling of HRT and expected to be completed without time and cost overrun.

INTRODUCTION

Pakal Dul (Drangdhuran) Hydroelectric project (4 x 250) is located in Kishtwar District of Jammu and Kashmir, UT of India. It is a storage scheme. The Gross storage of the reservoir is 125.4 Mcum.

A maximum gross head of 417 m between the dam site at Drangdhuran and power house site at Dul is to be utilized for power generation. The power house will have an installed capacity of 1000 MW.

The length of the Head Race Tunnels from power intakes to surge shaft is approximately 10 km (each). While the upstream approximately 2 km length of head race tunnel is proposed to be excavated by conventional Drill & Blast method and the downstream 7.7 km length is proposed to be excavated by deploying two nos. new independent TBM's. For the Drill and Blast reach, the size of the HRT has been kept as 7.20 m finished diameter horseshoe shaped while the size for the TBM reach shall be 7.20 m finished circular. The tunnel shall be concrete lined in its entire length.

Tunneling in Higher Himalayas for Hydropower project imparts innumerous extraordinary geological occurrence (EGO) like poor geological media, ingress of ground water, rock burst and squeezing ground condition, rock failures and geothermal conditions which delay the completion of Hydro Power Project and further leads to cost overrun. To mitigate such challenges and complete the tunneling work as per stipulate time period, tunneling by the TBM construction method is widely used in the construction of long tunnels because of its advantages of high advance rates due to continuous tunneling operation and high level of work safety for the machine personnel particularly in geological fault zones as compared to conventional methods like drill and blast. Double Shield Gripper type TBM is suitable for use in all kinds of rock and achieving very high tunneling performances by installing the concrete segmental lining parallel to mining/excavation are designed together with all required accessories and auxiliary equipment with diameter 8.33m for the boring in the Himalayas to overcome the problems in underground space.

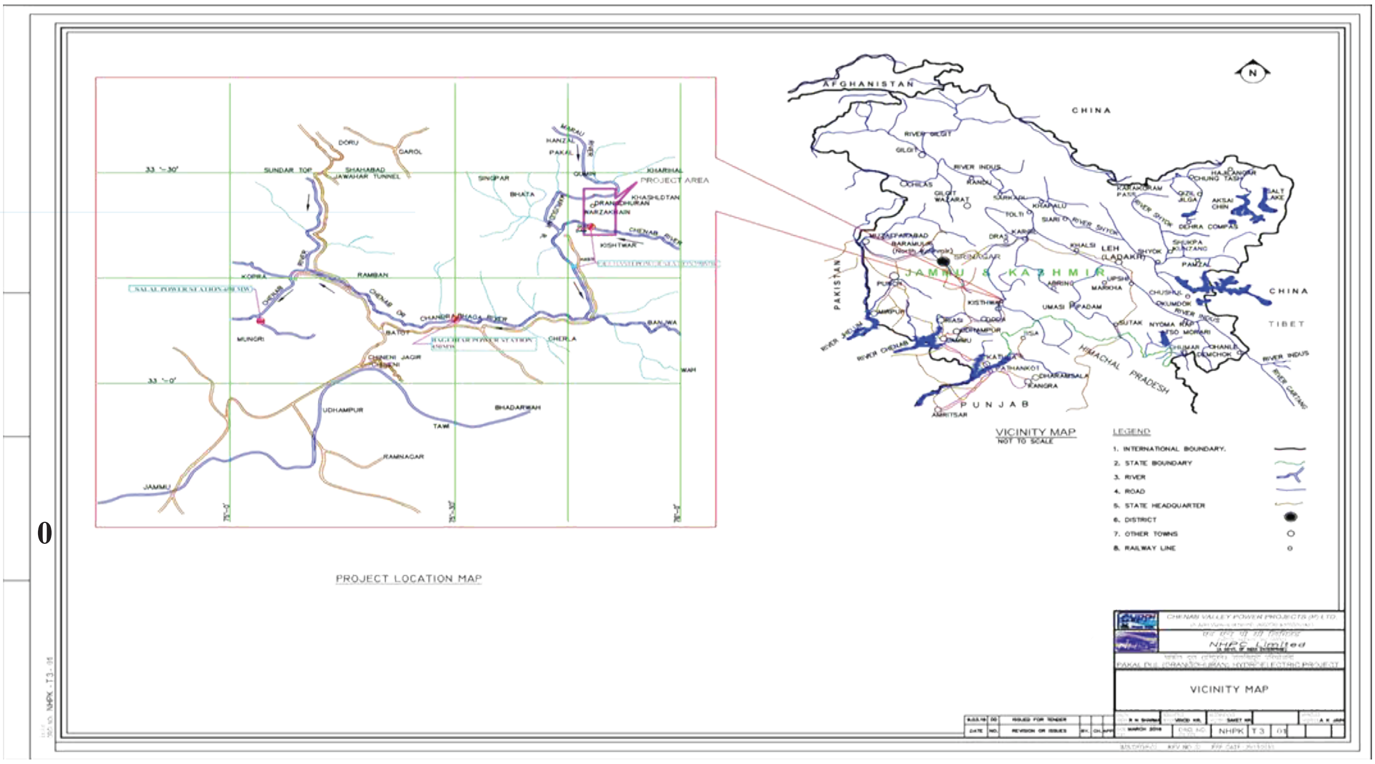


Figure-1 (Project Location Map)

GENERAL DESCRIPTION OF THE PROJECT

Pakal Dul Hydroelectric Project (4 x 250 = 1,000 MW) is a storage scheme in the Kishtwar District of Jammu and Kashmir, State of India. The gross storage of the reservoir is 125.40 MCM.

A maximum gross head of 417m between the dam site at Drangdhuran and powerhouse site at Dul is to be utilized for power generation. The powerhouse will have an installed capacity of 1,000 MW.

The project comprises the following main components:

167 m high concrete face rockfill dam;

A chute spillway on the left bank;

2 x two tunnel spillways 10.5 m diameter, 415 m and 435 m long respectively:

A diversion tunnel 11.0 m diameter, 800 m long:

The intake structure for the head race tunnel located just upstream of spillway on the left bank;

2 x headrace tunnels (HRT) 7.20 m internal diameter, each 10.0 km in length; an additional intake tunnel of 7.20 m diameter up to the gate shaft for 500 MW capacity addition in future;

2 x surge shafts 16 m diameter, each 200 m deep;

2 x steel lined pressure shafts 6.0 m diameter, each bifurcating at the top into two steel lined shafts of 3.9 m diameter,

Underground Power House

Tailrace system comprising of 4 x concrete lined horseshoe shaped tunnels, each 5.5 m diameter, 125 m long.

A surface Switchyard.

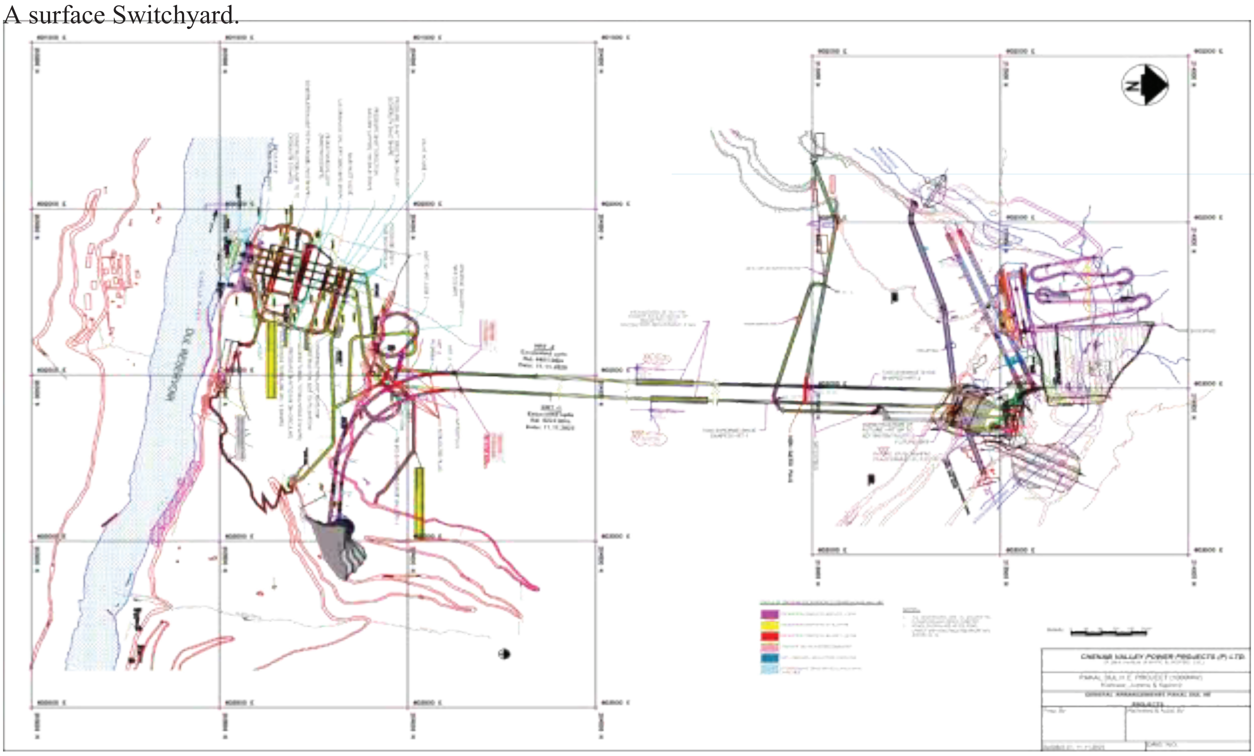


Figure 2-(Project Layout Plan)

BRIEF GEOLOGY OF THE PROJECT/HRT AREA

Pakal Dul project is located in the Kishtwar district of Jammu & Kashmir,UT of India. This region is located south-east of Kashmir valley and lies between Pir Panjal and Zanskar Range of Greater Himalaya. The different mountain ranges are dissected by major drainage system of Chenab and Marusudar Rivers. The Marusudar is the right bank tributary of Chenab River and joins it at Bhandarkot. Geologically the area falls within the "Kishtwar Window Zone" and exposes rocks of Salkhalas, Kibar & Dul Formations belonging to Precambrian age. Kibar and Dul Formations are considered younger to Salkhalas. The Salkhalas are separated from Dul formation by a regional fault/thrust known as Kishtwar thrust/Fault.

The rocks of Kishtwar area have litho-stratigraphically been classified into two groups viz. Older Kishtwar group and Younger Sinthan group, corresponding to the metamorphic rocks and younger sedimentary and volcanic rocks respectively.

The contact of these two groups is marked by a thrust designated as Chhattru Thrust, which has a sinusoidal alignment. This has been traced from N-S and NESW band near Chhattru. Other regional features viz. Kishtwar Fault/Thrust (runs almost N-S) and Daddhar Buzensheru Fault within Kishtwar Group have been mapped by various workers in Chenab and Marusudar basins. The project area falls within Kishtwar group.

The tunnel is anticipated to pass through various litho units comprising of gneissic granite/gneissic quartzite with schist bands of Kibar formation and phyllites, schists, slates with subordinate bands of quartzite (Lopara formation)

& Quartzite-Phyllite sequence of Dul formation with varying compressive strengths (20MPa – 250MPa). However, uniaxial compressive strength (dry) of rock material ranges from 35 Mpa (Phyllite) to 180 Mpa (Quartz-mica schist). The test results also indicate that the rock along tunnel alignment is mostly very abrasive (CAI ≥4), therefore, this condition shall be considered while designing the TBM and its cutters and any modifications if required, shall be incorporated

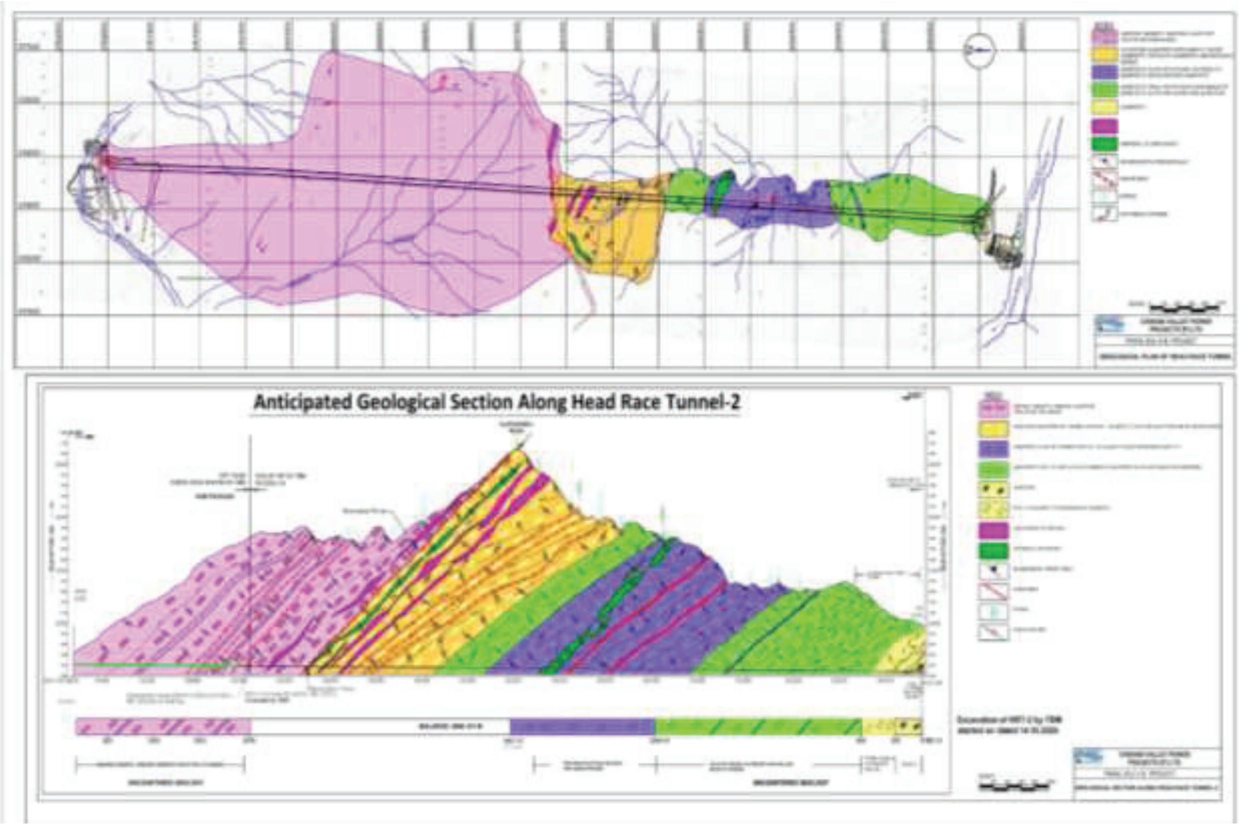


Figure 3-(Geological Section along Head Race Tunnel)

EVALUATION OF TBM DEPLOYMENT

The TBM should be capable of boring through heterogeneous rock mass with varying strengths and frequent occurrences of fractured, sheared, folded and faulted zones. It should also be capable to tackle the various nature of geo-hazards likely to occur during tunneling as under:-

a) Rock bursting/Spalling & Squeezing:

The general cover over the tunnel is expected to be ±400 to ±2000m, so the tunnel may experience incidence of rock stress problems such as rock bursting, spalling and popping in hard rocks viz. Gneissic granite/Gneissic Quartzite (Kibar form.) & Quartzite (Dul form.) and moderate to heavy squeezing conditions (convergence) in relatively softer rocks like phyllite, schist etc. The TBM should have capability to drive through such frequently varying hard and soft formation under high cover zones. Facilities for over boring may also be required under squeezing conditions.

b) Water inflow:

The superincumbent cover above the tunnel varies from ±400m to 2000m and as this area receives heavy rainfall & snow cover at higher elevations, heavy ingress of water is anticipated during tunneling. Accordingly, appropriate arrangements to tackle such eventualities should be made available.

c) Overbreaks:

Heavy overbreaks and cavity formations can occur in poor to very poor rock condition in Schist, Phyllite and slate units particularly in proximity of fault and intense folding, where rocks are supposed to be extremely closely jointed. This geo-hazard is particularly significant in the Daddhar- Buzensheru fault crossing area. Overbreaks owing to Slabbing along low dipping foliation planes in quartzitic slates/phyllites/schists is also anticipated.

d) High Geo-thermal gradient:

The possibility of encountering high geothermal gradient during tunneling under high cover zones may pose difficult tunneling conditions. Facilities for tackling such possible adverse conditions should be available at site.

Keeping in view of above discussed uncertain difficult geological conditions, the TBM, should be capable of boring through different litho-units having varying degree of competency from hard to soft rocks. It should have advance probing facilities including facility to probe all along the periphery of Tunnel face. Equipment should be capable of drilling holes approximately 60m ahead of TBM cutter face. Under special requirement, even core drilling machine, capable of drilling horizontal or sub horizontal holes up to 80-100m lengths should be readily available at site to undertake such drilling. TBM should be more useful to tackle varying degree of tunneling media in Himalayan condition which are known for heavy over breaks, cavity formation, debris flow condition and water seepage problems.

To treat water bearing or highly fractured zones ahead of cutter head, TBM should have facility to drill holes around the circumference of the tunnel face parallel to tunnel axis for pre-injection of grout, pre-drainage etc. as well as arrangement for shotcreting be kept. As some of the difficult zones may contain water charged fine particles in shear zone material, grouting with cement / micro fine cement/ ultrafine cement, silica fume, and chemical grout/foam may be required.

Facilities for pipe roofing in very poor rock and forepoling, or any other state-of-the art support system should be kept. TSP (Tunnel Seismic Prediction), a geophysical method may also be very useful to assess the geological conditions in critical zones.

BASIS OF SELECTION OF TBM

The TBM will be designed to perform under the following modes:

- Stable condition operating mode: The TBM will bore in Double Shield mode with the grippers extended with anchored ring to the rock and the segment erection is in parallel and to keep in place the last assembled ring during advancing of the rear shield (re-gripping). The advance rate is expected to be maximum in this mode of operation.
- Unstable rock operating mode: The TBM will operate without activating the grippers and without opening the telescopic joint. The segment erection will be done alternately and thus the advance rate will be limited.

The successful completion of the Kishanganga project also demonstrated that Double Shield TBM can be effective with proper planning in high overburden, high in-situ stresses, mixed geology, and water-charged formations in higher Himalaya. The experience of successful completion of HRT by DST in Kishanganga project helps in adoption of same mode of TBM for Pakal Dul HE Project as the site conditions are similar in nature.

ADVANTAGES OF DOUBLE SHIELD TBM

- A double shield TBM is generally considered to be the fastest machine for hard rock tunnels under the favorable geological conditions, the combination of methods allows for the installation of lining concrete segments parallel to Excavation/Boring. This type of TBM consists of a rotating cutter head, front shields, a telescopic shield (an inner shield that slides within the large outer shield), and a gripper shield together with a tail shield.
- While boring, gripper shoes radially press against the surrounding rock to hold the machine in place and take some of the load from the thrust cylinders. At the same time with TBM advancing, thrust cylinder is always protected by the telescopic shield.
- During TBM advancing, behind the gripper, installation of ring segment carried out by segment erector and auxiliary thrust cylinder. Because regripping is a fast process, double shield TBMs can almost continuously constructing the tunnel.

- In case TBM through the weak zone or fault, then main thrust cylinder retracted, and gripper cylinder also retracted keeping the gripper pad same level with front shield surface, and TBM advance using the auxiliary cylinder pushing the lining concrete segment behind, with this work mode TBM working like EPB machine and could not perform boring and erecting lining segment simultaneously.

Additional features designed specifically for excavation of twin HRT through TBM to face geological occurrences and monitoring geological condition during construction phase are as under:

- Equipped by Main Thrust and Auxiliary cylinders which allow to move each shield at any time and have more thrust available per shield surface.
- TBM conical shields design is to prevent shield from getting stuck under squeezing condition.
- Overcut boring of 160-180mm diametric through 8 shimming cutters + additional 2 overcut gauge cutters housing in Cutter head to compensate squeezing ground.
- Lubrication ports are installed on the forward and gripper shields to allow bentonite or polymer based lubricants to be injected around the extrados of the TBM.
- Shields are designed to cope with extreme loading in high cover/bursting/spalling ground.
- TBM is equipped with Dewatering Emergency pumps.
- Continuous probing ahead of face of 60-100m length.
- TSP has been adopted to predict geology of 100-150m ahead of face.

TUNNEL EXCAVATION AND INSTALLATION OF SEGMENTS

During excavation the front shield and the cutterhead advance while the gripper shield is stationary, firmly anchored by the gripper pads/shoes to the rock wall/surface. During operation of TBM, the following parameters are to be monitored:

- Rotation speed (rpm)
- Advance speed (mm/min)
- Penetration rate (mm/revolution)
- Thrust force (KN)
- Cutterhead torque (KNM)
- TBM position and the direction of the drive.

Concurrently with the boring stroke, the segments of the concrete lining can be assembled by means of the rotary erector, The other operations/ activities will be also performed simultaneously during the boring phase as under:-

a) Pea-gravel back filling of the annular space between the excavation and the segmental lining will be carried out in order to improve the lateral stability of the lining and also subsequent cement grout backfill.
b) Segment unloading by means of an overhead crane and transfer to the TBM erector

As soon as the boring stroke completed, the regripping phase started. The gripper shield advanced by reversing the main thrust cylinders operation and thrusting the auxiliary thrust cylinders against the installed segment ring. . After completion of the regripping phase, next boring cycle shall be carried out

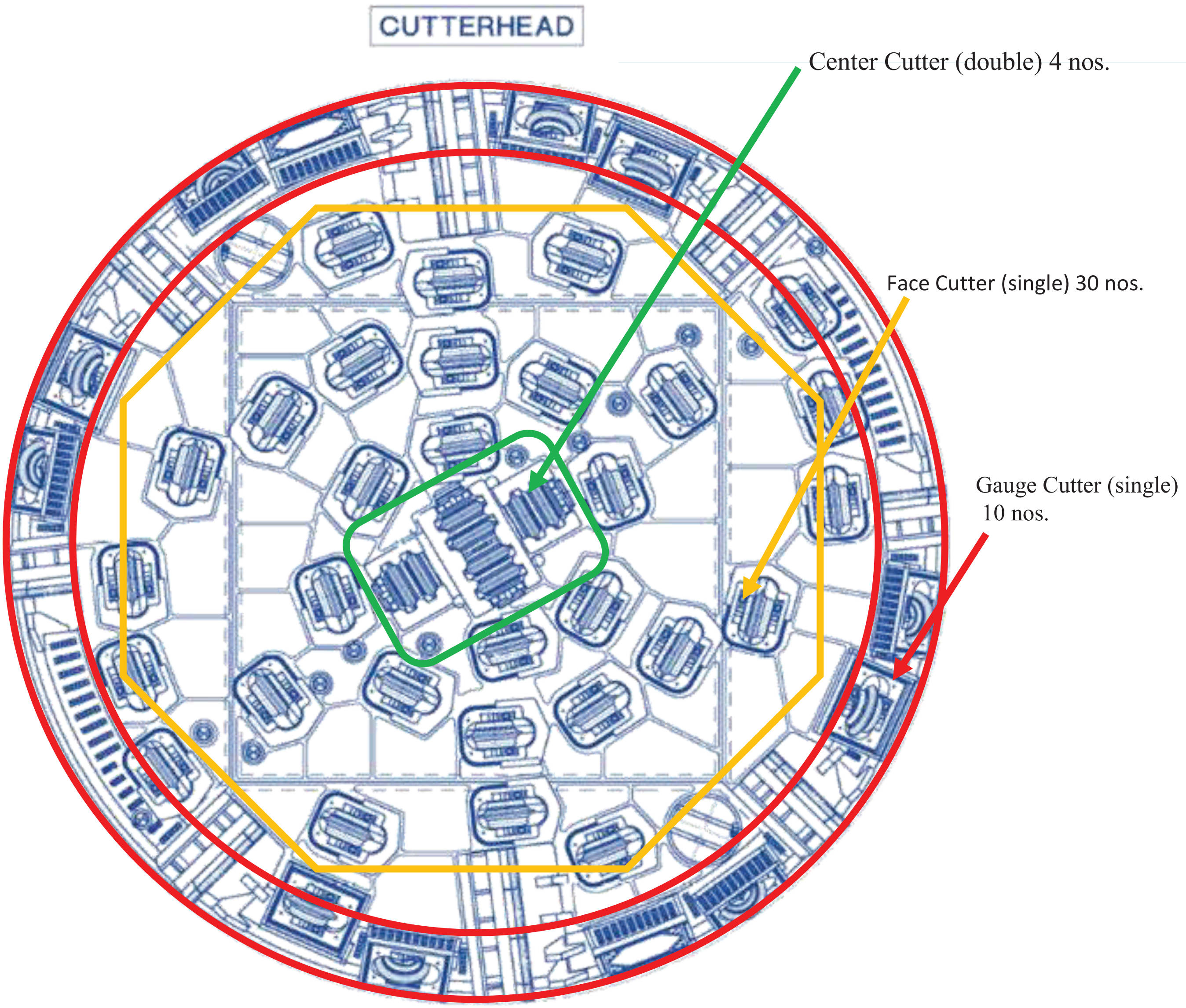
CUTTER RING WEAR AND REPLACEMENT

The maximum cutter wear limit varies depending on the position where the cutter is installing. In general, it divides into 3 cutter positions, i.e.

1. Center cutter, in the form of a double cutter mounted in the middle of the cutter head,
2. Face cutter, a single cutter is installed around the center cutter, and
3. Gauge cutter, is in the form of a single cutter that is installed in the outer circle of the cutter head. And if made a sequence for the cutter wear limit, then the gauge cutter is the cutter of the lowest limit of his wear,

then face cutter and center cutter. Because the gauge cutter is the outermost cutter of the cutter head, in charge for making the perimeter of tunnel, so need to keep a minimum wear, otherwise it will make the tunnel size smaller.

The cutter wear level also depends on each cutter position, distance from the center, more far from the center will take longer traveling in every revolution. Therefore, the gage cutter which is the outermost cutter, usually most often requires replacement because the wear rate is the highest, while the allowable limit is the lowest.



Total 44 Nos cutters, with 48 cutting tracks

Figure-4 (Details of cutter rings)

CUTTER RING CONSUMPTION

Sr,No,	Type of Rocks	Type of cutter used	Cutters Consumption per ring (1.5m)	Remarks
1.	Quartzite	Single disc HD Cutters	0.473767328	316 cutters consumed in 1000m mining
		Double disc HD Cutters	0.030707142	21cutters consumed in 1000m mining
2.	Slaty Quartzite/ Quartzitic Slate and Phyllite	Single disc HD Cutters	0.252470511	168 cutters consumed in 1000m mining
		Double disc HD Cutters	0.005862318	4 cutters consumed in 1000m mining

Double Shield TBM with Special Features to overcome Exceptional Geological Occurrence (EGO) & Other Potential Hazards for PAKAL DUL HEP.

S.No.	EGO Conditions	TBM Details with associated facilities
1.	**Geo-Thermal Condition: - High temperature working condition combined with geo-thermal seepage having temperature of 42°C or more.**	a) Provision of Air chillers in TBM to reduce the ambient temperature in the tunnel. b) Provision of Cold water spray on to the cutter-head along with forced air ventilation during interventions. c) Ice jackets can also be supplied to the Cutter-head crew.
2.	**High Ingress of Water:- Water ingress of 150 litre/sec at one location.**	a) TBM is equipped with Dewatering Emergency pumps Cap. 2x500 cum/hr emergency dewatering pump + 2x50 cum/hr submersible pump=306Lps incl. 1x10 cum cap. Sedimentation/dewatering tank + provision for additional pump as per requirement. The ingress water shall be routed through channels to the portal area. b) Muck entrance Bucket gates Regulation system to deal with the ingress of water/silt while boring the tunnel.
3.	**Rock bursting & Squeezing in high cover zone.**	a) The provision of double shields is to deals with the high cover/bursting/spalling ground. Shields protect the underlying equipment, manpower etc. and also supports the ground. b) Shields are designed to cope with extreme loading in these type of conditions.
4.	**Shear Zone:- Severe fault zone/Squeezing ground conditions**	a) **Smaller length of TBM –** 11.6m, 12.6m (incl. cutter head), Tot. length(with back up)-140m b) **Tapered/Stepped Shields-** Conicity of 330mm for shields w.r.t Cutterhead to aids better forward movement before the ground comes into contact with the trailing sections of the TBM shields. Conical design is to prevent shield from getting trapped in ground. c) **Overcut boring-**160-180mm diametric overboring through 8 shimming cutters + additional 2 overcut gauge cutter housing in Cutterhead to compensate squeezing ground (Max. Overboring dia-8510mm as per latest TBM design) d) **Shield Lubrication Ports –** 12 ports provisioned on the forward of gripper shield to allow bentonite or polymer based lubricants to be injected around the extrados of the TBM, which shall reduce friction between the shields and the converging ground. e) The shields are also designed to withstand heavy loading in Squeezing ground conditions.

		f) **High Thrust and Cutter-head Torque-** In Squeezing ground condition, TBM gripper shoes are retracted, the main cylinders are closed up and the Auxiliary cylinders are utilized to propel the machine forward by thrusting off the segmental lining. g) **Continuous probing ahead of face –** 60-100m length at a stretch with overlapping @ 10m cycles. 360^0 probing facility with permanent drill installed with ring carrier in shield area through 4xinclined ports +6xhorizontal ports in Front shield (latest), 16xinclined ports in Gripper shield (latest), & 4 horizontal ports through Cutterhead. h) **Ground improvement provisions-** 360^0 Drilling, forepoling (SDAs) & Piperoofing system incl. Grouting provision. Blowout preventers at all drill ports against water ingress. i) Regular interval TSP to predict geology 100-150m ahead of face. j) **Atlas Copco1838 Drill unit for drilling & core extraction –** Cap. 60-100m length minimum & Dufor Drill Units. Total 4 no. Drill Units.
S.No.	**Other Potential hazards**	**TBM Details with associated facilities + Mitigation measures**
1.	**Overstressed lining under excess ground pressure/excess squeezing condition:-**	**Mitigation measure-** Adoption of special segment design & Instrument Monitoring of Tunnel convergence in TBM back-up.
2.	**Jamming & Stucking of TBM:-**	a) Foam Injection through 4 ports in Cuter head (1cum Foam tank) b) Lubrication of shield c) Manual intervention in case spalling in front of cutter d) By-pass tunnel in the roof as last resort
3.	**Hazardous gas emission**	**a) TBM is equipped with Gas Detection Monitoring System in its Back-up to detect 5 gases-** 4xCH4, 2xO2, 2xCO, 2xCO2 and 2xH2S detectors with variable warning and cut-out levels. **b)** Provision of sufficient O_2 cylinders c) Bomb explosion type Fire Extinguishers provision & Alarm system for all electrical panel areas and other important areas.

ADVERSE GEOLOGICAL CONDITION ENCOUNTERED AT CHAINAGE 2852 TO 2858m

The misalignment of the front and gripper shields in TBM-1 occurred due to encountering of poor geological occurrence comprising jointed, blocky, and highly fractured rock mass. This led to the collapse above the TBM Gripper and front Shields, creating a cavity above and in front of the cutter head. At the end it causes shield gets struck at crown section from 10 o'clock to 2 o, clock. Also the machine is not getting compact support from the face and this vacant space at the front of cutter head is evident for this which makes the tendency of TBM-1 towards upside, which stopped the excavation for total 105 days.

In order to make the shield free from struck position and resume the excavation following actions and methodology was adopted

- First of all scaling will be done on the top of cutter head crown area and the shortcrete will be done in the crown area.
- Preparation of steel platform in the front of cutterhead towards crown side with roof sheeting
- Forpoling was done above the shield crown for safety 25mm dia 3.0m long SN rock bolt. Forpoling was done in 2 steps by providing overlap of 1.0m. Raisin capsule was used in place of grouting for making bond strength strong.
- Chipping of rock was carried out at the top of front shield along the length of 2.2m (10 o'clock to 2 o'clock) with the help of drilling machine, jackhammer, rock splitter and crackamite.
- Shotcrete the chipped area above front shield.
- Apply bentonite over shield to reduce the friction and removal of steel platform.
- Applied thrust from auxiliary cylinder (Single shield mode), it advanced and resumed normal operations.

TBM PRODUCTION IN EXCAVATED TUNNEL

Currently, more than 72% HRT-1with the help of TBM-1 and more than 62% HRT-2 with the help of TBM-2 have been excavated successfully using Double Shield Gripper TBM. To the end of 15th November, 2025, the TBM-1 and TBM-2 excavated a total length of tunnel of 5263m and 4530 m respectively, with a monthly average production of 350m/month from face. In the month of December, 2024, the maximum boring achieved by a TBM is 628 m per month with maximum production in a day of 49.50 m. This is an example of a great team working with meticulous planning, effective resource management, time cycle implementation and productivity improvement. The twin tunnel excavation will be completed well on schedule

KEY STRATEGIES FOR SUCCESS OF TUNNELLING BY TBM

- Thorough Geological Investigations: Detailed geological investigations are crucial at the planning stage to understand potential ground conditions and select the appropriate TBM.
- Arrangement of geological investigation prior to advancement of TBM:- (Probe Drilling, TSP and Beam), Regular probe drilling and TSP can help in identify and mitigate unexpected ground conditions.
- Risk Assessment and Mitigation: Implementing risk assessment and sharing protocols is vital for managing uncertainties and potential delays.
- Appropriate TBM Selection: Selecting the correct TBM design and concepts for Himalayan conditions is critical, as the industry has developed specialized machines for these challenging environments.
- TBM shall be designed with special features to handle all the geological occurrences and monitoring geological condition during construction phase

CONCLUSION

This paper outlines the selection of Double Shield Gripper type TBM is suitable for successful Tunneling of HRT in higher Himalayan region with special features/facilities by installing the concrete segmental lining parallel to mining/excavation by overcoming various extraordinary geological occurrence (EGO). The successful utilization of TBM in HRT, highlighting the need for advance geological investigation, probe drilling, and risk assessment to overcome geological complexities to ensure the success of TBM in the higher Himalayas.

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GEOPHYSICAL TECHNIQUES FOR SAFE AND SUSTAINABLE TUNNEL CONSTRUCTION

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ABSTRACT

In the pursuit of safe and sustainable underground infrastructure, geophysical investigations play an indispensable role in reducing geological uncertainties and enhancing project outcomes. Conventional tunnel site investigations- relying heavily on boreholes and limited geological mapping- often fall short in detecting hidden subsurface features such as faults, cavities, and water-bearing zones. These limitations can lead to cost overruns, delays, and unsafe construction conditions, particularly in challenging terrains or densely vegetated regions.

This paper advocates for a paradigm shift from traditional approaches to a geophysics-led investigative workflow, beginning at the early planning stage. Through real-world project experiences and cutting-edge methodologies, it showcases how techniques such as Heliborne TEM, Seismic Refraction Tomography, Passive Seismic Tomography, Electrical Resistivity Imaging (ERI), MASW/ ReMi, Sonic Tomography, and Ground Penetrating Radar (GPR) can offer rapid, continuous, and cost-effective subsurface insights. The paper also highlights advanced technologies like BEAM® (Bore-tunnelling Electrical Ahead Monitoring), which enables real-time ground prediction ahead of TBM excavation, ensuring uninterrupted and informed tunnelling.

Emphasizing a collaborative approach involving geologists, engineers, and geophysicists, this presentation makes the case for integrated geophysical-geotechnical models that reduce surprises, optimize borehole placement, and increase safety margins. It includes key recommendations and guidelines to ensure the successful deployment of geophysical techniques throughout the tunnel lifecycle- from alignment planning to construction monitoring.

INTRODUCTION:

Geophysical tests are indirect methods of exploration in which changes in certain physical characteristics such as magnetism, density, electrical resistivity, elasticity, or a combination of these are used as an aid in developing subsurface information. Geophysical methods provide an expeditious and economical means of supplementing information obtained by direct exploratory methods, such as borings, test pits and in situ testing; identifying local anomalies that might not be identified by other methods of exploration; and defining strata boundaries between widely spaced borings for more realistic prediction of subsurface profiles. Typical uses of geophysical tests include determination of the top of bedrock, the rippability of rock, the depth to groundwater, the limits of organic deposits, the presence of voids, the location and depth of utilities, the location and depth of existing foundations, and the location and depth of other obstruction, to note just a few. In addition, geophysical testing can also obtain stiffness and dynamic properties which are required for numerical analysis.

Geophysical testing can be performed on the surface, in boreholes (down or cross hole), or in front of the TBM during construction.

Sub-surface imaging by means of geophysical survey is a powerful tool for site assessment and mapping which historically has been under-utilized world-over. Continuing improvements in survey equipment performance and automation have made large area surveys with a high data sample density possible. Advances in processing and imaging software have made it possible to detect, display, and interpret small geological features with great accuracy.

Some of the unique advantages of geophysical survey:

- Geophysical methods are quick to apply, saving in terms of time and money.
- Light and portable equipment allows access to remotest of sites.
- Provides information on critical geological features like faults/ fractures/ weak zones/ shear zones, not visible from surface information

- Large areas mapped quickly and inexpensively
- Researchers can assess site conditions, and target specific locations for detailed investigations by drilling, while avoiding others.
- Geophysical methods can quickly produce subsurface geology avoiding delays during execution due to meeting the unexpected.
- Shear wave profiles can be quickly obtained for ascertaining liquefaction potential and earthquake response.
- Buried utilities, pipes and cables, can be detected before drilling/ excavation, avoiding damage to utilities and costly accidents.
- Concrete structures can be quickly scanned to ascertain integrity and detect defects like voids, honeycombing etc.

BENEFITS AND LIMITATIONS OF GEOPHYSICS

Geophysical surveys can offer considerable time and financial savings compared with borehole investigations. At an early stage of site investigation it may be beneficial to undertake a reconnaissance geophysical survey to identify areas of the site which should be further investigated using invasive techniques i.e. those where anomalies have been identified. Geophysics has a unique advantage of providing continuous profile of subsurface rather than discreet information as provided by boreholes. This is critical in areas with complex geology and in projects like tunnels, where a small shear zone can lead to major challenges during execution. Geophysical surveys can be used effectively to determine the geological, hydrogeological and geotechnical properties of the ground mass in which the engineering construction is taking place.

Using geophysical techniques to solve engineering problems has sometimes produced disappointing results, particularly when a method, which lacked the precision required in a particular site investigation has been used, or when a method has been specified that is inappropriate for the problem under consideration. In most of the cases these problems can be avoided by taking services of an experienced geophysicists and access to various techniques available. In other cases the geological conditions at the site have been found to be more complex than anticipated at the planning stage of the geophysical survey and hence interpretation of the geophysical data by the geophysicists has not yielded the information expected by the engineer. It is often advisable to undertake a feasibility study at the field site to assess the suitability of the proposed geophysical techniques for the investigation of the geological problem.

Once the geophysical data has been obtained, it is possible to produce a model of the geological structure, which gives a realistic correlation with the data. The best overall model is obtained by using all the available geological information from boreholes and field mapping. Without this input of precise information, which includes knowledge of the fundamental physical properties of the geological material at the site, the model cannot be constrained in practical terms. There needs to be close collaboration between site geologists, engineers and geophysicists in the interpretation of the geophysical data.

PLANNING AND PRE-CONSTRUCTION STAGE

Carefully planned and executed geophysical program can considerably reduce uncertainties associated with geological surprises encountered while executing an underground project. Geological mapping and conventional borehole programs can provide only limited and discrete information along proposed tunnel route. A geophysical program not only provides a much detailed and continuous information of subsurface, but also can be used effectively to plan boreholes at anomalous locations, thus enhancing the accuracy of subsurface investigation while at the same time reducing cost and time involved in obtaining such information.

The range of geophysics that can be used in the domain of underground engineering is very broad:

- gravity method
- magnetic method
- seismic refraction method
- seismic reflection method
- hybrid seismic method

- spectral analysis of surface waves
- multi-channel analysis of surface waves
- continuous surface wave system
- refraction micro-tremor
- borehole seismic method
- vertical seismic profiling
- seismic tomography
- electrical resistivity method
- spontaneous potential method
- induced polarization method
- electrokinetic probing
- ground penetrating radar
- transient electromagnetic method
- VLF method
- magnetotelluric method
- radiometric method
- Airborne/ Heliborne methods

Few of the above listed methods, proven to be extremely effective for tunnel investigations, are discussed hereunder:

Heliborne Time Domain Electro-Magnetic Method (TDEM)

The Electro-Magnetic (EM) method is based on the physical effect of electromagnetic induction where an electrical current is induced in the ground and thus a secondary magnetic field is created. This secondary magnetic field is governed by the electrical resistivity of the ground. EM systems measure the EM time decay or frequency response and the related resistivity distribution is subsequently obtained by inverse modelling. Time-domain systems (TEM) measure an EM step response decaying with time. They are generally well suited for deeper investigations due to the higher transmitter moment. Some TEM systems can provide highly accurate and well calibrated data.

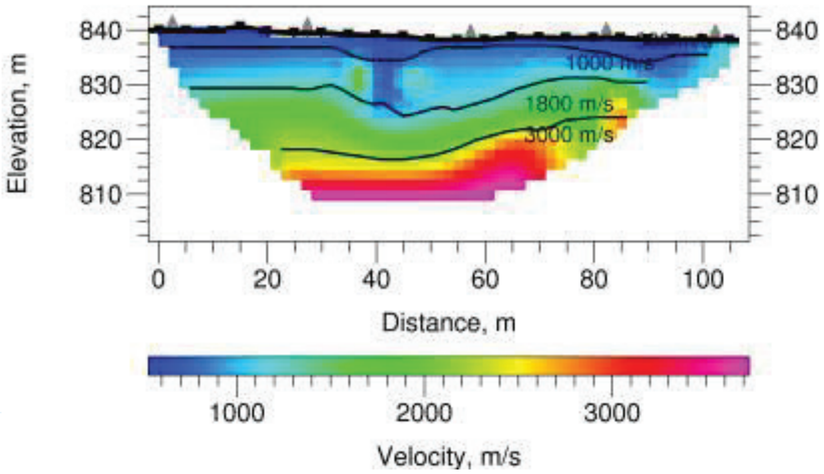
AEM (airborne electro-magnetic) data provides a powerful tool for geotechnical projects due to coverage and survey speed. Significant cost reductions can be achieved by planning geotechnical drillings based on the preliminary geological model derived from EM. Integrated with EM, limited drilling sites can be linked and combined to a model covering the complete area of interest.

Airborne or heliborne EM should be the first ground investigation step. Drilling locations can then be planned efficiently based upon AEM results. Subsequently drilling results should be incorporated in EM data interpretation and visualization leading to a combined geological model (e.g. bedrock topography). AEM is better suited for regional-scale projects rather than isolated projects because costs are relatively high for small surveys.

Deep seismic reflection surveying is the most advanced technique in geophysics today, thanks to its application on a huge scale for oil and gas exploration. This technique does, however, have other applications on a smaller scale, such as for civil engineering project site investigation. The methodology is identical, but the equipment and parameters are adjusted to provide a higher resolution at shallow depths.

In tunnel projects seismic reflection method can detect geologic structures in fault zones, find shallow, soft layers of underground earth materials, reduce mapping uncertainties and can greatly reduce the investigation costs of engineering projects.

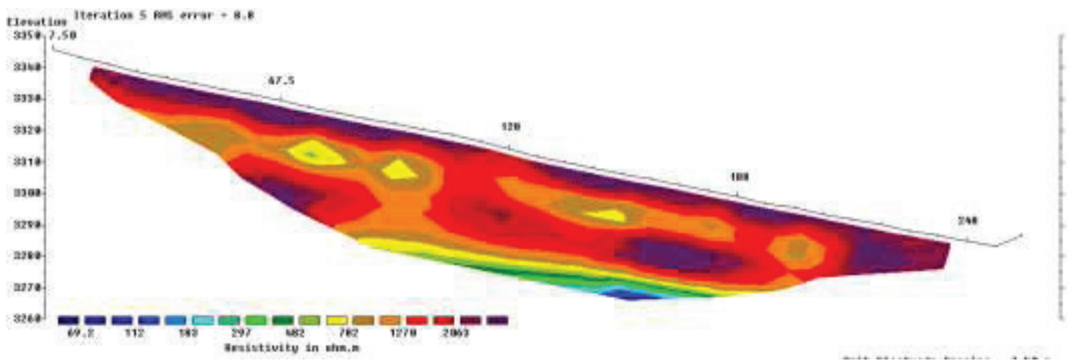
Seismic Refraction survey is an indispensable tool to determine bedrock profile, rock quality and depth, thickness of overburden, fractures and weak zones, topography of ground water etc. The method, like most other geophysical methods, provides continuous profile of subsurface, critical for engineering projects. Coupled with shear wave measurements, it also allows estimation of dynamic elastic moduli like Poisson's Ration, Young's Modulus, and Shear Modulus. Example below shows a 03-layer model obtained from seismic refraction, with last interface of 3000 m/s depicting topography of rock.



Interpreted Seismic Refraction Section Depicting Topography of Rock

Electrical Resistivity Imaging uses an array of electrodes (typically 64) connected by multicore cable to provide a linear depth profile, or pseudosection, of the variation in resistivity both along the survey line and with depth. The technique is extremely useful for investigations of important sites to get information on weak zones or buried channels, under the rock interface, which goes undetected in seismic refraction, which terminated at rock interface. Resistivity imaging can also be effectively used to determine rock profile along dam axis across high current shallow rivers where deployment of hydrophones is not possible restricting use of seismic refraction.

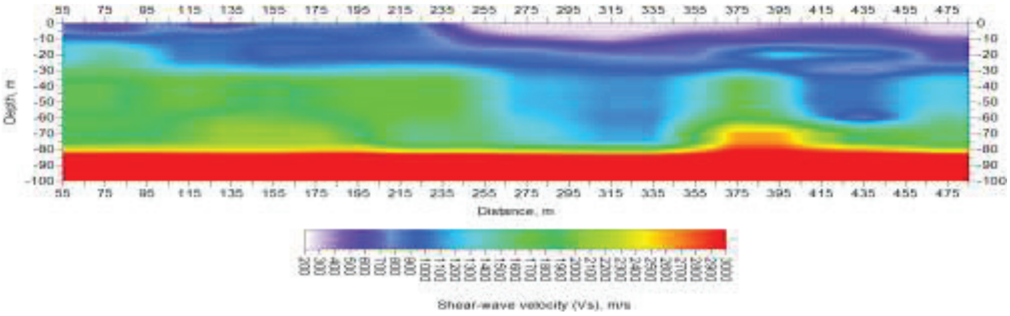
The example below shows use of ERI to detect a soft zone under rock cover, which would otherwise go undetected even by closely spaced boreholes:



Interpreted Electrical Resistivity Imaging Section Showing Soft Zone Under Rock

ReMi (Refraction Microtremor) can be performed under the same layout as used for seismic refraction, to obtain excellent shear wave velocity profiles of subsurface. ReMi is a new, proven seismic method for measuring in-situ shear-wave (S-wave) velocity profiles. It is economic both in terms of cost and time. Testing is performed at the surface using the same conventional seismograph and vertical P-wave geophones used for refraction studies. The seismic source consists of ambient seismic "noise", or micro-tremors, which are constantly being generated by cultural and natural noise. Because conventional seismic equipment is used to record data, and ambient noise is used as a seismic source, the ReMi method is less costly, faster and more convenient than borehole methods and other surface seismic methods, such as SASW and MASW used to determine shear-wave profiles. Depending on the material properties of the subsurface, ReMi can determine shear wave velocities down to a minimum of 40 meters (130 feet) and a maximum of 100 meters (300 feet) depth.

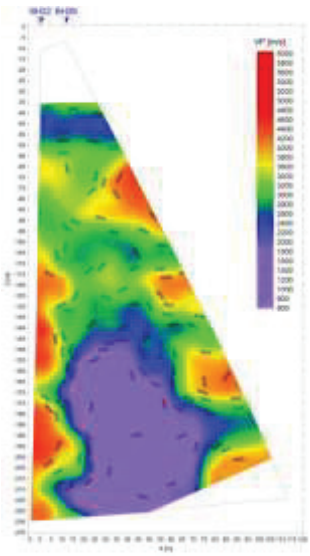
Typical 2D ReMi profile constructed using various 1D profiles is shown hereunder showing shear wave velocities up to a depth of 100m:



Example ReMi Result

Cross hole Seismic Tomography

The latest technique of seismic refraction tomography provides much more realistic and accurate subsurface velocity model compared to typical layered models obtained through conventional seismic refraction surveys. It is based on generation of elastic energy using various sources at predetermined depths in one bore hole and detecting it in another borehole through a chain of hydrophones. Velocity analysis involves estimation of time required to cross the distance between source and receiver depending on variations in elastic properties of material crossed. Deviation survey is carried out prior to tomography for determining alignment of bore holes. The set-up includes a source hole & a receiver hole. A tomographic section generated from survey between two non planar holes for a hydro project for detection of a cavity.



Tomographic section showing cavity

Various other geophysical methods like cross-hole seismic surveys, gravity, magnetic etc., can be applied to obtain critical subsurface information.

CONSTRUCTION STAGE

During construction stage of tunnels and underground projects, geophysical methods can be effectively used to predict unfavorable geological conditions (e.g. Tunnel Seismic Prediction ahead of tunneling) and to check and inspect quality of construction. As an example, a quick GPR run on concrete surface can effectively detect distribution of reinforcement bars, presence of honey combing and other similar defects.

MAINTENANCE STAGE

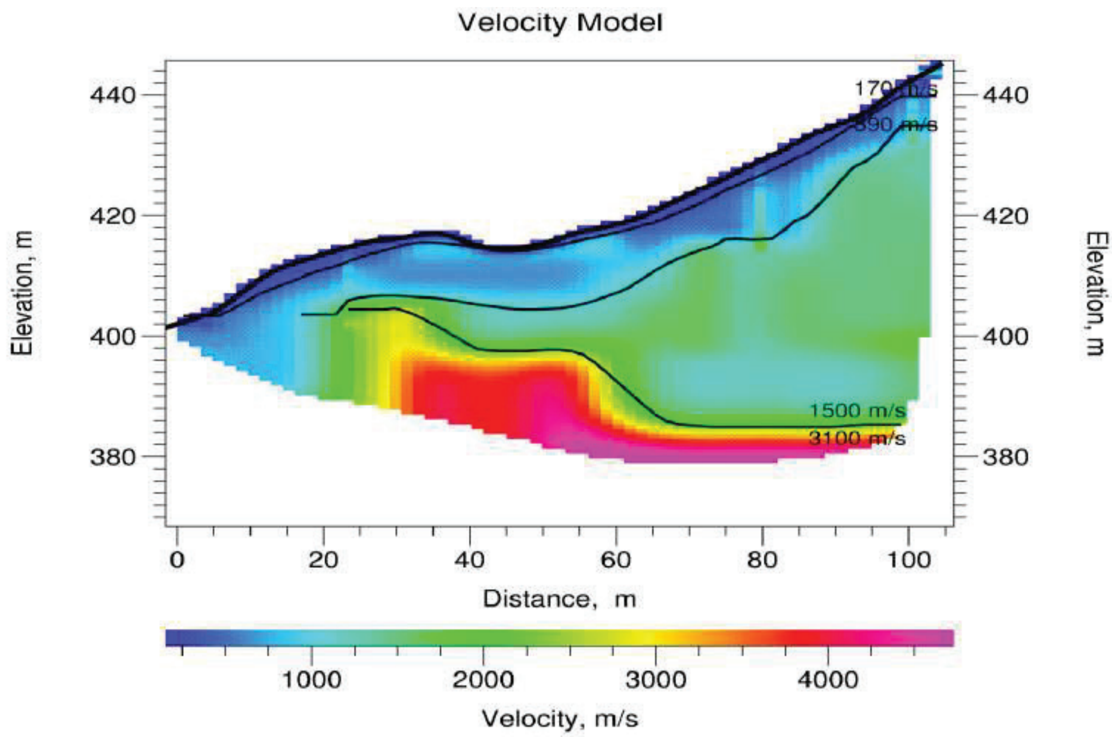
Ground Penetrating Radar is routinely used for health checks of tunnels in various ways:

- Provides Information on Construction & Condition
- Masonry Tunnels- mapping delamination in masonry arches, voids in and behind the brick lining, moisture variation due to leaking pipes, construction arrangement & unexpected changes in masonry and overburden thickness, mapping hidden blind construction shafts.
- Concrete Tunnels- Determine the thickness and arrangement of spayed, in-situ or pre-cast concrete. Voids within or behind the lining. Map variation along a tunnel, such as changes in moisture levels or geology.
- Unlined Tunnels- Maps voids, fractures and manmade features such as rock anchors within rock tunnels.

Electrical Resistivity Imaging can also be conducted along any line on the surface of the tunnel to detect features like cavities behind tunnel walls.

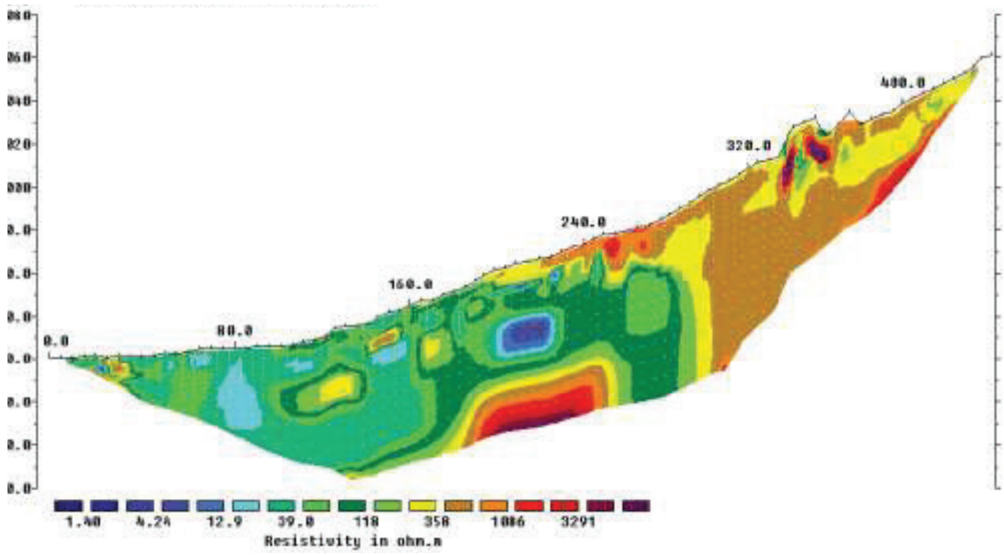
CASE STUDY 1

This case study is related to a railway line project. The proposed Railway line passes through foot hills of the Himalayas and dense reserve forest of Darjeeling district at West Bengal and East Sikkim district of Sikkim state. Geophysical seismic refraction survey was carried out at three tunnel portal locations to determine the stratigraphy of the proposed area. The study successfully revealed the highly undulating topography of rock, and abrupt thickening of overburden/ weathered zone.



CASE STUDY 2

This case study related to a tunnel project in the state of J&K where a shear zone's presence was suspected based on geological signatures. The main objective of investigations was to detect features like thrust, shear zones etc., upto a depth of 70m from the surface. Investigations lines were selected carefully to reveal information along the tunnel route. Site specific details and elevation details have been removed from images to maintain confidentiality of the project as per confidentiality requirement of the client.

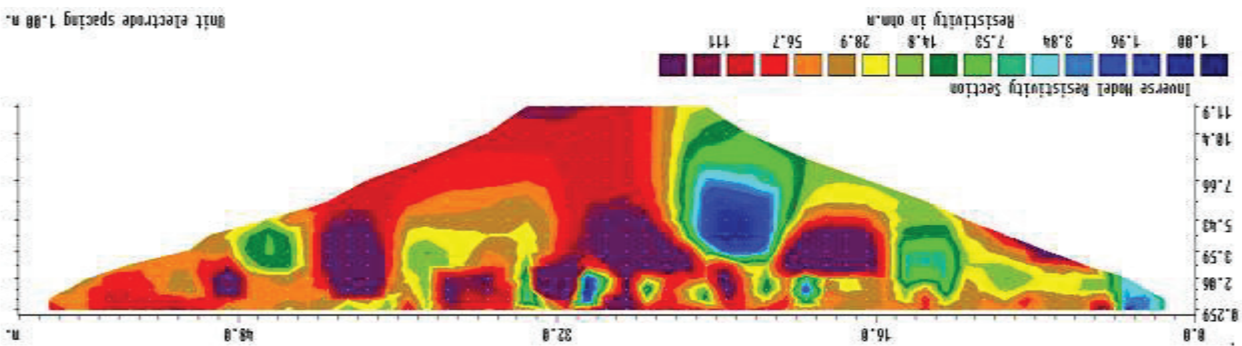


The subsurface resistivity section clearly demonstrates the geoelectrical and lithological layer sequence beneath the ground. The different colour contours represent the different lithological layers system within a depth of 78 m bgl. Following are the notable features of this profile (Elevation values changed).

- The rock (high resistivity- corresponding to quartzite as per local geology) seems to be lower than investigates depth upto RL 1860m, then coming up to RL 1900-1920 and abruptly terminating at Ch 250m. The rocky strata again appears Ch 290 onwards, and post this there is a distinct change in stratigraphy, with presence of high resistivity in all parts of profile.
- There is an extremely low resistivity zone (deep blue) between Ch 200- Ch 220m, with a thickness of around 7-10m. Such low resistivity zones can be typically associated with shear zones.

CASE STUDY 3

This case study related to investigations from inside a tunnel, to detect saturated anomalous zones responsible for seepage in the tunnel. Electrodes were planted along the length of tunnel, along various co-parallel lines, and zone responsible for seepage were clearly identified. One such section us presented here, with Blue zone being the source of seepage.



CONCLUSIONS:

Owners, planners, and designers frequently do not appreciate the vital importance of geophysical services to underground projects. It is well documented that insufficient investigation can result in misleading information and can substantially increase the risk of not finding hazards and unknown conditions that can seriously delay or stop construction, with costly consequences.

There exist various geophysical techniques to investigate the subsurface before finalizing the design to determine presence of anomalies like faults, shear zone, water table, bedrock profile, etc. The selection of techniques is governed by the objectives of study and geology. The information can be acquired quickly at a very small cost, and can save huge amount of money and time otherwise wasted when problems are not anticipated. For any further details author can be contacted at sanjay@parsan.biz.

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4. Airborne Electromagnetic Surveys for Carrying Out Feasibility Studies for Constructing Road and Rail Tunnels in Himalaya by Rajendra Bhasin, Thomas Pabst, Sara Bazin and Arnstein Aarset

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GEOTECHNICAL CHALLENGES IN SLOPE OF PORTAL-1, TUNNEL T-10 ON DIMAPUR–KOHIMA NEW RAIL LINE PROJECT

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ABSTRACT

The 78.40 km Dimapur (Dhansiri)–Kohima (Zubza) New Broad-Gauge Railway Line Project, being executed by Northeast Frontier Railway, is a vital connectivity aimed at linking Nagaland's capital with National Rail Network. Among its 20 nos. of tunnels, Tunnel T-10 (approximately 5.5km long) near Mengujuma presents one of the most challenging geological and geotechnical conditions. Portal-1, located on a steep and highly weathered slope, has encountered persistent instability, groundwater inflow and repeated slope failures above the portal.

Various slope support measures like shotcrete with wire mesh, rock bolts with PVC perforated drainage pipes etc. were adopted to stabilize the slope above portal. Despite these design provisions, the slope experienced instability due to continuous water ingress and inherently weak ground conditions. The sustained infiltration of water led to soil saturation, which significantly reduced its shear strength. This saturation induced a driving force along the rock–soil interface, initiating the slipping of loose overburden and weathered rock material. Furthermore, the presence of rolled boulder debris overlying the loose overburden mass escalated the instability and accelerated the sliding process.

To address these issues, a comprehensive slope stability analysis was carried out to identify critical slip surface and to assess the minimum FOS based on varying groundwater and loading conditions, to decide the stabilization measures.

Based on the analysis, a comprehensive and extensive stabilization scheme was finalized comprising concrete piles, struts, fully grouted rock bolts & cable anchors, with improved drainage measures, to enhance the factor of safety (FOS) ensuring long-term slope stability. Implementation of these measures is considered to contribute in improving the slope stability, reducing deformations and facilitating safe and efficient tunnel construction.

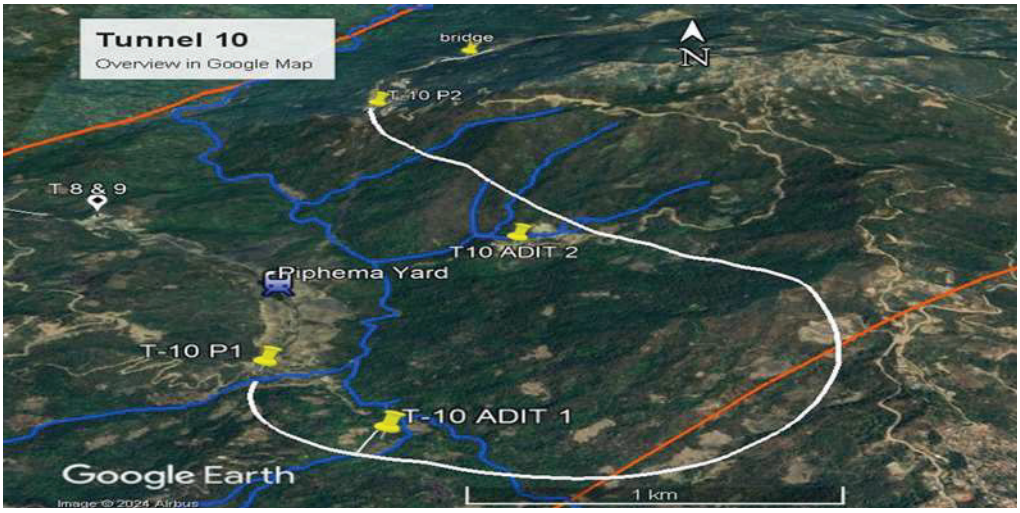
This case study of failure & revision of proposed stabilization scheme highlights the importance of detailed geotechnical investigation, flexible design adaptation and integrated slope stabilization planning for tunnelling projects in the fragile and geologically active hill terrains of North-Eastern India.

1. INTRODUCTION

The Dimapur–Kohima New Broad-Gauge Line Project, covering 78.40 km, is one of the most significant railway projects under the Northeast Frontier Railway (NFR). It aims to enhance regional connectivity and socio-economic growth in Nagaland state of the country. Tunnel T-10, situated near Mengujuma village, is approximately 5.5 km long and passes through steep and geologically complex terrain. The tunnel alignment is located between two major thrust zones, namely the Naga Thrust and the Disang Thrust. It traverses through the Disang and Barail formations consisting of shale, siltstone, and sandstone, all of which are highly weathered and jointed. The Portal-1 area is critical due to its proximity to an unstable slope and high intensity of rainfall in the area. The experiences at this location provide valuable insights into geotechnical risk management and adaptive tunnelling strategies in the Northeast region.

2. PROBLEM STATEMENT

2.1 Salient Features of Tunnel T-10



Tunnel 10 comprises a single-track rail tunnel to be constructed using NATM construction methods in variable and difficult ground condition. Table-1 below provides the salient features of Tunnel 10:

Table 1: Salient Features of Tunnel 10

State	Nagaland
District	Dimapur
Main Tunnel Length	5589 m
Escape Tunnel Length	5618 m
New Portal Chainage	Portal 1 at Ch: 60+760
Formation Level	Portal 1 at Ch: 685.33 m
Seismic Zone	V
Maximum overburden over formation level	360 m

2.2 Geology of Area

The geological composition of the area is summarized as follows:

- **Top Soil:** This is the uppermost layer of the ground, consisting of organic material, vegetation, and loose particles. It is typically composed of a mixture of mineral particles, such as sand, silt, and clay, along with organic matter.
- **Overburden Material:** Below the topsoil, there is an overburden layer composed of boulders, cobbles, and gravel-sized fractured rocks. This layer also contains infilling of clayey silt, which is a fine-grained sedimentary material with a higher clay content.
- **Fractured Shale Rock:** Deeper within the geological profile, there is a presence of fractured shale rock. This shale rock mass is highly weathered and occasionally altered, indicating that it has undergone significant physical and chemical changes over time. The shale rock is described as very weak and semi-consolidated decomposed shale, meaning it has started to break down and is partially solidified.

- **Rock Mass Classification:** The rock mass in the area is categorized as rock class VII, which signifies a very poor rock mass quality (Q value ranging from 0.001 to 0.01). This classification indicates that the rock masses are thinly bedded, highly fractured, having larger aperture and slickensided surfaces due to frictional movement between rock joints, making them structurally weak and unstable.

Overall, the geological composition of the area consists of a relatively thin layer of topsoil, followed by an overburden layer of boulders, cobbles, and gravel-sized fractured rocks with infilling of clayey silt. Deeper layers contain highly weathered and occasionally altered fractured shale rock, which is classified as a very poor rock mass due to its weak and fractured nature.

2.3 Geology Along Tunnel Alignment

Tunnel No. 10, specifically at Portal-1 (chainage Km 60+760), comprises geological formations belonging to the Surma and Barail groups. Along the tunnel alignment, mainly the Surma Group of geological formation is encountered, which consists mainly of medium to coarse-grained sandstone interbedded with shale. These formations exhibit sedimentary structures including current bedding, flute casts, flaser bedding, ripple marks, and convolute laminations, indicative of dynamic depositional environments. Also, the whole tunnel alignment traverses along the ridges and spurs with rugged and steep topography, which further contributes to the challenging geological conditions in this section.

At Portal-1, the slope gradient is mild, with an overburden thickness of approximately 360 meters above the formation level. The exposed geology at this location consists mainly of weak to very weak, laminated shale with minor sandstone bands, which are moderately to highly weathered. In some sections, colluvial deposits are present, comprising rock fragments of shale and sandstone intermixed with silty clay, reflecting past slope movement and weathering processes.

[Fig. 1: Geological formation and rock condition along the tunnel alignment]

2.1. Stability Issue at Portal-1 Location: -

The initially proposed design for the Portal-1 cut slope comprised a 1V:1H (45°) slope angle, supported by a 200 mm thick layer of shotcrete reinforced with double-layer wire mesh, 12 m long rock bolts spaced at 1.5 m × 1.5 m, with drainage arrangement to facilitate subsurface drainage.

Despite these design provisions, the slope experienced distress and subsequent destabilization, primarily due to persistent water ingress and weak ground conditions. The principal cause of cracking and slope movement was traced to a natural surface drain located on the right-hand side of the slope, which carries perennial flow. Although the drain was diverted during execution, residual seepage continued to percolate through the slope mass, leading to saturation of the soil and weathered rock, thereby reducing their shear strength parameters (c and φ). This reduction in shear resistance, combined with hydrostatic pressure buildup, generated a driving force along the rock–soil interface, initiating the sliding of loose overburden and highly weathered rock material.

The affected slope remained in a marginally stable condition for a period before eventual failure, as the saturated, loosely compacted colluvial material lacked sufficient cohesion and internal friction to resist movement. The disintegration of the soil mass and the presence of rolled boulders and debris accelerated the failure process. Overall, the low shear strength, high moisture content and heterogeneous composition of the colluvial and weathered materials led to progressive slope movement and localized failure near the portal area.

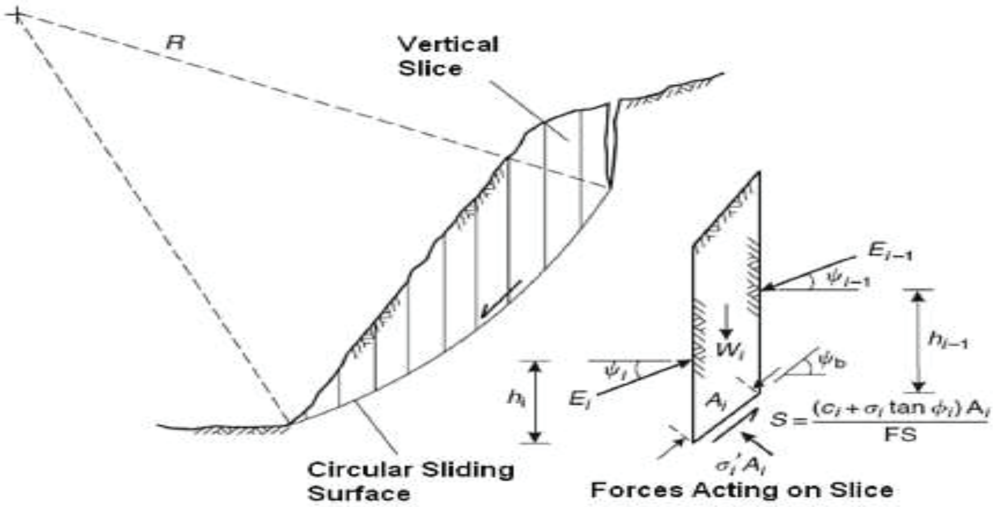
[Fig. 2: Stability problems on slope of Portal 1 of Tunnel T-10]

3. ANALYSIS AND RESULTS

3.1. Failure Philosophy for Slopes

The rock mass condition at the site is characterized by closely fractured and highly weathered material, indicating a significant loss of intact rock structure. Under such conditions, a well-defined structural pattern ceases to exist, allowing potential failure surfaces to develop along the path of least resistance within the slope mass.

Field observations and geotechnical experiences indicate that in highly weathered and decomposed rock masses, the failure surface tends to assume a circular or near-circular geometry, similar to that observed in soil slopes. Consequently, most analytical methods for assessing slope stability under these conditions—such as limit equilibrium analyses—are based on the circular slip surface model. Figure 3 illustrates a typical circular failure mechanism anticipated in the current slope conditions at Portal-1.



[Fig. 3: The shape of typical circular sliding surface with the detail of forces on slice]

Soils composed predominantly of sand, silt, or other fine-grained materials also display circular failure mechanisms, even for relatively low slopes of a few meters. Similarly, highly weathered, altered, or closely jointed rock masses with randomly oriented discontinuities lose their inherent rock-like behavior and tend to fail in a circular or rotational manner. Therefore, for design and stability assessment purposes, it is prudent to treat such slopes as soil-like materials and evaluate them using methods based on circular slip surface analyses, such as limit equilibrium or numerical slope stability modeling.

3.2. Slope Stability Evaluation

A detailed analysis is performed to evaluate the stability factor against possible mode of failures surfaces. Necessary geotechnical and shear strength parameters for the rock are derived from Geological Investigation Report (GIR). Several computations are made to assess the least expected FOS for the proposed geometry of slope. Attempts are made to stabilize the slope by means of concrete piles, struts, fully grouted rock bolts and improve the FOS.

The stability analysis is carried out considering contributions of:

- Dead mass of the sliding material – the weight of the material is considered in the software with respect to the unit weight of the material. Due to the small influence of the sprayed concrete and reinforcement weight with respect to the overall weight of the sloping ground body, the weight of the support structure is neglected in the analyses.
- Ground water – the ground water table is taken based on the borehole data for the excavated slope.
- Seismic load that may be generated at extreme loading conditions.

For the most critical slope, the potential for total global slope failure has been investigated by considering circular failure mechanism. The global stability analysis has been carried out in both static loading condition and seismic loading conditions with the later case adopting a pseudostatic approach of analysis.

3.3. Seismic Consideration

As per IS 1893 (Part 1): 2002 – Criteria for Earthquake Resistant Design of Structures, the project site falls under Seismic Zone V, which represents the region of highest seismicity in India. Accordingly, a zone factor (Z) of 0.36 has been adopted for design calculations. The FOS for seismic condition has been evaluated in accordance with the provisions and guidelines specified in IS 1893:2002 for earthquake-resistant design of structures.

3.4. Geotechnical Design Parameters

The geotechnical design parameters for the slopes are considered in the conservative side based on the previous slopes excavated in the same geology and present site condition at the proposed portal location. The overburden material up to height of 20m approx. was available at this location, which was completely disturbed.

A summary of the design parameters used in the analysis is as shown in table below:

Table 2: Geotechnical Design Parameters for Portal

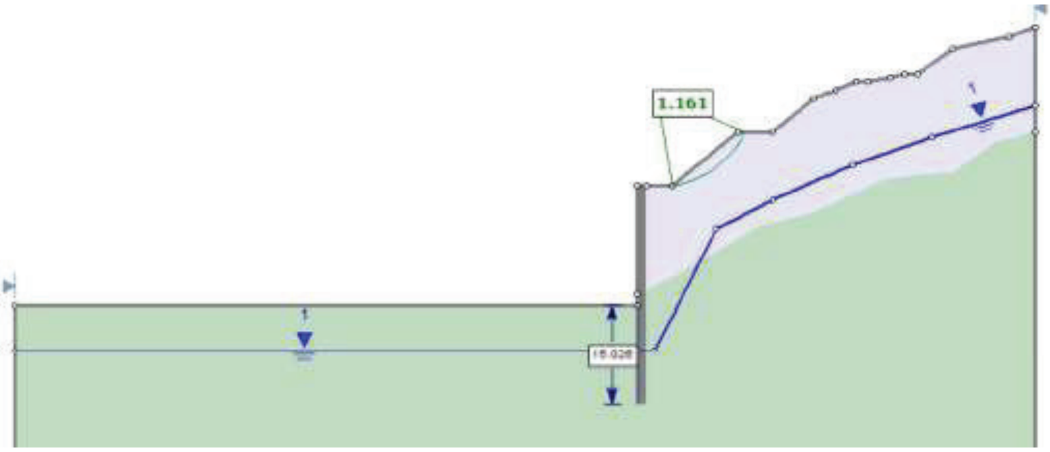
Material	Bulk unit weight	Cohesion	Friction Angle
Overburden/Soil	20 KN/m^3	5KPa	30 degrees

Geotechnical inputs considered to determine rock mass parameters for rock/ shale based on the Lab test results are below in the table:

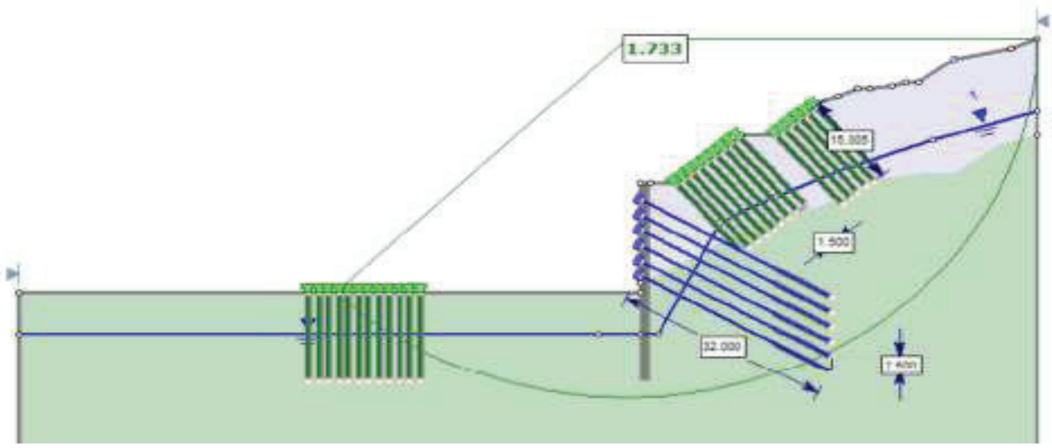
Rock type	Bulk unit weight	UCS	Geological Strength Index (GSI)	Intact rock
Shale	25 KN/m^3	13MPa	10	2823MPa

3.5. FOS for Portal Slope

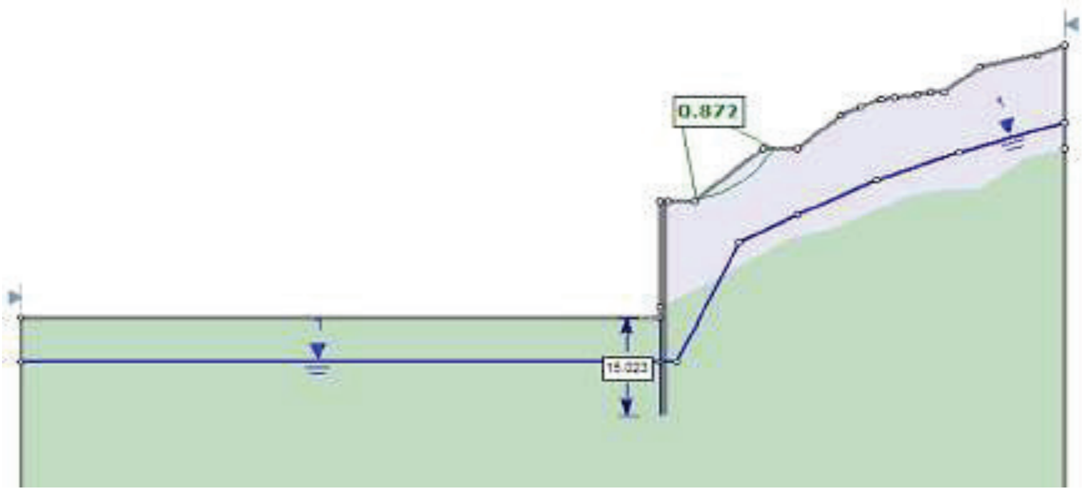
The critical section analysed is cutting slope at Ch: 60+760 at the Portal of Tunnel 10 portal 1 Slope. The design parameters are taken from table 2 above. The results of the slope analysis using Limit State Equilibrium Analysis for face slope and side slopes are as shown below:



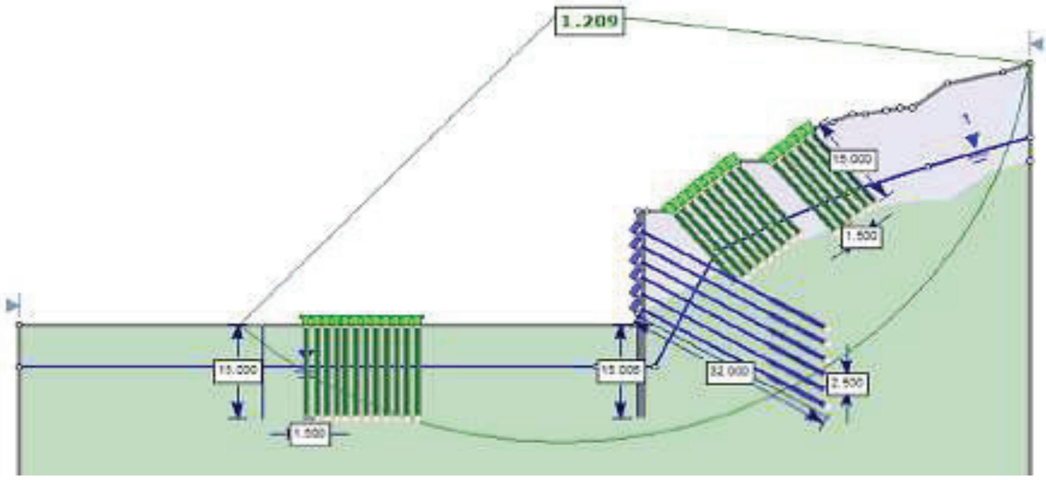
a) FOS for face slope unsupported-main tunnel in static condition is 1.161



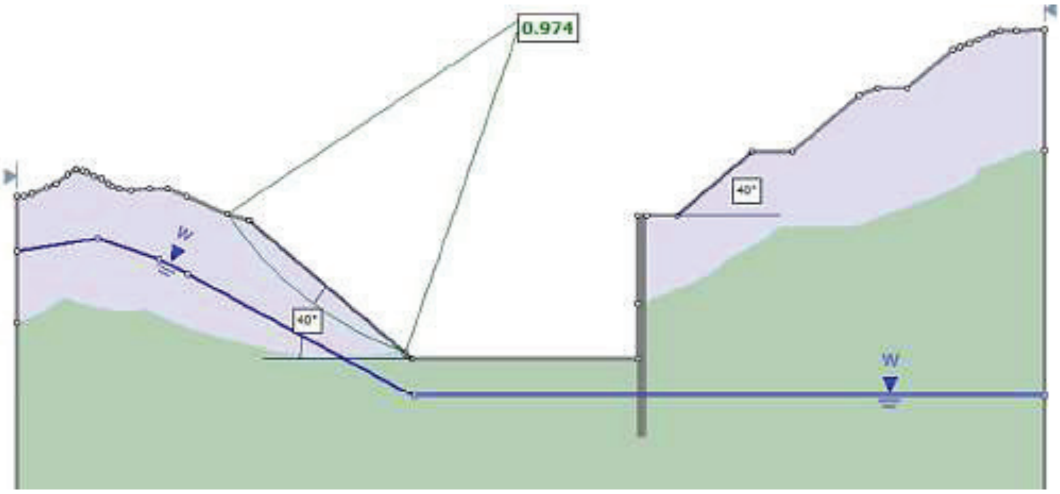
b) FOS for face slope supported -main tunnel in static condition is 1.733



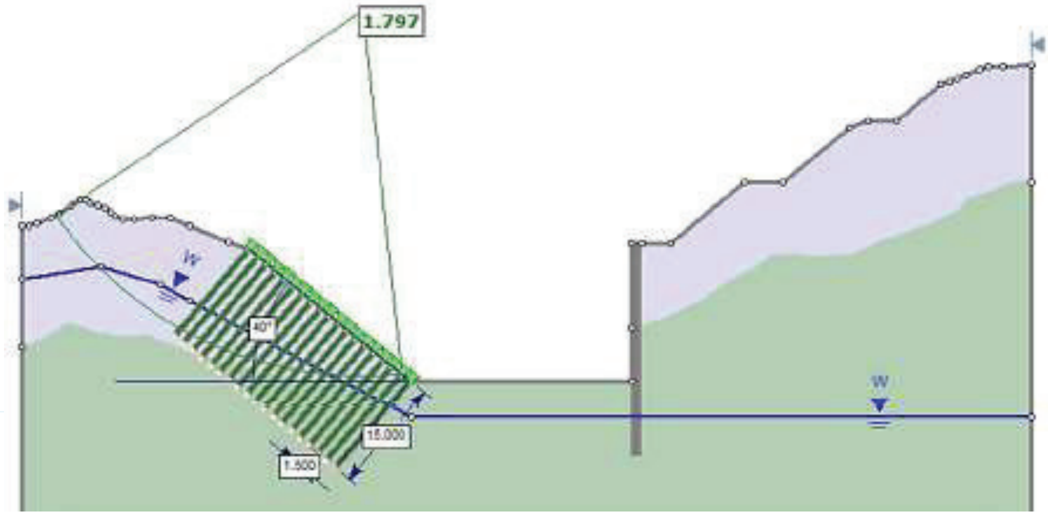
c) FOS for face slope unsupported -main tunnel in seismic condition is 0.872



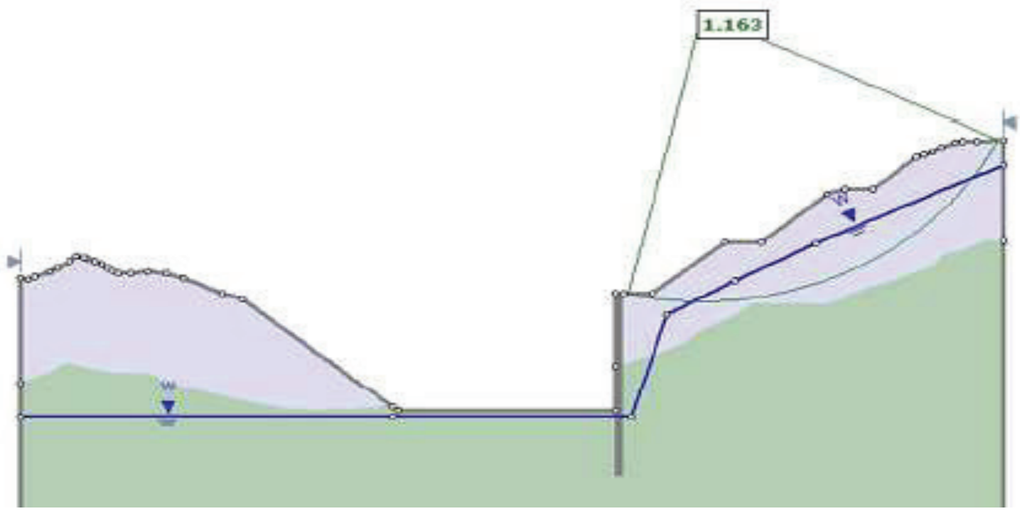
d) FOS for face slope supported -main tunnel in seismic condition is 1.209



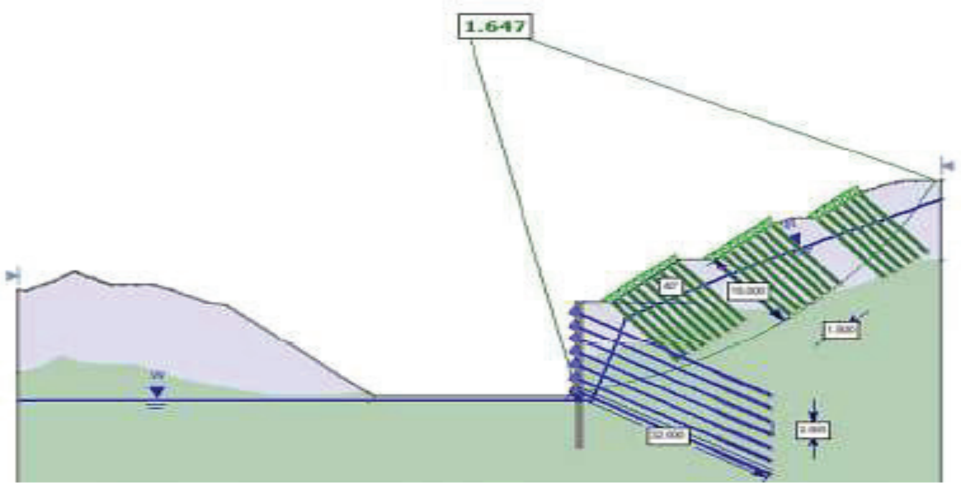
e) FOS for LHS slope unsupported in static condition is 0.974.



f) FOS for LHS slope supported in static condition is 1.797



g) FOS for RHS slope unsupported in static condition is 1.163



h) FOS for RHS slope supported in static condition is 1.647

[Fig. 4 (a) to (h): FOS for face slope main tunnel, LHS & RHS (unsupported & supported)]

The summary of the FOS obtained for face slope and the side slopes are as shown in table below:

Table 3: Summary of FOS Obtained

Section	Condition	Required FOS	FOS (Unsupported)	FOS (Supported)	Remarks
Face Slope-Main Tunnel	Static	1.5	1.161	**1.733**	Stable after support [Fig. 4 (a) & (b)]
	Seismic	1.2	0.877	**1.209**	Stable after support [Fig. 4 (c) & (d)]
Face Slope-Escape Tunnel	Static	1.5	1.159	**1.698**	Stable after support
	Seismic	1.2	0.873	**1.204**	Stable after support
LHS Slope	Static	1.5	0.974	**1.797**	Stable after support [Fig. 4 (e) & (f)]
	Seismic	1.2	0.707	**1.211**	Stable after support
RHS Slope	Static	1.5	1.163	**1.647**	Stable after support [Fig. 4 (g) & (h)]
	Seismic	1.2	0.747	**1.249**	Stable after support

3.6. Summary of Support Systems Adopted

Based on the above analysis, summary of supports will be provided are as mentioned in the table below:

Table 4: Summary of Support Systems

Portal	Slope	Recommended Support Systems
Portal 1	**Face Slope Main Tunnel**	1.2m dia pile with embedment length of min 15m in rock, 32m length of 6 strand cable anchor @ spacing of 2.7m with waler @ spacing of 2.5m vertically and struts @ spacing of 3.6m in the horizonal direction. On top slope, 76mm dia SDR (Self Drilling Rockbolts) of 15m length @ 1.5x1.5m spacing.
	Face Slope Escape Tunnel	1.2m dia pile with embedment length of min 15m in rock, 32m length of 6 strand cable anchor @ spacing of 2.7m with waler @ spacing of 2.5m vertically and struts @ spacing of 3.6m in the horizonal direction. On top slope, 76mm dia SDR of 15m length @ 1.5x1.5m spacing.
	LHS Slope	200mm shotcrete with double layer wire mesh, 76mm dia SDR of 15m length @ 1.5x1.5m spacing.
	RHS Slope	1.2m dia pile with embedment length of min 15m in rock, 32m length of 6 strand cable anchor @ spacing of 2.7m with waler @ spacing of 2.5m vertically and struts @ spacing of 3.6m in the horizonal direction. On top slope, 200mm shotcrete with double layer wire mesh & 76mm dia SDR of 15m length @ 1.5x1.5m spacing.

3.7. Execution Scheme for Slope Rectification Works at Portal P1 of Tunnel T-10: -

The slope rectification at Portal P1 will be executed through a combination of structural, drainage and grouting measures to ensure long-term slope stability and safe tunnelling operations. The execution methodology to be adopted is outlined as follows:

- The face slopes of both the Main and Escape Tunnels and RHS slope of Portal 1 will be supported by 1.2 m diameter piles embedded 15 m into rock, along with 32 m long 6-strand cable anchors at 2.7 m spacing, interconnected with walers and struts.
- Self-drilling rock bolts (76 mm dia) of 15m length at 1.5 m × 1.5 m spacing and 200 mm thick shotcrete reinforced with double-layer wire mesh will be used on face slope and RHS slope above the pile level and on LHS

slope to provide surface support on the slopes. The slope protection work will commence from top and subsequent slopes will be excavated after protection of the excavated slope is done as per design.

- Subsurface drainage will be ensured through 75 mm perforated PVC pipes wrapped in geotextile and installed with a 5° upward inclination, while peripheral/berm drain at the top and toe of slopes will control surface runoff.
- Excavation will proceed in stages, maintaining interim stability and all support and drainage elements will be adjusted on-site in consultation with the Engineer and DDC. Continuous monitoring of slope behavior will be carried out during and after construction.
- The excavated areas will be backfilled immediately after completion of the Cut & Cover tunnel.

3.8. Additional Measures to Enhance Long-term Stability of Slopes:

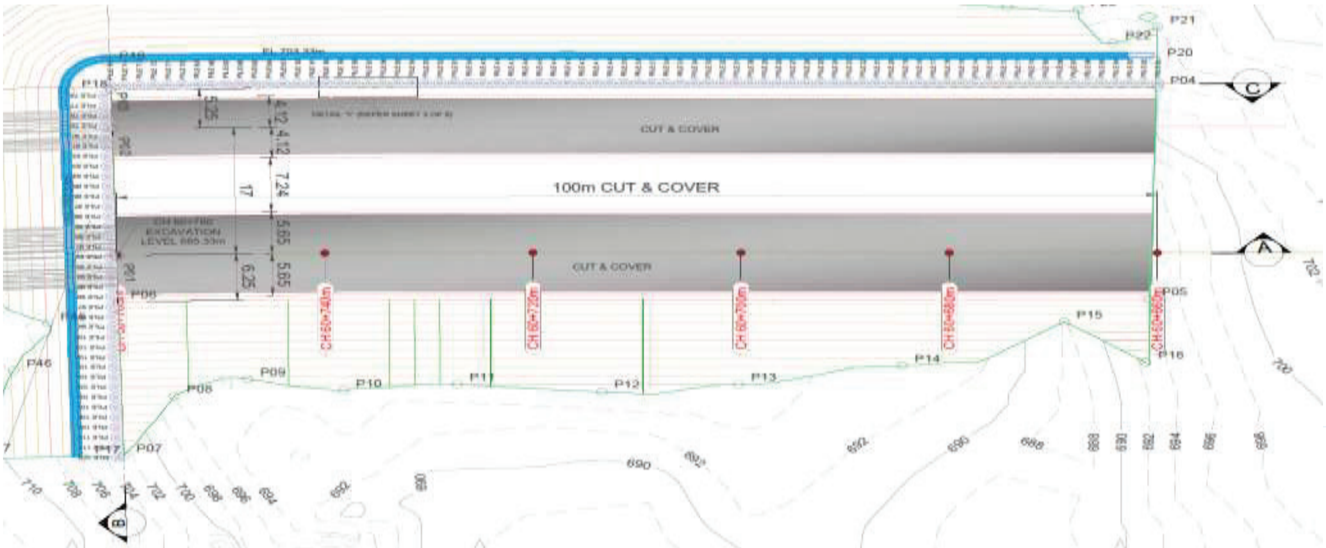
The detailed analysis, based on the prevailing geotechnical and geological characteristics of the slope, is carried out to evaluate the global stability factor against various possible modes of failure. The analysis considered both static and seismic conditions, incorporating ground parameters. In addition to the recommended support system as above, following supplementary measures are suggested to enhance the long-term stability of the slope:

- Installation of 15m long perforated drainage pipes of 76 mm dia wrapped with Geotextile will be provided @2.7m (h) x 2.5m (v) c/c Spacing at an inclination of 5 degrees upward, to efficiently dissipate pore water pressure (Fig-8).
- The efficiency and bond strength of all rock bolts and 6-strand cable anchors should be verified through standard pull-out tests before installation. Displacement/elongation within 40 mm under 375 KN load for 76 mm dia SDR and 32 mm under 600 KN load for 6-srand cable anchors will be ensured in pull out tests to ensure adequate bond strength and load transfer for the proposed slope support design. The cable anchors will be locked at 500 KN load for prestressing purpose.

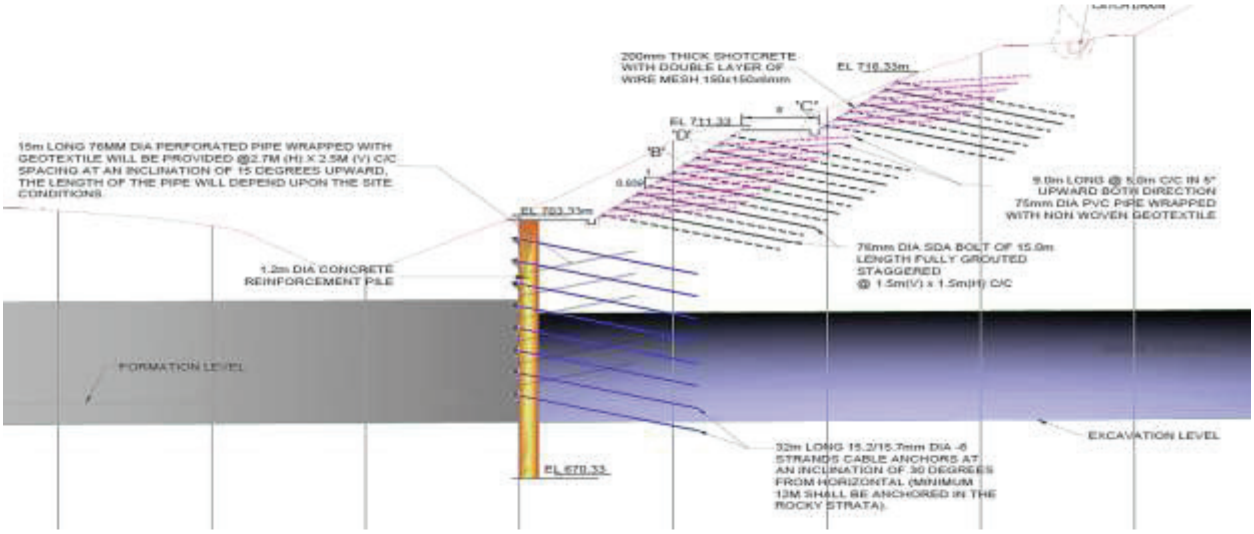
For accurate performance assessment and safe execution, certain field considerations are crucial and thus, have been integrated as design and construction directives:

- The depth of overburden material and groundwater table position used in the analysis are indicative and may vary during excavation. Any increase in these parameters or alteration in slope geometry can reduce the achieved FOS below the design threshold. Therefore, continuous verification of these parameters is essential, and any deviation must be considered for the design re-evaluation.
- Surface water management is critical to slope performance. Runoff from the catchment area must not be allowed to enter the excavation zone. The highly weathered shale–sandstone material loses significant shear strength when saturated. Hence, all surface runoff should be effectively diverted through natural streams or berm drains.

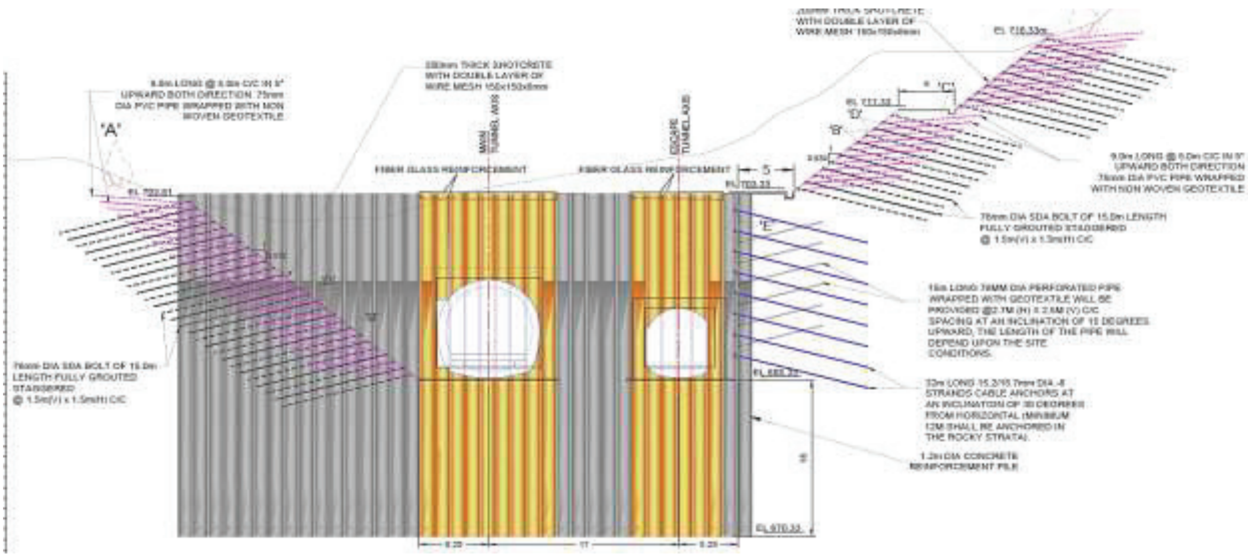
  The surface and berm drains are designed only for residual flows that are not intercepted by primary diversion drains. These drains are to be maintained and cleaned periodically to ensure unobstructed flow and minimize hydrostatic pressure buildup behind the slope.



[Fig. 5: Excavation Plan for Tunnel T-10 Portal P1]



[Fig. 6: Tunnel-10 Portal-1- L Section A-A]



[Fig. 7: Tunnel-10 Portal-1- L Section B-B]

3.9. Implementation of Slope Rectification scheme at Portal P-2 of Tunnel T-16:

A similar rectification scheme has been successfully implemented at Portal P2 of Tunnel T-16 on the Dimapur–Kohima New Rail Line Project, where it has proven effective in stabilizing the slope.

[Fig. 8: Figure showing 'Similar' rectification measures used in Portal 2 of T-16]

4. CONCLUSIONS

The geotechnical challenges encountered at Tunnel T-10, Portal-1, underscored the critical necessity for detailed investigation and flexible design adaptation in tunnelling projects in the geologically active terrains of North-Eastern India. The initial slope failure, driven by persistent water ingress and weak, fractured ground conditions, necessitated a comprehensive re-evaluation using limit equilibrium analysis that treated the rock mass as a soil-like material and using a circular slip surface model for stability analysis. The revised stabilization scheme, which combines structural supports like concrete piles, cable anchors, and struts with improved subsurface and surface drainage measures, successfully enhanced the Factor of Safety (FOS) from a critically low, unsupported value of 0.87 in seismic conditions to a stable value of 1.21, thus validating the integrated approach for ensuring long-term stability and safe execution of the rail line projects.

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Sameer Singh: Graduated in Civil Engineering (B.Tech) from IIT Roorkee, Uttrakhand, India. He is an IRSE (Indian Railway of Service Engineering) officer of 2007 batch and have 16 years of working experience at different positions in Indian Railways, in Construction as well as Open line maintenance. Notably, he has been associated with the construction of a 10.2 km long tunnel T-48 between Sangaldan & Sumber Railway stations of Ramban District (J&K) under the prestigious Udhampur–Srinagar–Baramulla Rail Link (USBRL) Project. In his present position as Director in RDSO, he is engaged in the field of Geotechnical engineering.

TUNNELING IN MIXED GROUND AND CHALLENGING GEOLOGIES

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ABSTRACT

Construction of tunnels by using tunnel boring machines is very common in urban areas, and metro rail is essential for urban areas as a mode of public transport, especially in cities like the Silicon City of India, which got 4th place on the traffic rankings in India. It plays a primary role in urban cities like Bangalore metros and is most essential for daily commuting of millions of passengers per day. Also, it helps in the development of business, less road traffic during peak hours, and is an environmentally friendly, safe, and reliable transport system. Tunneling in Bangalore geology is one of the challenging tasks for the tunnel engineers. The majority of the Bangalore region is covered by the Peninsular Gneissic Complex, which represents remobilized parts of an older crust with abundant additions of newer granitic material. The rocks essentially consist of granites and gneisses intruded by a number of basic dykes. Tunneling in mixed ground conditions with the presence of boulders like gabbro or dolerite below densely populated areas and busy roads was a challenging endeavor. These issues impact the delay in progress as well as cause heavy damage to cutting tools. This paper presents a hindrance faced during tunneling due to the presence of boulders along the alignment and action taken at the site.

Keywords *: TBM, Boulders, Jaw Crusher, Cutters.*

1 INTRODUCTION

In India, tunneling works are going on in all major cities to ease the traffic movement. Bangalore Metro is the combination of elevated and underground structures. Generally, elevated metro is constructed on the road or parallel to the road based on site conditions, and underground is proposed wherever issues arise in acquiring land and high-rise buildings in densely populated areas.

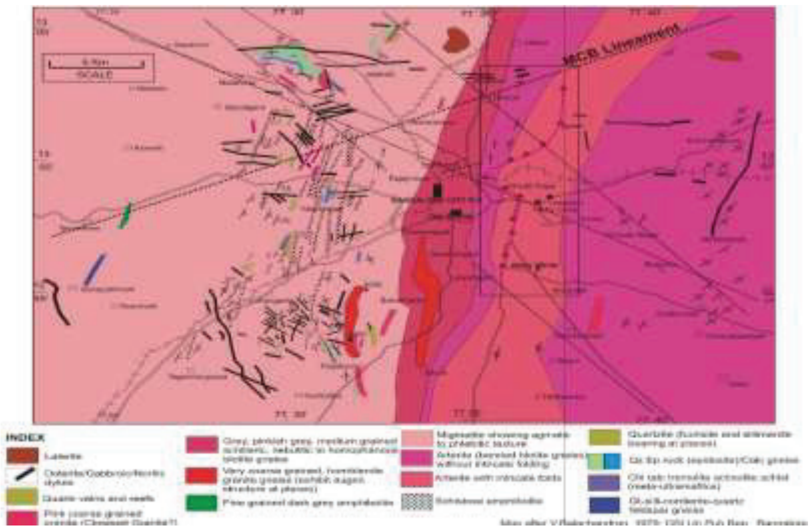

Fig. (1) Geological Survey Map of Bangalore

Fig. 1 indicates geological survey map of Bangalore region. Currently, Phase 2 metro construction is in progress in Bangalore with the combination of elevated and underground alignments. One stretch of the tunneling work and underground stations has been awarded to L&T (Larsen & Toubro). Bangalore city falls under Zone II of the seismic map as per IS: 1893 (Part I)-2016. The soils of Bangalore district consist of red laterite and red, fine loamy to clayey soils with a wide variation of overburden thickness. This article explains the difficulties faced due to the presence of boulders during tunneling between Shivajinagar and MG Road and the methodology to overcome those issues. Slurry TBM had been launched for this mixed geological condition.

2 TUNNEL DRIVEN BETWEEN SHIVAJINAGAR AND M.G. ROAD:

2.1 Overview of the Drive:

This paragraph explains the summary of the tunneling between Shivajinagar and M.G. Road of the Bangalore metro alignment of Phase II. The length of the tunnel drive was 1085 m (with the combination of 1.5 m, 1.4 m, and 1.2 m rings). Tunneling was done below the hostel building, mosque, densely populated commercial buildings, defense quarters, a few residential buildings, and a high-traffic road. Overburden varies from 9 m to 18 m approx. along the alignment. 453 days taken to complete the drive. Average progress for the stretch was 2.4 m per day. The slurry TBM encountered a boulder zone for a length of approximately 255 m during its journey, which was the significant challenge of this drive. Fig. 2 indicates the layout of drive-1 of TBM from Shivajinagar to M.G. Road station.

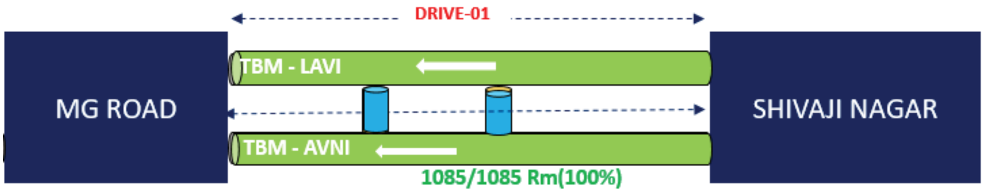

Fig. (2) General Layout of Drive 1

2.2 Slurry Tunnel Boring Machine:

Slurry TBM named AVNI launched in northbound from Shivajinagar station and continued the mining towards M.G. Road Station. Face pressure is to be greater than active earth pressure to avoid face loss. It was achieved by circulating slurry in the excavation chamber to counter the active earth pressure. Slurry penetrates the excavation surface and forms a thin slurry film with low permeability, which effectively holds the excavation surface in order to avoid the face collapse.

The geology was anticipated as mixed soil condition. Hence, the contractor decided to launch a slurry TBM for this stretch. In general, slurry plays a major role as a lubricant system and helps in reducing the wearing of cutting tools during the handling of mixed conditions of geology with the presence of hard rock. Slurry systems avoid direct handling of excavated muck, and excavated earth is managed by a slurry treatment plant.

3 GEOLOGICAL CONDITION:

3.1 Anticipated Geology:

The inferred geology was one that was interpreted or anticipated as per soil investigation reports. Soil investigations and confirmatory boreholes had been done along the alignment based on feasibility before the start of tunneling work. It is found that the expected overall geology from bore logs for this stretch was a mixed condition of silty sand to soft weathered/moderately weathered rock. It is observed at site that some traces of Gabbro and Dolerite are observed visually. It is observed that there is not much variation in the properties between Gneiss and Gabbro. Gabbro and dolerite are among the hardest rocks, which were expected at few locations. Fig. 3 indicates the anticipated geology based on soil investigation reports for a certain distance.

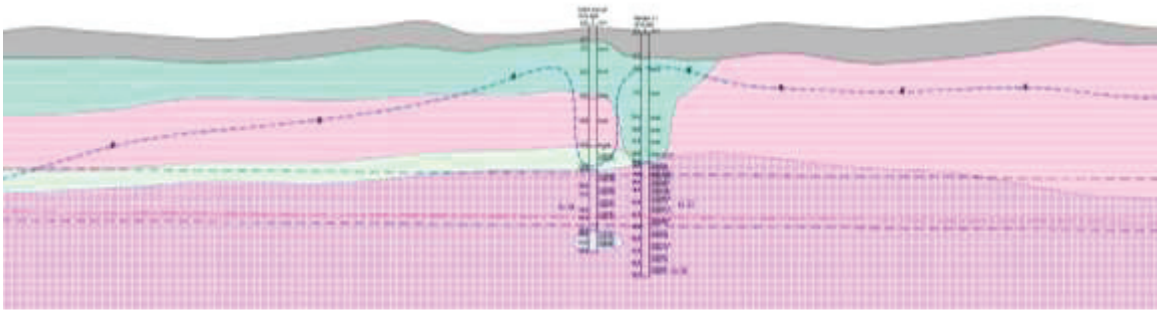

Fig. (3) Mixed Geological condition

3.2 Actual Geology:

The actual geology was one that was encountered during tunneling. The hardest rock of gabbro/dolerite had been encountered in the form of moderately weathered to slightly weathered boulder conditions during tunneling for the length of 255 m from R220 to R371. The rock mass condition was spheroidal weathering type, a form of chemical weathering. This condition affects jointed bedrocks, which results in the formation of spherical layers of highly decayed rock. The presence of boulder rock masses got stuck in front of the cutter head, which rotated in an uncontrolled manner. Fig. 4 indicates the presence of boulders from the excavation face during tunneling.

Fig. (4) Boulders from excavation face

4 CONSEQUENCES DUE TO GEOLOGY:

4.1 Damaging Cutting Tools and Other Parts:

The presence of boulders damaged the cutting tools of the TBM. Mixing arms got broken, cutter discs were severely damaged, and frequent cutter replacements had been conducted. According to the operation parameters, intervention had been executed to identify the cause and it was found that the cutter disc falling off. The fallen cutter entered the excavation chamber and got stuck between the active mixing bar on the outer periphery of the cutter head and the shield invert. When the cutter head rotates, the active mixing bar on the outer periphery of the cutter head is broken. Fig. 5(a) indicates the broken mixing arm and Fig. 5(b) indicates the damaged/worn out cutters.

Fig. (5a) Broken mixing arm

Fig. (5b) Damaged Cutters

A jaw crusher is installed in the bottom of the front shield, and the main function is to crush the large stones and slurry masses. Due to obstruction caused by hard objects such as cutter wedges, the movement of jaw crusher got restrained, which led to the development of cracks at the connection of the jaw crusher bracket and the ring plate of the front shield body. This further tilting of the jaw crusher had happened. Fig. 6 indicates the occurrence of cracks in the jaw crusher.

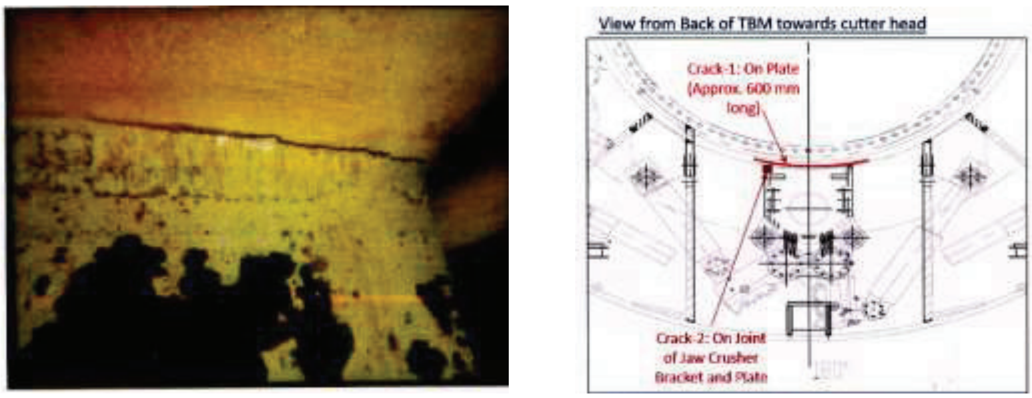

Fig. (6) Cracks in the Jaw Crusher

4.2 Frequent Intervention:

Cutter head interventions had been conducted very frequently due to the presence of boulder conditions of mixed geology for the stretch of 255 m. A total of 193 cutterhead interventions were conducted from Shivajinagar to M.G. Road. However, 51 interventions were executed for the length of 255 m within 155 days of duration, and 87 cutter discs had been changed due to the presence of boulder conditions. Fig. 7(a) indicates the face condition of the particular ring, which shows a mixed combination of fresh rock and highly weathered rock with the presence of boulders. Fig. 7(b) indicates the presence of a boulder that got stuck between cutting tools.

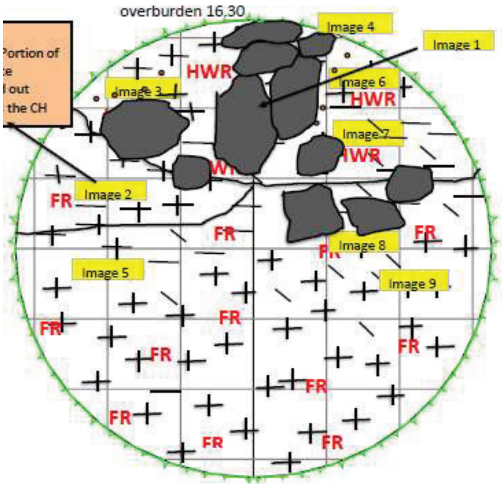

Fig. (7a) Face Mapping

Fig. (7b) Boulders between cutting tools

As the opening ratio is 30%, big-sized boulders entered in the chamber and got collected in the jaw crusher. Fig. 8 indicates the big-sized boulders that were retrieved from the jaw crusher. Also, it was not able to crush to the small size, which was cleared from the excavation chamber.

Fig. (8) Retrieved boulders from chamber

4.3 Delay in Progress:

This mixed-condition boulder appeared below commercial high-rise buildings and the busiest road. Big pieces of boulders entered in the excavation chamber due to an opening in the cutter head, which was the major cause for the severe damages in the cutters, the breaking of the mixing arm, and the development of cracks in the jaw crusher. Presence of boulders trapped in the excavation chamber, discharge slurry pump lines, and a quarry box, which led to frequent stoppages and frequent interventions for cleaning and maintenance. Also, the presence of boulders and cobbles resulted in severe wear and damage in cutting tools, which led to replacement for the same. Hence, it resulted in more interventions and maintenance. Frequent intervention led to a delay in productivity. Further, the components such as the mixing arm, jaw crusher, and return hose to the jaw crusher, which were located inside the excavation and plenum chamber, got damaged. The repair work of the jaw crusher, mixing arm, scrappers, and other maintenance work itself took more time than mining in the boulder geological condition. Since the geology was in mixed condition, intervention had been carried out in hyperbaric mode. All the major repair works, such as welding for the jaw crusher and fixing of mixing arms, had been conducted in compressed mode.

4.4 Corrective Measures:

The new mixing bar was inserted into the inner hole and welded firmly, and both sides are welded with ribs to strengthen the structure. Broken mixing arms had been fixed in the cutter head by mixed gas arc welding. When welding, cutting, grinding, and other operations were carried out near the processing surface, asbestos cloth was kept to wrap or cover the processing surface and assembly surface to avoid damages. Fresh air supply for the miners from the outside supplied by a compressor.

The opening ratio of the cutter head was 30% for the slurry TBM. Hence, the opening ratio of the cutterhead was reduced by installing 16 wedges in between all openings of the cutterhead to avoid big-sized boulders entering inside the chamber. Fig. (9) indicates the installation of wedges in the opening area of the cutter head of the Avni TBM.

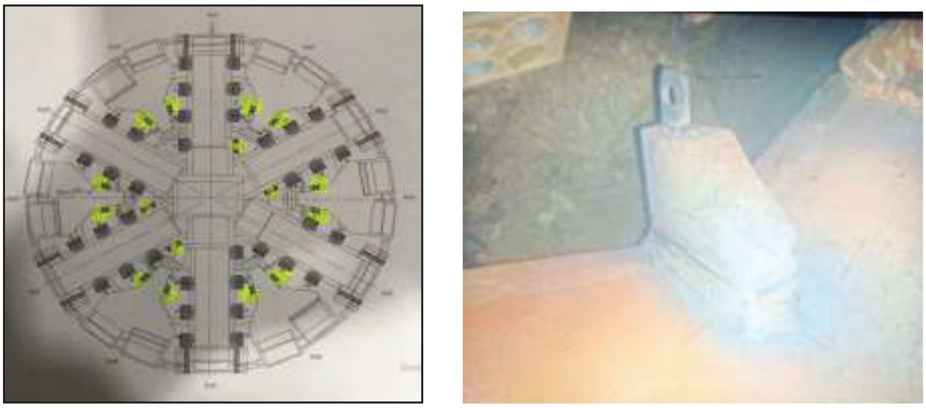

Fig. (9) Installation of wedges in the opening of cutter head

A jaw crusher is installed in the bottom of the front shield and it consists of a hydraulic cylinder, a rotating plate, a center plate, and an assembly of jaw plates. The main role of the crusher is to crush the big size stones and slurry masses and pass to discharge slurry pipe. Due to obstruction caused by hard objects such as cutter wedges, the movement of jaw crushed got restrained, which led to the development of cracks. Further, cracks in the jaw crusher were rectified by welding and gouging. To avoid the tilting of the jaw crusher, additional ribs were added behind the center seat of the jaw crusher. During welding, fire-proof materials were used to protect the main drive, hydraulic pipelines, and other sensible parts of TBM. After completion of welding, a dye penetration test was conducted to identify the presence of cracks. Further, the necessary repair works, such as replacing discs and wedges, etc., were carried out in hyperbaric mode with the help of a trained team of compressed workers. Subsequently, Avni TBM continued its drive with frequent intervention carried out to ensure the conditions of cutting tools. Fig. 10 shows additional ribs were installed in the center seat of the jaw crusher.

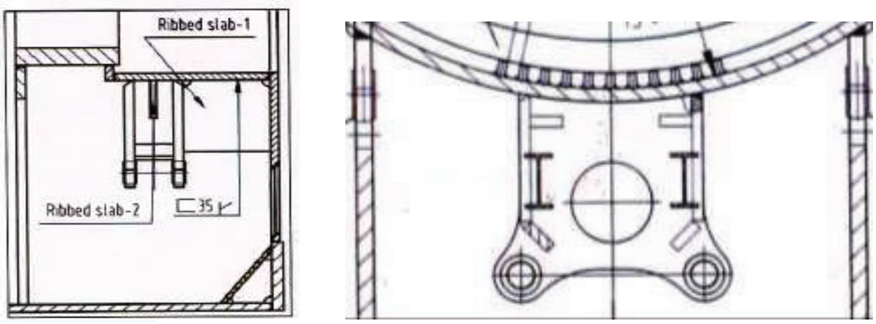

Fig. (10) Installation of wedges in the opening of cutter head

The necessary repair works were carried out in compressed air mode. To ensure the smoke emission in the chamber and the air circulation for human breathing, the exhaust pipeline was set at the welding position in the excavation chamber, which passed through two chamber diaphragm and was connected to the tunnel. The pipeline exhausts and the Samons system supplied gas for circulation. The work had been executed that the gas in the chamber can meet the requirements during operation.

5 CONCLUSION:

Bangalore geology is unpredictable in a short span of length. Always, there will be surprise factors that suddenly occur at the site during tunneling. Mining below the busiest road, densely populated buildings, and significant structures is a very challenging and risky task. In addition to this, the presence of mixed geology conditions is always an enormous challenge for engineers. In Bangalore metro, issues with respect to mixed geological conditions were different from one site to another. Corrective measures had been adopted based on the site conditions. According to this site condition, the opening ratio of the cutter head was reduced by installing wedges, the installation of additional ribs in the jaw crusher, and frequent interventions had been done to continue the drive smoothly to avoid severe damage to the cutter head. After crossing this boulder zone, the drive was smooth and completed the breakthrough at M.G. Road station.

Geology changes within a short span, which is inevitable. Hence, identifying the changes in geology by the conventional soil investigation method is not feasible at all times. In mixed ground conditions, it is better to carry out soil investigation in a short span of length, approx. 20 m. In addition to this method, advanced methodologies like geophysical surveys and ground penetration radar can be adopted.

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APPLICATION OF AN INNOVATIVE DRAINAGE SYSTEM IN THE TOYO ROAD TUNNEL, COLOMBIA

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ABSTRACT

Effective control of water ingress in underground structures is a major challenge. Among other geology-related hazards, sudden ingress of groundwater during construction or once the infrastructure is in operation may result in difficult situations such as flooding, accidents, damage to electromechanical equipment, risk of fire, among others. In order to guarantee a water-free surface, waterproofing or water drainage designs are developed depending on the expected groundwater conditions, ground type, and project characteristics. Fortunately, there are a number of products available for such designs, each with its own advantages and, in some cases, specific applications. Hence, finding the most suitable concept for a given project requires a clear target for the water management, a good understanding of the underground condition, as well as a thorough exchange between all parties involved, i.e., owners, designers, contractors and applicators. Typical components for waterproofing concepts include sheet membranes, geotextiles and sprayed membranes. A common feature is that they prevent water from passing from the rock mass to the final lining of an infrastructure. However, their installation requires high expertise, special equipment and logistics and is time-consuming. Moreover, if water is already present in large volumes - e.g., as seepage or heavy dripping - the excess water must be controlled as much as possible before installing these water management components. An interesting alternative is the drainage system Dolenco Drain. The system consists of a modular conduit network that can be installed onto the rock surface, a layer of shotcrete, or concrete slabs. Each 1 m² module is fixed to the surface using 30 nails and can be easily installed by a small crew using a battery-operated nail gun. Depending on the application, a second layer of shotcrete, which completely covers the network, may be required. The horizontally and vertically interconnected channels are then able to collect and to divert the water by gravity to a designated main drainage. Compared to traditional techniques, it systematically relieves the water seeping to the base and permanently prevents water pressure on the lining. The system can be used as a stand-alone solution or combined with other waterproofing elements. This work presents the application of this solution in the Toyo Road Tunnel Project. The usages demonstrate the versatility and flexibility of the solution. During tunnel excavation, due to water ingress problems, Dolenco Drain was installed after the first shotcrete layer to facilitate the application of the final shotcrete. Thus, making the combined lining of primary and secondary shotcrete layers into a permanent lining.

1. INTRODUCTION

At 9,73 km length The Guillermo Gaviria Echeverri Tunnel, known as the Toyo Tunnel, is the longest road tunnel on the continent, which also reaches a depth of 900 metres.

It joins the Pan-American Highway, which connects America from north to south and the integration will boost connectivity, facilitating trade and strengthening the transport infrastructure of the continent.

2. DESIGN CHANGE

The initial design for the lining of the tunnel was made as a Double Shell Lining (DSL) with a sheet membrane.

By the changing of design, there are significant savings in concrete, time and construction cost. The change of solution increases the clearance height of the tunnel and being a permanent solution, it extends longevity from 50 to 120 years.

By managing serious water inflow (figure 1) with Dolenco Drain during construction, it added to the convenience of construction.

Figure 1 Initial water flow problem

The resulting design:
- Primary shotcrete layer, no fibre
- Dolenco Drain (14mm)
- Cover with ca 2 cm shotcrete, no fibre
- 10mm rubber hose to lead running water into Dolenco Drain strings, where needed
- Ca 4 cm shotcrete with fibres
- Ca 4 cm shotcrete with fibres

Figure 2. Final design

The main advantage of integrating Dolenco Drain into the design is to systematically preventing pressure on the lining and the opportunity to make a monolithic lining.

When Dolenco Drain is installed, the water will automatically find the nearest string of the draining mesh and run to the floor level, and thus permanently alleviate the pressure from the lining. The strings are interconnected vertically and horisontally, and the water will always go to the easiest path.

With 70% of the primary shotcrete layer surface exposed, the secondary layer shotcrete has sufficient adhesion to create a monolithic lining. And since both primary and secondary lining can be considered permanent lining, there is basis for designing a slimmer overall lining, assuring maximum clearance in the tunnel.

3. WATER MANAGEMENT DURING CONSTRUCTION

In the sections with most severe water ingress, a number of options was defined to address this:

- Plugging or grouting/injecting
- Directing the water with Dolenco Line Drain (high flow)
- Installing larger and smaller pipes/hoses (medium/low flow)

“Smaller” seepages are usually managed by installing Dolenco Drain and cover with shotcrete.

With this in mind, a section with most severe inflow was selected and a few hundred sqm test area was made just with Dolenco Drain and shotcrete. The experience from this was to decide how to go forward.

The result was that most seeping stopped and the remaining seeping points were managed by installing 10mm rubber hoses.

It was considered to combine with a spray membrane, but as the work progressed, it was decided not to.

4. INSTALLATION PROCEDURE

When making the initial test area, the inspection included assuring proper installation of Dolenco Drain in accordance with guidelines, before covering with shotcrete.

Points for visual test and inspection of installation:

1. The concrete surface must be level without holes or protrusions, which prevents the proper installation of Dolenco Drain.
2. Modules must be nailed to the surface at all dedicated nailing points.
3. Modules must be nailed to the surface with no more than 5mm distance from the surface, to prevent clogging by shotcrete.
4. Modules must have overlapping of vertical and horisontal connections to secure water flow.
5. If horisontal connections are not overlapping, they must be nailed shut, to avoid possible clogging by shotcrete.
6. Ends of modules at the base must be installed, so water can flow freely out. Typically, by mounting a strip of geotextile over the ends to secure that shotcrete has not blocked the ends of the modules.
7. After the appropriate layer of shotcrete has been applied over Dolenco Drain and geotextile has

been placed, Dolenco Drain must be covered entirely with shotcrete, with no penetrations and no visible drain parts.

8. After finished installation, water should be visually flowing from the ends.

5. INITIAL SHORTCRETE LAYER

When mounting Dolenco Drain, it is not a problem to have seeping water. The procedure starts with applying the initial, primary lining on the surface to ensure a solid basis.

Figure 3. Initial shotcrete layer applied onto the rock surface

The surface is levelled out, so there are no sharp edges, but a wavy contour (Figure 3). This assures that Dolenco Drain can be mounted correctly on the surface.

Cavities are filled out to assure proper mounting (Figure 4) of Dolenco Drain no more than 5mm from the surface.

Figure 4. Mounting distance criteria

6. MOUNTING DOLENCO DRAIN

Following application of the initial shotcrete layer, Dolenco Drain modules is mounted onto the surface.

It was done partly from a scaffold and partly from a scissor lift (Figure 5), with a handheld, battery driven nailgun, typically with 30 mm nails.

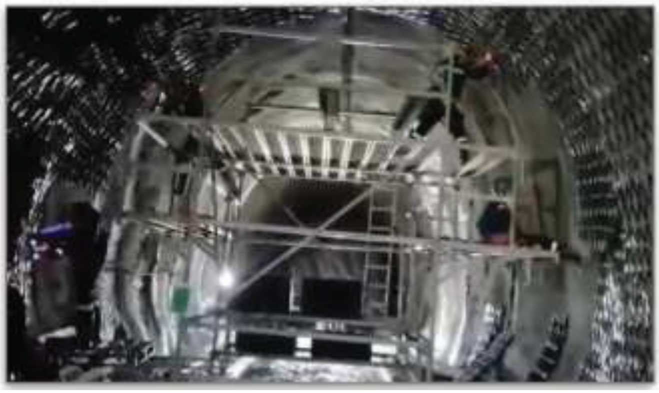

Figure 5. Mounting partly from a scaffold and partly from a scissor lift

The modules flexibility made it possible to mount on very uneven surfaces (Figure 6) and to adapt around other installations and for example anchors.

Figure 6. Mounting on uneven surface

Since Dolenco Drain modules are light weight (660g/module), they can be mounted quickly without any heavy lifting. For making faster installation, 5-6 modules were fastened together in a row (Figure 7), before mounting.

Figure 7. A row of modules mounted onto a wavy surface

7. COVERING DOLENCO DRAIN WITH SHORTCRETE LAYERS

First a thin layer of 20mm unreinforced (no fibres added) shotcrete was applied to cover the modules (Figure 8). Hereby the primary function of leading the water inside Dolenco Drain strings is achieved.

Figure 8. First 20mm shotcrete applied over Dolenco Drain in cross passage

It does however, not secure against all seeping through the lining. Specially, where there is significant water flow. Where waterflow at this point was significant and damaging the shotcrete layer, a piece of rubber hose was installed by drilling into the structure and fixing the rubber hose in the hole with a glue.

Guiding the end of the hose into one of the strings of a Dolenco Drain module, the water flow is led to the base of the wall through the Dolenco Drain draining network channels (Figure 9).

Figure 9. Installing rubber hose

After installing the rubber hose, it is possible to apply the next 40mm fibre reinforced shotcrete, which can harden and fully cover areas with even severe seeping.

Finally, another layer of 40mm fibre reinforced shotcrete is applied. By applying in two layers of 40mm each, any defects in the casting of the first layer is countered, when applying the second layer.

Water has thus been fully managed and there is no pressure on the waterproof lining. At the same time the concrete lining thickness has been reduced.

The inclusion of Dolenco Drain has permanently prevented pressure on the lining and secured a robust solution for the lifetime of the structure. In addition, Dolenco Drain can be 100% recycled.

8. DISCHARGE OF THE SEEPING WATER

At the base of the wall, water is seeping out of the open ends of the modules, which has been exposed by mounting a strip of geotextile over the ends, before covering with shotcrete.

Below, a trench has been dug out to contain the water on both sides of the tunnel and for the water to move horisontally. At the end of the tunnel a pipe has been installed to lead the water to a designated location.

Figure 10. The final result of changing design to prevent water pressure

9. CONCLUSION

Changing the design with the principle of permanently preventing pressure with Dolenco Drain highlight the many advantages of this approach.

A more robust solution, extending the lifetime of the lining from 50 to 120 years. Improving clearance height. Easy and simple installation during, and easy management of, inflow of water.

Significant reduction in concrete consumption, reduced construction time and construction cost. Reducing future maintenance.

By managing even serious water inflow with Dolenco Drain, a permanent, robust solution is achieved for the future.

USE OF ULTRA HIGH PERFORMANCE MACRO POLYPROPYLENE FIBER IN PRECAST FINAL LINING OF WATER TUNNEL

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ABSTRACT

The application of Ultra High-Performance Macro Polypropene Fiber in the final lining of tun-nels presents a promising alternative to traditional reinforced concrete linings, particularly in sce-narios where the stresses and moments observed on-site, remain within permissible limits. This study focuses on a live project case involving a D-shaped waste-water tunnel. The project includes a cut-and-cover section approximately 18 kilometers in length, with an overburden of 12 meters, backfilled using the same excavated soil. To accelerate construction progress, the secondary lin-ing is planned using precast segments, which are casted in yard, transported & erected at the site. Finite Element Modeling (FEM) was conducted using RS2 software, complemented by analysis in STAAD.Pro. Various conditions were considered in the tunnel analysis, including empty tun-nels, filled tunnels, and segments during lifting or erection. Before implementing Polypropylene Fiber Reinforced Concrete (PPFRC) in the segmental lining, a series of mechanical tests were conducted at varying fiber dosages. These included Crack Mouth Opening Displacement (CMOD), split tensile strength, compressive strength, and flexural strength tests. The test results were compared against FEM simulations.

The findings of this study provide valuable insights and serve as a potential reference for the effective use of Ultra High Performance Macro Polypropylene fiber (Tensile strength- 650 MPa, Modulus of elasticity- 9 GPa) in tunnel segment linings. This material offers a viable alternative to traditional steel reinforcement, delivering impressive cost savings of approximately 70% with-out compromising structural integrity, safety, or durability.

***Keywords**: Tunnels, Tunneling using precast panels, Ultra High Performance Macro Polypro-pylene fibers in secondary lining, RS2, STAAD.Pro, Cost savings, Steel replacement*

1 INTRODUCTION

Concrete structures primarily experience either tension or compression. The compres-sive forces are handled by the concrete itself, while the tensile forces are resisted by reinforcement. Even when the tensile forces are minimal, a minimum percentage of steel reinforcement must still be provided, as specified in various codes.

But can this tensile force be resisted by a ma-terial other than steel? The answer is yes — fibres mixed into concrete in the right proportions can effectively handle tensile stresses.

However, it also depends on the magnitude of the stresses acting on the lining, which must be determined based on actual site conditions and kept well within the capacity of fibre reinforced concrete.

The addition of fibres enhances the ductility and structural integrity of concrete. Fibres may be made of steel, glass, synthetic materials, or natural substances. The properties of fibre-reinforced concrete vary depending on the type of fibre used, as well as its orientation and distribution within the mix. Fibre reinforcement helps control cracking, especially in the early stages of concrete curing, and improves impact and fatigue resistance. By selecting the appropriate fibre type and dosage, engineers can tailor concrete performance to suit specific structural or environmental demands.

In this research, a study was conducted using macro polypropylene fibre—Bajaj Fibre Tuff in the secondary concrete lining of a water tunnel. The total length of the tunnel is 30 km, consisting of an 18 km cut-and-cover section and a 12 km NATM (New Austrian Tunneling Method) section. The proposed tunnel has a D-shaped cross-section with an overburden of 12 meters. The hydraulic characteristics of the tunnel include a discharge of 40 cumecs at a flow velocity of 2.1 m/s. The tunnel operates under gravitational flow, with a cross-sectional height and width of 5.1 meters.

2 GEOLOGY

The location of the project is on the Malwa Plateau, an extension of the Deccan Traps, formed between 60 and 68 million years ago during the late Cretaceous period. This plateau is situated north of the Vindhya Mountain ranges and is elevated above the northern plains, with an average elevation of 494 meters (1,620 feet). However, depending on local geomorphology and fluvial influence, the basaltic bedrock is overlain in many areas by significant alluvial and colluvial deposits, including sandy soils and silt-clay mixtures, particularly in valleys and low-lying areas.

Several boreholes were drilled along the tunnel alignment. In the cut-and-cover section, sandy soil was encountered up to a depth of 18 meters, beneath which massive basalt was found (Image 1).

Depth	Run	Core No.	Core Recovery	RQD	Structural Details (Jt, Fr, Bedding, Fault etc.)		Log	Lithology
(m)	(m)		(%)	(%)	Symbol	Description		
1.5	1.5							
3.0	1.5							
4.5	1.5							
6.0	1.5							
7.5	1.5							
9.0	1.5							Yellowish brown sandy soil
10.5	1.5							
12.5	2.0							
14.0	1.5							
15.0	1.0							
16.5	1.5							
18.0	1.5							
18.5	0.5		56	0				Massive basalt boulders

Image 1

Interpretation was made by referring bore-logs. (Image 2).

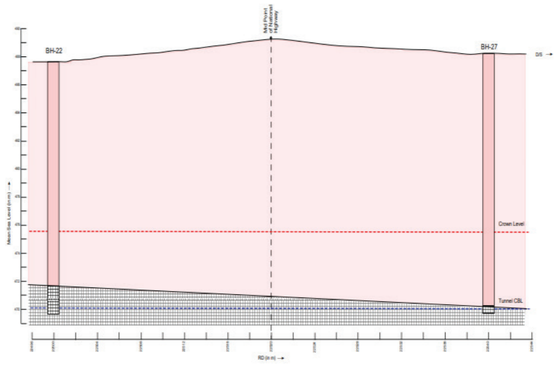

Image 2

The presence of approximately 18 meters of sandy soil, there is a high potential for surface settlement and a significant risk of ground loss, particularly if the water table is present. Under such conditions, NATM or conventional tunneling methods are considered unsafe without prior ground improvement. Thus, precast frames of D shape are planned within this 18 km stretch of cut & cover.

3 DESIGN ASPECT

To determine the stresses acting on the final lining, a Finite Element Method (FEM) model was developed using RS2 software (Image 3). In the model diagram, the light grey layer represents sandy soil, while the light green layer depicts basalt rock. The boundaries on both sides are restrained in X direction whereas for base of the model it is restrained in the X and Y directions to simulate support conditions. Material is considered as homogeneous.

Following four stages are considered in RS2 model:

1. **In-situ Condition**: This initial stage establishes the natural stress state of the ground before any excavation begins.
2. **Excavation Stage**: In this phase, the excavation process is simulated. To extract the maximum stresses arriving on final lining, the sandy soil is modeled as an elastic material, whereas the basalt rock is treated as a

plastic material to capture maximum stress concentration.

3. **Installation of Concrete Lining**: After excavation, the concrete lining is applied on the excavated periphery of the tunnel with a Polypropylene fibre-mixed concrete property of M-35 grade, fibre dosage 7 kg/m3.
4. **Application of Internal Water Pressure under Seismic Conditions**: In the final stage, internal water pressure of 0.04 MPa is applied along with seismic loading to assess the structural response of the lining under combined effects.

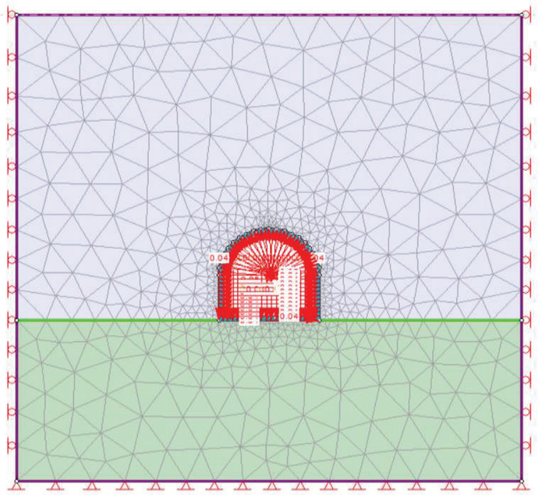

Image 3

In obtained results, maximum axial is found as 0.210 MN (invert corners), maximum bending moment is found as 17.67 kN-m & maximum shear is observed as 0.113 MN.

To reverify the results obtained from RS2, an analysis was also conducted using STAAD.Pro, incorporating spring supports to simulate ground interaction. The same conditions were considered in this model with two loading scenarios: tunnel in empty condition, and second tunnel filled with water.

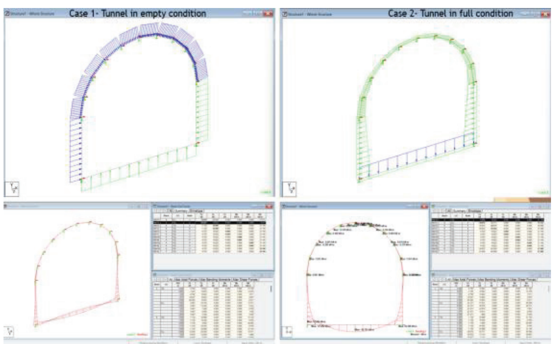

Image 4

Among these two cases, the maximum bending moment observed was 57 kNm (Image 4). In addition to the analysis, a site visit was carried out to observe key construction activities such as reinforcement tying, concrete casting, and the erection process of the precast tunnel segments. One critical condition to assess was the lifting scenario, as the D-shaped precast frame is lifted during erection using two slings attached at the crown to make segment vertical.

Image 5

To simulate this condition, a separate STAAD model was developed specifically for the lifting case. The analysis showed a maximum bending moment of 74 kNm, resulting in a peak bending stress of 2.7 MPa. This case of lifting was found to be the most critical, as the observed stresses were high as compared to those recorded in the other scenarios. Therefore, the performance of the fiber-reinforced concrete must be sufficient to safely withstand these elevated stress levels.

4 MACRO POLYPROPELENE FIBER PROPERTIES & TEST RESULTS

A Macro Polypropylene Fiber, improves the performance of concrete by imparting strength to concrete, reduce cracking, increasing toughness, and enhancing durability. The key features are listed in table as below (Table 1).

Sr. No.	Characteristics	Performance
1.	Length of fiber	54 mm
2.	Dia. of fiber	0.84 mm
3.	Tensile strength	650 MPa
4.	Modulus of elasticity	9 GPa
5.	Effect on strength of concrete at 7 kg/m3	Peak 5 MPa, No crack

Table 1

Furthermore, by adding fibre Bajaj Tuff at a dosage of 7kg/m3 in a concrete with a grade of M-35, we have conducted a test on composite material of fiber. The length of fiber is considered as 54 mm and results are analyzed under CMOD (Crack Mouth Opening Displacement) test trough IIT-Hyderabad (Image 6).

The Crack Mouth Opening Displacement (CMOD) test measures the displacement at the mouth of a crack in a material under loading to assess fracture toughness and crack growth behavior. It is commonly used in concrete to evaluate how cracks develop and propagate under stress. Average Peak flexural strength is observed as 5 MPa.

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भारतीय प्रौद्योगिकी संस्थान हैदराबाद
Indian Institute of Technology Hyderabad

FIBER REINFORCED CONCRETE TEST REPORT AS PER EN-14651

Reference No: IITH/BRPL/Beam-7kgm/16122024-1

Description: Evaluation of beam specimens with Macro Fiber as per EN-14651

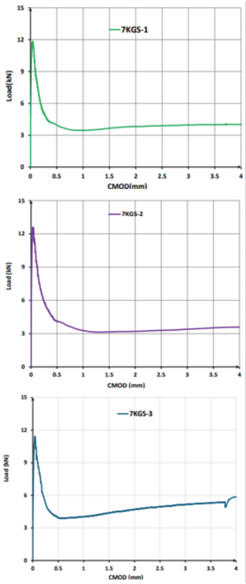
Test details as specified by the client

Flexural Residual Strength (CMOD) as per EN-14651

Sample description

Sl.No	Particular	Description
1	Fiber type	Macro Fiber #
2	Fiber Length	54 mm #
3	Fiber dosage	7.0 kg/m³ ##
4	Type of Specimen	150*150*700 mm³ (4. Nos) supplied by the client
5	Date of casting	08-10-2024
6	Curing age	-
7	Defects in specimen before the test	-
8	No of the specimens tested	3
9	Date of Notching	7/12/2024
10	Test date	7/12/2024
11	Testing Machine	300 kN Flexure Testing Machine of CONTROLS make

As Specified by Ms. Bajaj Reinforcement Pvt. Ltd.
Beams were tested in the 'as-received' condition. The fiber dosage is as per the client. The fiber dosage cannot be independently verified.



Observed and calculated Parameters

S.No.	Particular	Beam -1	Beam -2	Beam -3	Average
1	Average width of the specimen.b(to the nearest 0.1mm)	150.2	150	150.1	
2	Average Distance from the Tip of the notch and the Top of the Specimen(to the nearest 10.1mm)	124.9	125	124.8	
3	Width of the notch.x((to the nearest 0.1mm)	4.1	4.3	4.2	
4	Span length l(to the nearest mm)	700	700	700	
5	Rate of increase to the CMOD (mm/min)	0.05(CMOD<0.1mm) 0.2 (CMOD≥0.1mm)			
6	Max Load Corresponding to the LOP F_L (to the nearest kN)	11.83	12.6	11.44	11.95
7	LOP, FL (F_L (to the nearest 0.1N/mm²)	4.9	5.2	4.8	4.96
8	Load Corresponding to the $CMOD_j$, F_j				
8.1	At CMOD1=0.5mm (to the nearest kN)	3.97	4.12	3.94	4.01
8.2	At CMOD2=1.5mm (to the nearest kN)	3.6	3.15	4.37	3.70
8.3	At CMOD3=2.5mm (to the nearest kN)	3.89	3.29	4.94	4.04
8.4	At CMOD4=3.5mm (to the nearest kN)	3.99	3.52	5.29	4.26
9	Residual flexural tensile strength values F_{rj}				
9.1	At CMOD1 to the nearest 0.1N/mm²)	1.65	1.71	1.64	1.65
9.2	At CMOD2 to the nearest 0.1N/mm²)	1.5	1.31	1.82	1.48
9.3	At CMOD3 to the nearest 0.1 N/mm²)	1.62	1.37	2.05	1.68
9.4	At CMOD4 to the nearest 0.1 N/mm²)	1.66	1.46	2.20	1.77
10	Uniformity of fiber distribution at the fracture surface	Uniform	Uniform	Uniform	

Image 6

5 CONCLUSION

In this study, the use of high-performance polypropylene (HPP) fibers in the concrete lining of water tunnels was investigated. The advantages and shortcomings of using HPP fibers for concrete linings are also discussed. Based on the results, the incorporation of HPP fibers significantly improved the concrete's flexural toughness, reduced permeability, and enhanced resistance to chloride penetration compared to plain concrete. The findings further indicate that the use of HPP fibers can improve the durability and serviceability of concrete linings. The experimental investigation on high-performance polypropylene (HPP) fiber-reinforced concrete and steel fiber-reinforced concrete with various fiber volume fractions revealed that the addition of HPP fibers slightly reduces workability due to good interlocking between aggregates and fibers. However, the workability can be improved by increasing the slump.

The results obtained from the design analysis indicate that the structural performance remains well within the capacity limits of the composite material used—specifically, a mixture of a ultra-high strength polypropylene fiber added at a dosage of 7 kg/m³, with a length of 54 mm combined with M-35 grade concrete. Presents a highly effective alternative to conventional steel reinforcement, offering substantial cost savings of approximately 70% without sacrificing structural integrity, safety, or durability. In addition to its economic advantages, the fiber-reinforced concrete also simplifies the construction process and reduces labor requirements. This innovative approach contributes to a more sustainable and efficient construction methodology.

6 REFERENCES

1. **EN 14889-2:2006** – This European standard specifies the requirements for polypropylene (PP) fibers used in concrete, including definitions, specifications, and conformity criteria.
2. **ASTM C1116/C1116M-06** – This standard specification from ASTM International covers the use of fibers in concrete, including PP fibers, for various applications such as crack control and reinforcement.
3. **BS EN 14845-1:2007** – This British standard provides test methods for fibers in concrete, including those made of polypropylene, to assess their performance in concrete mixes.
4. **IRC SP 46** – This Indian Road Congress publication provides guidelines for the use of fibers in concrete, including polypropylene fibers, for road and bridge applications.
5. **"Utilizing Polypropylene Fiber in Sustainable Structural Concrete Mixtures"** – This study published in MDPI discusses the use of polypropylene fibers in concrete mixtures, highlighting their benefits in terms of sustainability and structural performance.

MANAGING GEOLOGICAL COMPLEXITY IN TAIL RACE TUNNELS AND THE OUTFALL STRUCTURES OF TEHRI PUMPED STORAGE PROJECT-A CASE STUDY

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ABSTRACT

The Tehri Pumped Storage Plant (PSP) Project is one of the prestigious ongoing mega projects in the state of Uttarakhand in Tehri Garhwal District, India. Prior Estimate of uncertainties and risks involved in the construction process is central to decision making in the underground projects. This Project aims to generate 1000 MW of power by harnessing the Hydro power potential of the mighty Bhagirathi River- the main tributary to the river Ganges.

The main feature of this project is its Tail Race Tunnels and Outfall structures, which will function as Tail Race Tunnels and Outfall in generation mode and as Head Race Tunnels and Power Intake structures in the pumping stage.

The present paper discusses advance identification of geological risks, design updating, and measures applied to reduce the risks in the construction of Twin Tail race Tunnels and outfall structures of Tehri Pumped Storage Hydro Project in the Indian Himalaya. The Paper also underlines the meticulous construction planning, on the spot resolution of the problem and visualization of geological risks involved at every stage of design and execution.

1 INTRODUCTION

The Tehri Dam Project, a prestigious hydropower and pump storage plant, is located on the left bank of the Bhagirathi River in Uttarakhand, India, between 78°30'–79°00' E longitudes and 30°30'–33°30' N latitudes. The project site is about 82 km from the nearest railhead at Rishikesh. As part of the Tehri hydro power complex, it serves as a peaking power plant located between the Tehri and Koteshwar dams, with a combined installed capacity of 2400 MW, including Tehri Hydro Power Project, Koteshwar, and the 1000 MW (4x250 MW) Pumped Storage Plant (PSP), currently under construction.

Major components under construction include the Upstream Surge Shafts, Upper Chambers, Butterfly Valve Chamber, Penstock Assembly Chambers, Machine Hall, Downstream Surge Shafts, Tail Race Tunnels (TRTs), Outlet Structure, Bus Bar Tunnels/Shafts, and Access Tunnels. The outlet structure, critical to the Tehri PSP, requires safe cut slopes based on rock type and geotechnical parameters for its stability. Detailed geological and geotechnical investigations were conducted to design slope stabilization measures in phyllitic rock and areas with dumped material, which presented unexpected geological challenges. This paper outlines the system behavior in the Tail Race Tunnel outfall, verifying the effectiveness of the primary rock support through data from deformation monitoring instruments, excavation records, and quality control reports for the applied supports.

2 GEOLOGICAL OVERVIEW OF THE AREA

The Indian Himalayas present some of the most challenging geological conditions for infrastructure projects. The Tehri region is characterized by complex geological formations, including highly folded and faulted rock masses. The region is also seismically active, which adds another layer of complexity to tunnel construction. The geological strata in the area include phyllites, schists, and quartzites, interspersed with shear zones. The varying rock mass conditions required extensive geological mapping and in-situ testing to determine the most suitable tunnel alignment and support systems.

Excavation in rock included Phyllite, Quartzite Phyllite, and Argillaceous Phyllite. Logs from Adit 8 and 9, which run parallel to the proposed TRTs for a considerable stretch, and six drill holes in the TRT outfall area (three on each TRT alignment) revealed faults and thrusts that might influence tunnel alignment and outfall areas. A weathered zone on the firm bedrocks was identified to be approximately 20m thick, with the zone being relatively thin on the moderate to steep hill slope around the outfall area.

Tunnel	Geology	Joint Set Details
TRT-3	Aligned through a combination of PQM (Phyllite quartzite massive), PQT (Phyllite quartzite thinly bedded), QP (Quarzitic phyllite), and SP (Sheared/schistose (foliated) phyllites) rocks. 3D geological logs indicated the tunnel encountered PQT, PQT+PQM/PQM+PQT, PQT+PQM+SP, PQT+SP, and SP. Several shear zones were mapped, mostly aligned along foliation orientation, with varying thicknesses from minor to mega. These shear zones were clay-filled and wet to damp at places, causing cavity formation at some locations due to the intersection of shear zones	**J1**: N180°-210°/45°-55° (bedding) **J2**: N130°-165°/35°-45° (foliation) **J3**: N010°-030°/60°-65° **J4**: N270°-310°/60°-70°
TRT-4	Aligned through a combination of PQM, PQT, QP, and SP rocks. 3D geological logs indicated the tunnel encountered PQT+PQM, PQT+PQM+SP, PQT+SP/SP+PQT, PQT, PQM, and SP. Several shear zones were mapped, mostly aligned along foliation orientation, with varying thicknesses from minor to mega. These shear zones were clay-filled and wet to damp at places.	**J1**: N180°-210°/45°-55° (bedding) **J2**: N130°-165°/35°-45° (foliation) **J3**: N010°-030°/60°-65° **J4**: N270°-310°/60°-70°
TRT-Outfall Slope	The slope consists of PQM+PQT, QP with PQT bands, and SP, with SP occupying weak phyllite zones up to 1.5m thick. PQT+PQM are exposed near the TRT-3 portal and extend to the EA-08 portal. The slope dips SW (N225°), concave, and oblique downstream at 40°-45°. Shear bands and kink zones could cause planar failures. Overburden near the TRT-3 and TRT-4 portals, mostly colluvium, is less than 3m thick. The slope dips SW (N225°), concave, and oblique downstream at 40°-45°.	**J1:**N195°/45°-55° **J2**:N150°/35°-45° **J3**:N020°/60°-65° **J4**:N295°/60°-70°

Further rock mass in TRTs and associated structures was classified into five rock classes using RMR (Rock Mass Rating) geo-mechanical system. Assessment has been made using the information collected from the 3D geological mapping of the excavated reach of the TRTs which indicate that the TRTs mostly falls in three rock classes viz. Class III (Fair), Class IV (Poor) and Class V (Very Poor). Some of the rock types like PQT+PQM and PQT+PQM+SP falls in two rock classes Class III and Class IV.

PQT+PQM mostly fall in Class III, but it has been downgraded to Class IV in some places. Similarly, PQT+PQM+SP mostly falls in Class IV but sometimes it falls in Class III. PQT+SP also shows similar trend and is classified into two rock classes IV and V. Other rock types like QP+SP and SP falls in Class IV and V.

3 DESIGN METHODOLOGY

Planning and design decision are strongly influenced by the quality and quantity of Geological information available before construction, this information has a significant impact on controlling and minimizing geological risk during execution.

The two Tail Race Tunnels, TRT 3 (1081 m long) and TRT 4 (1174 m long), are connected to the Outfall /Intake structure, which is equipped with four sluice gates and trash racks.

3.1 TRT Outlet

The TRT Outfall of Tehri PSP is not a typical outfall structure, it is highly critical as it also functions as an intake structure during pumping mode. To accommodate the hydro-mechanical and electro-mechanical components at the outlet structure, safe cut slopes were maintained to ensure the proper functioning of the outfall structure.

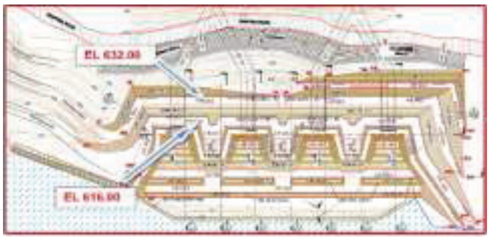

Figure 1. Plan view of TRT outfall cut slopes Figure 2. TRT outfall natural slope (i.e. before excavation)

Joints and Shear Zones:

Table 1. The shear strength properties of joints

Joints Details	Shear Parameter	Values
argillaceous fillings	Friction Angle	27°, - For J1, J2 and J3.
	Cohesion	0 kPa.
"dry" joints (no filling- no opening	Friction Angle	35°, For J4.
	Cohesion	0 kPa.
Shear Zones	Friction Angle	27°
	Cohesion	0 kPa.

3.1.1 Support system installed at TRT outfall area

Detail design analysis has been carried out to finalize the support system on TRT outfall slope using global and local stability in dry, wet, and saturated ground water conditions (Table) summarized the support recommendations for outfall slope.
It has also been recommended that all the activities for stabilization of slope to be carried out from top to bottom elevation with proper monitoring.

Table 2. Sequence of Support System recommended

S. No.	Support	Description
1	Slope Excavation	Above EL 616.0 m, the cut slope should be excavated in 1(H):3(V) slope, and below EL 616.0 m, the cut slope should be excavated in 1(H):5(V) slope.
2	Rock bolts and Cable anchors at main cut Slope face	8 rows of 120 Ton Capacity Cable anchors @ 2.0 x 2.0 m c/c spacing (staggered) above EL 608.0 m shall be provided. Apart from cable anchoring area, pre tensioned (with 12 Ton) fully grouted rock bolts of 310 KN and 12 m length @ 1.5 x 1.5 m c/c spacing shall be provided.
3	Rock bolts at downstream and upstream cut slope face	Pre tensioned fully grouted rock bolts of 310 KN and 12 m length @ 1.5 x 1.5 m c/c spacing shall be provided.
4	Shotcrete at all slope face	100 mm thick shotcrete with 1 layer wire mesh shall be provided.
5	Drainage System	Drainage holes of 76mm diameter shall be provided (inclined upwards by 200) to avoid rock mass saturation near slope face.
6	Monitoring of Slope	Proper monitoring of the slope is very essential during as well as after the construction and shall be provided based on-site conditions.

3.2 Tail race tunnels

Considering complex Geological conditions, bigger diameter of tunnel and minimize excavation influence of one tunnel to another parallel tunnel it was planned to excavate both tunnels with heading and benching method.

As per Geological model of preconstruction phase, it was assessed that maximum portion of tunnel will cross with Fair to poor rock class and few intermittent zones of very poor rock class of Sheared Phyllites rocks. Therefore, support system designed considering following parameters for different rock units.

3.2.1 Support system installed in TRT

Table 3. Support System recommended as per rock class

Sr. No.	Rock Class	Support
1	Class-III	25mm diameter, 6m long Rock bolts at 1.5 m c/c spacing along with 100 mm thick wire mesh reinforced shotcrete.
2	Class-IV	25mm diameter, 6m long Rock bolts at 1.0 m c/c spacing along with 150 mm thick wire mesh reinforced shotcrete.
3	Class-VA	ISMB 350 @ 0.4 m c/c along with 50 mm thick shotcrete in crown. 32mm diameter, 6m long Rock bolts at 1.0 m c/c spacing along with 200 mm thick wire mesh reinforced shotcrete in walls.
4	Class-VB	ISMB 350 @ 0.4 m c/c along with 50 mm thick shotcrete in crown. 32mm diameter, 6m long Rock bolts at 1.0 m c/c spacing along with 200 mm thick wire mesh reinforced shotcrete in walls combined with a 7 m thick pre-grouting/consolidation grouting annulus. Alternately, ISMB 500 @ 0.5 m c/c along with 50 mm thick shotcrete can be provided without grouting annulus.

Parametric analysis carried out of all geo tech parameters obtained from in situ and lab tests results and a range of design parameters determined for each rock segment to analyse behaviour of tunnel and design suitable support system. Conservative design adopted based on worst expected geotechnical parameters of rock mass of tunnel. Support types and quantity adjusted and revised according to recorded deformation and potential failures mechanism of rock mass.

4 GEOLOGICAL RISKS IDENTIFICATION

The Tehri PSP project is situated in a geologically complex region characterized by a folded sequence of low-grade metamorphic rocks, including phyllites, quartzites, and schistose phyllites. The presence of major tectonic features such as the Shrinagar Thrust, along with minor faults and shear zones, posed significant geological risks during excavation. Detailed geological investigations, including surface mapping, drill holes, and exploratory drifts, revealed the presence of various lithological units and identified potential hazards such as:

- **Sheared/Schistose Phyllites (SP)**: These rocks, often associated with fault zones, presented weak and highly fractured conditions, leading to increased risk of tunnel collapses/excessive deformations.
- **Fault Zones and Shear Zones**: The identification of these zones was crucial as they were likely to be water-bearing and could cause unexpected inflows during excavation, leading to potential stability issues.
- **Cavity Formation**: Particularly in TRT-3 and TRT-4, cavities were observed due to the intersection of shear zones, which created voids that posed a risk of sudden ground collapse.

5 DESIGN OPTIMIZATION AND UPDATING

Given the complex geological conditions encountered at the Tehri PSP site, the design of the tunnels and other underground structures required continuous optimization. The key aspects of this process included:

- **Rock Mass Classification and Support Design**: The rock mass was classified into different support classes (Class-III to Class-VB), with each class corresponding to specific rock support measures based on the anticipated geological conditions. For example, areas with more stable rock

required lighter support (Class-III), while highly fractured or sheared zones required heavier support, such as ISMB 350 or ISMB 500 beams, combined with shotcrete and rock bolts (Class-VA and Class-VB).
- **Dynamic Updating**: The design was updated to reflect the real-time data as the excavation progressed and actual geological conditions were encountered. For example, where shear zones were thicker or more extensive than initially expected, the support system was enhanced to ensure safety and structural integrity.
- **Tunnel Alignment Adjustments**: The alignment of tunnels like TRT-3 and TRT-4 was carefully adjusted to navigate through more competent rock and avoid major fault zones where possible. This helped in minimizing the risks associated with weaker rock formations.

6 MEASURES TO REDUCE RISKS

Several measures were implemented to mitigate the geological risks identified during the Tehri PSP project:

- **Advanced Geological Investigations**: Extensive drilling and exploratory adits were used to map the geological conditions ahead of the excavation front. This proactive approach allowed for the identification of problematic zones before they were encountered during tunnelling.
- **Pre-Grouting and Consolidation Grouting**: In areas where weak zones or water-bearing strata were identified, pre-grouting and consolidation grouting were employed to strengthen the rock mass and reduce water inflow. For instance, in the Class-VB support areas, a 7m thick pre-grouting/consolidation grouting annulus was used to stabilize the ground before excavation.
- **Enhanced Support Systems**: The implementation of robust support systems, such as the use of ISMB 350 or ISMB 500 beams combined with thick layers of shotcrete and closely spaced rock bolts, provided additional stability in zones of weak rock. This reduced the risk of collapses and deformations, particularly in shear zones and areas with high joint densities.
- **Water Management**: Proper drainage systems, including perforated PVC pipes and collector drains, were installed to manage water ingress effectively. This helped in preventing the build-up of hydrostatic pressure, which could otherwise destabilize the rock mass and increase the risk of failures.

7 CONSTRUCTION PLANNING

This paper provides an overview of the construction procedures for the Twin Tail Race Tunnels (TRT 3 and 4) and the TRT Outfall structure, emphasizing detailed planning. Direct excavation from the TRT outlet was impractical due to the time required for full excavation and rock support. Instead, excavation was initiated through ADIT EA-7, accessed via ADIT-9. ADIT EA-7 connects both TRT tunnels, allowing work to proceed towards the downstream surge and TRT outlet simultaneously. According to the schedule, both TRT tunnels were to be excavated up to the bifurcation from ADIT EA-7, with the outlet excavation starting after portal development.

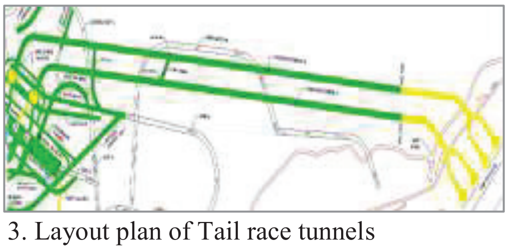

Figure 3. Layout plan of Tail race tunnels

Figure 4. Layout plan Access Road to TRT Outlet

7.1 Construction methodology for excavation of TRT

The Tail Race Tunnel (TRT) excavation employed advanced equipment and techniques for safety and precision. It began with Access Adit EA-7 using a two-boom drill jumbo for drilling and an excavator for scaling. Muck was removed by loaders and dumpers, with support systems applied per Good for Construction (GFC) drawings.

The main TRT excavation had two stages: heading and benching, using drilling, blasting, and support treatments. Rock bolts, shotcrete, and ventilation ensured stability, while surveying ensured alignment. Equipment like a two-boom drill jumbo, ROC-202, jackhammers, and a shotcrete machine was selected for the task.

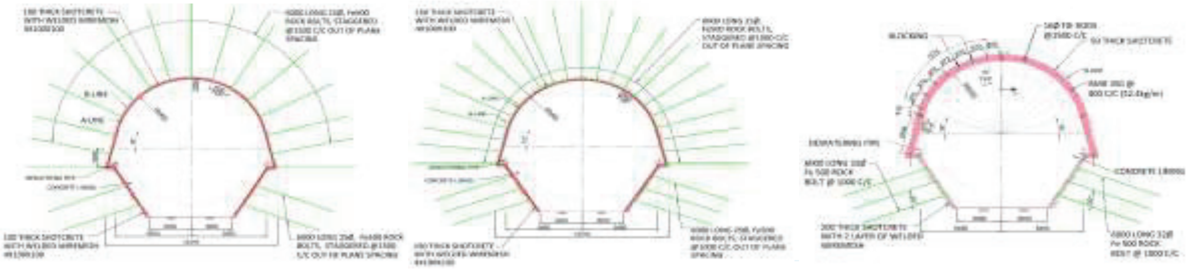

Figure 5. Showing Rock support in Fair, Poor and Very Poor Rock Class

7.2 Construction methodology for excavation of TRT outlet

The TRT outlet excavation began with preparing the area from EL 660.00 down to the existing road level. The process included installing rock bolts, drainage holes, and a 500mm thick cladding wall, which required a level base with leveling concrete. Staging was used for drilling grouted rock bolts (25mm diameter) for stability.

Excavation below EL 660.00 proceeded in bench-wise steps from EL 660.00 to EL 608.00. Each step involved cutting the slope in 2 to 3-meter increments and installing rock supports based on geology. Adjustments to rock bolt orientation and location were made as needed. Gravel-filled perforated PVC pipes were installed in drainage holes for effective water drainage.

From EL 624.00 to EL 610.00, cable anchors were installed as per the design. These cable anchors had a capacity of 120 tonnes, a length of 20 meters, and were placed at 2-meter intervals, staggered, and inclined at 10 degrees downwards. This anchoring system ensured additional stability in this section.

Slope supports at Face:

Elevation		Shotcrete	Wire Mesh	Rock Bolts		Drainage Holes	Cable Anchors
From	To			Type	Length		
660.00	644.00	100mm	1 layer	350 kN 32mm dia Pretensioned grouted rock bolts	12m	From EL 660 up to EL 617 - 76mm dia. 10m long 20 degree inclined upwards	
644.00	632.00	100mm	1 layer		12m		
632.00	616.00	100mm	1 layer		12m		From EL 624 up to EL 610 - 120 Tonne 20m long 10 degree inclined downwards
616.00	608.00	100mm	1 layer		12m		
608.00	598.00	100mm	1 layer		12m		

Slope supports at Face:

Elevation		Shotcrete	Wire Mesh	Rock Bolts		Drainage Holes	Cable Anchors
From	To			Type	Length		
660.00	644.00	100mm	1 layer	350 kN 32mm dia Pretensioned grouted rock bolts	12m	From EL 660 up to EL 617 - 76mm dia. 10m long 20 degree inclined upwards	
644.00	632.00	100mm	1 layer		12m		
632.00	616.00	100mm	1 layer		12m		From EL 624 up to EL 610 - 120 Tonne 20m long 10 degree inclined downwards
616.00	608.00	100mm	1 layer		12m		
608.00	598.00	100mm	1 layer		12m		

Table 4. Rock supports at different levels

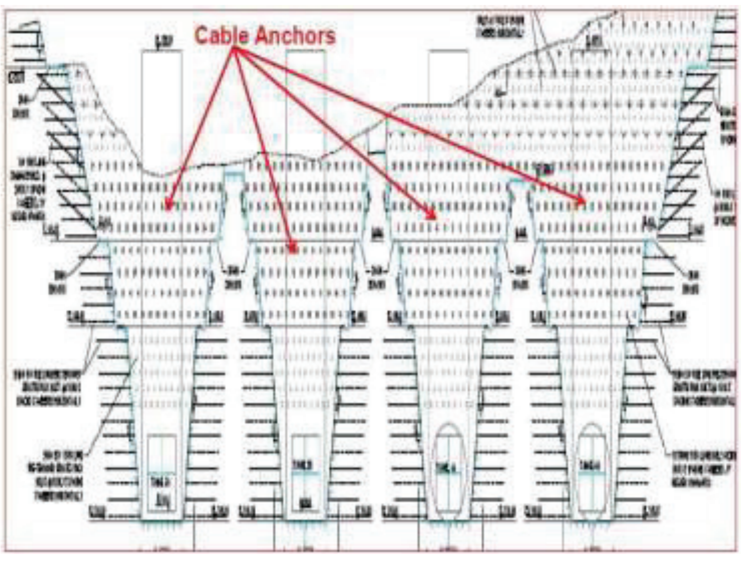

Figure 6. Rock supports at different levels

To manage water drainage effectively, collector drains of 300mm x 300mm were constructed at each berm level. These drains had a longitudinal slope of 1:100 to direct water away from the excavated area. Finally, access roads were constructed to reach the Gate Operating Platform at EL 633.50 and the Trash Rack Top at EL 616.00, ensuring proper access to these critical areas during and after construction.

8 CHALLENGES ENCOUNTERED AND INOVATION DONE DURING EXECUTION

8.1 Cavity formation during excavation in TRT-3 and TRT-4

During the Tail Race Tunnel excavation, heavy water ingress in rock class IV at RD 737.0 (TRT-3) and RD 730.0 (TRT-4) caused significant crown flow. To address this, sacrificial steel fiber reinforced shotcrete, rock anchors, and steel ribs were applied, but cavities up to 15 meters high formed. To stabilize the rockfall, excavation faces were filled to the crown level, and bulkheads with fore poles were installed to seal openings for backfill concrete. Pipe roofs (76 mm diameter, 12 meters long with a 4-meter overlap) were installed and grouted at the crown. Excavation resumed with immediate remedial measures to prevent further complications and chimney formation.

Mitigation measures taken:
During excavation, both tunnel faces advanced 15 meters from the initial cavity zone before halting for mitigation. Backfill concrete was placed 848 cubic meters in TRT-3 and 660 cubic meters in TRT-4. Despite this, voids between the concrete and rock led to a new cavity forming 15 meters above the filled area. To address these voids, contact grouting with cement mortar mixed with sand was performed. Drill rigs created 76 mm diameter holes averaging 19 meters in length, using 9,492 bags of cement and 74.55 metric tons of sand. After grouting, excavation resumed with RIBs and pipe roofs until reaching improved rock conditions.

8.2 Challenges in slope stabilization at EL 660.0 m

During excavation from EL 660.00 to 646.00 m, unexpected geological conditions led to ineffective shotcrete and soil nails, causing instability and wastage. A revised method using Non-Woven Geo green Erosion Control Blankets, High Tensile Rolled Cable Nets, and 12.0 m deep Self-Drill Anchors was implemented. However, the steep 75° slope and persistent flow of sand and boulders continued to challenge the support system. Heavy rains on August 18, 2019, worsened the situation, causing a cavity and damaging protective work, which created unsafe conditions.

Figure 7. RBM and Muck Fill Deposits and Loose fall behind erosion control blanket forming Cavity upto EL 660.0 m.

Mitigation measures taken:

To address the issues, remedial measures included applying reinforced cladding to stabilize the slope, extending drainage holes to 15.0 m, and reducing rock bolt spacing to 2.0 m. After cladding, 12.0 m deep rock bolts and 15.0 m drainage holes were installed for better water management. Ongoing slope protection work from RD 49.00m to 87.00m continued, with additional support measures proposed for saturation-related failures between RD 60.00m and 70.00m. These included installing 32 mm diameter SDR rock bolts, 12.00 m long, to enhance stability and safety.

8.3 Challenges in constructing concrete outfall structure on weak ground

The construction of the TRT outfall structure at Tehri PSP encountered challenges due to weak ground conditions, particularly PQT and SP rock types with low UCS (<25-40 MPa). The poor load-bearing capacity and instability made conventional reinforcement ineffective.

Mitigation measures taken:
Micro-piles were used to transfer loads to deeper, more stable strata. A total of 560 micro-piles were installed in a staggered pattern, spaced 3 meters apart, with depths of 8-10 meters depending on the rock quality. Casings were used during drilling to prevent hole collapse. The design and installation of

micro-piles were closely monitored and adjusted, effectively stabilizing the ground and supporting the heavy structure.

8.4 Challenges due to the flood protection wall placement at outlet structure

The TRT outfall works faced significant challenges due to the flood protection wall and access road. The TRT bottom was at EL 589.5 m, while the reservoir level of the 1000 MW Hydro Power project was at EL 612.0 m. To prevent water ingress, the client built a coffer wall at EL 622.0 m. However, the flood protection wall encroached into the construction area, requiring dismantling down to EL 616.0 m to complete the end wall and piers. The access road also posed a challenge. Completely removing it would restrict manpower and machinery movement, while keeping it in place would obstruct construction. Strategic planning was needed to balance access with construction requirements. These constraints are illustrated in the picture below.

Mitigation measures taken:

To address the challenges at the TRT outfall, a 3-D model of the outlet structure and flood protection wall was created, and a detailed excavation methodology was developed. The slope portions of the flood protection wall hindering construction below EL 616.0 m were excavated, and sacrificial shotcrete with wire mesh was applied to maintain stability. The access road was strategically repositioned, with a 600 m stretch excavated from EL 670.0 m to the bottom of the structure. This allowed 75% of the structure to be clear of obstructions, with only 2-3 m of the final portion requiring staged concreting.

With these obstacles resolved, the concreting of the TRT outfall progressed using a lift-wise approach to reinforcement and boom placers for precise concrete placement in difficult areas. This method ensured safe and efficient completion of the TRT outfall works at Tehri PSP.

9 INNOVATIONS DONE

9.1 Optimized Concreting Sequence for TRT 3 & 4: Invert First Approach

The construction of TRT 3 and TRT 4 at Tehri PSP faced challenges due to multilayer reinforcement in Rock Classes IV and V, making traditional concreting methods impractical. To overcome space constraints and the complexity of handling up to five reinforcement layers during kerb casting, an invert-first approach was adopted.

This method involved cleaning, reinforcing, and concreting the invert first, followed by installing the overt lining gantry and completing the overt section. This approach simplified reinforcement placement, optimized space usage, and improved workflow, leading to a safer and more efficient construction process for TRT 3 and TRT 4.

Figure 8. Invert and Overt Rebar and Gantry Shutters

9.2 Deployment of tower crane to execute the concreting works of TRT Outlet structure

During the concrete works at the outlet structure, a boom placer at EL 616.0 m could not reach the bottom raft and pier portions, requiring additional equipment. Two tower cranes were proposed, and after client approval, they were installed with the client covering the extra costs. The tower cranes significantly improved the transportation of reinforcement, shuttering, and concrete to the lower sections, leading to the successful completion of the structure. Once finished, the flood protection wall was dismantled, and the debris was cleared from the site.

10 CONCLUSION

The construction of the Tail Race Tunnel (TRT) and outfall structure at Tehri PSP faced significant engineering challenges due to complex geological conditions. Advanced excavation techniques, including drilling, blasting, rock bolts, and shotcrete, were used for stabilization, with careful excavation from Access Adit EA-7 to the main tunnel.

Challenges like cavity formation and water ingress required innovative solutions such as sacrificial shotcrete, rock anchors, pipe roofs, and grouting. Weak ground conditions at the TRT outlet necessitated the use of micro-piles for stabilization.

Innovations like the invert-first approach and the use of a tower crane improved efficiency. Rigorous safety measures and medical support ensured worker safety throughout. The project showcased advanced excavation and reinforcement techniques, setting a benchmark for future geotechnical engineering projects in challenging environments.

DESIGN & CONSTRUCTION OF TUNNEL IN EXTREMELY POOR GEOLOGY IN SHALLOW OVERBURDEN : CASE STUDY OF TUNNEL FOR A PSP IN INDIA

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ABSTRACT

Construction of underground tunnel portal in shallow rock cover in fragile geological formations possess many geotechnical challenges. Managing these challenges often requires support of numerical and analytical approaches. This paper presents a case study of Main Access Tunnel (MAT) of 9 m finished D shape, for a pumped storage project in India, which is under construction. The tunnel was to be made in extremely poor rock class with shallow rock cover which is about 4.5 m. The geological strata consisted of highly weathered shale, characterized by its heterogenous and anisotropic nature. Rock mass properties of shale also adversely affected under saturated condition. Numerical analysis of the tunnel was carried out, with regular site mapping and field test results of the rock mass. Numerical modelling techniques are effectively used for understanding the rock mass behaviour. The tunnel has been excavated using conventional method of tunnel with short round lengths and cautiously moving ahead. The conventional support system in the form of pipe roofing, SFRS, rock bolts and steel ribs have been provided. The support system was reviewed timely, based on strata encountered ahead of the tunnel face and modified as per site conditions. The tunnel was also provided with adequate in-strumentation for monitoring purpose which comprises of MPBX, Inclinometer, targets and load cells. This paper highlights the case study of design and successful completion of the initial reach of 40 m length tunnel in poor geology and shallow overburden, with conventional tunnelling techniques.

1 INTRODUCTION

1.1 *General*

As a part of green India initiative, green energy will play a critical role as a part of decarbonizations. Pumped storage schemes are the game changers for clean energy initiative. One of the pumped storage projects in India with an installed capacity of 1440 MW is in construction stage and during execution, the challenges encountered in the construction of the main access tunnel (MAT) are presented in this paper. This tunnel was provided for transiting Electro-Mechanical equipment to the Power house, which is seated inside a 85m deep pit, and was very important to facilitate erection works inside the powerhouse and it's functioning.

Tunneling in weak rock mass presents special challenges to the geotechnical engineer since misjudgments in the design of support systems can lead to under-design and costly failures or over-design and high tunneling costs. In order to understand the issues involved in the process of designing support for these tunnels, it is necessary to examine some basic concepts of how a rock mass surrounding a tunnel deforms and how the support system acts to control this deformation. The excavation of a tunnel causes the deformation of ground and formation of plastic zone from some distance ahead of the tunnel face which has been illustrated by Hoek (2001), Vlachopoulos and Diederichs (2009). When the excavation is through poor geological conditions, minimizing the deformations ahead of the face is important in terms of tunnel safety as well as limiting the surface settlements. Usually, the tunnel supports such as shotcrete or rock bolts are installed behind the tunnel face and does not help in reducing the ground displacements ahead of the face, which is critical. In such cases, pre-excavation support measures are generally adopted. In the present case study, portal of main access tunnel of a pumped storage project is described which was excavated in poor geology with very low cover of 4.5 m. The pre-excavation support system in the form of pipe roofing, rock bolt, GFRP, SFRS, steel sections and gradually taking smaller pulls were recommended. Rock face after each pull was critically examined, geologically evaluated before taking next blast and thus tunnel initial reach was successfully executed.

1.2 *Project Background*

The pumped storage project envisages creation of new Upper Reservoir, Water Conductor System, pit Powerhouse and Tail Race Channel. The scheme involves the construction of about 6.3km long Geomembrane Faced Rockfill Dam for creation of upper reservoir with 1.9 TMC gross capacity. The lower reservoir is an existing reservoir with 258.47 TMC gross capacity. The scheme envisages utilization of 121.15 m rated head and design discharge of 1368 cumecs for generation of about 1440MW. The project comprises following major components:

- Power intake structure.
- Buried Penstock
- Vertical & horizontal pressure shaft and
- Power House
- Tail Race Tunnel
- Lower Intake/ Outlet structure
- Main Access Tunnel (MAT)

Main Access Tunnel of length of 526 m of D shape of 9 m x 9 m was proposed near road to powerhouse till the service bay level with a longitudinal gradient of 1 in 17.10. The rock cover over the MAT portal varies from 4.5 m at the portal and up to a maximum of 25 m. Geological conditions were very challenging as rock type was weak shale. The plan of MAT portal presented in this paper is shown in Figure 1.

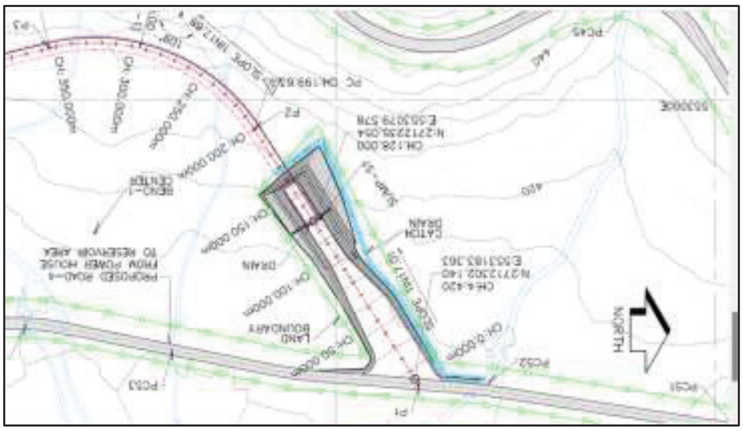

Figure 1 Plan showing MAT Inlet Portal

2 GEOLOGICAL SETTING

2.1 *General*

MAT is aligned along north to south direction. MAT portal is explored by 20 m deep drill hole, bored from EL 414.73m which encountered 6.78 thick overburden material having sandy soil fragments of Quartzitic sandstone and shale rock. Further down this hole has encountered choco-late colored shale, which is moderately weathered, closely jointed rock. The core recovery was 80-95% in the shale portion but the RQD was NIL throughout the length of the hole indicating closely/thinly bedded shale rock. As per investigation data, the portal was located in poor to very poor rock conditions.

Due to unavailability of alternate location this was the only location for MAT. The tunnel was expected to negotiate through weak, thin to medium thickly bedded and jointed shale rock throughout its length. The cover along MAT varies from 4.5 m near the portal to about 25 m (maximum) cover towards the junction with bottom penstock. The condition of shale rock mass was expected to be in improved condition with increase in length from the portal end. During investigation stage to negotiate about 90% length of class-IV, 10% of class-V rock as per RMR method.

As per bore holes study of Powerhouse area at this location, it was interpreted that the anticipated rock would be comprising shale and shale with sandstone interbedding (generally with 5 mm to 20 mm). The shale is moderately to highly weathered, weak, fractured in the first 5 m to 20 m depth from the ground level in most of the holes, olive green and chocolate colored, weak with interbedding of light grey to greyish white colored, fine grained, weak rock. With increase in depth, the condition of the shale improves compact and medium thick to thick, gently dipping bedding planes. Figure 2 shown geological profile along with rock class during construction.

Details of Drill hole:
- Overburden Depth: 6.63 m
- Bed rock: Thinly bedded weak shale
- Lugeon value: 6.63-12.42

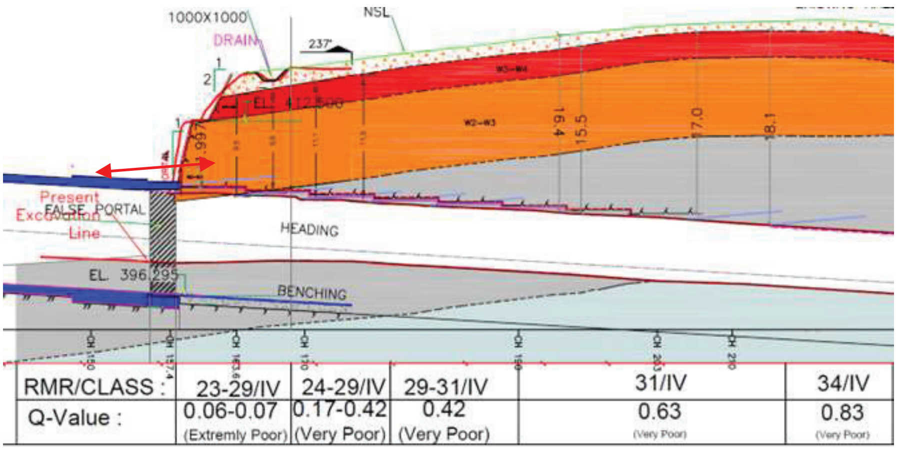

Figure 2 Geological profile (Construction stage)

Table 1. Details of rock joint sets at the MAT portal

Joint	Altitude of Discontinuity		Joint Parameters			
	Dip	Direction	Persistence (cm)	Spacing (mm)	Roughness	Infilling
S0	270	6	2-15	1-6	SR-RP	None
S1	130	80	0.5-5	5-10	SR-RP	Soft
S2	076	63	0.5-2	7-20	SR	None
S3	240	65	0.5-2	15-30	RP	None

2.2 *Geological parameters*

When the tunnel is shallow or within superficial deposits, soft ground tunneling technique such as Excavation and Lateral Support System (ELS), Tunnel Boring Machine (TBM) or mechanical excavation with forepoling and face support is normally adopted. In this case, design governing criterion will normally be settlement controlled rather than stresses induced.

The design is normally based on empirical rule and numerical modelling and observation behavior during excavation stage. As the rock mass was highly to moderately weathered equivalent continuum modelling was best suited and structural discontinuity based failure was ruled out.

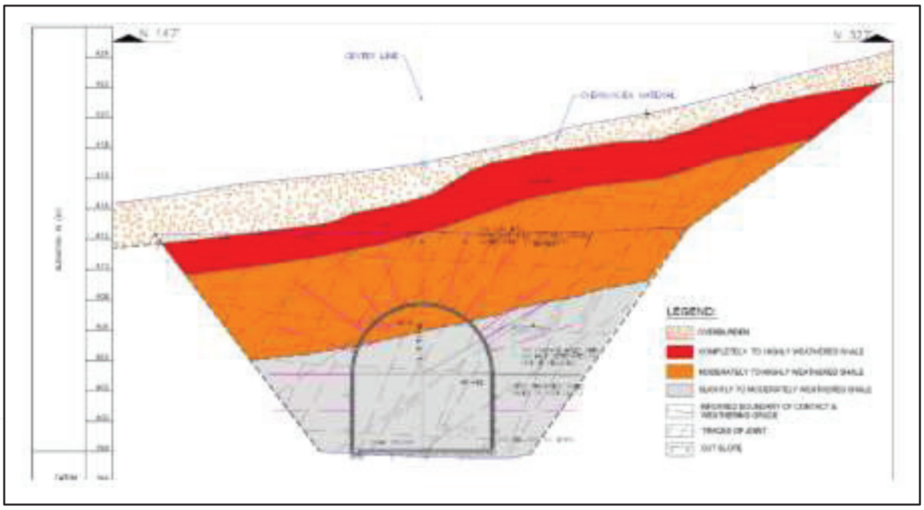

Figure 3 Geological face mapping data

Figure 4 Figure showing Portal Location before excavation

From Figure 3 and 4 it was observed that MAT inlet portal lies in Moderately Weathered (MW) to Highly Weathered (HW) along with Slightly weathered (SW). For SW- MW RMR value was given greater than 28 (Class IV) and for HW-MW it was 20 with Q value 0.07 (Extremely poor). From geological section it was observed that majority of the portion is in SW to MW shale. However, being conservative considering the thinly bedding of shale and low cover zone, the RMR of 20 was adopted or both MW-HW and MW-SW weathered shale. Face mapping data was complimenting the drill hole data and based on face mapping data, design was carried out.

3 NUMERICAL ANALYSIS

3.1 *Model geometry*

The model geometry was considered same as the geological profile given in figure 5 for the minimum cover. The finite element numerical analysis was carried out in RS2, a software by Rocscience. Six noded uniform mesh was used to finite element modelling. The RS2 model is shown in Figure 6. The lateral boundary constraints were kept sufficiently far in order to take into account far field stresses. The vertical boundaries were allowed to deform in downward direction and restraint horizontally. The bottom most boundary was kept fixed. RS2 model is presented in Figure 5 and Figure 6 showing various stages of numerical modelling.

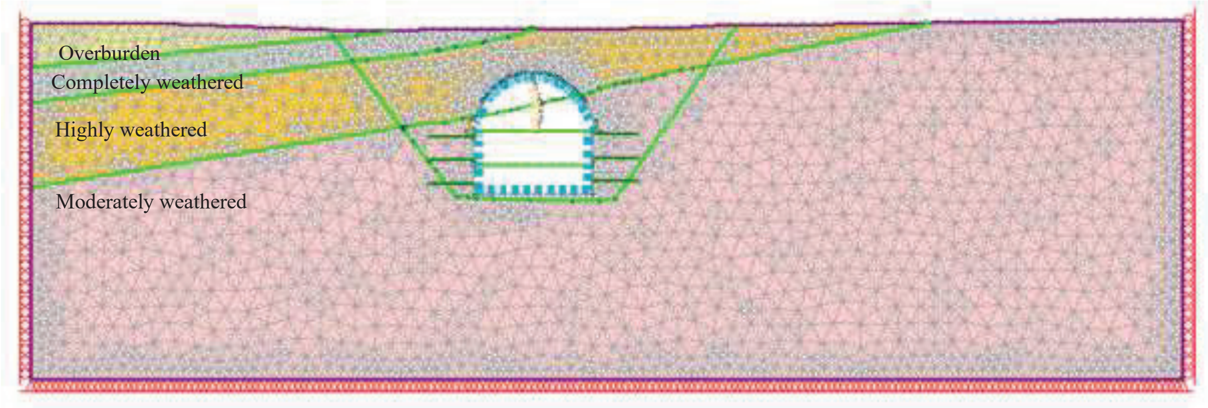

Figure 5 RS2 model for MAT portal

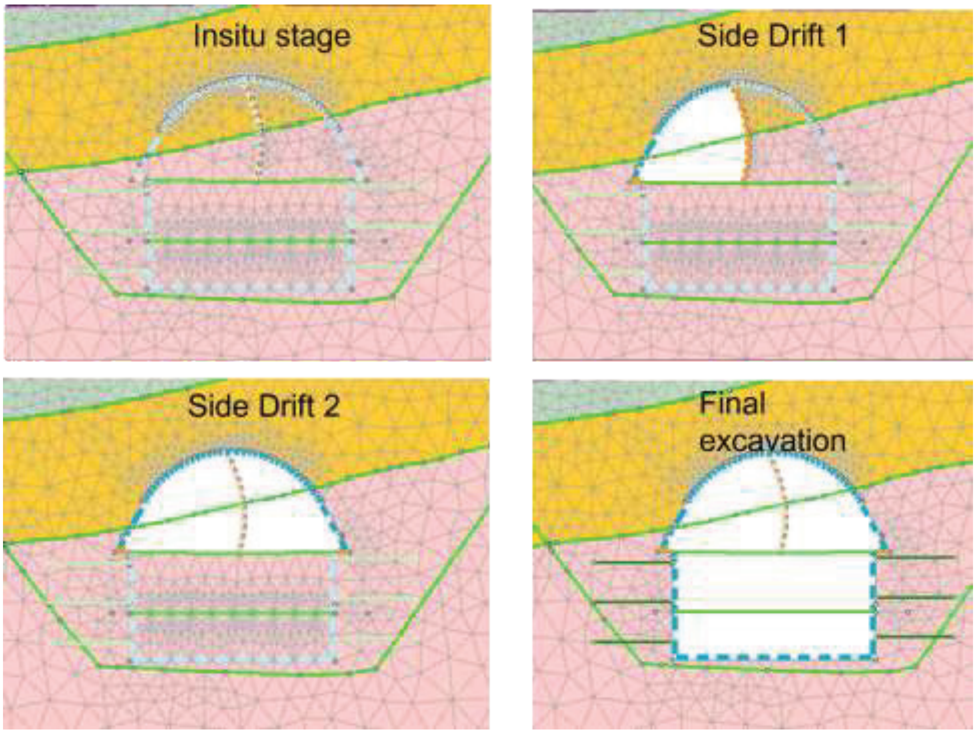

Figure 6 various stages of FEM analysis

The primary support estimation was done using Indian standard guidelines based on roof pressure calculation and requisite amount of support pressure. However, these methods have limitations as these assume hydrostatic stress field around the circular openings. When support types are combined, the total available support pressure can be estimated by summing the maximum allowable pressures for each system. However, in making this assumption it must be noted that these support systems do not necessarily act at the same time and that it may be necessary to check the compatibility of the systems in terms of deformation and stresses developed. However, since it only gives an estimate of the final support capacity, it cannot be used to investigate the details of the excavation and support installation sequence required to deal with very difficult tunnelling problems such as those under consideration in this paper. Hence stage numerical analysis to study the stress deformation behavior is of utmost importance.

3.2 *Geotechnical rock mass properties*

It was evident from the geological mapping that the portal majorly falls in Highly to Moderately weathered fine closely spaced shale. The GSI quantification varied from 18-25 and Q value 0.07. Although the rock mass below springing line of tunnel was marked as slightly weathered, for design, the entire tunnel was considered with GSI quantification of 20. Generalized Hoek Brown criteria was used as constitutive model for numerical analysis. The primary inputs for this constitutive model are unconfined compressive strength (UCS) and Geological strength index. As the sample for UCS was difficult to make owing to the thinly bedded shale, two sets of point load tests were carried out from available samples and the value was, 8.26 MP and 10.24 MPa. As the sample value is to be applied to the whole mass, it was decided to go for least value. For Completely weathered material, GSI was reduced to 10 and UCS was considered from Hoek and Marino's literature for very weak rock as 2.5 MPa. As this material was above the crown, its influence was on the settlements above tunnel face.

In tunneling through weak ground, it is generally accepted that the stability of the face depends upon the area exposed. Consequently, one commonly used technique for maintaining stability is partial face excavation in which the tunnel is driven in stages such that the area of each face is small enough to control. In this case top heading was excavated in two drifts with temporary wall supported with SFRS and GFRP's while maintaining sufficient lag between the faces. Excavation of the bench was further carried out to form a complete profile. It was recommended to maintain the round length within 1 m until the initial reach was completed. The in-situ stress field is hydrostatic, in other words the stresses are the same in all directions. This is a reasonable assumption for very weak rock as it is incapable of sustaining significant stress differences. Hence, even if the far field stresses are asymmetrical, the stresses within the weak rock are likely to be approximately hydrostatic. Geotechnical parameters considered in design are tabulated in Table 2.

Table 2. Geotechnical properties

Parameter	HW-MW Shale	
	Unit	Value
Density	kN/m3	25
UCS	MPa	8.26
GSI		19
Intact Modulus	MPa	1000
Poisson's ratio	MPa	0.25
Deformation Modulus	MPa	71.3
Hoek-Brown	mi	6
mb		0.332506
s		0.000123
a		0.546749

3.3 *Results*

Preliminary and unsupported analysis was carried out and the model failed to converge and due to poor strength of rock mass, full yielding was observed. From the RS2 analysis and results, it is inferred that due to poor geological conditions, it is safe to excavate the MAT portal area in two drifts. Support was installed sequentially in drift of heading followed by benching and invert closure. Maximum displacement from numerical analysis was 60 mm (about 0.5 % of tunnel diameter) and maximum extent of plastic zone was 2.8 m. The support capacity plots were plotted for the Steel ribs section WPB 300 and were found to be within permissible limits for safety factor of 1.4. Results of numerical FEM analysis are presented in Figure 7 (Plasticity developed) and Figure 8 (Total displacements). The constructed portal is also shown in Figure 9.

Support measures adopted and executed:

- Mechanical excavation was proposed
- Two side drifts with shorter pull of 0.5-0.8 m with advance in top heading and excavating the side gullets first. The lag between the drifts were maintained as 5 m at least.
- Pipe roofing with 114 mm dia, 6 mm thick, 9 m long @ 500 mm c/c spacing with minimum overlap of 3 m, ahead of the face before advancing for face stabilization, inclined at angle of 5 degrees with the horizontal. Pipe roofing was calculated using analytical beam model approach.

- Protecting the face with GFRP of 4 m long, seven (7) numbers (350 kN) and sealing shotcrete of at least 50 mm thick.
- Enlarged elephant footing was proposed for load transfer
- Spot bolting to arrest any local wedge failures, and 25 mm dia. Rock Bolt 4 m long @ 2m c/c spacing in the wall below springing line.
- WPB 300x100.84 @ 400 mm c/c spacing fully embedded in SFRS
- 150 mm thick SFRS should also be provided at the temporary side drift wall as soon as the excavation is done.
- 76 mm dia. 6 m long drainage holes during construction stage to release any excess pore pressure.
- Settlement pins, extensometers and inclinometers were installed on the top of the tunnel to keep a check for deformation and load response (as shown in Figure 10)

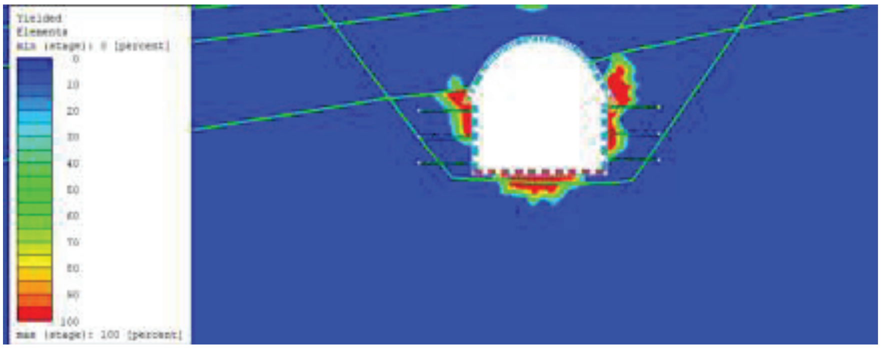

Figure 7 Plastic elements plot

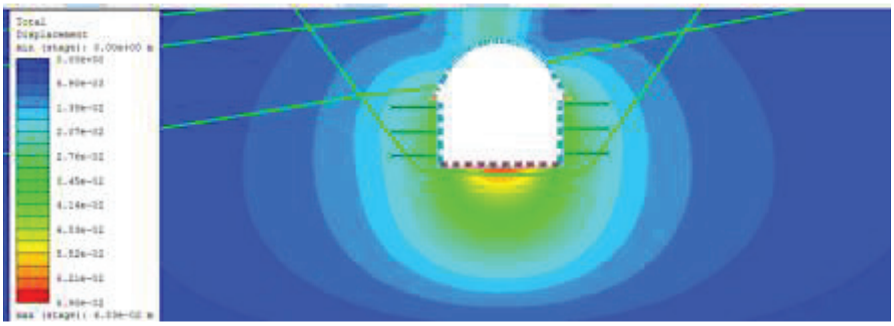

Figure 8 Total displacements

Figure 9 Constructed Tunnel Portal

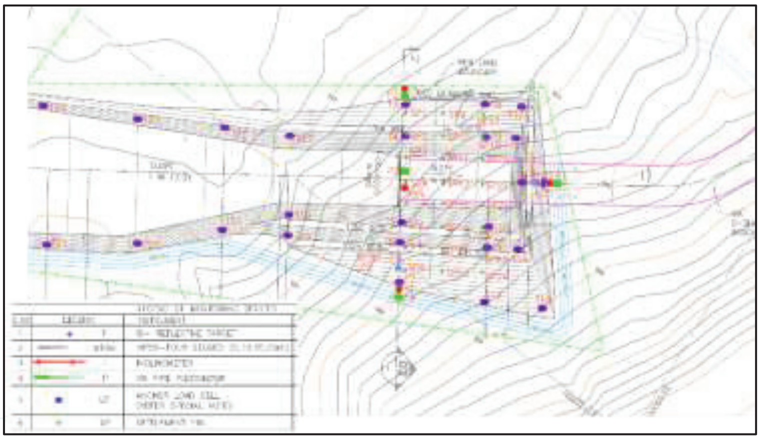

Figure 10 Monitoring and Instrumentation Plan at the MAT portal

3.4 *Conclusions*

The various challenges encountered during tunnel portal excavation in shallow overburden in shale rock type were described in this paper. Several alternative methods for maintaining stable face and tunnel stability were implemented before further advancing the tunnel. Despite challenging geology and low cover, the excavation was completed successfully by following methodological approach of taking small pulls, installation of support closer to the face temporary support measures and partial excavation. The finite element based numerical analysis are great aid to modern tunnelling requirements along keeping the check on heath of tunnel by means of monitoring regularly. This shows that tunnel advance rates under such conditions seldom exceed about 1 m per day. To save time and money and negligence in adopting inadequate solutions invariably may lead to even costlier failures.

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RS2: Elasto-plastic Finite Element stress analysis program for underground or surface excavations in rock or soil.

IMPACT ASSESSMENT OF TUNNEL CROSSING IN CHENNAI METRO USING FINITE ELEMENT SIMULATION

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ABSTRACT

This paper presents a case study of a tunnel crossing in Chennai Metro Rail Phase 2, where the new alignment intersects perpendicularly with the existing Phase 1 tunnel near an underground station. The analysis addresses a critical condition in which both tunnels are positioned one above the other, making assessment of soil–structure interaction essential for safe operation. Three-dimensional finite element modelling was carried out using PLAXIS 3D to evaluate the combined influence of tunnel excavation and train movement loads on both tunnel systems. The numerical framework incorporated construction sequences, site-specific geotechnical parameters, and operational scenarios with and without dynamic train loading. Through these simulations, the study quantified ground settlements, tunnel deformations, and lining stresses at the crossing, identifying the zones most affected by excavation and movement. The results highlight how sequence of construction and load application directly influence tunnel behaviour and overall stability. Beyond addressing this project-specific challenge, the study provides a structured methodology for analysing and simulating similar critical scenarios in urban environments. By integrating excavation effects with moving train loads, it establishes a reference framework for engineers and planners to predict tunnel–soil–structure interaction and ensure reliable underground transit development.

Keywords *: Metro rail, tunnel crossing, PLAXIS 3D, soil–structure interaction, finite element analysis*

1.0 Introduction

The rapid expansion of urban metro systems has become a defining feature of modern cities seeking sustainable, high-capacity transport solutions. In India, where metropolitan regions are experiencing simultaneous growth in population, land-use intensity and private vehicle ownership, underground metro rail has emerged as a vital response to increasing congestion and environmental pressures. Because these systems are capital-intensive and time-consuming to build, they are typically developed in multiple phases. Once the first corridors become operational and attract substantial ridership, subsequent phases are planned to extend coverage and improve network connectivity.

This phased approach, while operationally and financially pragmatic, introduces a new generation of underground design challenges. Future alignments must weave through complex subsurface environments and often pass very close to, or even cross, existing operational tunnels, especially in dense urban corridors. In such situations, the key question is simple but critical: can we build and operate the new tunnels without compromising the safety and serviceability of the existing line? Answering this requires a realistic assessment of soil–structure interaction and a clear understanding of how excavation, volume loss and train loads from both lines will affect the behaviour of the interacting tunnel systems.

This paper presents a case study from Chennai Metro Rail focusing on a critical tunnel crossing zone. Chennai, the fourth-largest metropolis in India and capital of Tamil Nadu, has an operational underground twin-tunnel system constructed under Phase I. Near Kilpauk Medical College (KMC) station, the Phase II twin tunnels cross almost perpendicularly beneath the Phase I tunnels, forming a vertically stacked arrangement in a dense urban environment. This configuration creates a highly sensitive interaction zone, where excavation for the new tunnels can induce ground movements and stress redistributions affecting the existing tunnels. Once Phase II becomes operational, the superposition of train-induced dynamic loads from both levels must also be considered. At this "no-compromise" location, conservative assumptions alone are inadequate; a project-specific, evidence-based assessment is required to verify safety and long-term serviceability.

To address this, the study employs a three-dimensional finite element analysis using PLAXIS 3D to simulate the complex soil–tunnel interaction. The analysis considers multiple construction stages and loading conditions, incorporating site-specific geotechnical parameters derived from borehole data, surcharge loads, dynamic train loads and a calibrated volume loss approach for enhanced realism.

The results indicate that the induced displacements, axial forces, bending moments and shear forces in the existing Phase I tunnels remain within acceptable limits across all analysed stages. The study captures the combined influence of staged construction and operational loads, with the aim of minimising ground settlement and ensuring seamless construction of the Phase II tunnels without compromising the structural integrity and operational safety of the Phase I tunnels. The findings support the efficacy of the adopted tunnelling methodology and contribute to a better understanding of complex underground infrastructure interactions in urban environments, offering a practical reference for similar tunnel crossing problems in industry.

1.1 Tunnel Crossing Section

The proposed Phase II twin tunnels cross the existing Phase I twin tunnels near Kilpauk Medical College (KMC) station at chainage +11,660 m, forming a perpendicular, vertically stacked arrangement. With Phase I already operational and Phase II planned directly beneath at a deeper level, the configuration creates a sensitive interaction zone where excavation and train loads from the new tunnels can influence the behaviour of the existing ones. Owing to the limited vertical clearance, even small additional settlements or distortions may be significant, making this crossing a critical location requiring detailed assessment.

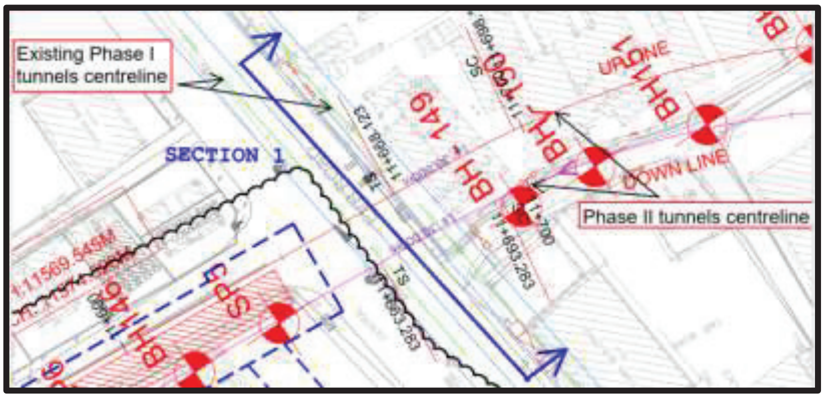


Fig. 1: Section considered for analysis

1.2 Objectives and Scope of the Study

Within this context, the objectives and scope of the study are as follows:

1. To quantify construction-induced effects of the Phase II excavation on the existing Phase I tunnels, in terms of ground settlements, tunnel displacements and changes in axial force, bending moment and shear force.
2. To simulate the behaviour of the Phase I tunnels under train-induced loads from Phase II, using a three-dimensional numerical model in PLAXIS 3D. The following cases are analysed: Case 1: Impact on Phase I tunnels due to train movement in Phase II tunnels; Case 2: Combined impact due to train movement in both Phase I and Phase II tunnels.
3. To determine and summarise ground settlements and tunnel segment forces for these cases as obtained from PLAXIS 3D, and to compare the results with commonly adopted serviceability and strength criteria for metro tunnels.
4. To present a clear, practical workflow for three-dimensional analysis of perpendicular tunnel crossings that can serve as a reference for similar urban projects.

The following sections describe the project setting and ground conditions, the numerical modelling approach, the construction and loading scenarios, and the main findings from this Chennai Metro case study.

2.0 Project Context and Site Characterization

A reliable three-dimensional numerical model depends critically on an accurate representation of both the tunnel geometry and the subsurface conditions. For the present study, particular attention was given to the exact layout of the tunnel crossing near KMC station and to the site-specific geotechnical parameters derived from nearby boreholes. This section summarises the geometric configuration of the intersecting tunnels and the ground profile adopted in the analysis, which together form the core inputs for simulating soil–structure interaction.

2.1 Geometric Configuration

The analysis focuses on the perpendicular intersection of the Phase I and Phase II tunnels near KMC station at a chainage of +11,660 m. The vertical alignment places the new tunnels significantly deeper than the existing ones, creating a direct undercrossing scenario.

Table 1: Tunnel axis levels at the crossing location

Parameter	Elevation (mRL)
Ground Level (GL)	+6.10
Phase I Up Line Tunnel Axis	−8.416
Phase I Down Line Tunnel Axis	−8.099
Phase II Twin Tunnels Axis	−21.57

2.2 Subsurface Conditions and Geotechnical Parameters

Chennai’s regional geology is predominantly composed of sand deposits, clays, granite, gneiss, and local occurrences of shale and sandstone. Broadly, the city can be divided into sandy, clayey and hard-rock zones. The KMC area falls within a transition setting where near-surface soils overlie weathered to moderately weathered rock at depth.

The site investigation methodology was based on information from boreholes drilled in the vicinity of the tunnel crossing. Two primary boreholes, BH-149 and BH-150, located near the point of intersection, were used to derive the design parameters for the soil and rock strata.

The site geology near KMC station is characterised by a sequence of distinct layers. The profile begins with a surface layer of Fill, followed by layers of Clayey Silt (CI) and low to medium plasticity Clays (CL). Below this, extensive layers of Silty Sand (SM) are present, extending to a considerable depth. This sandy stratum transitions into completely and highly weathered Sandstone (Grade V), which in turn gives way to moderately weathered rock (Grade III) at greater depths.

The key design parameters derived from the site investigation for each stratum are summarised in the following table.

Table 2.1: Summary of design parameters for KMC at chainage 11,660 m

Elevation (mRL)		Strata	SPT N60	γsat (kN/m3)	Cu (kPa)	Øu (˚)	Eu (MPa)	C' (kPa)	Ø'(˚)	E'(MPa)	Erm (MPa)	µ	Ko
From	To												
6.1	3	Fill	10	18	-	-	-	5	-	15	-	0.2	0.5
3	1	CL	12	18	60	-	-	-	-	18	-	0.2	0.7
1	-3	SM	12	19	-	-	-	1	32	18	-	0.2	0.47
-3	-6	SM	18	19	-	-	-	1	32	18	-	0.2	0.47
-6	-8	SM	31	19	-	-	-	1	34	46.5	-	0.2	0.44
-8	-16	SM	60	19	-	-	-	1	38	90	-	0.2	0.38
-16	-20.5	CL	100	20	500	-	-	-	-	150	-	0.2	0.7
-20.5	-24	Sandstone G(V)	-	20	-	-	-	35	45		150	0.3	0.29
-24	-50	Grade 3	-	22	-	-	-	90	46		450	0.25	0.28

These derived geotechnical parameters form the core inputs for the constitutive material models detailed in the following section, where the non-linear behaviour of the soil strata is simulated using the Hardening Soil model and the rock mass is represented by the Mohr–Coulomb model.

3.0 Numerical Modelling Methodology

To assess the interaction between the existing Phase I tunnels, the new Phase II tunnels and the surrounding ground, a three-dimensional numerical model was developed in PLAXIS 3D. A three-dimensional approach was adopted in preference to two-dimensional analysis because the perpendicular crossing and vertically stacked configuration generate inherently non-planar stress and deformation patterns that cannot be captured adequately in plane strain. The modelling strategy was designed to represent realistic soil–structure interaction, staged construction and operational loading with sufficient fidelity for serviceability assessment.

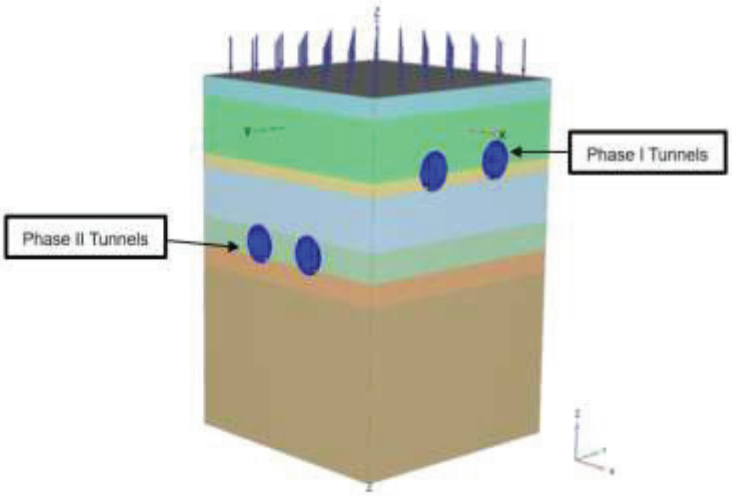


Fig. 3: Plaxis 2D Numerical Model

3.1 Constitutive Models and Parameters

The behaviour of the geological materials was simulated using appropriate constitutive models to reflect their real-world response to loading and unloading. The Hardening Soil model was adopted for all soil layers to capture their non-linear stress–strain behaviour, including stress-dependent stiffness. The behaviour of the rock strata (sandstone) was simulated using the Mohr–Coulomb model, a well-established model for brittle materials.

Phase II twin tunnels are planned to be excavated using Earth Pressure Balanced Tunnel Boring Machines (EPB-TBMs) operating in closed-face mode, with the drives advanced sequentially. In the numerical analysis, the tunnelling-induced volume loss (VL) and corresponding K parameters were defined using the matrix-based procedure of Chiriotti et al. (2001), which relates VL to the ground conditions at the tunnel face and the combined face/overburden configuration. The TBM excavation was represented through this VL-based approach to capture the inevitable, localized ground loss around the tunnel perimeter. Accordingly, the following VL values were adopted:

- 1.5% for the proposed Phase II tunnels, as a conservative estimate.
- 0.5% for the existing Phase I tunnels, reflecting their original construction conditions.

The model also included a surface surcharge of 20 kPa to account for traffic loading, derived in accordance with IRC 6 standards. Both the Phase I and Phase II tunnel liners were modelled as plate elements with a thickness of 275 mm, representing M45 grade concrete.

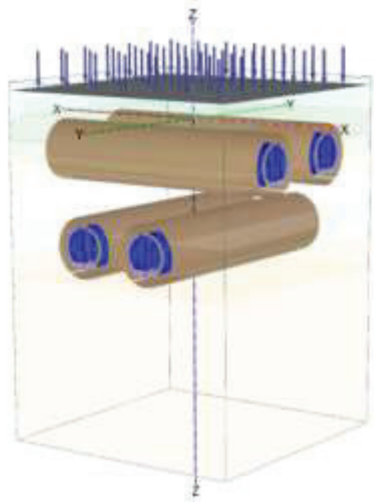
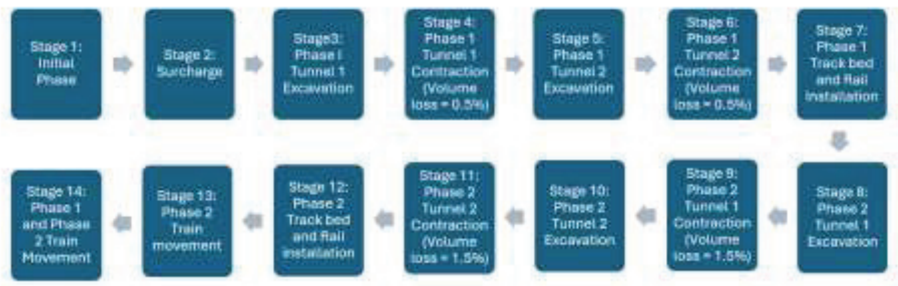


Fig. 3: Model showing Phase II tunnels and Phase I Tunnels with loadings.

3.2 Construction Sequence

To assess the cumulative impacts on the existing infrastructure, the analysis was performed in a series of stages that logically replicate the construction and operational timeline. This sequenced approach ensures that the effects of each activity are correctly superimposed and that the stress history associated with excavation, lining installation, surcharge application and train loading is realistically represented in the model.



3.3 Moving Load Simulation

To assess the impact of the operational metro system, train movements were simulated to analyse real case scenario. This analysis was conducted for two primary scenarios:

- Train movement in the Phase I tunnels only.
- Combined train movement in both existing Phase I and new Phase II tunnels simultaneously.

The simulation was based on Chennai Metro Rail Limited (CMRL) loading details for a standard train speed of 70 km/h, providing a realistic assessment of the operational vibration environment. As per design manual for underground station structures, loading details are shown in figure 2.

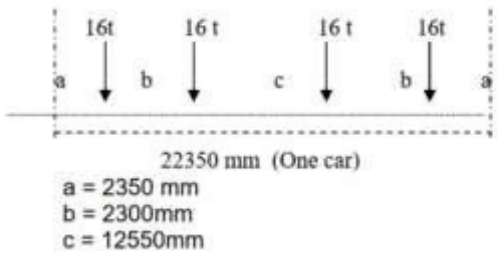


Fig. 4: CMRL Loading details

4.0 Results and Discussion

This section presents the quantitative outputs from the PLAXIS 3D simulation. The results are critically evaluated to assess their implications for the structural integrity of the existing Phase I tunnels and the overall feasibility of the proposed construction. The analysis covers both the structural forces induced in the tunnel lining and the associated displacements.

The numerical analysis quantified the total displacements and the structural forces (axial force, bending moment and shear force) induced in the existing Phase I tunnels at key stages of construction and operation. A summary of these results is presented in Table 3.

Table 3: Summary of PLAXIS 3D analysis results for Phase I tunnels

Stage	Vertical Displacement (mm)	Axial Force (kN/m)	Bending Moment (kN·m/m)	Shear Force (kN/m)
Phase I tunnels after Phase I tunnel construction	23	110	31	24
Phase I tunnels after Phase II tunnel construction	17	901	84	79
Phase I tunnels after Phase II tunnel construction and live load in Phase I tunnel	17	937	90	87
Phase I tunnels after Phase II tunnel construction and live load in Phase I and Phase II tunnels	18	931	87	87

4.1 Bending Moment, Shear Force and Axial Force

Under initial conditions (after Phase I construction), the Phase I tunnel carries relatively modest internal forces, with an axial force of 110 kN/m, a bending moment of 31 kN·m/m and a shear force of 24 kN/m.

After excavation of the deeper Phase II tunnels, stress redistribution in the ground causes these values to increase to 901 kN/m, 84 kN·m/m and 79 kN/m, respectively.

With live load in the Phase I tunnels, the internal forces reach their maximum values: axial force 937 kN/m, bending moment 90 kN·m/m and shear force 87 kN/m. When live load is applied in both Phase I and Phase II tunnels, the response remains of similar magnitude, with only slight redistribution (axial force 931 kN/m, bending moment 87 kN·m/m and shear force 87 kN/m), showing that combined operation does not cause any significant amplification beyond the worst single-line loading case. These peak values were verified against the design capacity of the existing lining segments and found to be within allowable limits, confirming that the Phase I tunnel lining is structurally adequate for the additional loads from Phase II construction and operation.

4.2 Vertical Displacement / Settlement Behaviour

A critical parameter for serviceability is the induced tunnel displacement. The simulation predicted a maximum cumulative displacement in the Phase I tunnel of 18 mm under all operational loads for volume loss of 1.5%. This final value is the governing displacement for assessing serviceability and is well below the permissible limit of 25 mm specified by the Land Transport Authority (LTA), Singapore, in its Civil Design Criteria. This confirms that the predicted settlement is tolerable and does not compromise the operational safety or serviceability of the existing metro line.

It is also noted that a volume loss lower than the assumed 1.5% would further reduce the induced settlements. The selected value is therefore conservative with respect to both anticipated construction practice and the resulting tunnel movements. The distribution of forces and displacements in the Phase I tunnel lining after the construction of the Phase II tunnels is visualised in corresponding numerical output figures, which illustrate the concentration of effects in the crossing zone and support the overall conclusions of the study. The Plaxis results for the final stage of Phase I tunnels after Phase II tunnel construction and live load in both Phase I and Phase II tunnels are presented below:

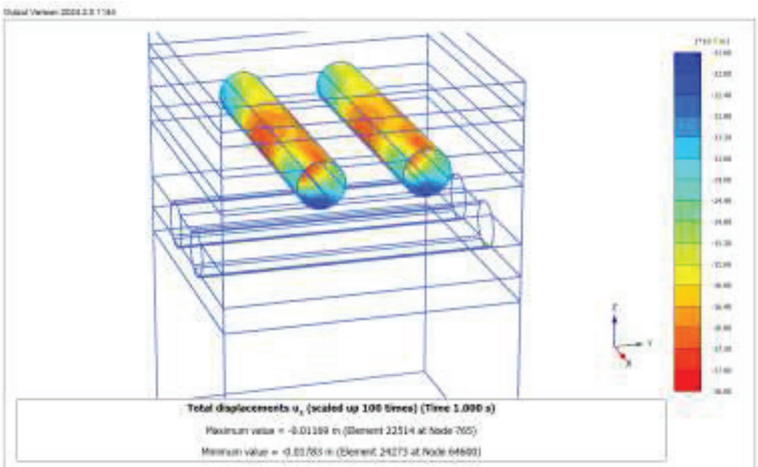


Fig. 5: Vertical Displacement / Settlement of Phase I tunnels

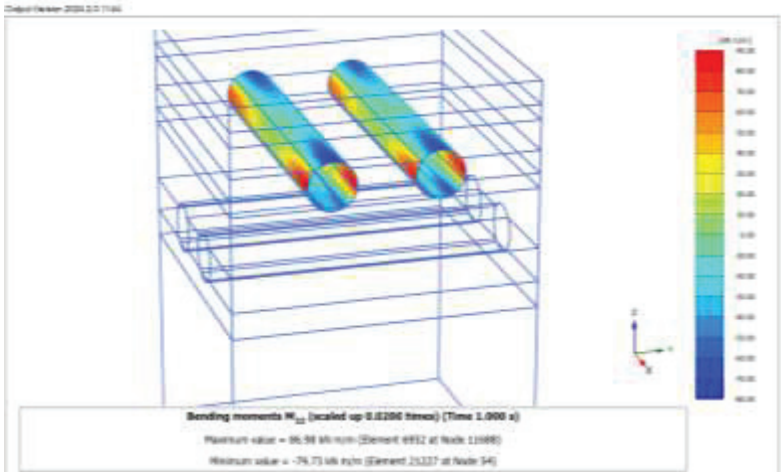


Fig. 6: Bending Moment of Phase I tunnels

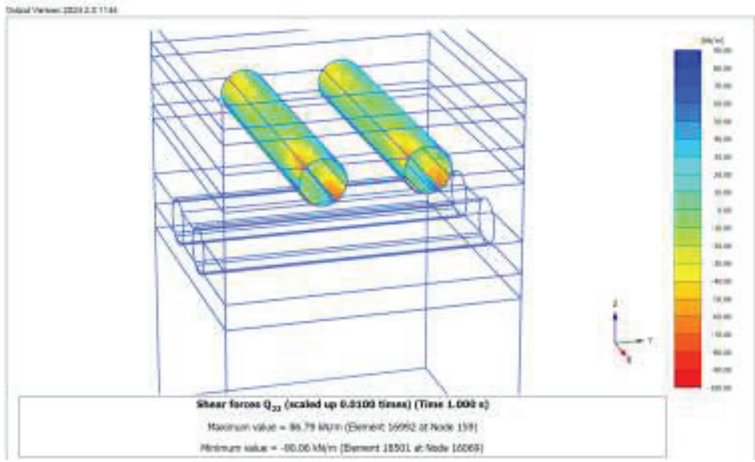


Fig. 7: Shear force of Phase I tunnels

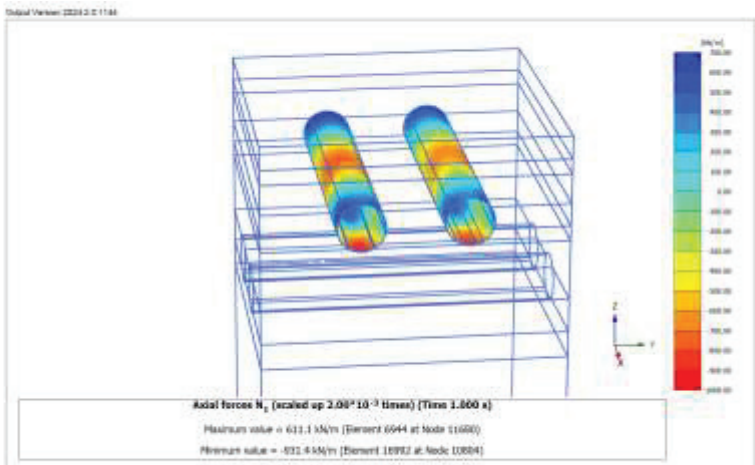


Fig. 8: Axial Force of Phase I tunnels

5.0 Conclusions

A three-dimensional finite element study was undertaken to evaluate the impact of Phase II tunnel construction and operation on the existing Phase I tunnels near KMC station on the Chennai Metro. Based on the numerical simulations and the comparison with established design criteria, the following conclusions can be drawn:

- The maximum internal forces induced in the Phase I tunnel lining as a result of Phase II excavation and combined train loading—axial force of about 937 kN/m, bending moment of about 90 kN·m/m and shear force of about 87 kN/m—remain within the capacity of the installed segmental lining. The existing Phase I tunnel structure is therefore capable of safely accommodating the additional loads associated with the underpassing Phase II tunnels.
- For a conservative assumed volume loss of 1.5% during Phase II TBM excavation, the maximum vertical displacement of the Phase I tunnels under the most adverse combined loading condition is approximately 18 mm. This is significantly less than the 25 mm limit adopted from LTA Civil Design Criteria, indicating that the serviceability of the existing tunnels is maintained. In practice, lower volume loss would lead to even smaller settlements, further increasing the safety margin.
- Comparison of the load cases shows that introducing train loads in the Phase II tunnels in addition to the Phase I tunnels does not substantially increase the internal forces or displacements in the Phase I tunnels beyond those observed when only Phase I carries trains. The combined operation of both tunnel systems, as modelled, therefore does not impose an unacceptable additional demand on the existing infrastructure.
- The case study demonstrates that perpendicular tunnel crossings in dense urban environments can be safely realised when supported by appropriate 3D numerical analysis, careful design and proactive monitoring. For the assumed ground conditions, construction methodology and operating loads, no major adverse impact on the existing Phase I tunnels is expected due to the construction and operation of the Phase II tunnels at the KMC crossing.

6.0 References

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LESSONS LEARNT FROM TUNNELLING THROUGH FAULT ZONE IN ATAL (ROHTANG) TUNNEL PROJECT

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1. Introduction

Atal (Rohtang) tunnel has been constructed under Pir panjal range near Rohtang pass. Tunnel has connected Manali to Lahaul valley. It has reduced the road length of Manali-Sarchu-Leh axis by 46 Km and established throughout the year connectivity between manali and lahaul & spiti district of Himachal Pradesh. South Portal of Atal tunnel is located at a distance of 25 Km from Manali and at an altitude of 3060m. North portal of Atal tunnel is located near village Teling, Sissu in Lahaul valley at an altitude of 3071m. Total length of Atal tunnel is 8.802 Km. It is horse shoe shaped, single tube two lanes along with raised footpath on both sides of the tunnel. It has semi transverse ventilation system at top and Egress tunnel at the bottom of the carriageway for maintenance and emergency exit. Drill and Blast method of tunneling with NATM philosophy has been used for the construction of Atal tunnel. Figure 1 is the plan of the area showing Manali-Sarchu road and Atal tunnel alignment. Figure 2 shows the longitudinal cross-section of the total length of tunnel and also indicates main fault structures in the alignment. Figure 3 shows the longitudinal section of south portal drive of Rohtang tunnel.

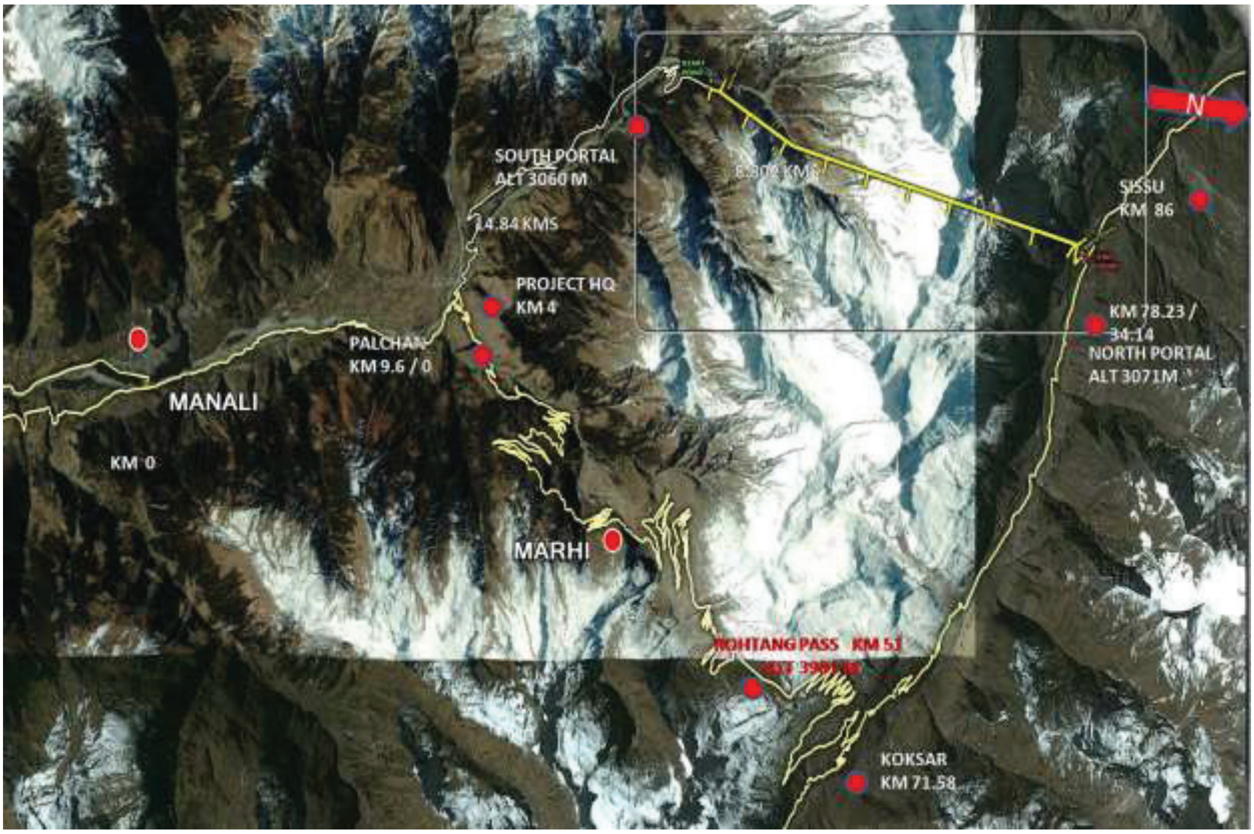
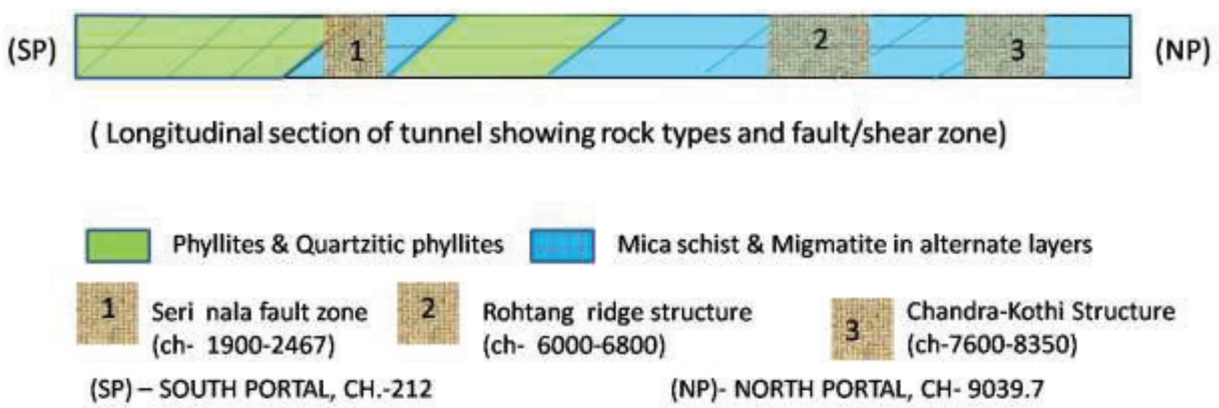


Fig 1: Plan of the area showing the existing Manali-Sarchu road and Atal tunnel alignment

2. Geology of Rohtang Tunnel

Rohtang tunnel project is located within 'Central crystalline group' litho-tectonic group of Himalayas. The regional geological succession at the project site comprises the Tandi formation, Batal formation, Salkhala group and the Rohtang Gneiss conmplex. The Rohtang tunnel alignment is mainly through Salkhala group (Age- Precambrian). Main rock types along the alignment are Phyllites, Quartzites, Mica schist, Migmatite and Gneiss. Major Geological structures in the area are- Seri nala fault, Chandra-Kothi structure, Rohtang Ridge structure, Dhundi structrе, Palchan structure, Palchan fault, Sundar nagar fault and Main central thrust fault.



Overburden (metre) along the tunnel alignment

Chainage (m)	212	500	1000	1500	2000	2500	3000	3500	4000	4500	5000	5500	6000	6100	6500	7000	7500	8000	8500	9000	9039
Over-burden(m)	0 (SP)	147	340	311	273	242	460	833	1120	1128	1320	1524	1770	1908(max)	1699	1334	1107	747	429	103	0(NP)

Fig 2: Longitudinal cross section of the tunnel showing major rock types main structures along the alignment and overburden along the tunnel alignment

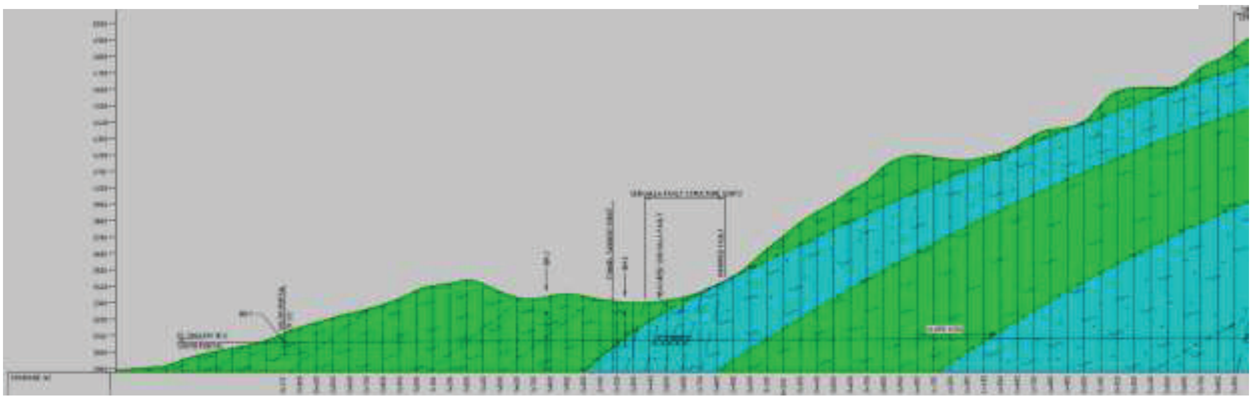


Fig 3: Longitudinal section of south portal drive of Atal tunnel showing Seri nala fault zone

3. Construction of Atal tunnel: South Portal drive

3.1 Location and Geology of South Portal

South Portal of Atal tunnel is located at 25 Km NNW (North North West) of Manali in the state of Himachal Pradesh. Approach Road to South Portal (ARSP) connects Palchan village (situated on Manali-Sarchu road at Km 10 from Manali) to South Portal. Total length of ARSP is 14Km. Dhundi structure, Beas nala fault structure and Seri nala fault structure are main geological structures in the locality. Tunnel alignment is passing under Seri nala through Seri nala fault structure. Seri nala is a small rivulet having surface gradient of approximately 10% . 570 metre length of tunnel alignment has been excavated in the loose strata due to Seri nala fault structure. Nearby area of South portal is covered with fairly dense Kail. Rohtang ridge structure is approximately 5.8 Km from South Portal. Seri nala is having strike of N47°E and at chainage 2500, tunnel is just below the Seri nala.

3.2 South Portal drive and encounter of Seri Nala Fault Structure

First blast at South portal was done on 30.08.2010. By the end of year 2011, 1900 metre length of tunnel was excavated before entering in Seri nala fault zone. As per design drawing, Seri nala fault zone was to be encountered in between chaingae 2200-2800. But fault zone was hit at chainage 1900 in left hand side. By the chainage 1950, full face of the tunnel was occupied by loose strata with water. Loose strata and ingress of water was due to seri nala fault zone. It was hit 300 metre before as anticipated in drawing. Alignment of the tunnel was N29°E. Angle of intersection of tunnel alignment and fault line was 16°. By March 2012, heading excavation was being carried out in step by step. Fixing of one complete Lattice Girder was being done in 4-6 steps as only partial excavation was allowed to minimize the risk of face flow. Earlier 4 parts of lattice girder were being assembled and installed in one single step. At chainage 2049(April 2012) first major flow of face occurred. After which there was no other option than Pipe roof umbrella -single and double layer. At chainage 2077 in September 2012 second major and biggest face flow till now was seen in south portal drive. Fig 4 shows the situation after face flow at chainage 2077.Pipe roof umbrellas were installed at chainage 2049 onwards. At some location forepoling by Self Drilling Rock bolt (SDR) were also used. Figure 5 and Figure 6 show Single layer and Double layer Pipe roof umbrella. Figure 7 shows the perspective view of Pipe roof umbrella. From change 2385 onwards Drift method of tunneling was used under Pipe roof umbrella (September 2014 onwards) for about 20metre length. Intensive grouting with OPC, MFC with Sodium silicate was used with good result. There were several minor flows from the face during excavation. In early 2015, proposal of changing the tunnel alignment was voiced from contractor's side. Tunnel alignment since last 2011 was continuing in loose strata and it was not known when this condition was going to end. Tunnel alignment had not yet reached nala on the surface.So contractor proposed to change the alignment in right side and crossing the nala upstream of Gorge structure visible on the surface. It was also discussed in high level meeting. Experts from Designer and Engineer were also saying that it could last for further several hundred of metres.

Author interpreted the data collected during the track to Seri nala. Dip direction, dip visible on the mountain side, alignment of tunnel, overburden of tunnel were analysed properly. It was determined by the author that we should hit the hard rock in the existing alignment nearby to chainage 2427. Also changing the alignment in right side will lead to loss of hundreds of crores as well as time. If change of alignment is to be done, it should be in LHS and not in RHS direction. Proposal of core drilling in right side perpendicular to alignment was also not recommended by the author as it could not produce any fruitful result. In March 2015, it was decided for core drilling at chainage 2410. In May 2015, it was confirmed by core drilling that alignment is hitting the hard rock at chainage. So prediction by author came into reality. If the proposal of changing the alignment was agreed, it could have cost crores of rupees extra and additional time to complete the project. In this way money and time both were saved. Figure 9 shows Seri nala fault zone.

Fig 4: Flow of face at Ch 2077

Fig 5: Pipe roofing umbrella (Single layer)

Fig 6: Pipe roofing umbrella (Double layer)

Fig 7: Pipe roofing umbrella (Perspective view)

Fig 8: Drift method of tunneling in loose strata at ch 2410.

Fig 9: Seri nala fault zone

3.3 Seri Nala Fault Structure – Analysis

Figure 9 shows the Seri nala fault zone between two parallel lines. The width of Seri nala fault is 180-200metre as determined by the author. Figure 10 shows the set-up of core drilling starting at chaingae 2410. Figure 11 shows the hard rock obtained from chainage 2454 onwards. It was concluded by the author that hard rock would be hit in LHS of tunnel alignment at chainage 2445 and the full face will be covered by the hard rock at chainage 2467.

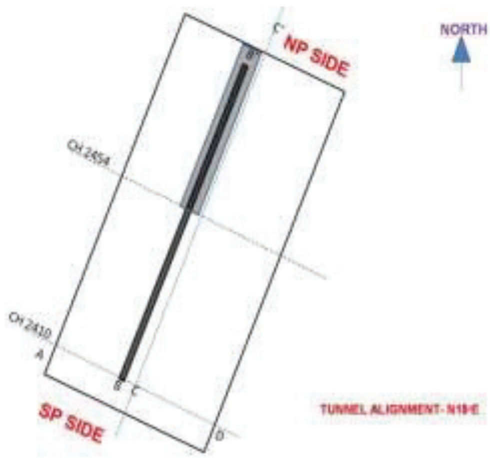


Fig 10: Layout of Core drilling at Ch 2410

Fig 11: Hard rock found at chainage 2454 by core drilling

4. Lesson learnt from Seri nala fault structure/zone-

1. Alignment of the tunnel should be chosen in such a manner that it crosses the fault zone perpendicularly especially when a rivulet is flowing on the above surface. If necessary two curves can be provide in the alignment- one before the fault zone and second after that. Wrong selection of alignment can lead to loss of hundreds of crores rupees and valuable time also.
2. Detailed study of water ingress should be estimated and proper design of drain pipe (at the bottom) should be carried out. Enough factor of safety should be taken for estimated ingress of water through the tunnel.
3. In the loose strata there is chance of undercut. Longitudinal gradient of tunnel should be kept near to 3% so that in the case of undercut it can be reduced upto 0.5%. If longitudinal gradient is kept 0.5% there is no possibility to further reduce it if required.
4. Q system of Rock Mass Classification is failure in fault zone with water. So classification system in these zones should be different. Accordingly BOQ items should be included in contract. Rock class which cannot be defined by Q-system can be included in the contract.
5. In DRESS method of tunneling, drainage of rock mass ahead of face is most important. So proper specifications of drainage pipes as that manufactured by DSI should be included in the contract. Specification of pipes of pipe roofing umbrella must be including in contract.
6. In loose strata grouting can play a vital role in improving the rock mass/soil quality. Sieve analysis of soil/rock sample to determine groutability, water pressure test etc. should be carried out regularly before application of any grouting material. It was concluded that PU is not successful in the condition of seri nala fault zone. MFC, OPC with sodium silicate, multistage grouting were proved to be effective upto certain extent.
7. Side drift method of tunneling was found to be effective in the loose strata with water.
8. Some Emergency measures must be kept near to face to handle the situation of face flow and cavity formation.
9. TSP is almost failure in loose strata. TSP could not even detect hard rock some 50-60 metre ahead. It was only confirmed by core drilling.
10. It should be noted that there is very much difference in rock class 7 under high overburden and class 7 in fault zone with water. Proper support design for rock class 7 or the poorest class of rock in fault zone with water must be there in details.

OVERVIEW AND CHALLENGES & INNOVATION ON LARGE DIAMETER TBM TUNNEL WORKS IN MUMBAI COASTAL ROAD PROJECT (SOUTH)

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ABSTRACT

MCRP4 project (Mumbai Coastal Road Project Package IV from Princess Street Flyover to Priyadarshini Park) comprises the design and construction of twin bored road tunnels, 11 m internal diameter (finish) accommodating carriageways for two lanes together with an emer-gency lane and 2.072km of tunnel (2 drives) drives using single Slurry TBM (12.19m in diameter) which is India's largest as of now. This TBM Tunnel includes 900m long undersea tunnel (during high tide) and 1.0km long tunnel under Malabar hill (max overburden is 67m). The TBM tunnel started on 11.01.2021 (first drive, RHS tunnel) and final breakthrough (second drive, LHS tunnel) was completed on 30.05.2023. This paper presents an overall description of TBM tunnel works and shares the information on challenges, lesson and learns during the journey of India's first large diameter TBM tunnel drive.

1 OVERVIEW

1.1 *Project Background*

Mumbai is reckoned as the financial capital of India & economic powerhouse. Mumbai has been constantly evolving and globally engaged city over the past 150 years & has made a rapid economic transition from trade focused to services focused. It is the second most populous metropolitan area in India , with a population of 21 Million as of 2022. Mumbai is the financial , commercial & entertainment capital of India. The City houses the headquarters of over 400 business houses. It houses important financial institutions & Corporate headquarters of numerous Indian & multinational corporations. It is also home of some India's premier scientific & nuclear institutes. Mumbai's business opportunities as well as potential to offer a higher standard of living , attract migrants from all over the India making the City a melting pot of many communities & culture.

Speaking about major challenges to city's expansion are numerous. It houses a population of 21 million besides a large floating population, in a small area of 437sqkm and is surrounded by sea and has nowhere to expand.The constraints of the geography and the inability of the city to expand have already made it the densest metropolis of the world. High growth in the number of vehicles in the last 20 years has resulted in extreme traffic congestion. This has lead to long commute times and a serious impact on the productivity in the city as well as declining quality of life of its citizens. The extreme traffic congestion has also resulted in Mumbai witnessing the worst kind of transport related pollution.

However , Mumbai city flanked by the coast on the east & the west, Mumbai's space for expansion is limited. The roads are already congested and there is no ample space available for the Mumbai city to expand. Traffic-related problems are very common for the residents of Mumbai. This presents a number of challenges including pressure on land for drainage , water supply network , road & public transportation networks & lack of open spaces. Further, utility box within the tunnel will cater the need of utilities through out the coastal road.

The twin tunnels of Costal Road (South), running underneath the Girgaon Chowpatty Beach as well as the depths of Malabar Hill connecting Marine Drive to Priyadarshani Park are boon for the commuters of Mumbai, reducing the travel distance as well as commute time between the Southern Mumbai to Northen Mumbai.

Coastal Road is unique solution in terms of transportation mobility & availability of much needed recreational spaces by way of reclamation. However, South Mumbai is most developed part where major scarcity of land to develop any infrastructure project. Therefore, Coastal road comprises of tunnels , road on reclamation, bridges & Interchanges etc. is access control, signal free will enhance travel speed. Being toll free it will encourage common citizens of Mumbai.

Creation of huge green open space to the magnitude of 70 ha at one place will enhance the environmental conditions which shall be utilized for development of green space / landscape for residents of Mumbai for recreational purpose/activity e.g.Biodiversity Park, Butterfly garden, Amphitheatre, Jogging/cycling track, 7.5 km continuous sea promenade etc. shall be developed on the reclaimed land which shall bring a different paradigm of urban comfort.

1.2 *Project Description*

MCRP4 (Mumbai Coastal Road Project Package IV) from Princess Street Flyover to Priyadarshini Park Comprises of design and construction of twin bored transportation tunnels under coastal area equipped with all automation and tunnel ventilation systems within it.

The work includes cut & cover tunnels and transition ramps at either end of the bored tunnel. The transition ramp and cut & cover tunnel at the South end passes through a prominent VVIP road known as Marine Drive (Netaji Subhash Chandra Bose Road).

Package IV, Princes Street Flyover to Priyadarshini Park consists of grade road, transition ramp, cut & cover tunnel, TBM tunnel, reclamation and seawall. This package extends from CH 1+850 to CH 5+900, and is mostly underground road, except entry and exit of the tunnel.

Twin Bored Tunnels of 11m internal diameter (finish) accommodating carriageways for two lanes together with an emergency lane.

Total length of this project is 4.050 km out of which 2.072km (2 drives) is planned to build using Slurry Tunnel Boring Machine (TBM, 12.19m)) which is India's largest as of now. This length of TBM Tunnel includes 900m tunneling under coastal area and about 1.0km tunneling works under Malabar Hill (max overburden is 67m) which has water storage reservoir below hanging garden on top of the hill (Figure 1 and Figure 2).

The first drive of TBM Tunnel had been completed (RHS Drive) which started from Priyadarshini Park (Launching Shaft) till Girgaon Chowpatty (Retrieval Shaft) and completion of RHS took exactly 1 year (November 1, 2021 to January 10, 2022). The second drive of TBM tunnel (LHS tunnel) started from the Retrieval Shaft with TBM U-Turn technique on April 08, 2022 and the breakthrough completed on May 30, 2023 which took 1.2 year.

The average advance rate of TBM during tunnel drive recorded as 5m/day in rock class III or IV and 11m/day in rock class I or II.

Figure 1. MCRP4 (Mumbai Coastal Road Package IV)

1.3 *Geological Setup*

TBM is predominantly through UG3d-UG4d-UG4b ground (Figure 2). However, overburden on top of the tunnel at the initial TBM launching zone is only around 5.0 m and the class IV weathered Basalt (UG3b) is overlaid by loose boulder zone (UG2c) and then lagoon area is coming on top of the UG3b which might be prone to the direct groundwater risk during TBM Tunneling.

The half of the tunnel ground faced during the mining was full face with slightly weathered Breccia, while other half was mixed with Shale, Breccia, and Basalts, thus depending on the amount of shale content in the ground, frequent cutter head intervention had been occurred due to clogging.

On top of the Malabar Hill section, Hanging Gardens Reservoir (14,700m^3 capacity of potable water storage) exists, hence the stability of such structure shall be ensured during mining and also during tunnel operation.

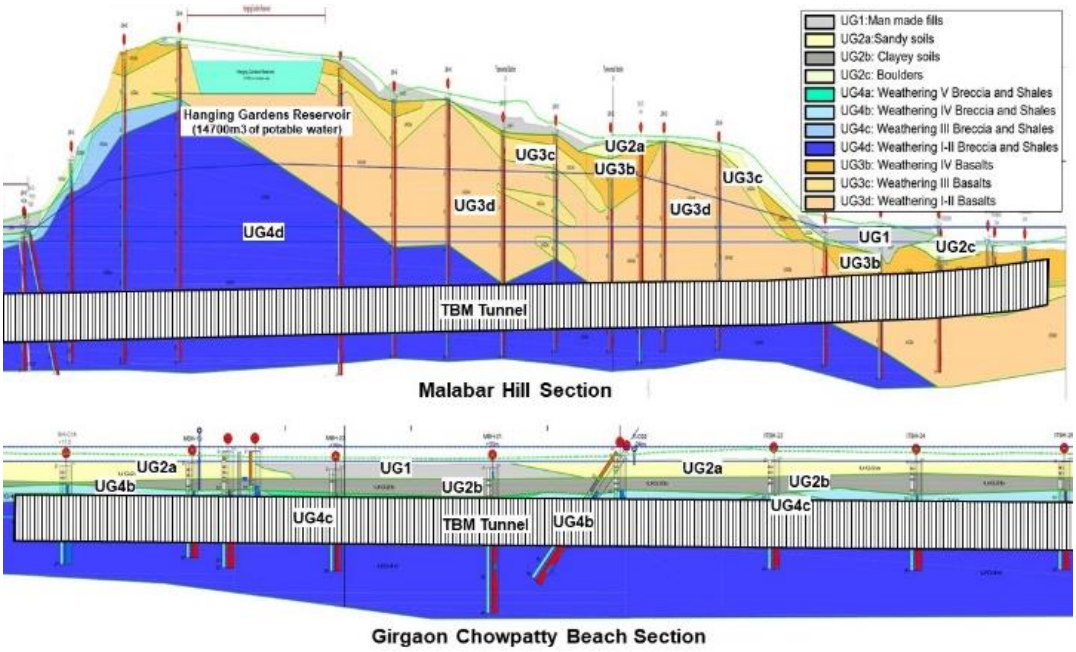


Figure 2. Geological Setup for TBM Tunnels

2 TBM TUNNEL DESIGN

2.1 *Tunnel Space Proofing and Segmental Lining*

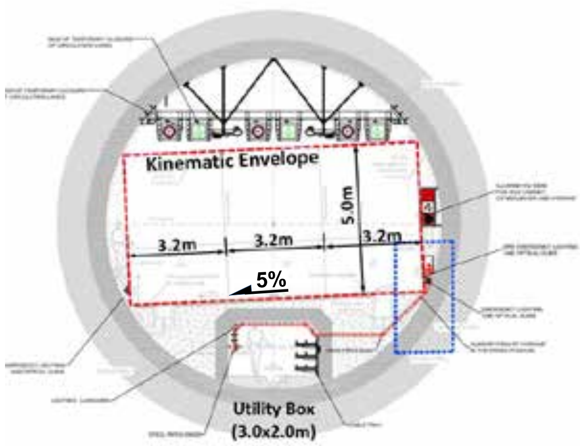
Figure 3 shows the space proofing and tunnel design considerations adopted in MCRP4.
As the minimum radius of the curve in horizontal tunnel alignment is 387m, the maximum super elevation over the tunnel increases up to 5%.

Saccardo system had been adopted for the tunnel ventilation thus jet fans are no need to control both ventilation and smoke during fire incidents, which in turn ensures more space to be utilized inside tunnel.

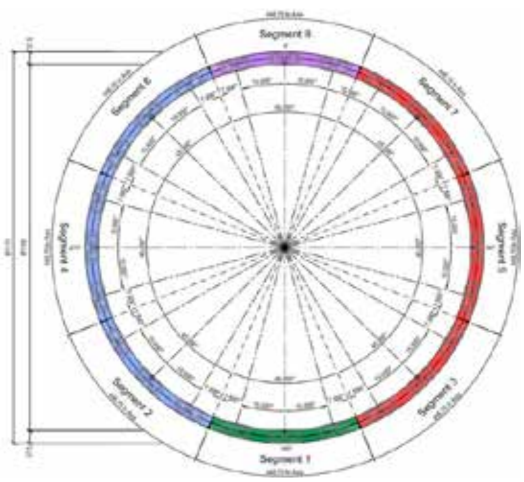
Cross passages are designed to be installed in every 300m interval as per NFPA 502 in bored tunnel section, which are 4 nos. of pedestrian CPs and 2 nos. of Motorable (i.e. vehicle) CPs for emergency vehicle access.
As the low point of the vertical alignment of the tunnel is located at CP no. 3 (Figure 5), tunnel water coming from intermittent wall washing, hydrant water during fire etc. will be collected into the low point sump installed in CP no.3 and pumped out through the vertical shaft installed at the same location (Figure 5).

Fire board has been adopted in mainline tunnel for the protection of structural integrity during fire incident, and the efficiency of the board on the fire has been verified though the full scale test, while PP(Polypropylene) fiber has been included in the final lining of the CP design as CPs are protected by the fire door as a first barrier.

Items	Design Consideration
Carriageway	3.2m per lane x 2 + 3.2m for emergency
Clearance of the Envelope	5.0m
Min. Radius of Alignment	387m
Max. Vertical Gradient	3.0% (down grade to low point at CP3)
Super elevation	2.5% ~ 5.0%
Walkway for maintenance	500mm at both sides
Drainage System	400mm RC Pipe, collected at low point sump of CP3
Ventilation System	Saccardo System (i.e. without Jet fans)
Utility Box	3.0m x 2.0m
Underground Cross Passage	4 nos. of Pedestrian, 2 nos. of Motorable, 300m interval
Structural Fire Protection	Fire Board (Mainline), PP fiber (Cross Passages)



(a) Tunnel Space Proofing



(b) Segmental Lining Scheme

Figure 3. Tunnel Space Proofing and Segmental Lining Scheme

The maximum overburden considered for the segment design is 67m which is estimated as per the actual overburden when TBM passes underneath Malabar Hill area.

Hydrostatic pressure underneath seabed has been adopted in the design and structural calculation over all possible load combinations has been carried out.

Water tightness of EPDM gasket has been verified through the tests carried out at manufacturer's laboratory and up to 20mm offset of gaskets had been tested considering the segment installation error. Also, hydrophilic cord had been included in the gasket system (Figure 4) and the effectiveness of the watertight on this material had been tested at the laboratory using saline water as the expansion features of the cords shall be verified when it is exposed to high content of saline water under seabed.

Universal segment (i.e. 375mm in thickness, 2.0m in width, 7+1) has been adopted in design for the efficiency of the segment production at the casting yard. And the erector was designed sufficiently to hold such dimension of segment (i.e. 8.7 MT per segment). Segments are tightening up with spear bolts and guide rods were installed for the better assembly of segments and rings were connected with cone dowels for the resistance of shear movement.

(a) Anchored EPDM Gasket (b) Hydrophilic Cord

Figure 4. Water tightness with EPDM Gaskets

2.2 *Tunnel Drainage System*

Tunnel drainage system has been designed to cater water inflow due to fire-fighting operations (i.e. hydrant system), water ingress due to intermittent washing of tunnel walls, seepage through tunnel segments and cut and cover construction joints, and vehicular induced water ingress during rainy days.

Tunnel drainage consists of surface drainage, firetraps, and sub-surface drainage as shown in Figure 5. The surface drainage can be covered with PC slot drainage system which surface water is collected by gravitational flow along super elevation (i.e. max. 5%).

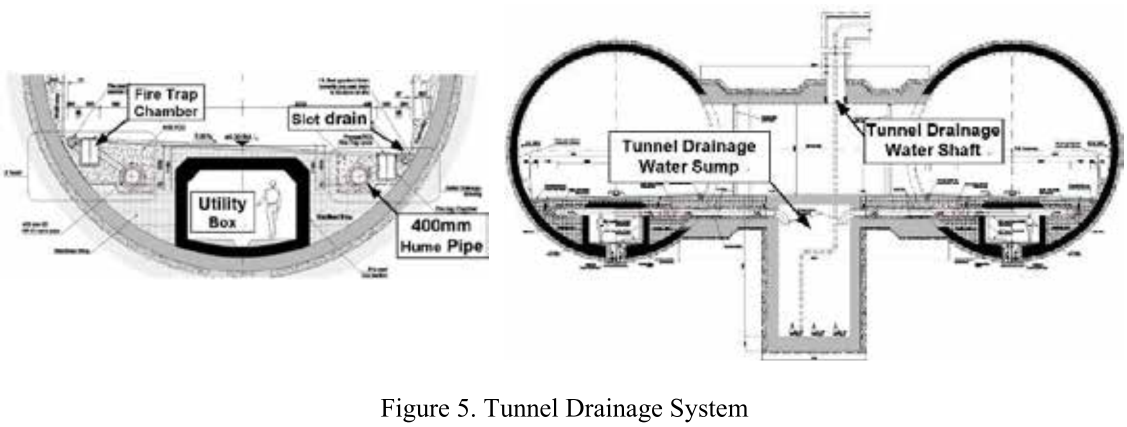


Figure 5. Tunnel Drainage System

The firetrap contains oils and debris and does not allow these to enter the main subsurface drainage system. Firetrap pit consists of a cast-iron partition which divides the pit into two chambers and creates a 'U' flow of water. Firetraps are provided at 50m intervals. The sub-surface drainage

system which surface water is collected through slot drain serves as the main tunnel drainage system and is composed of 400mm RCC Hume pipes.
As the lowest point presents inside the tunnel, water is collected into the sump at the lowest point of both tunnels at CP-03 at Ch. 4+175 where it will be pumped out of the tunnel through a shaft to the surface (Figure 5).

3 CHALLENGES DURING TUNNELING WORKS

3.1 *Geotechnical Risks during Tunneling Work*

As shown Figure 2, due to the constraint in alignment design, tunnel cover during initial drive was limited to 5~6 m only with crushed loose basaltic rock strata (i.e. UG2c, UG3b) which is risky and unfavorable ground for tunnel boring. TBM is also supposed to drive underneath lagoon area with low overburden of around 5~8m, which might have ended up with whole flooding inside the tunnel if water ingress is allowed.

To overcome this challenge, 350mm concrete slab has been installed at the surface of the low cover zone and at the bottom of lagoon area.

Extensive tam grouting was also applied on the area to fix the loose ground and minimize the water ingress by filling the cement material and blocking the possible water channelization from the lagoon area (Figure 6).

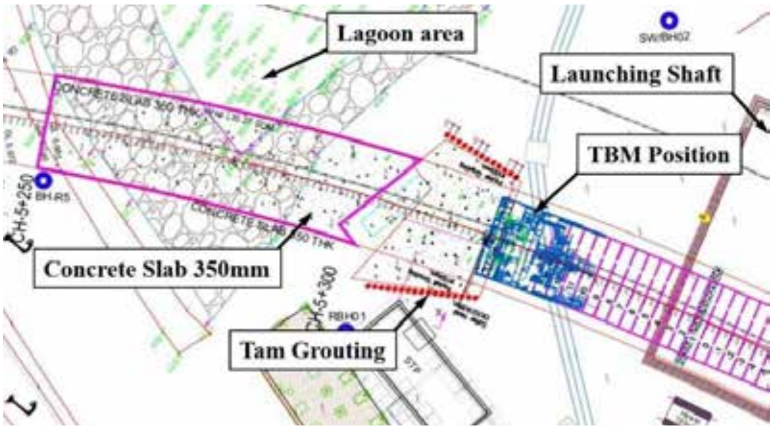


Figure 6. Ground treatment for loose rocks and lagoon area

Continuous instrumentation and monitoring has been done, and with the help of tam grouting and continuous monitoring works, TBM could move forward without any concern.

The geological-geotechnical model shows (Figure 2) that the project alignment extends through a quite variable types of materials ranging from loose soils to rocks with different degree of strength, fracturation and weathering. Therefore, it was foreseen that TBM excavating the tunnel section will have to cope with mixed face conditions, that is the occurrence at the face of materials with a large difference on strength. Due to the mixed face conditions, tunnel advancing with TBM had been struggling to clogging problems which results in frequent cutter head intervention or multiple halt points in advancement of TBM tunneling. Such halt points had been occurred not only because of the cutter head clogging but also clogging in stone collector screen.

During LHS tunneling work in MCRP4, such clogging issue in stone collector rapidly increases the pressure in working chamber and slurry material filled in the chamber subsequently infiltrated into the main bearing seals thus main bearing system was stopped and as a result, TBM works were halted almost 2 months until the main bearing system is fully recovered.
To avoid such an extreme clogging failure, clogging shall be monitored very carefully whenever TBM is halted for cutter head intervention.

3.2 *TBM driving under Hanging Garden at Mallabar hill*

Water reservoir located in Mallabar hill was also one of the most critical structures to be checked for the impact of TBM driving and was also put in the list of possible challenges. As shown in Figure 2, this reservoir at Hanging Gardens contains 14,700㎥ volume of potable water, impact on this reservoir due to tunneling may end up with highly critical disaster.

As the rock ground condition over this hill area is mostly high class (I, II) of Basalt, Breccia and shales, also slurry type TBM is always advancing with slurry pressure on face, moreover overburden on top of the tunneling work is more than 60m, risks on this reservoir was expected less.

However, expected impact on the structure had been investigated through numerical analysis and convinced that there was no significant impact on it. Also, extensive instrumentation and monitoring had carried out in those particular areas.

3.3 *Groundwater Control during Cross Passage Works*

As shown in Figure 1, almost half of MCRP4 tunneling works were under beach area at Girgaon Chowpatty area which is practically the same as subsea tunnel as such area will be covered with sea water during high tidal time.

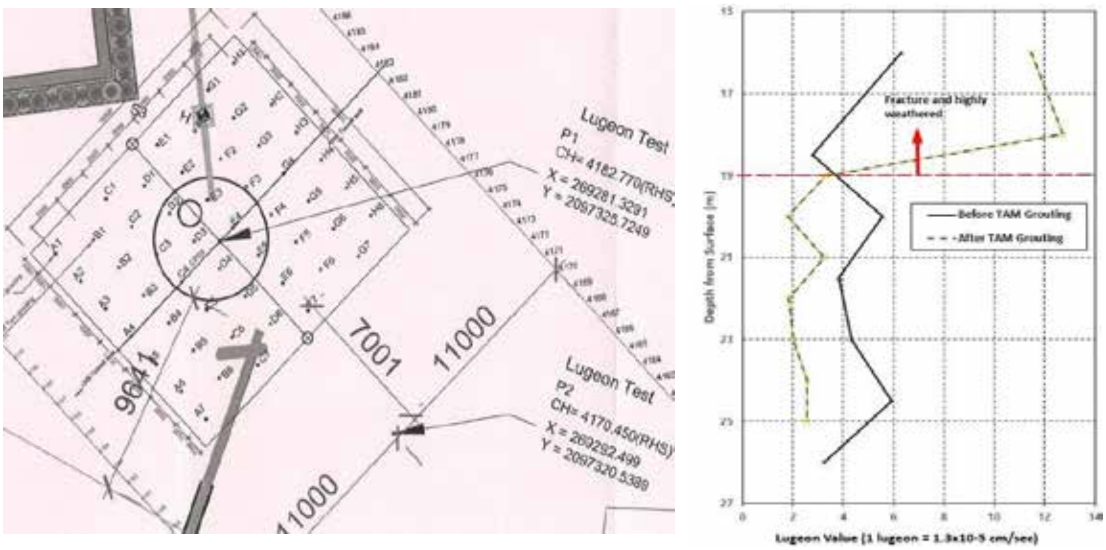
In TBM tunneling works, inundation risks of tunnel due to water infiltration through the face or walls always present. Inundation risks through tunnel face is somehow mitigated as TBM is advancing with pressurized slurry at working chamber unless slurry mud filter cake at the face is broken up due to highly fractured or loose rocks.

However, as tunneling works for cross passages relies on conventional tunneling, tunneling works are more vulnerable to the groundwater risks especially for the CP (Cross Passage) works under beach area.

In CP3 tunneling works, significant volume of water infiltration was expected when probe drilling through the segment was carried out before the actual excavation and removal of concrete segments. To control such groundwater infiltration, surface grouting works were carried out to reduce the permeability of rocks (Figure 7 a) and verification on this grouting effect on rock through rock permeability test (Lugeon test, Figure 7 b). Figure 7 b shows that permeability of rock is somehow reduced after the grouting works except fractured and high weathered ground as such grouting works may not be effective on such a high fractured and permeable ground.

Other than surface grouting works, pre-grouting for waterproofing was also carried out separately before the excavation of Cross Passage.

Due to the pre-grouting work both on surface and in tunnel respectively, groundwater was controlled successfully and no significant infiltration occurs during the CP works.



(a) Tam grouting and lugeon test holes (b) Lugeon test results

Figure 7. Ground treatment on surface and Permeability test for verification

3.4 *Relaunching of TBM*

After the breakthrough of the first TBM drive (RHS) on the twin tube tunnel on January 10, 2022 U-turn of 12.19 m diameter of slurry TBM for its parallel drive had been completed an on February 4, 2022. The slurry TBM is the largest and heaviest (1800 MTONS) of its kind engaged in India and the U-turn of the machine in the limited space at retrieval shaft is another technical first in India.
After the breakthrough of RHS tunnel, TBM was supposed to be dismantled, transported to the Pryadharshini Park site piece by piece, and reassembled for re-launching at the same place for second drive. However, based on the internal analysis, it is concluded that such scheme may take more than 4~5 months compared with adopting U-turn scheme in the retrieval shaft located in Girgaon Chowpatty site. Hence, the original relaunching scheme of TBM had been changed into the U-turn and re-launching at Girgaon Chowpatty site.

Hydraulic skidding system had been introduced for the effective U-turn technique of TBM in the re-launching scheme. The skidding system has two major components, linear skidding system which moves the TBM in linear direction such as forward or transversal movement and the circular skidding system which rotates the TBM up to 180 degrees for U-turn.

The skidding system is composed of skid track, skid pad, skid jacks, and skid shoe. Skid track is supposed to be seated on the steel plate installed at the bottom of the shaft and skid shoe (rubber plate with Teflon and pasted with grease) is installed on top of the track and skid pad is supposed to be rode up on the shoe. Multiple skid jacks (i.e. Max. 25MT for linear skidding for each, Max. 80MT for circular skidding for each) are fixed on the track and connected with the skid pad and then TBM is supposed to be rode up on the pad. Once the jacks are pushed the connected skid pad, TBM machine will start to slide through the greased skid shoe. 16 numbers of vertical hydraulic jacks which are 250MT each were installed for slightly lifting the TBM to secure the space for the installation of skidding system and ride up the TBM on the skid pad. TBM can be moved in max. 550~600mm for one stroke and then jacks are relocated and reinstalled for the next move.

On the linear skidding scheme, TBM moves forward 5m away from the head wall and then moves 3.5m left transversally along the pre-installed skid tracks. After the removal of those linear tracks, circular tracks for circular skidding system are installed and the circular skidding is activating up to the rotation of 180 degrees with 4 numbers of 80MT skid jacks (Figure 8, b). The system has been strictly simulated at the factory (Figure 8, a) before the installation on site.

After the completion of circular skidding, backup gantries for TBM was pulled out and lifted on surface by the Mega lift crane and a few more linear skidding for TBM was carried out for the next re-launching sequence.

(a) Skidding system simulation at the factory *(b) 180 degree skidding work on TBM*

Figure 8. TBM Tunnel U-turn Scheme with hydraulic skidding system

4 VENTILATION AND FIRE PROTECTION SYSTEMS

4.1 *Tunnel Ventilation System*

The ventilation system adopted for MCRP4 tunnel is Saccardo Ventilation System. In both tubes, a technical room (i.e. tunnel operation room) with ventilation station was installed at the beginning of the tunnel, at a distance of 100 m from the beginning of the cut & cover tunnel section. Fresh air supplies all the way through the tubes through the Saccardo nozzles (Figure 9). The system has been used over the world but first introduced and adopted in India.

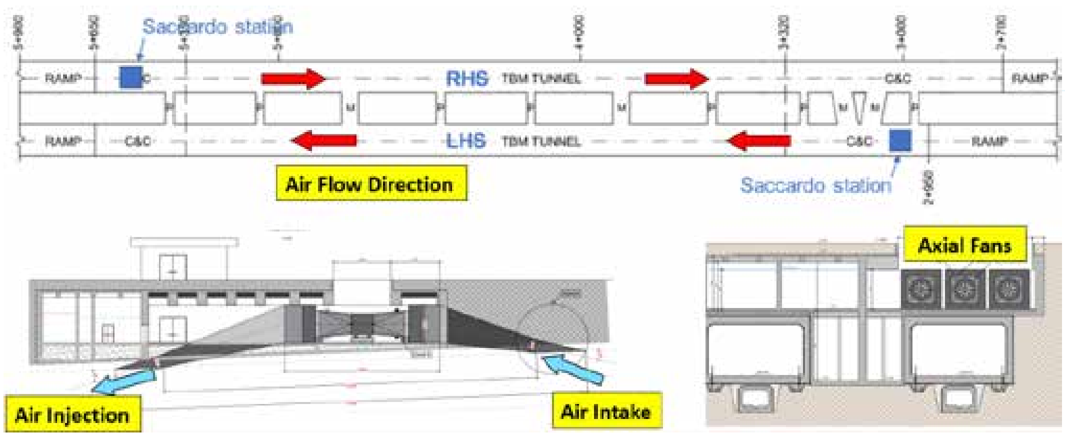


Figure 9. Saccardo Tunnel Ventilation System adopted for MCRP4

While the Saccardo ventilation system was first introduced as a longitudinal tunnel ventilation idea driven by a single injection point at the end of 19th century, its positive features have not been highlighted adequately. Hence, its practical approach had been studied and adopted at the latter quarter of 20th century.
Figure 10 shows the case examples which Saccardo ventilation system had been adopted worldwide in both railway and road tunnels.

Tunnel	Location	Year	Reference	Type
Procchia	Procchia, Italy	?	Saccardo (1894)	?
Coen	Amsterdam, Holland	1960s	Costeris & Sweetland (1994)	road
Benelux	Rotterdam, Holland	1960s	"	road
Tijsmans	Antwerp, Holland	1980s	"	road
Cochem	Kolbenz-Trier, Germany	?	Felli (1976)	rail
SURS	Singapore	pending	Kennedy, Colino & Elpidorou (1994)	road
Liberty	Pittsburgh, Pennsylvania, USA	1920-40s	"	road
Wheeling	Wheeling, west Virginia, USA	1960-65	"	road
New River	Fort Lauderdale, Florida, USA	1965-70	"	road
Kan-etsu	Minakami, Central Japan	1985	Baba, Ohashi & Jyozuka (1988)	road
Tsuruga	Hokuriku Express, Japan	1980	Baba, Ohashi & Uesaki (1982)	road
Ena-San	Japan	1982	Japan Highway Public Corporation	road
Yanagase	Japan	1975	Konda & Mizutani (1976)	road
Higo	Japan	1984	Japan Highway Public Corporation	road
Thames	Greenhithe (CTRL), UK	2006	Tabarra, Matthews & Kenrick (2000)	rail

Figure 10. Saccardo Ventilation System adopted worldwide [1]

A comparison of the technical and economic features of Saccardo ejectors with respect to jet fans reveals that: [1]

- Jet fans have little or no civil engineering costs for installation, but they will have significant electrical cabling costs. On the other hand, Saccardo fans require expensive civil engineering work to install the fans at the tunnel portal, with no cabling distribution costs.

- Routine maintenance or emergency repair work on jet fans will interfere the normal tunnel operation; this is not the case for Saccardo fans.
- The absence of electrical cabling within the tunnel in Saccardo fans is a clear safety advantage.
- Jet fans take up some space in the tunnel ceiling which limits the effective kinematic envelope of the traffic, whereas Saccardo ejectors are located outside the tunnel making them ideal in tightly spaced tunnels.
- Saccardo ejectors deliver their thrust at a single point, making them quite vulnerable to local tunnel fixtures. For example, a badly placed traffic sign, LED display, lighting equipment or any significant form drag near the outlet of an ejector will cause a dramatic drop in ejector performance, whereas jet fans are less affected, as their thrust is distributed.
- Jet fans are not only vulnerable but also derated when operating at elevated temperatures in a fire environment (lower density), whereas ejectors are both safely outside the fire's reach as well as immune to thrust reduction by virtue of using fresh air for primary intake. This makes Saccardo ejectors ideal for emergency smoke clearance.

Other than the features illustrated above for the Saccardo system, it is also true that there has been a criticism about the system itself when it is subjected to fire incidents such as:

- As the system operates always in single direction continuously and only controlled in operation room, flexible operation which requires different flow direction to control the smoke or independent flow operation due to local air leakage is limited.
- As the system is supposed to be activated by Saccardo fans in ventilation station at C&C section, if one of those fans are failed, tunnel ventilation quality might be degraded and also such situation can be critical one at the time of fire incident.

In MCRP4 tunnel, those risks pointed above had been reviewed and mitigated through 1D and 3D CFD (Computational Fluid Dynamics) analysis (Figure 11). Most possible critical fire scenarios had been reviewed case by case and the results showed somehow positive in pressure and critical velocity inside the tunnel during fire incidents.
To mitigate the risks on axial fan failure in Saccardo system, emergency axial fans were provided for extra, thus two axial fans are operational in normal condition, thus even if one of fans are failed, extra fans can be activated without degrading the ventilation quality (Figure 11).

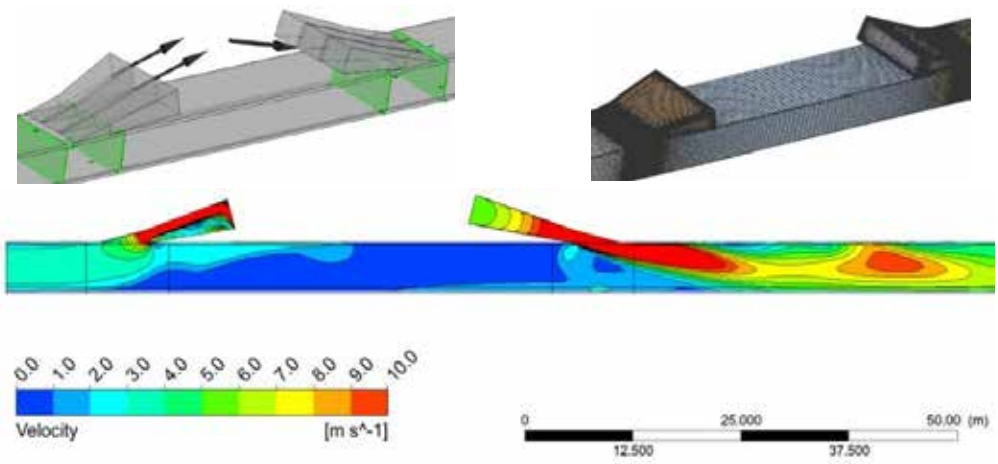


Figure 11. 3D CFD Analysis on Saccardo Tunnel Ventilation System adopted for MCRP4

4.2 *Structural Protection System during Fire Incident*

In case of fire incident, structural protection on tunnel lining is one of the big concerns, as the elevated temperature from fire incident may damage the segment tunnel lining significantly and such damage may cause a falling of hanging structures and highly dangerous to fire fighters. Moreover, considering the recovering time to tunnel operation due to the structure damage, structural protection shall be ensured at any cost.

In MCRP4, requirement for the structural protection was maximum 350°C in concrete interface temperature and steel reinforcement temperature shall be below 200°C for 180min of fire exposure in accordance with Rijkswaterstaat (RWS) time/temperature curve, which is little more conservative than that of NFPA 502 which are 380°C and 250°C respectively.
To reach this requirement, structural protection system has been ensured by adopting Fire Board which encapsulating the concrete tunnel lining with Calcium Silicate Board. As such system has been introduced for the first time in India, verification for the material need to be carried out.
Real scale verification test on fire board has been carried out at CBRI, (Central Building Research Institute), IIT, Roorkee, India (Figure 12). Fire exposure during the test had been maintained up to maximum 1100°C as per HC (Hydrocarbon) curve and the temperature data was converted into RWS curve.
Figure 12 (a) shows the test setup with actual concrete segment encapsulated by the fire board and Figure 12 (b) shows the status of fire board after the completion of the test. Some cracks on the board had occurred due to the tension contraction of the board during the cooling process after the test completion. However, no cracks on the board were found during the fire test and no cracks or any damages were found on the concrete segment as well. The interface temperature of concrete and steel reinforcement were maintained below 350°C and 200°C respectively during the test.

(a) Fire Board Testing Setup *(b) Fire Board Samples after the Test*

Figure 12. Fire Board Test for Structural Protection

Figure 13 shows the installation process of the fire board and completed section. Note that inside of the TBM bored tunnel was almost fully covered with the fire board from walkway to walkway thus concrete segmental linings can be protected during fire incidents, and structural integrity will be ensured up to 3 hours as per the project requirement.

(a) Fire board installation (b) Fished section of fire board installation

Figure 13. Fire Board Installation in TBM Bored Tunnel

5 CONCLUSION

MCRP4 (Mumbai Coastal Road Project Package IV) from Princess Street Flyover to Priyadarshini Park Comprises of design and construction of twin bored transportation tunnels under coastal area equipped with all automation and tunnel ventilation systems within it.

During the journey of TBM driving in last two years, there have been many challenges on risks working under sea area and first experience with large diameter of TBM in India.

Nevertheless, those challenges and experiences will be a good lessons and learns in tunnel engineering industry and some of outcomes and challenges during this project are summarized as following.

- Geological condition in MCRP4 was not highly favorable to tunneling works for TBM tunneling as it shall advance under sea area and tunnel alignment extents through a quite variable types of materials ranging from loose soils to rocks with different degree of strength, fracturation and weathering. Hence, TBM works had been continuously coped with mixed face which makes them struggle with clogging problems with frequent cutter head interventions. Also, highly loose rocks with thin overburden was high risks especially during TBM driving on high tidal time. Ground water control during cross passage excavation was also high risks when working under beach area. Those risks had been always challenged but mitigated with ground treatment on surface and pre-grouting for waterproofing before the excavation commenced.
- Hydralic skidding system had been introduced for the effective U-turn technique of TBM for re-launching in MCRP4 tunnel. By adopting such a construction technique, TBM re-launching for LHS tunnel could be commenced within a month from the breakthrough of RHS tunnel with significant saving in time and cost. There have been some efforts to adopt such a U-turn technique worldwide but most of those techniques has been limited to using turntable or on relatively smaller TBM. The hydraulic skidding system adopted in MCRP4 is for one of largest and heaviest TBMs and it is believed to be one of the successful construction records.
- Saccardo ventilation system adopted in MCRP4 project has been used over the world

but first introduced in India. While the Saccardo system was first introduced 100 years back, its positive features have not been highlighted adequately and there has been growing interest and usage on this system in last decades. By taking sophisticated CFD analysis and QRA (Quantified Risk Analysis), major risks adopting this system has been predicted, mitigated and convinced for the stability of the system.
MCRP4 tunnels are now officially opened to public and operational since March 11, 2024 (Southbound lane) & June 10 ,2024 (Northbound lane).

- In case of fire incident, structural protection on tunnel lining is one of the big issues in design and construction to resolve, as the elevated temperature from fire incident may damage the segment tunnel lining significantly and such damage may cause a falling of hanging structures and highly dangerous to fire fighters. Moreover, considering the recovering time to tunnel operation due to the structure damage, structural protection shall be ensured at any cost. MCRP4 tunnel should be highly secured structure in Mumbai considering its importance and role as a VVIP road around the area and risks passing beneath the sea area. The Calcium Silicate Fire Board adopted as a structural protection system in MCRP4 is costly option among other fire protection systems such as PP fiber or fire resistant coating and introduced for the first time in India.
  Real scale verification test on the fire board has been carried out at CBRI, (Central Building Research Institute), IIT, Roorkee, India, and it was convinced that the system could protect the structure as per the requirement and adequately protect the structure under extreme fire situation.

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CHALLENGES AND REMEDIAL MEASURES FOR CAVITY TREATMENT DURING TUNNELLING IN THE HIMALAYAN REGION: INSIGHTS FROM THE ARUN-3 HYDROELECTRIC PROJECT (900 MW), NEPAL

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ABSTRACT

Tunnelling and underground works for hydropower projects in the Himalayan region present significant geological and construction challenges. The complex tectonic setting, intense jointing, continuous deformation, contrasting rock types, clay-filled fault zones, shear zones, and substantial groundwater ingress collectively create adverse conditions that hinder progress and demand continuous adaptation of design and construction methodologies.

During the construction of the Head Race Tunnel (HRT) of the Arun-3 Hydroelectric Project (900 MW), several challenges were encountered, including high overburden stresses, weak rock masses, shear zones, cavity formation, squeezing ground conditions, and heavy water inflows. This paper specifically focuses on the challenges associated with cavity formation during HRT excavation and the mitigation measures adopted to overcome them. The cavity developed at Face-7, between RD 575 m and RD 605 m, where the rock mass comprised Augen gneiss, gneiss, and a thick weak band of sericite-chlorite schist with highly clayey characteristics. Continuous water inflow led to progressive deterioration of the rock mass, further aggravating the instability. The paper discusses the evolution of rock conditions during excavation and illustrates how the design approach and construction techniques were modified in response to the encountered ground behaviour.

GENERAL DESCRIPTIPON OF ARUN-3 HYDROELECTRIC PROJECT

The Arun-3 Hydroelectric Project (900 MW) is located on the Arun River, a major tributary of the Sapta Koshi River, in the Sankhuwasabha District of eastern Nepal. It is an export-oriented project currently in an advanced stage of construction, with the excavation of the Head Race Tunnel (HRT) fully completed. The project is designed to utilize a discharge of 344.68 m^3/s and a net head of 286.21 m to generate approximately 3,924 million units (MU) of electricity annually.

The project comprises an 80 m high concrete gravity dam, two intake structures, and a 12 km long Head Race Tunnel (HRT) of 9.5 m diameter, excavated through four construction adits. The water conveyance system leads to a 155 m high, 28 m diameter surge shaft, followed by two underground pressure shafts of 5.5 m diameter each, which further bifurcate into four branch pressure shafts of 4 m diameter each, connecting to the underground powerhouse complex. The powerhouse houses four Francis turbine-generating units, each rated at 225 MW. The discharge from the turbines passes through draft tube tunnels to the tailrace system, which leads to the Tailrace Pond. From there, water is diverted to the Lower Arun Hydroelectric Project (669 MW), which has been planned as the tailrace development of the Arun-3 HEP and will operate in tandem with it. The layout of project is shown in Figure 1.

HEAD RACE TUNNEL

The Head Race Tunnel is 9.5m diameter horse shoe shaped concrete lined of length 11.8km. To facilitate the construction of Head race tunnel four number construction adits are constructed as per the details shown in the figure-2. Each Adit provides two numbers of faces to facilitate the excavation of HRT, thereby total 8 numbers faces were available for the speedy construction of HRT. All the project components are located on the left bank of the river Arun. The HRT alignment crosses through the various nallahs/kholas, and the cover / overburden along the alignment is ranging from as high as 1km in the reaches of Face-2& face-3 and lowest value of cover is 80m in the Khola reaches.

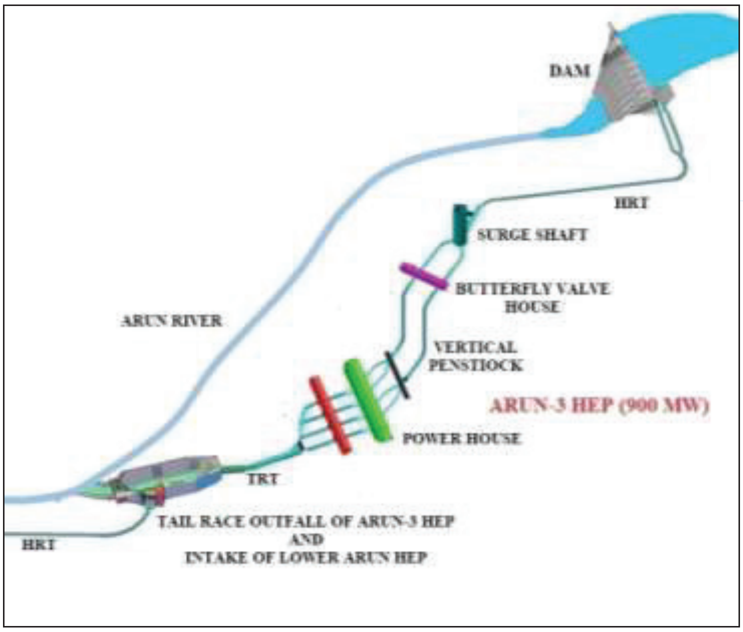

Figure 1: Layout of Arun-3 HEP

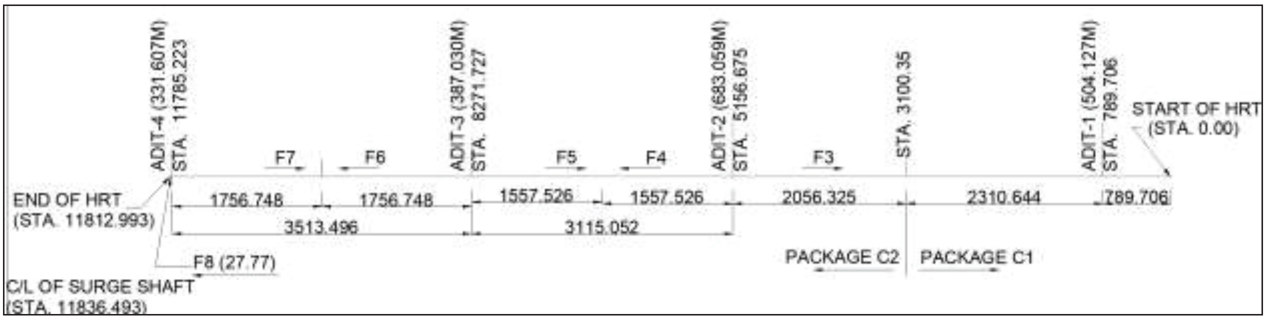

Figure 2: Line diagram of HRT showing face length arrangements.

CAVAITY FORMATION IN HRT U/S OF ADIT-4 AT RD 575M TO 605 M

During the heading excavation of Face-7 from the upstream reach of Adit-4, approximately 8 m of heading excavation was completed up to RD 575 m. Subsequently, another round of blasting for the next 2 m pull was carried out. At that time, installation of the support system comprising steel rib erection and an umbrella of steel channels was in progress for the prevailing Rock Class VI conditions.

During these operations, small rock fragments began to detach from the crown portion of the heading. This progressive detachment eventually led to a major collapse, resulting in complete blockage of the tunnel face and the formation of a cavity. Approximately 400–500 m³ of muck was deposited in front of the tunnel face as a consequence of this collapse.

Cause of the Cavity Formation

The HRT reach from RD 545 m to RD 575 m had been supported with designed support comprising steel ribs at 1 m center-to-center spacing and backfilled with concrete due to poor rock conditions. Excavation from RD 575 m to RD 577 m revealed that rock fragments had begun detaching from the crown. The detached material was extremely weak and disintegrated into a soil-like substance, indicating very low uniaxial compressive strength of the rock mass (refer to Figures 3 and 4).

The rock cover in this reach ranges between 400 m and 450 m, corresponding to an estimated vertical stress of 11–15 MPa, based on the overburden and rock density. Field observations confirmed that the rock mass was highly weathered and soft, with a soapy texture that exhibited swelling upon water absorption due to its clayey composition. The rock's strength was inadequate to sustain the overburden stresses, particularly under conditions of minor water ingress, which was further aggravated by heavy rainfall in the region. Consequently, the combination of high overburden pressure, low rock strength, and water-induced weakening led to the collapse and subsequent cavity formation in this reach.

Figure 3: Before collapse

Figure 4: After collapse

Figure 5: Channel umbrella

Figure 6: Depostion of muck infront of face-7

Geology of the reach

Approximately 10 to 15 m in the preceding reach of the tunnel encountered a similar problem, characterized by a thick and weak band of sericite chlorite schist at the crown. The thickness of this weak band varies between 12 m and 14 m. This band was exposed in the tunnel crown from RD 575 m onward and subsequently collapsed due to its self-weight and the low compressive strength of the rock. The foliation joints in this zone dip in the direction of 110°, with a dip amount of 10 to 15° (indicating a very low dip). The tunnel excavation was oriented along N70°E, forming an angle of approximately 50° between the strike of foliation and the tunnel axis, with the foliation planes becoming flatter near the crown.

The major rock types encountered in this reach include augen gneiss, gneiss, and a thick band of sericite chlorite schist, the latter being highly clayey in nature. Multiple intersecting joint sets were observed, resulting in fragmentation of both the augen gneiss and gneiss rock masses.

The rock mass was classified as Class VI in this section; however, it deteriorated further under the influence of water ingress and flowing conditions. Considering the foliation dipping toward the face at a low angle, regression of the cavity toward the excavated face was anticipated. Therefore, continuous monitoring of the support system in the rear reach of the tunnel was deemed essential.

MITIGATION MEASURES

Following the collapse at RD 575 m of Face-7, which resulted in the formation of cavity, detailed deliberations were held considering the prevailing geological conditions, construction schedule, and excavation methodology. To address the issue of cavity formation and ensure the safe and timely completion of the Head Race Tunnel, two alternative approaches were evaluated, as described below.

Alternative–1: Bypass Tunnel Alignment

An alternative tunnel layout was proposed to bypass the collapsed reach of the Head Race Tunnel (HRT), as illustrated in Figure 7. This option aimed to maintain construction progress without undertaking immediate treatment of the existing cavity. Under this arrangement, the heading excavation from Face-7 would have continued along the revised alignment. However, this alternative required an additional 152 meters of tunnel excavation and the introduction of two additional bends in the HRT alignment.

The increased tunnel length and curvature would have resulted in higher construction costs and additional head losses, leading to a perpetual reduction in power generation during the operational phase of the project. Considering the cost–benefit implications and long-term operational efficiency, this alternative was deemed uneconomical and subsequently rejected.

Alternative-2 treatment of the collapsed reach

A joint site inspection was conducted with representatives from the design, geology, and project implementation teams to assess the cavity conditions and finalize the appropriate mitigation strategy. Based on detailed discussions and site observations, it was collectively decided to adopt the following methodology for the treatment of the cavity:

1. *Assessment of Cavity Extent*

Carefully open the cavity near the crown to determine its extent, where feasible.

2. *Initial Support Installation*

If no further rockfall occurs from the crown, install steel ribs at spacing determined by prevailing geological conditions, followed by concrete backfilling.

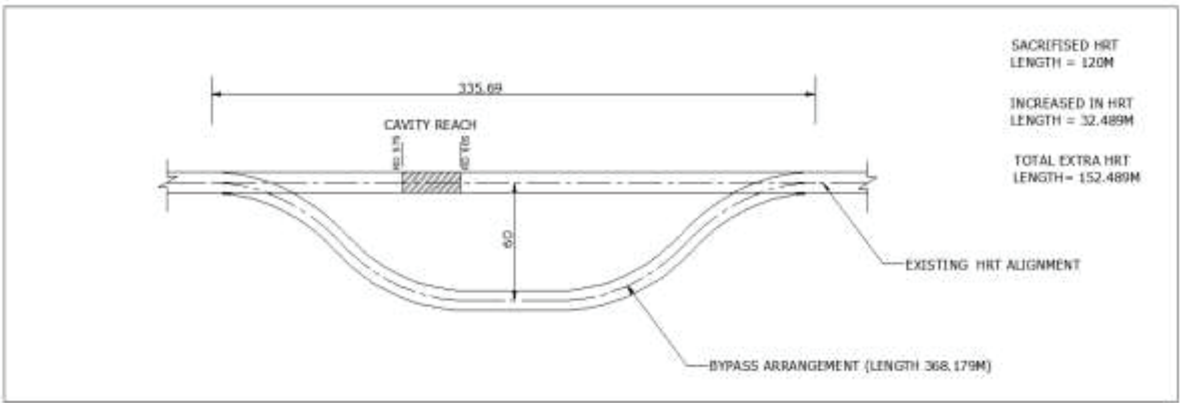

Figure 7: Cavity bypass arrangement

3. *Grouting in Case of Continued Rockfall*

If rock detachment continues, perform systematic grouting through the steel ribs already installed near the cavity to stabilize the surrounding rock mass.

4. *Forepoling and Umbrella Arch System*

   Install a forepole umbrella at the crown, consisting of grouted steel pipes (32 mm diameter) with rock bolts inserted into the pipes to enhance structural integrity. The forepoles should be inclined at an angle of 10°–15° to the horizontal, with lengths varying between 6 m and 12 m, depending on machine drillability and rock conditions.

5. *Steel Rib Support and Backfilling*

   Provide steel ribs at spacing of 0.75 m to 1.0 m, reducible to 0.5 m in weaker zones depending on stand-up time and rock mass quality, followed by concrete backfilling.

6. *Controlled Face Excavation*

   Conduct face excavation in short advances of 1.0–1.5 m (or less, as required) to allow safe erection of steel ribs and ensure face stability.

7. *Umbrella Overlap and Crown Grouting*

   Maintain an overlap of 3 m between successive forepole umbrellas and grout the overlapping rock mass in the crown to improve bonding and reduce seepage.

8. *Monitoring of Deformation*

   Install bi-reflex targets between RD 547 m and RD 577 m to monitor deformation and movement of the rock mass during and after treatment activities.

CHALLENGES FACED DURING TREATMENT OF THE CAVITY REGION

During the treatment and stabilization of the cavity zone as per alternative, several significant challenges were encountered owing to the complex geological and geotechnical conditions prevalent in the area. The forepoles provided as pre-support along the longitudinal direction failed in the central portion of the crown, extending partially to both sides, primarily due to inadequate bonding in the weak rock mass. During the removal of large boulders, muck flow was observed from the face and crown**,** indicating continued instability and movement within the loosened rock mass ahead. The movement of loose material from the cavity zone caused disturbance to the installed support system in the rear reach, compromising its effectiveness. This movement led to additional loading on the erected steel ribs**,** particularly in the crown area, further reducing their stability and performance. As a result, both the excavation face and the immediate support system were found to be vulnerable and at risk of progressive deformation or collapse under prevailing stress and geological conditions.

ADDITIONAL MEASURES ADOPTED TO TACKLE THE CAVITY

Considering the above challenges and the continuing instability of the rock mass, a series of additional stabilization measures were implemented to ensure safe and effective treatment of the cavity reach. The rear zone support was strengthened using 32 mm diameter, 6–7 m long fully grouted rock bolts, combined with an additional layer of shotcrete to embed and reinforce the exposed portions of the steel ribs**.** To prevent uncontrolled muck flow while handling large boulders within the slumped mass, a hydraulic breaker was used instead of the excavator bucket, allowing controlled fragmentation and minimizing sudden collapse. Based on site conditions, further excavation was carried out in a central gullet configuration of 6–7 m span, allowing controlled advance and systematic support installation (refer Figure 8). Steel ribs of size ISHB 200×200 mm or higher were installed at 0.75–1.0 m spacing wherever available. In constrained conditions, ISHB 150×150 mm sections were used at closer spacing of 0.5–0.75 m to enhance overall structural stiffness.

Figure 8 : Excavtion in form of the central Gullet

Figure 9 : Face treatment

A forepole umbrella system was adopted using 12 m long, 114 mm diameter steel pipes, each fitted with 32 mm diameter rock bolts grouted inside, placed at close spacing of 30–40 cm, to reinforce the crown region and minimize ground relaxation. Grouting of the slumped mass with thick cement grout was undertaken to consolidate loose material, enhance bonding, and reduce permeability in the weak zones. Water ingress, if any, was carefully collected and channelized away from the excavation face to prevent further deterioration and softening of the rock mass**.** Following the successful excavation and stabilization of the central gullet, the excavation of tunnel beyond the gullet was excavated with full section of heading. Though the cavity was stabilized successfully, for the safety of workers and from experience gained in blockage of many tunnels in India, a provision of pipe was kept to evacuate the workers in the eventuality of future instability of the cavity regions. (See figure 10).

WIDENING OF CAVITY REGION

After the completion of excavation of tunnel beyond the gullet systematic widening operations were undertaken to achieve the full tunnel cross-section. These works commenced immediately after the completion of the overt lining and invert concrete near the starting point of the gullet, in strict compliance with approved safety and engineering standards. Prior to widening, the stability of the central gullet was confirmed through Deformation Reading Target (DRT) measurements, while loose materials along the gullet walls were cleared and excavation boundaries for the final profile were clearly marked. The existing support system, including fore-poling, steel ribs, grouting, and tie rods, was inspected and found satisfactory for proceeding. Widening began with the excavation of the unexcavated portion left on both the left and right sides, during which ISHB 150 × 150 mm steel ribs were installed at 500 mm center-to-center spacing up to the spring line level to ensure stability. The exposed rock surface was covered with wire mesh, followed by plain shotcrete and concrete backfilling behind the vertical ribs, while 25 mm diameter tie rods were installed below the spring line to anchor the support system. The same procedure was followed for the opposite side to complete the full lateral section, ensuring alignment and continuity with previously installed supports. Subsequently, the remaining unexcavated crown portion was removed, and arch ribs were installed and joined with the vertical ribs to complete the full tunnel profile. Concrete backfilling was carried out above the crown to fill voids and strengthen the cavity zone, while 60 mm diameter HDPE drain pipes were provided to facilitate drainage from the crown area.

Figure 10 : Widening of central Gullet with Safety Pipe at left side.

Figure 11: full section width after widening

The widening of the center Gullet to the full section was successfully completed (See Figure 11) as per design support including ribs, tie rods, wire mesh, shotcrete/SFRS and drainage ensuring the structural stability and safe working conditions throughout the operation.

CONCLUSION

Tunneling through weak and sheared rock formations in the Himalayan region poses major geological and construction challenges. The cavity encountered in the Head Race Tunnel of the Arun-3 Hydroelectric Project between RD 575 m and RD 605 m demonstrated the complex interaction of poor rock quality, groundwater ingress, and high in-situ stresses. Through coordinated efforts of the design, geology, and construction teams, the zone was successfully stabilized using controlled excavation using multi drift method, close-spaced steel rib supports, systematic forepoling, and continuous grouting, enabling safe resumption of tunneling. Despite the inherent risks of working in a cavity-prone section, the adoption of an integrated system involving umbrella forepoling, wire mesh, ribs and tie rods, and real-time deformation monitoring ensured safe and effective tunnel advancement. The mechanical execution of each activity not only facilitated progress but also enhanced worker safety and long-term tunnel stability. The experience emphasizes the importance of proactive geological assessment, adaptive design, and integrated monitoring for managing adverse ground conditions in Himalayan tunneling projects.

PARAMETRIC INVESTIGATION OF TUNNEL EXCAVATION EFFECTS ON GROUND BEHAVIOUR THROUGH NUMERICAL ANALYSIS

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ABSTRACT

This paper presents parametric evaluation of the ground movements through Finite Element Model (FEM) by using PLAXIS-2D for a typical tunnel of 5 to10 m diameter at a depth of about 9 m from the surface. The need for tunnels has been noticed, with the rise in population and rapid urbanization for frequent communications like road or rail networks, sewage disposals, pipelines, water supply in urban areas and also for various functions in mining sector. Tunneling leads to ground movements with relatively change in displacements, which are evident to the damages caused to the super-structures, sub-structures, surface and sub-surface. For planning and designing purpose, the underground structures like tunnel, there should be knowledge of the results obtained of ground displacements. The ground movements can also arise from the over-excavation of the tunnels. The total displacements encountered due to the damage caused by the construction of tunnels are required to be predicted for proper design of tunnels to mitigate any unstable situations. For the lining in the tunnel, shotcrete lining has proved to be more cost-effective and durable than the traditional concrete lining. A parametric study is carried to find out the influence of various geometrical and geotechnical parameters on the behavior of the surface and sub-surface due to tunnel construction, and validated with the measured parameters.

Keywords *: Total displacements, PLAXIS, shotcrete, geotechnical and geometrical parameters, numerical modeling.*

1. Introduction

The influence of various parameters on ground settlement, tunnelling, and ground control problems in tunnel construction is complicated, involving a complex interplay of geological, mechanical, and operational factors. These parameters can significantly affect the stability and safety of tunnel projects, especially in urban and high-stress environments. Understanding these influences is crucial for optimizing tunnel design and construction methods to mitigate risks and ensure structural integrity.

Recently, in November 2023, a tunnel collapse occurred at the Silkyara Bend-Barkot Tunnel construction site in Uttarakhand. Over 40 workers were trapped inside as the tunnel caved in during routine construction work. The accident was caused by a sudden collapse during construction, likely due to geological instability and inadequate safety measures which might have contained a hidden loose patch of fractured or weak rock, undetectable during construction (Anon,2023). So, for many years, there has been a maximum amount of interest from the researchers in developing numerical methods for generating total displacement due to tunneling. To measure the ground movement during tunneling, numerical methods like Finite Element Analysis (FEM) are used. The numerical methods can predict the interaction between the surrounding soil, subsoil, and tunnel, as well as the elastic plastic behavior of the substructure and the complexity during the tunnel construction underway. The construction of tunnels underneath the surface involves a large number of problems.

As per the IS 5878-3 (1972) [Code of practice for construction of tunnels], the practice of tunneling in soft strata would differ with the softness of the strata, subsoil water pressures, and other facilities available for construction like tunnel boring methods (TBM). The technology that would be applied to the soil strata for tunneling purposes would decide the tunneling method, which in turn would depend on the area of tunnel sections. The face from which the tunnel construction starts has required to be decided concerning the rock cover provided to the tunnel. The minimum cover that has been provided to the tunnel depends on the structure, the type of rock mass, and the shape and size of the tunnel [1]. Additionally, depth-related stresses and instability in natural supports during excavation becomes more prominent under higher stresses and in-situ conditions [2].

According to the IS 4880-2(1976) [Code of practice for the design of Tunnels conveying water], the geometric design of the tunnel depends on factors like geological, hydraulic, structural, and functional factors. So, the shapes that are generally used for tunneling are circular sections, D sections, horse-shoe sections, modified horseshoe sections, and egg-shaped sections. In this study, the circular section has been adopted for the tunnel cross-section. The circular cross-section is the most suitable consideration from a structural point of view [3].

The main purpose of this work is to analyze the influence of tunnel construction on the ground in terms of the total displacement developed and the horizontal movements without the existence of any superstructures, with the help of numerical methods i.e. FEM based on the Mohr-Coulomb behavior criterion.

1.1 The Key Parameters and Their Impacts on Tunneling and Ground Control Issues

- ***Soil and geological conditions*** like soil properties and geological variations. For example, factors such as the elastic modulus of gravel and coarse sand have a major impact on settling results in soft soil conditions [4]. Different deformation and internal forces within the tunnel structure may result from the existence of multi-layered earth formations and the transition between different types of soil [5]. The stratigraphy and shear strength of soil layers also play a vital role in ground movement, particularly in mixed face conditions [6].
- ***Tunnel design and construction parameters*** like tunnel lining and grouting; and tunnel geometry. The stiffness of tunnel lining and strength of grout used for construction are very critical in control over ground settlement. These phenomena dictate much of the tunnel's capacity to resist external pressures and remain stable [7]. Distribution of internal forces in shaft is also depended on diameter and depth of the tunnel as well. Greater diameters and deeper depths of tunnels are sensitive to changes in the underground environment [5].
- ***Operational factors*** such as shield machine parameters; and advance rate and pitching angle. The operational parameters of the shield machine, such as its drilling speed and earth pressure in excavation chamber are significant for soil-tunnel interaction and minimizing settlement [8], [9]. In the soft soil situations, where exact control over the shield machine's movement helps lower settlement hazards, the advance rate and pitching angle parameters are important [10].
- ***Ground control strategies*** such as monitoring, adjustment, and support systems. Choosing suitable support systems, including shotcrete shells and energy-absorbing rock bolts, is crucial for controlling rock burst and compressing ground conditions in high-stress situations [11]. For the reinforcement strategies like side bolting with wire mesh are effective in limiting spalling and enhancing natural support stability in stressed geological conditions [12]. For efficient risk management and settlement control, tunnelling parameters like grouting pressure and advance speed must be monitored in real time and managed adaptively [9].

These parameters were used to analyse the behaviour of the soil and tunnel under different conditions, with results showing how variations in these factors affected tunnel stability. Even if the previously mentioned components have a significant impact on ground settlement and tunnelling difficulties, it's important to take the entire tunnel construction context into account. Also, the tunnel interacts with the surrounding soil, the type of tunneling such as slurry shield machines or earth pressure balance, can also have an effect on settlement results [13]. Furthermore, incorporating innovative predictive models like Artificial Neural Networks (ANN) can improve the precision of settlement predictions and guide better choices throughout the design and building of tunnels [8].

1.2 Objectives

The primary objective of this study is to:

1. Assess ground movements brought on by tunnelling and looking into geotechnical and geometrical influence to the surrounding strata
2. Enhance the planning and design of underground structures
3. Simulate the tunnel behavior

2. Materials and Methods

The tunnel considered for the research purpose is the Line-4 Tehran, Iran subway. The yellow line is shown in the Figure 1 is the Line-4 Tehran Subway of Iran and its junction with other lines. In previous studies of this particular tunnel by other researchers, numerical analysis was done to show sufficient tunnel resistance against axial, shear, and bending forces. This tunnel has been bored in the sand formation, and this formation includes two well-graded gravel layers (GW Layers) and one silty gravel layer (GM Layer). After drilling, the tunnel cross-section was 57.82m² which was further reduced to 56m² as the final cross-section of the tunnel. The length of the tunnel drilling was 2.5km [14]. A conceptual figure has been drawn and showed the two different layers of strata and their respective thickness, including the tunnel diameter in the Figure 2, also there has been a repetition of GW layer at two sections and the tunnel has been drilled at the GW layer.

Figure 1: Plan view of line-4 of Tehran Subway and other lines.

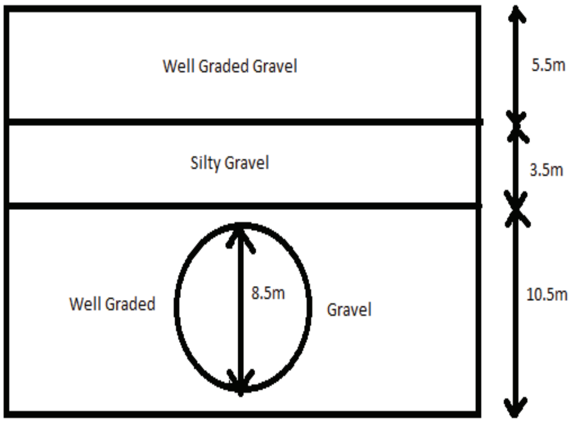

Figure 2: Geometry of different soil layers and the tunnel.

2.1 Materials Used for Tunnel Lining

1. Sprayed concrete, which is also known as shotcrete, is a type of concrete that is sprayed using a high-pressure pneumatic tube.
2. The materials that are composed to make shotcrete are sand, cement, water, and additives (Mineral fines such as micro silica).

3. It is preferred because of its high flowability; provides immediate and flexible support; cost-effective compared to traditional concrete; with better durability; and fire resistance.
4. Material Properties of Shotcrete such as density, Modulus of elasticity, E Poisson's Ratio, are 2500 kg/m³, 3000000 kN/m², and 0.2, respectively.

2.2 Methodology:

2.2.1 Finite Element Modelling (FEM) with PLAXIS-2D:

1. PLAXIS-2D version 20 software has been used to show soil-structure interaction. To understand the stability and performance of underground structures requires the ability to study the soil-structure interaction in detail, which PLAXIS makes possible.
2. The objective is to evaluate total displacements and the respective influence of geotechnical and geometrical parameters on the underground structures like tunnel.
3. The tunnel that has been studied and took as a case study was from Tehran Subway Line-4, which was drilled in sand formation with gravel layers.

2.2.2 Numerical Modelling:

1. The model included the soil profile, the shotcrete lining and the illustrated tunnel.
2. Geometrical parameters include on the diameter of the tunnel; and for the geotechnical parameters, friction angle (φ), Poisson's ratio (ν), and earth pressure coefficient at rest (K_o) has been considered.

2.2.3 Physical and Geotechnical Properties:

1. Soil Types: Well-graded gravel (GW) and silty gravel (GM) layers.
2. Geotechnical Parameters of Soil Layers:
   - *GW Layer:* Friction angle of 45°, unit weight of 22 kN/m³, modulus of elasticity 98,066 kN/m² and cohesion 49.033 kN/m².
   - *GM Layer:* Friction angle of 40°, unit weight of 20 kN/m³, modulus of elasticity 68,646 kN/m² and cohesion 12.75 kN/m²

2.2.4 Tunnel Specifications:

1. Cross-sectional Area= 56 m²
2. Diameter= Varied from 5 m to 10 m in different simulations.
3. Lining Parameters=
   - Axial Stiffness (EA)= 8×10^6 kN/m
   - Flexural Rigidity (EI)= 80,000 kNm²/m
   - Weight (W)= 8.4 kN/m/m
   - Poisson's Ratio (ν)= 0.15

3. Results and Discussion

This stage deals with the analysis of the behavior of the surrounding soil with the digging of the tunnel. Initially, the objective of this study was to evaluate the total displacements (horizontal and vertical shown in Figures 3 and 4), but later the influence of different geotechnical and geometrical parameters on the construction of the tunnel will be projected at various intervals in terms of displacements on the surrounding surface, as the construction of the underground structures and its safety factor is related to the displacement to restraint. Also, different stress conditions created after drilling of the tunnel which showed failure in structure are simulated using PLAXIS-2D software.

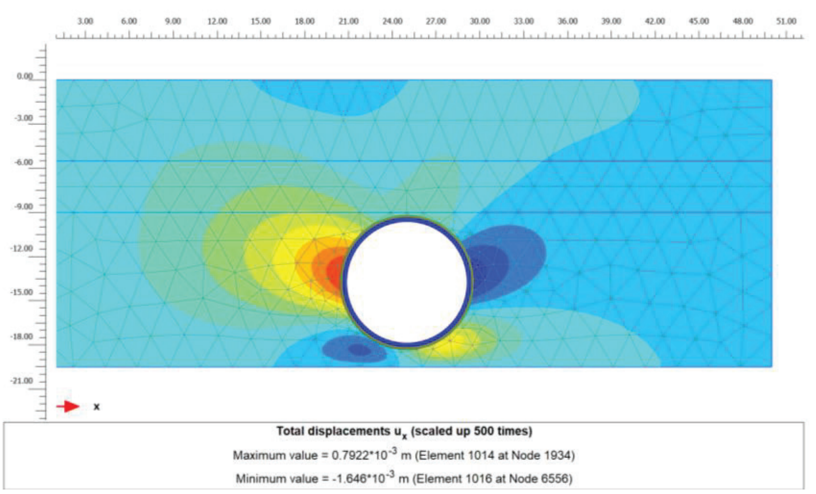

Figure 3: Horizontal displacements of the referred tunnel

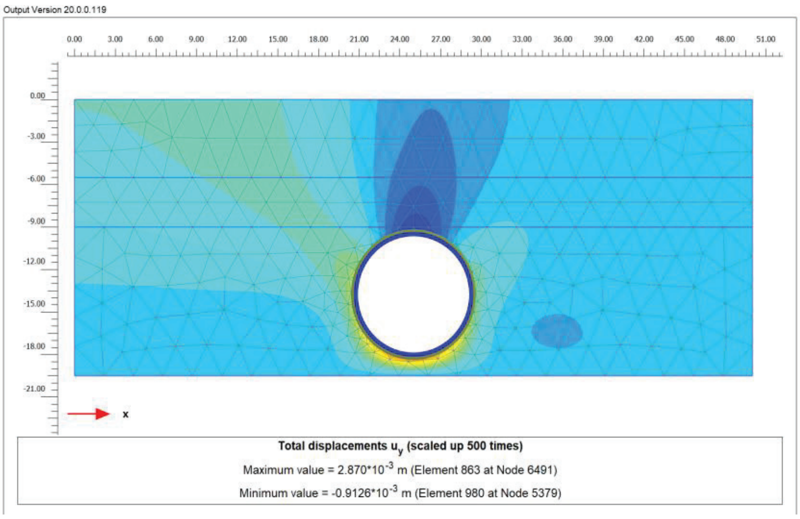

Figure 4: Vertical displacements of the referred tunnel

In the Figure 5, the axial force of the segment is decreased due to the horizontal movements. And the negative sign in the analysis shows the induced compressive stresses. The maximum axial force is found to be -746 kN/m and the minimum value is -1120 kN/m. In Figure 6 and 7, the maximum and minimum rate of shear force are 15.82 kN/m and -15.77 kN/m, respectively; and the maximum rate of induced bending moment has been simulated to justify the previous work of the referred authors [14].

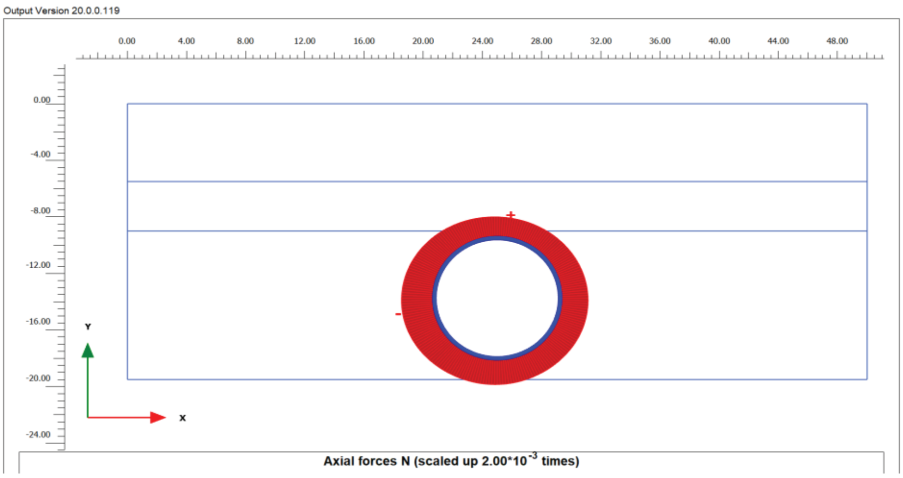

Figure 5: Impact of axial forces around the tunnel lining

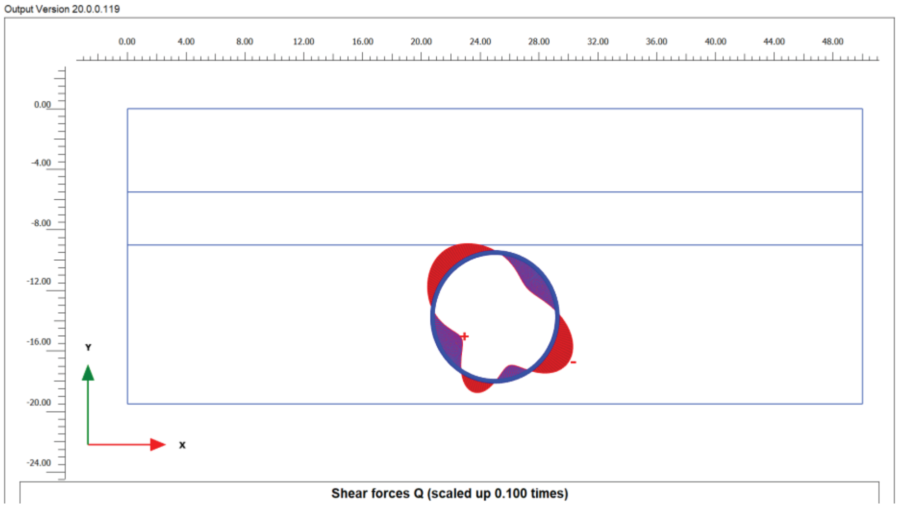

Figure 6: Induced Shear forces around the axis of the tunnel

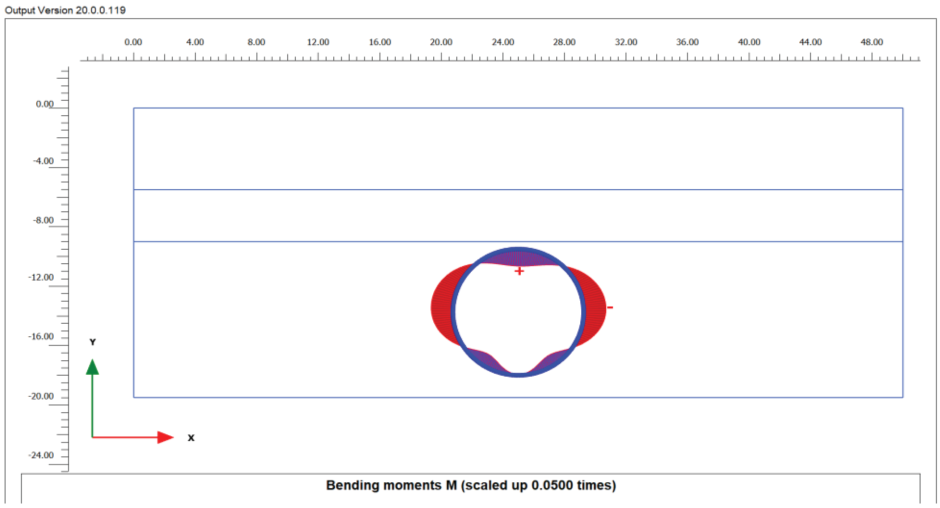

Figure 7: Induced bending moment around the axis of the tunnel

3.1 Influence of Geotechnical and Geometrical Parameters

The significant effect due to tunneling on the soil and its strata depends on different factors, such as the diameter of the tunnel, other mechanical properties like the coefficient of earth pressure at rest, the angle of internal friction, and the Poisson's ratio.

3.1.1 Influence of Diameter of the Tunnel

Five different calculations of different diameters which are lower, reference, and higher are considered, to study the influence of varying diameters on the movement of the ground. The other data will stay the same as those of the reference diameter (D = 8.5m). In the Figure 8., a graph has been derived which showed the various diameters' influence on the total displacements of the tunnel, and Table 1, comprises the values of total displacements and it showed clearly that the displacements around the tunnel are influenced by the varying diameter. From the analysis, it has been found that an increase in tunnel excavation diameter, increased the deformation in tunnel lining, but decreased the axial force, shear force and bending moment as shown below in Table 1.

In Figure 9, a clear analysis has been shown for the referred tunnel diameter influence and likewise other diameters' impact has been analyzed.

Table 1: Influence of the diameter of the tunnel

Diameter (m)	Total displacement (mm)	Axial force (kN/m)	Shear force (kN/m)	Bending Moment (kN m/m)
5	1.731	-540	42.98	52
6.5	2.473	-655	26.74	40.89
7.5	2.778	-709.1	20.22	32.36
8.5 [14]	3.044	-746.4	15.82	23.11
10	3.542	-760.9	12.44	16.7

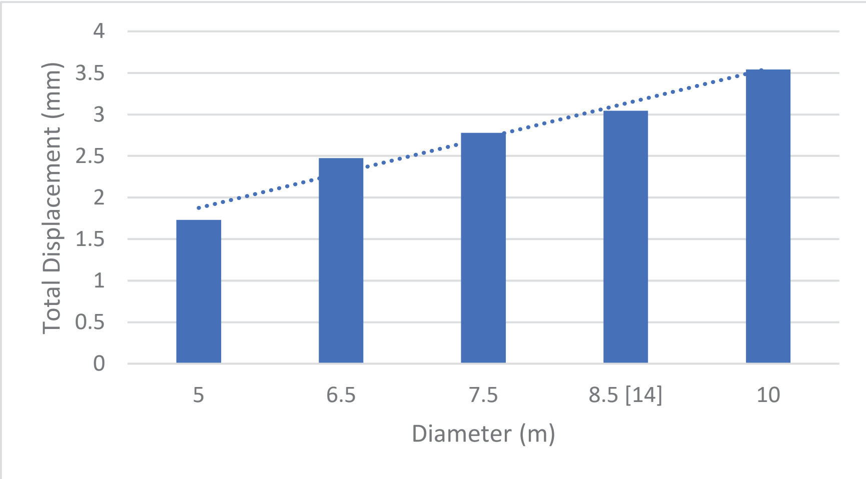

Figure 8: Influence of Diameter on total displacement of tunnel

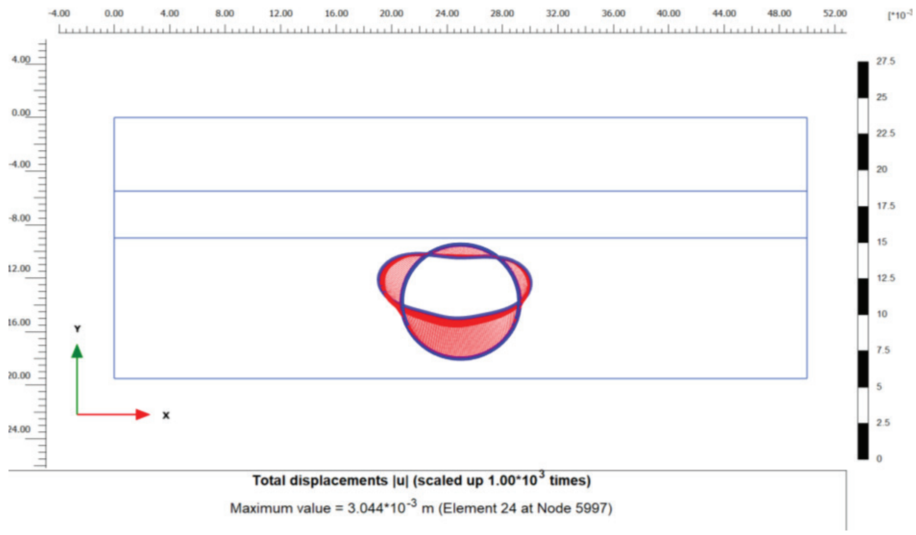

Figure 9: Total displacements in the tunnel (D = 8.5m)

3.1.2 Influence of Angle of Friction (φ) of the Ground

When an excavation takes place, the influence of this angle of friction on the ground has been derived with a graph in Figure 10. And the results obtained are shown in Table 2. There is an increase in the total displacements shown in the above graph. However, opposite results have been expected is the decrease in total displacements with the increase in friction angle, but the opposite results are obtained which is the increase in total displacements with the increase in friction angle by the numerical simulation. This is because PLAXIS used Jacky’s formula for the calculation of φ.

Table 2: Influence of angle of friction on the ground

Sl No.	Angle of friction, φ	Total Displacement (mm)
1	42°	2.244
2	45° [14]	3.044
3	48°	3.257
4	53°	3.381

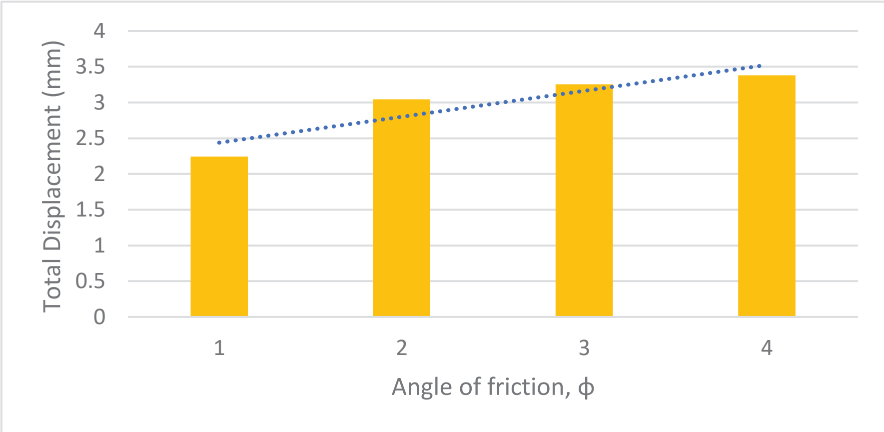

Figure 10: Influence of Angle of Friction on total displacement of tunnel

3.1.3 *Influence of Coefficient of Earth Pressure at Rest (K_o)*

This parameter defines the natural state acting on the soil. With regards to the digging of the soil, the geotechnical parameters have a serious impact on the soil behavior. After studying the influence of K_o, which is represented in Table 3 and a graph plotted in Figure 11, it has been found that, with the increase in K_o value, the displacements in the tunnel seem to be affected which is decreased accordingly.

Table 3: Influence of K_o

Sl. No.	Coefficient of earth pressure at rest, K_o	Total displacements (mm)
1	0.2014	3.381
2	0.2568	3.257
3	0.2929 [14]	3.044
4	0.3309	2.244

3.1.1 Influence of Poisson's Ratio, ν

This parameter determines the characteristics of the elastic behavior of a material (here soil). Two more calculations have been carried out other than the reference (ν=0.3), to study the influence of Poisson's ratio on the behavior of the soil. The results are shown below in Table 5 and the graph has been plotted in Figure 12. The results clearly showed that with the increase in Poisson's ratio, total displacements of the tunnel decreased. Below figures verified the results obtained.

Table 5: Influence of ν

Sl. No.	Poisson's ratio, ν	Total displacement (mm)
1	0.1	3.566
2	0.3 [14]	3.044
3	0.4	2.350

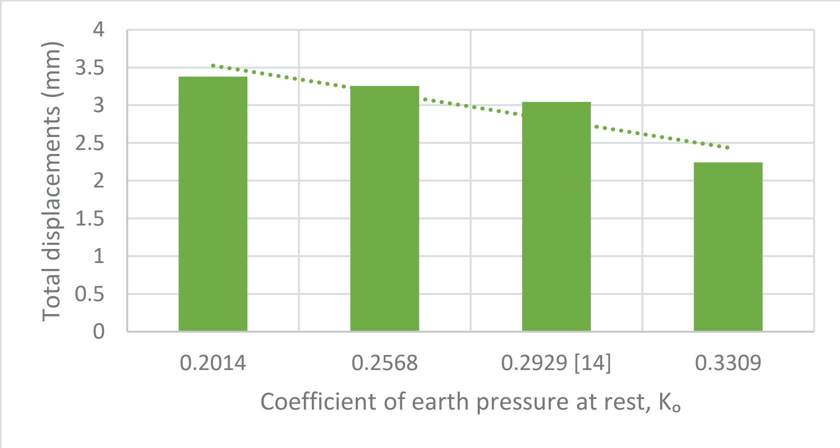

Figure 11: Influence of Coefficient of Earth Pressure at Rest on total displacement of tunnel

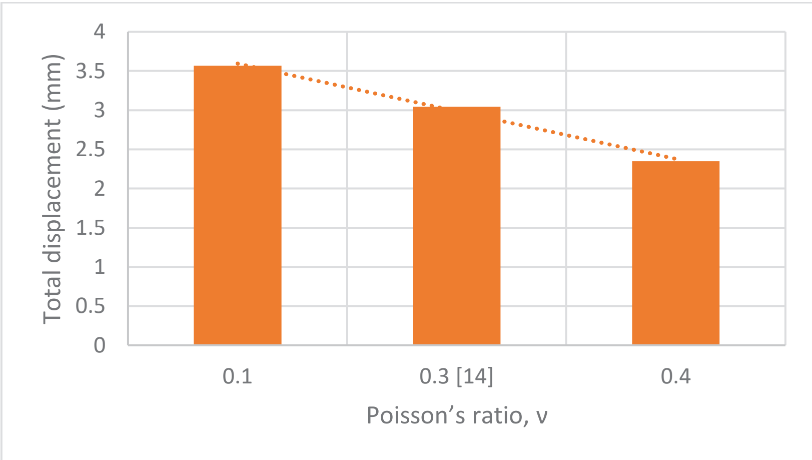

Figure 12: Influence of Poisson's Ratio on total displacement of tunnel

4. Conclusions

Based on the detailed numerical analysis and validation of results with measured displacements around typical tunnel, following are the conclusions:

a) Soil and rock properties like cohesion and angle of friction, has significant influence on the ground deformation and stress distribution around tunnel, but the variation of Poisson's ratio has no ostensible effect.
b) Tunnel diameter, shape and depth significantly influence ground stability, with larger diameters leading to more ground movement. The least displacement for the tunnel diameter of 5 m is 1.731 mm, as compared to 3.044 mm for the reference tunnel of diameter 8.5 m.
c) The increase or decrease of displacements, axial force, bending moment and shear forces of the tunnel is not only dependent on the thickness of the lining but also largely dependent on horizontal and vertical position of the tunnel. Effects of these parameters are clearly shown in the present study.
d) Additional studies can include groundwater and dynamic loads for more accurate predictions and resilient designs of tunnels and other civil and mining engineering structures.

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LIFE-CYCLE OPTIMIZATION OF TUNNEL CONSTRUCTION : A GREEN ENGINEERING PERSPECTIVE FOR SUSTAINABLE INFRASTRUCTURE

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ABSTRACT

The global tunnelling industry is at a decisive juncture where sustainable engineering practices are no longer optional but essential. Conventional tunnelling methods, while technologically robust, continue to impose significant environmental burdens through energy-intensive processes, high embodied carbon materials, and inefficient resource use. This paper presents a comprehensive framework for life-cycle optimization of tunnel construction through a green engineering perspective, integrating material innovations, energy efficiency, digital technologies, and circular-economy principles. Drawing insights from major Indian metro projects—Delhi, Mumbai, Chennai, and Pune—as well as select international benchmarks, the study quantifies potential reductions of up to 35% in embodied carbon and 25% in operational energy through sustainable interventions. The proposed framework highlights the necessity for the systematic adoption of life-cycle assessment (LCA) tools and renewable integration strategies to transform underground construction into an environmentally resilient and economically viable sector.

Keywords: *Tunnel engineering; Life-cycle assessment; Sustainable construction; Circular economy; Renewable energy; Digital optimization*

1. INTRODUCTION

Rapid urbanisation and surface-level congestion have made underground infrastructure indispensable to modern cities. India's expanding metro and utility tunnel programmes have advanced quickly but at a high environmental cost. Traditional tunnelling methods still rely heavily on Portland-cement concretes, diesel-powered machinery, and non-recyclable materials—factors that together generate most the sector's greenhouse-gas emissions [1,2]. Recent analyses estimate that nearly 70% of a tunnel's life-cycle carbon emissions occur during the construction stage, primarily from material production and power consumption [3].

In response, green tunnelling has appeared as a multidisciplinary approach that combines engineering efficiency with environmental stewardship. It promotes optimisation across all stages—design, excavation, lining, and operation—while encouraging the adoption of renewable energy and recyclable materials [4,5]. Within the Indian context, sustainability in tunnelling has gained policy momentum through national programmes such as the National Infrastructure Pipeline (NIP) and the PM Gati Shakti Mission, which emphasise carbon reporting, energy audits, and life-cycle performance metrics for major public projects [6].

Indian metro corporations have started incorporating sustainability measures at scale. The Delhi Metro Phase IV, Mumbai Metro Line 3, and Chennai Metro Phase II projects have all introduced low-carbon concrete, energy-efficient tunnel boring machines (TBMs), and digital monitoring platforms to improve transparency and performance [7–9]. However, current practices stay fragmented and lack an integrated framework that connects material choice, energy efficiency, and operational sustainability.

This paper addresses that gap by proposing a comprehensive framework for life-cycle optimisation in tunnel construction. It assesses the carbon and energy performance of emerging materials and technologies [10,11], integrates life-cycle assessment (LCA) into design and construction workflows [12,13], and illustrates, through Indian and international case studies [14,15], how green engineering principles can enhance both environmental and economic performance.

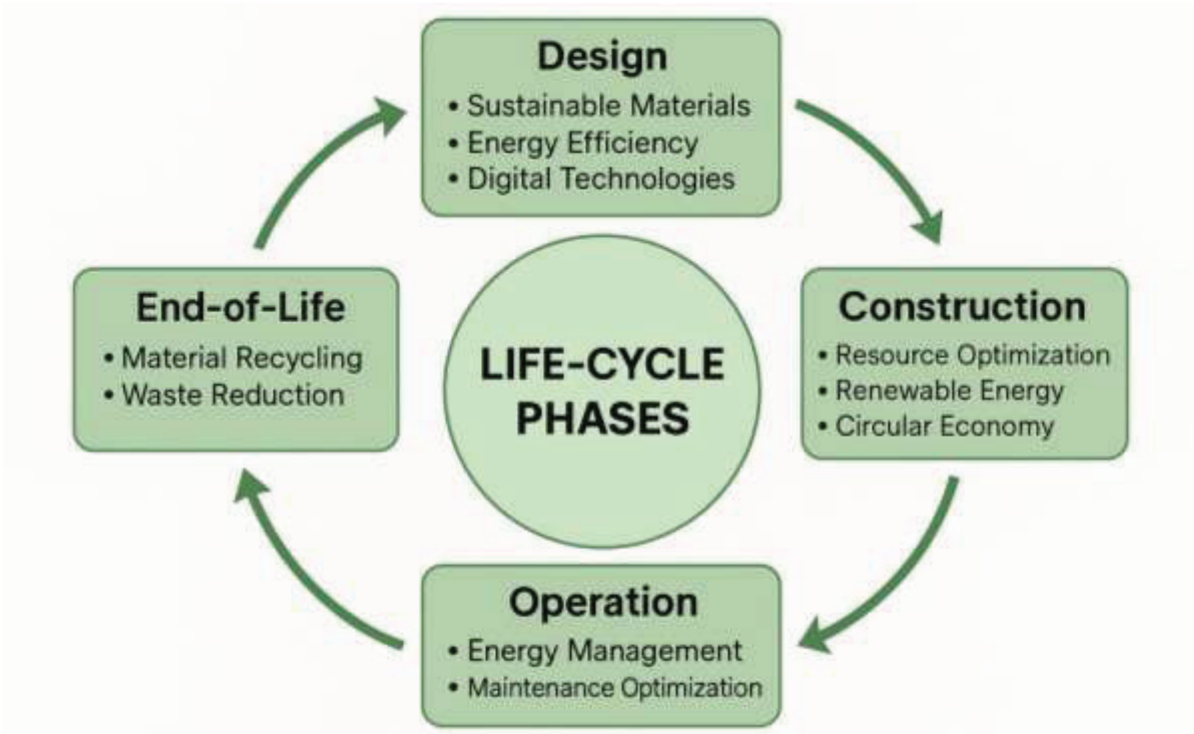

Figure 1. Life Cycle Phases of a Tunnel Project

2. GREEN CONSTRUCTION MATERIALS AND TECHNOLOGIES

The material composition of tunnels fundamentally decides their long-term sustainability profile. Concrete and steel dominate both the embodied energy and carbon footprint of underground works, making material innovation central to any life-cycle optimization effort. In Indian metro tunnels, recent advancements in low-carbon binders, reinforced composites, and data-driven mix design show how environmentally responsible solutions can be achieved without compromising safety, cost, or performance.

2.1 Low-Carbon Cementitious Systems

Ordinary Portland Cement (OPC) is still the most widely used binder in tunnel construction, but its production contributes nearly 8% of global CO_2 emissions [3]. For large-diameter metro tunnels—where each kilometre requires roughly 25,000 m^3 of concrete—the embodied carbon burden becomes immense. Replacing OPC with supplementary cementitious materials (SCMs) such as fly ash, ground granulated blast-furnace slag (GGBS), or calcined clays (LC^3 systems) offers immediate and measurable reductions in life-cycle emissions.

In the Mumbai Metro Line 3, substituting 50% OPC with GGBS achieved a 27% reduction in embodied CO_2 while preserving mechanical strength and durability. Similarly, the Delhi Metro Phase IV project incorporated 35% fly ash replacement, reporting compressive strengths of 55 MPa at 28 days—comparable to standard OPC mixes [4]. Early trials with LC^3 concrete in Pune Metro's UGC-02 package proved potential carbon reductions of 25–30%, confirming the viability of locally sourced, low-clinker alternatives.

Table 1. Comparative performance of cementitious materials used in tunnel linings.

Binder System	OPC Replacement (%)	28-day Strength (MPa)	Embodied CO_2 (kg/m³)	Field Application Example
OPC (Baseline)	–	55–60	320	–
Fly Ash Concrete	30–40	50–55	220	Delhi Metro Phase IV
GGBS Concrete	40–60	55–60	210	Mumbai Metro Line 3
LC³ Concrete	30–50	52–58	240	Pune Metro UGC-02
FRC Segments	–	60–65	280	Chennai Metro CP-06

2.2 Reinforced and Recycled Composites

Hybrid and fibre-reinforced concrete (FRC) systems allow for partial or full replacement of conventional reinforcement cages, reducing embodied energy. Tests performed in Chennai Metro's CP-06 package showed that hybrid FRC segments achieved an 18% reduction in steel content and improved crack resistance under cyclic loading [5].

Recycled aggregates from tunnel spoil and demolition waste (RCA) have also proven practical for secondary concrete layers. In the Pune Metro UGC-02 pilot, 25% of coarse aggregates were replaced with RCA while meeting IS 383:2016 standards. This integration not only offsets virgin aggregate demand but reduces disposal by nearly 50%.

2.3 Smart and Data-Driven Material Design

Machine-learning (ML) algorithms now predict compressive strength and permeability of SCM-based concretes based on mix proportions and curing conditions [6]. Paired with IoT-enabled batching systems, predictive models dynamically adjust water-cement ratios and admixtures to ensure consistency.

Nanomaterials and admixtures like nano-silica further refine pore structure and extend service life, particularly in groundwater-exposed tunnels. While they slightly increase initial costs, long-term maintenance savings significantly reduce life-cycle emissions.

3. ENERGY OPTIMIZATION IN TUNNELING

Energy consumption forms one of the most decisive components of a tunnel's life-cycle footprint. Excavation, lining, and auxiliary systems collectively decide total energy demand. Indian metro projects are now targeting efficiency through technological retrofits, renewable integration, and intelligent energy management systems.

3.1 Energy Demand and Mechanized Excavation

TBMs, slurry systems, and dewatering units together consume between 400–450 MWh per kilometre of excavation [7]. TBM propulsion alone may account for nearly half of that. The installation of variable-frequency drives (VFDs) on cutterhead motors, conveyors, and pumps enables torque modulation, thereby cutting idle losses.

In Chennai Metro CP-06, VFD integration reduced power demand by 18% while keeping advance rates above 10 m/day. Delhi Metro Phase IV combined 15% solar power in its TBM supply mix, cutting CO_2 intensity by ~35% (≈320 t CO_2/km).

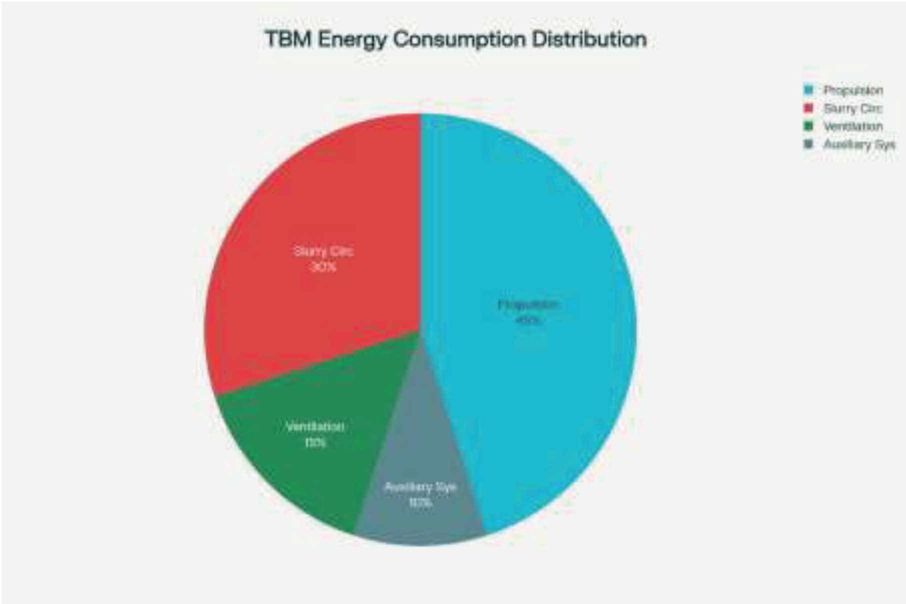

Figure 2. Indicative distribution of TBM energy consumption (Propulsion 45%, Slurry Circulation 30%, Ventilation 15%, Auxiliary Systems 10%)

3.2 System-Level Efficiency in Ventilation and Lighting

Adaptive ventilation, using real-time CO and dust sensors, enables variable-speed fan control, resulting in ~15% annual savings in fan hours. LED lighting reduced tunnel lighting loads by 45% while improving visibility.

Table 2. Comparative energy performance of sustainable tunnelling practices

Project	Intervention	Energy Savings (%)	CO_2 Reduction (t/km)
Delhi Metro Phase IV	Renewable grid + VFDs	35	320
Mumbai Metro Line 3	Adaptive ventilation + LEDs	20	240
Chennai Metro CP-06	VFD retrofits + efficient slurry pumps	18	190
Pune Metro UGC-02	Closed-loop dewatering system	15	110

3.3 Intelligent Energy Management and Recovery

Modern energy management systems (EMS) integrated with SCADA platforms enable predictive maintenance and load balancing [8]. Waste-heat recovery systems that capture the heat from TBM drives can prewarm ventilation air or process water, achieving up to 12% energy recapture in pilot projects.

4 DIGITAL INTEGRATION AND SMART TECHNOLOGIES IN TUNNELLING

Digitalisation has become a cornerstone of modern tunnel construction, transforming traditionally mechanical operations into data-driven systems. Through the integration of Internet-of-Things (IoT) networks, Building Information Modelling (BIM), Artificial Intelligence (AI), and Supervisory Control and Data Acquisition (SCADA) systems, tunnelling projects now achieve measurable gains in safety, productivity, and sustainability. These technologies collectively enable real-time sensing, predictive analytics, and energy-optimised decision-making [6–8].

4.1 Digital Transformation in Tunnelling

The tunnelling industry is undergoing a change in basic assumptions from isolated instrumentation toward interconnected digital ecosystems. IoT sensors embedded in lining segments, slurry pipelines, and TBM frames continuously measure strain, vibration, pressure, temperature, and gas concentrations [16, 17]. Data transmitted through wireless gateways allows engineers to watch performance remotely and show early signs of instability or equipment fatigue. AI-assisted algorithms convert these readings into actionable insights, automatically adjusting TBM torque or ventilation speed to keep safe and energy-efficient conditions [7, 8]. Such automation has reduced unplanned stoppages by up to 15 % and improved safety indices across several Indian metro projects.

4.2 Integrated Smart Monitoring and Control Systems

The integration of predictive analytics within the control architecture enables "smart tunnelling." Machine learning models forecast cutterhead torque, advance rate, and ground loss, allowing real-time parameter tuning and stable excavation fronts. Reinforcement-learning frameworks further refine predictions by learning from operational feedback. In the Chennai Metro CP-06 project, AI-based torque prediction achieved 10% energy savings and improved face-pressure stability. IoT data streams are combined into central dashboards, providing continuous situational awareness for site managers. These connected systems also support adaptive energy management—optimising ventilation and pump operation according to environmental and load conditions [11–13].

4.3 Digital Twins and Cyber-Physical Integration

BIM platforms now serve as the foundation for fully digital project delivery, linking design, construction, and operation within a single collaborative environment. When coupled with IoT sensors, they evolve into *digital twins*—dynamic virtual replicas of tunnel assets that synchronise design data with live field performance [9, 10]. This integration allows for simulation of excavation progress, carbon and energy tracking, and predictive maintenance scheduling. Projects such as the Mumbai Metro Line 3 and the Singapore Downtown Line have shown 15–20% reductions in material waste and downtime through the coupling of BIM and IoT.

Cyber-physical integration extends these benefits to the operations layer. SCADA systems integrate electrical, mechanical, and safety subsystems under a single interface, while cybersecurity protocols safeguard interconnected TBMs and cloud-based infrastructure from intrusion [11–13]. Recent implementations by Siemens Mobility (2023) and BAE Systems (2022) have set up encrypted data channels and redundancy mechanisms to ensure data integrity. Guidelines issued by DNV (2021) and DEFRA (2022) outline interoperability standards for such digital-twin frameworks in asset-intensive industries [14, 15].

4.4 Sustainability Impacts and Benefits of Digitalisation

Digital technologies contribute directly to sustainability outcomes by reducing energy consumption, resource use, and emissions. AI-controlled TBMs operate closer to their best efficiency, cutting power requirements by up to 20%. Automated ventilation systems that respond to IoT sensors save 15% of fan energy annually, while predictive maintenance extends part life cycles. Cloud-based carbon dashboards embedded in BIM environments simplify compliance with PAS 2080 and CEEQUAL sustainability frameworks, enabling real-time carbon accounting and transparent reporting. When integrated effectively, digitalisation can lower overall construction-stage emissions by roughly 25% and operational energy by 20%, positioning it as a key enabler of life-cycle optimisation in tunnelling.

Table 3 – Key digital technologies and sustainability outcomes

Technology	Application	Sustainability Outcome	Implementation Example
IoT Networks	Real-time structural and environmental monitoring	Early fault detection; 15 % downtime reduction	Delhi & Chennai Metro

BIM + Digital Twin	Integrated design and operation	20 % less material waste; predictive maintenance	Mumbai Metro Line 3; Singapore Downtown Line
AI-Assisted TBM Control	Real-time parameter optimisation	10 % energy saving; stable face pressure	Chennai Metro CP-06
SCADA + Cybersecurity	Unified control and secure data exchange	Continuous monitoring; data integrity	Siemens (2023); BAE Systems (2022)
Cloud Dashboards	Carbon & energy performance tracking	Automated sustainability reporting	Delhi Metro Phase IV

5 CIRCULAR ECONOMY AND RESOURCE RECOVERY IN TUNNELLING

The circular economy (CE) is a shift from the traditional *take–make–dispose* model to one that prioritises resource efficiency, material reuse, and waste minimisation. In tunnelling—where material throughput is exceptionally high—the CE approach offers an opportunity to close the loop between excavation, construction, and decommissioning, supporting life-cycle optimisation and carbon-neutrality goals [18–22].

5.1 Principles and Relevance of Circular Economy

Circularity in tunnelling focuses on designing assets that enable reuse, remanufacturing, and recycling throughout their life cycle. A single kilometre of metro tunnel can generate more than 500,000 m^3 of spoil and consume thousands of tonnes of concrete and steel [23]. By adopting circular principles, projects can reduce virgin-material demand and embodied emissions by 20–40 %.

Policy and procurement frameworks increasingly promote this approach. PAS 2080 and CEEQUAL reward verified carbon reduction and material efficiency, while the European Investment Bank now evaluates tenders using circular-performance metrics [24–26]. India's *National Tunnelling Policy (2023)* has also begun incorporating sustainability scoring for major infrastructure projects. Together, these policies set up a solid foundation for circular practices across the construction supply chain.

5.2 Reuse and Recycling of Tunnel Materials

Material recovery forms the backbone of CE implementation. Excavated spoil, concrete debris, and steel reinforcements offer significant reuse potential if properly segregated and treated. Spoil conditioning with lime, fly ash, or bentonite improves workability, while selective segregation during TBM discharge allows reprocessing into aggregates and fill materials [27]. Indian projects such as Delhi Metro Phase IV, Mumbai Line 3, and Pune UGC-02 have already achieved reuse rates of 25–30 %.

Table 4 – Examples of excavated-material reuse strategies

Project / Location	Material Treated	Reuse Application	Reported Benefit
Delhi Metro Phase IV	Muck + fly ash	Segment-casting aggregate	25 % less virgin aggregate
Mumbai Metro Line 3	Marine clay + lime	Station backfills	18 % cost saving
Pune Metro UGC-02	Basaltic muck	Road sub-base layers	30 % material recovery
Lyon–Turin Base Tunnel	Spoil → geopolymer binder	Secondary lining of concrete	22 % lower CO_2

Steel from decommissioned segments can be recovered and re-rolled with minimal quality loss, while fibre-reinforced polymer linings enable partial or full reuse. Geopolymer concretes utilising aluminosilicate-rich spoil and industrial by-products further enhance sustainability by reducing clinker dependency and embodied carbon [28–30].

5.3 Design for Circularity and Digital Traceability

Designing tunnels for future adaptability is a key enabler of circularity. Modular precast segments with standardised connectors simplify maintenance and eventual disassembly [31]. Material passports embedded within BIM environments record composition, source, and placement data for every part, ensuring long-term traceability. Emerging blockchain-based procurement systems extend this transparency across the supply chain, preventing data tampering and improving accountability [32, 33].

Digitally managed inventories also allow real-time assessment of material availability and carbon performance during construction. When paired with LCA databases and BIM-linked dashboards, such systems create feedback loops that support material-efficient design revisions before manufacture, reducing waste at the source.

5.4 Challenges and Implementation Pathways

Despite encouraging progress, the adoption of circular economy practices in tunnelling continues to face practical and institutional barriers. Variations in spoil composition, the absence of regional processing hubs, and limited acceptance of recycled aggregates restrict large-scale application. Fragmented contracting structures often do not assign clear responsibility for material recovery, reducing the motivation for innovation among contractors and designers.

Moving forward, a coordinated approach that aligns technical standards, infrastructure, and incentives is essential. Standardising specifications for recycled aggregates and geopolymer concretes—harmonised with BIS and PAS 2080—will build confidence in performance. Establishing regional spoil-treatment and recycling centres near major tunnelling corridors can lower logistics costs and promote local reuse. Equally important, procurement frameworks should link contractor rewards to verifiable sustainability outcomes, rather than focusing solely on the lowest-cost bids, thereby fostering genuine innovation.

When embedded into design, construction, and policy frameworks, these measures can transform circular-economy principles from experimental trials into mainstream tunnelling practice. The integration of digital tools, material innovation, and supportive policy ensures that circularity strengthens both environmental and economic resilience.

6 CASE STUDIES: INDIAN AND INTERNATIONAL APPLICATIONS

6.1 Indian Metro Applications

Four flagship Indian metros illustrate scalable sustainability:

- **Delhi Metro Phase IV:** 35 % fly-ash concrete + IoT energy tracking → 15 % lower power use [40].
- **Mumbai Metro Line 3:** BIM-driven coordination + spoil reuse → 27 % less embodied CO_2 [41].
- **Chennai Metro CP-06:** AI prediction of TBM parameters → 10 % energy savings [42].
- **Pune Metro UGC-02:** Recycled-aggregate concrete → 25 % virgin-aggregate substitution [43].

Table 5 – Sustainability performance of Indian metro projects

Project	Core Innovation	Quantified Outcome	Verification
Delhi Metro IV	Fly-ash concrete + IoT tracking	15 % power reduction	Internal LCA
Mumbai Line 3	BIM + spoil reuse	27 % CO_2 cut	Third-party
Chennai CP-06	AI TBM prediction	10 % energy saving	Academic
Pune UGC-02	Recycled aggregates	25 % substitution	QA tests

6.2 International Benchmarks

The Lyon–Turin Base Tunnel repurposes spoil into geopolymer binders [44]; Oslo's Fornebu Line uses renewable energy and smart ventilation (30 % energy cut) [45]; Singapore's Downtown Line employs digital-twin maintenance reducing downtime by 25 % [46].

6.3 Comparative Evaluation

Integrated frameworks linking materials, energy, and data management yield the highest benefits [47]. India aligns with global practices in digital integration and low-carbon materials, while the expansion of CE planning and renewables is still the next frontier [48].

7 FUTURE DIRECTIONS AND CONCLUSION

Sustainable tunnel engineering is transitioning from isolated initiatives to a systemic discipline grounded in life-cycle optimisation [49]. Key priorities include:

1. Embedding LCA tools within BIM workflows for real-time carbon assessment [50].
2. Deploying AI-enabled digital twins for operation and maintenance [51].
3. Developing regional spoil-processing supply chains [52].
4. Aligning procurement policies with verified sustainability outcomes [53].

For India, future research should show benchmark datasets for embodied carbon in underground works and develop optimized LC^3 and SCM formulations [54]. Collaborative platforms linking academia, industry, and government can accelerate these advances [55].

In conclusion, when life-cycle assessment, digital integration, and circular economy principles are implemented together, tunnel projects can achieve up to 35% lower embodied carbon and 25% lower operational energy without compromising safety or productivity [56].

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CONSTRUCTION OF RAILWAY TUNNELS FOR EAST COAST RAILWAY FROM ADANIGARH TO PURANA KATAK-ISSUE AND CHALLENGES FOR FASTEST CONSTRUCTION-A CASE STUDY

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ABSTRACT

Adanigarh to Puranakatak railway tunnel project is one of the prestigious ongoing projects in the state of Orissa in Boudh district. Owner of the project are East Coast Railway (EcoR), the branch of Indian Railways. Objective of the project is to connect the remote area people of Eastern Orissa to Western Orissa and to the rest of the country. This will also give the shortest access to the Visakhapatnam Port.

Adanigarh Puranakatak Railway tunnel Project under Pkg-5 of the Khurdha Road–Bolangir rail line project in Odisha, India, exemplifies innovation and efficiency in large-scale infrastructure development. Project is having 4 tunnels viz. T4, T5, T6, and T7 spanning a length of 7.492 kms and one escape tunnel. The project implemented a mix of Cut & Cover and NATM tunnel method, to overcome geological and logistical challenges. Key innovations included the use of Polyfibre Reinforced Shotcrete (PFRS) instead of traditional shotcrete and wire mesh, and the introduction of polyform concrete lining, enhancing cost-effectiveness and safety. Proactive solutions, such as resolving land acquisition issues and optimizing access routes, minimized delays and reduced the costs. The project achieved 1.5 million safe working hours and preserved nearly 3,000 trees, underscoring its commitment to safety, environmental sustainability, and advanced engineering practices.

Project is in advance stage of construction and entire project is to be completed in 30months time.

Author wants to highlight the challenges faced by the working team in speedious mobilization, communication, wildlife problems, construction techniques for fastest construction and cut and cover methodologies and on the spot innovative solution adopted to complete tunnels ahead of schedule. Project is executed using NATM technology.

1. INTRODUCTION

1.1 Overview

Indian Railways, one of the world's largest networks, spans 67,956 km with 13,169 passenger trains and 8,479 freight trains, transporting 23 million passengers and 3 million tonnes of freight daily from 7,349 stations. It aims to add 1.5% to India's GDP by supporting 45% of the freight share. In 2020-21, freight revenue increased by 3%, with a 1.93% rise in goods loaded.

The Khurdha Road–Bolangir rail line will connect Khurdha Road to Bolangir in a shorter way through Daspalla. The project aims at construction of Four tunnels including approaches in general area between Km153.0 and Km 180.00 in the state of Odisha between Adenigarh-Charichak - Purunakatak station in connection with construction of khurdha Road-Bolangir new BG rail link project of East Coast Railway on Engineering Procurement and Construction Contract. The Site of the Railway Project comprises the execution of Tunnels between km.153 to km 180 i.e., the Adenigarh Purunakatak section in the State of Odisha in the East Coast

Railway zone. The Project includes Design, Construction and instrumentation to be provided for monitoring of New Single-track tunnels BG line between Khurdha road and Bolangir connecting rail line from Khurdha road to Bolangir in a shorter way through Daspalla, which includes construction of 4 Tunnels and 1 Escape Tunnel i.e. T-4, T-5, T-6, T-7 and Escape tunnel with total length of 7.492 Km. Out of the total length of 7.492 Km, 1.742 Km length will be executed by Cut & Cover tunneling methodology due to shallow depth and rest will be executed as per Conventional tunneling and shaft driven method for construction works in all underground condition using drilling and blasting or by manual means.

1.2 Description of Project

- **Project Location**:

  State : Odisha

  District : Boudh

Fig. 1 : Layout of the project

  Location of Tunnels: Adenigarh-Charichak (T-4, T-5 & T-6), Charichak- Purrunakatak (T-7)

  Nearest Rail head : Bionda Railway station 60 Km

  Nearest Airport : Bhubaneshwar Airport 160 km

  Nearest Road Head: NH-57

- Components of the Project:

  The project comprises the following major components:

  Tunnel – 4 Start Chainage 156.195 Km To 160.38 Km = 4.185 Km (main Tunnel modified horseshoe shape) Proposed cut & cover length = 1.312 Km.

  Tunnel – 5 start Chainage 164.406 Km to 164.723 Km = 0.317 Km (main Tunnel modified horseshoe shape).

  Tunnel – 6 start Chainage 164.941 Km to 165.256 Km = 0.285 Km (main Tunnel modified horseshoe shape) Proposed cut & Cover length = 0.03 Km.

  Tunnel – 7 start chainage 168.35 km to 170.325 Km = 1.975 km (main tunnel modified horseshoe shape) Proposed cut & cover Length = 0.100 km.

  Escape Tunnel – 0.700 Km length proposed cut & cover length (Modified Horseshoe shape).

2. GEOLOGY OF THE AREA

According to the National Center for Seismology, the state of Odisha is situated in two earthquake zones, with the construction site falling within Zone II. Data from the National Center for Seismology show no recorded earthquakes in the past 10 years within a 50 km radius of the construction site. The studied area is part of the Eastern Ghat Mobile Belt (EGMB), a Proterozoic mobile belt characterized by intensely deformed granulite facies gneisses, schists, migmatites, and magmatic complexes (Ramakrishnan et al., 1998). It is bordered to the north and west by the Archaean Singhbhum and Bastar/Bhandara cratons, respectively. EGMB is described as a heterogeneous collage of several lithozones (Ramakrishnan et al., 1998), terranes (Chetty, 1998), provinces, or domains (Dobmeier and Raith, 2003), separated by major shear zones and lineaments (Nanda,2008). Chetty (2007) described a section across the northern EGMB, consisting of several subparallel shear zones, as a crustal scale ‘flower structure’. (Fig. 2)

The geological map shows the distribution of lithological domains around the study area (Chetty, 2010). The Main Shear Zone (MSZ) separates the northern segment of the Eastern Ghats Mobile Belt (EGMB), including the Rengali Province, Angul, and Tikarpara domains, from the southern main belt of EGMB. The MSZ trends WNW-ESE, contrasting with the NE-SW trend of the main EGMB block. It extends over 200 km and is 3-4 km wide, starting from Khurdha (20°11’08”:85°37’38”) and stretching through Kantilo to Ganian. Beyond Ganian, it branches into the Kantilo Fault (northeast) and the Ranipathar Shear Zone (west), continuing to the Satkoshia gorge and the northern Phulbani plateau. The MSZ is a dextral shear zone with ultra-mylonites and pseudo-tachylyte veins (Chetty, 2010).

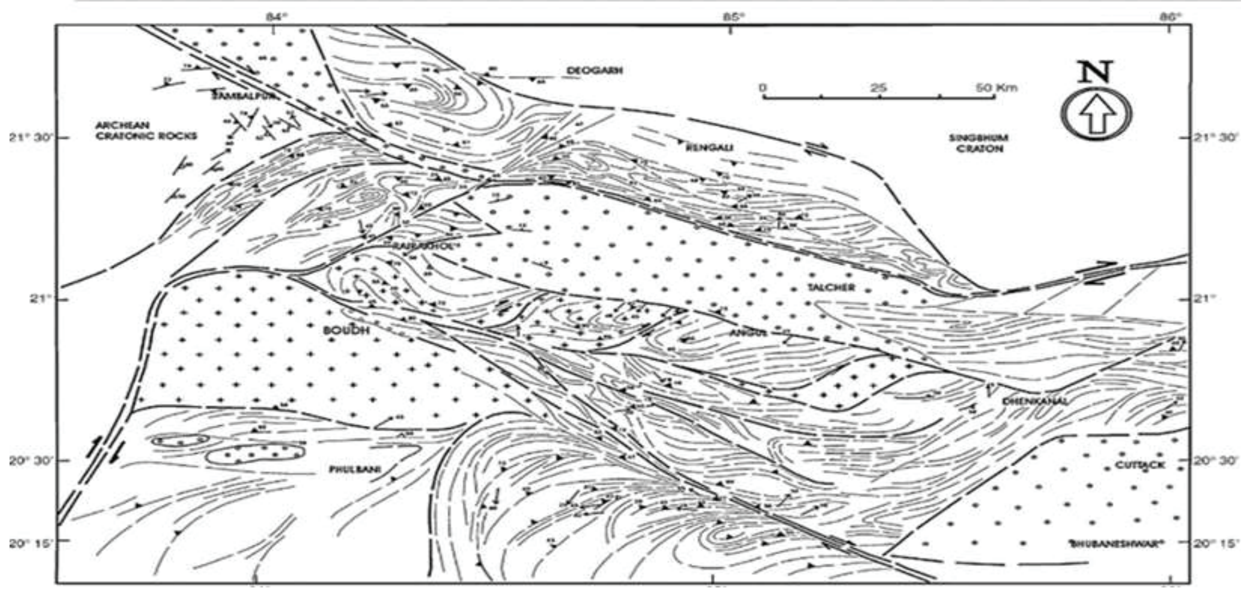


Fig. 2 : Geological map of the area

3. LOCAL LITHOLOGY

The region around the site of construction is highly metamorphosed terrain. The rocks are metamorphosed in granulite facies mainly comprising basement of Khondalite with some patches of charnokite at places around the Tunnel 4 and the region around Tunnels 5 and 6 are characterized by homogeneous lithology comprising feldspar garnet gneiss.

Khondalite is metasedimentary rock also referred as quartz garnet gneiss. Generally brownish grey in color. Presence of grains of garnet and less prominent gniessosity was the corroborative criteria for identification of khondalite in the field. Charnokite is also considered as metasedimentary rock formed in granulite facies of metamorphism at high temperature and pressure which leads to the fine-grained texture of the rock. Generally leucocratic in nature due to presence of mafic minerals, especially orthopyroxene which is striking feature of the rock. At some places interfingering of both Charnokite and khondalite were encountered with evidence of migmatisation which leads to the development of the leucocratic Quartz- Feldspathic Gneiss (QFG). QFG contains biotite, amphibole in a small amount in melanocratic bands and alkali feldspar in leucocratic bands.

The degree of weathering in southern portion of site is very high such that thick layer of soil up to 30m covers the region and clay minerals such as goethite, kaolinite, gibbsite is formed along the foliation plane of khondalite. Charnokite and less weathered Khondalite are found at the top of the hillocks from BH-9 to BH-15 of tunnel 4 and highly weathered khondalite are found in northern most portion of the tunnel up to BH-18 of Tunnel 4. Few photographs of for encountered lithology at tunnel locations are shown in Figure 3:

4. COSTRUCTION PLANNING

This paper’s goal is to give comprehensive work procedures that were used for construction of portals, Main Tunnel in underground portion, Cut and cover portion and Escape Tunnel.

A B C D

Fig. 3 : A. Highly weathered Khondalite from Borehole, B. Charnokite from borehole C. Less weathered khondalite from borehole and D. Feldspar garnet gneiss from bore hole

4.1 Construction sequence for Portal development

4.1.1 *Open Excavation*

Open excavation at each tunnel portal was conducted according to the levels provided in the drawings or as determined by the survey. This included cutting trees and preparing access roads to the portals, ensuring a smooth gradient for the movement of vehicles and equipment. Activities involved:

Deforestation: Clearing vegetation to create access for the approach road to the portal, with measures to minimize environmental impact.

Initial Survey: Conducting topographical and Geotechnical surveys to measure the volume of earth to be excavated, assessing underground Rock conditions, ensuring accurate resource estimation, and planning.

Approach Road Construction: Excavating and constructing a stable approach road, involving site clearing, soil excavation, compaction, and road surfacing.

Excavation in Soft Soil: Safely excavating soft soil using specialized equipment and tech-niques to prevent soil collapse.

Excavation in Soft/Hard Rock: Employing appropriate machinery to excavate both soft and hard rock, ensuring safe and efficient removal.

Drilling and Controlled Blasting: Utilizing drilling rigs and controlled blasting to break down hard rock, followed by post-blast safety inspections.

Mucking: Efficient removal of excavated material (muck) from the site to maintain a clean construction area.

Material Dumping: Transporting and disposing of excavated materials at designated dumping yards, with erosion control and environmental compliance.

4.1.2 *Portal Construction*

Before opening the faces of the tunnel, portal slope excavation, stability, and protection works were done with shotcrete and rock bolting to avoid any slope failure. The sequence adopted in the portal construction is explained below:

Initial Excavation: Excavated 1-2 m below the first rock bolts level, then trimmed the face to be smooth and regular to reduce shotcrete wastage.

Shotcrete Application:

In Soil: 100 mm total (50 mm 1st layer, 50 mm 2nd layer after wire mesh installation).

In Rock: 75 mm total (25 mm 1st layer, 50 mm 2nd layer after wire mesh installation).

Rock Bolt Installation: Drilled and cleaned holes, installed rock bolts, and performed gravity cement grouting.

Sub-Horizontal Drains construction: Constructed concrete drains connected to the main drain, with 50 mm drainage/relief holes every 4 m c/c. Used 50 mm perforated PVC pipe wrapped in geotextile for drain holes.

Wire Mesh Installation: Used 150 x 150 x 6 mm wire mesh with 300 mm overlap and installed rock bolt bearing plates (200 x 200 x 10 mm) and nuts.

Next Excavation: Repeat excavation 1-2 m below the next rock bolts level, ensuring a smooth and regular face. (Fig. 4)

Upon Completion of open excavation up to portal invert level the tunnel faces were supported for a 5-m reach to create a false portal, with the installation of 89 mm diameter pipe roofing, lattice girders, wire mesh, and shotcrete to ensure a secure and safe entrance to the tunnel before the main tunnel construction activities began.

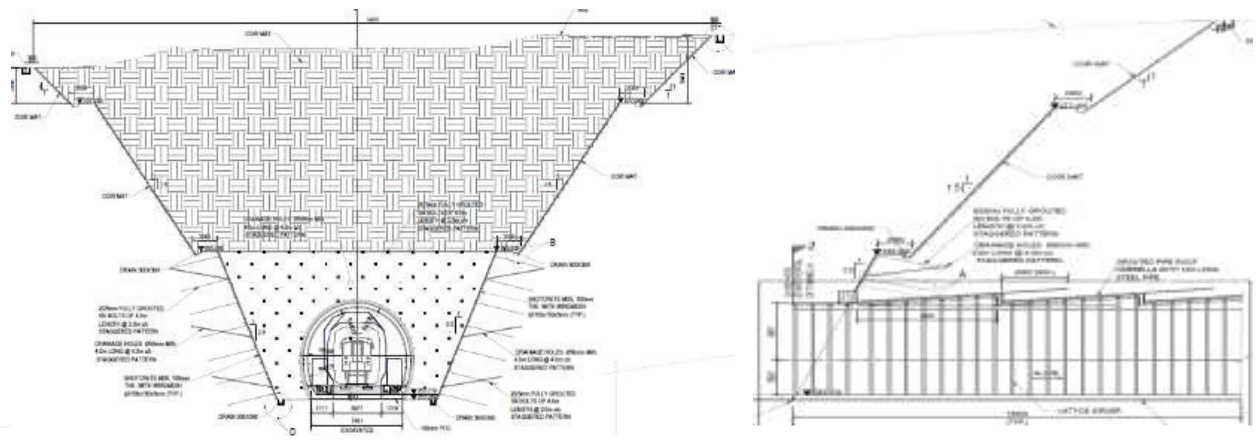


Fig. 4 : Cross-section and Longitudinal section of Portal

4.2 Construction sequence for underground section

4.2.1 *Construction of Tunneling*

On completion of Portal construction activity, the next face of Underground tunneling was started using NATM method. The New Austrian Tunnelling Method (NATM) is a tunnelling technique that focuses on the efficient and safe excavation and support of underground tunnels. Developed in the 1960s, NATM emphasizes the use of natural ground strength and adapting construction methods to varying geological conditions. The approach involves continuous monitoring and adaptation to ensure stability and minimize deformation. Key Principles of NATM:

Ground Monitoring: Continuous monitoring of ground conditions is essential. This includes using instruments to measure ground movements, stress, and other parameters to adapt construction methods accordingly.

Support System: NATM uses a flexible support system that evolves as excavation progresses. The support system typically includes shotcrete, rock bolts, and steel supports to stabilize the excavation and take advantage of the natural strength of the ground.

Sequential Excavation: The excavation is often done in stages, with careful attention to the stability of the rock mass and surrounding ground. The sequence includes drilling, blasting, and immediate application of support measures.

Incremental Construction: The method involves incremental construction where initial support is followed by further reinforcement and final lining, allowing for real-time adjustment based on observed ground behaviour.

Adaptation to Ground Conditions: NATM emphasizes adapting to the encountered ground conditions. The method is flexible and allows for adjustments in excavation and support techniques based on ongoing geological observations.

The typica sequence of operation adopted for tunnel excavation using NATM method is described below:

- **Excavation**: Excavate the tunnel face using controlled methods such as drilling and blasting or mechanical means.
- **Fore-Poling/ Pipe-roffing**: Install temporary support elements to stabilize the excavation face before the main support is applied.
- **Drilling**: Drill holes for explosives or support elements as required by the excavation plan.
- **Charging and Blasting**: Charge and blast the drilled holes to break the rock and create the desired excavation profile.
- **De-fuming**: Use ventilation systems to clear smoke and gases resulting from blasting.
- **Scaling**: Remove loose or unstable rock from the excavation face to prevent collapse.
- **Mucking**: Remove excavated rock and debris from the tunnel.
- **Shotcrete Application**: Apply shotcrete to provide immediate support and stabilize the excavation face.
- **Rock Bolting**: Install rock bolts to further reinforce the tunnel and stabilize the surrounding rock mass.
- **Steel Support**: Add steel supports as necessary to provide additional structural stability.
- **Lining**: Pour and set concrete lining to provide the final, permanent support and complete the tunnel.

NATM is characterized by its adaptability and reliance on real-time monitoring to manage the stability of the tunnel, making it suitable for various geological conditions. The following image presents a typical “drill and blast” sequence of excavation. (Figure 5)

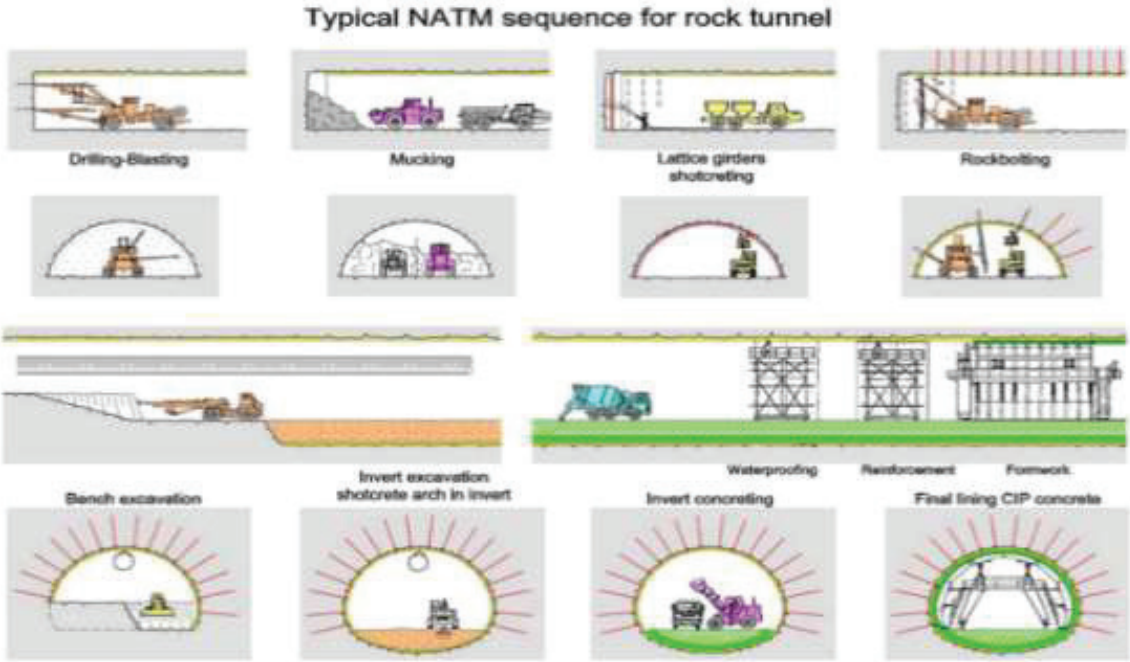


Fig. 5 : NATM sequence of excavation and Rock Support

4.2.1.1 *Excavation Profile and Cross section*

The tunnel excavation was carried out in accordance with the approved drawings. The blasting pattern depended on the actual rock conditions encountered on site. The alignment and profile were checked and recorded by the surveyor with the help of calibrated equipment. The cross-section of the excavation profile is shown in Figure 6 and table 1.

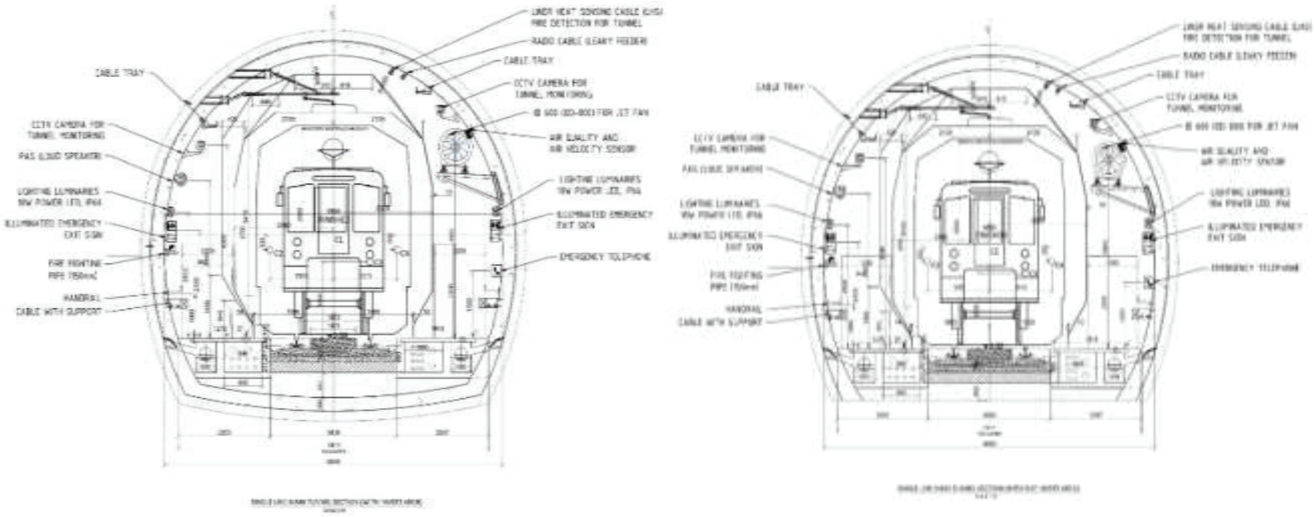


Fig. 6 : Cross section of excavation profile

Table 1 : Rock Class as per the feasibility report

S. No.	Location	Length of Tunnel (excl. C&C)	Rock Class I	Rock Class II	Rock Class III	Rock Class IV/V
1	Tunnel - 4	2873	0.0%	48%(1379m)	48% (150m)	10% (287m)
2	Tunnel - 5	317	0.0%	45% (143m)	48% (159m)	5% (15m)
3	Tunnel - 6	315	0.0%	45% (142m)	48% (158m)	5% (15m)
4	Tunnel - 7	1875	0.0%	45% (843m)	48% (938m)	5% (94m)
Total		**5380**	**0.0%**	**2335**	**2634**	**411**

- **Excavation Method:**

Full-Face Excavation: Given that 75% to 80% of subterranean excavation was in rock classes 2 and 3, full-face excavation was executed. This method involved excavating the entire tunnel section at once and was preferred for its faster advancement and easier construction. However, the decision took into consideration the geology, opening size, and stand-up time.

Heading & Benching: For the remaining 20% to 25% of excavation in rock classes 4 and 5, where rock cover was less, the heading and benching method was proposed. This involved excavating the top heading first and then the bench after securing the top heading, which allowed for longer bench increments if wall stability issues were managed. (Table 2)

Table 2 : Excavation Method

Type of Rock class	Excavation area (m^2)	Excavation Circumference (M)	Round length	Excavation Method
Class-I	58.64	20.52	3	Full Face
Class-II	58.64	20.52	3	Full Face
Class-III	59.67	20.78	3	Full Face
Class-IV	60.71	21	1.5	Heading & Benching
Class-V	68.5	21.2	1	Heading & Benching

4.2.1.2 *Drilling Works for Underground Excavation*

Face drilling was carried out using an electrically operated Twin Boom Drill Jumbo, with a 16 ft feeder for 4.6 m holes. The Tunnel Engineer and geologist set the drilling pattern, executed by skilled operators. Line drilling and dummy holes minimized overbreak, while exploratory and post-excavation vertical drilling up to 4 m addressed geological conditions.

- **Drilling Patterns:**

Burn or parallel hole cut: The Burn-Cut or parallel hole cut pattern, used in rock classes 2 and 3, involved closely spaced parallel holes drilled perpendicular to the face for deep excavation. Larger, uncharged central holes created zones of weakness, enhancing the effectiveness of adjacent charged holes. This method was favored for its ease of placement and alignment in suitable rock masses. (Figure 7)

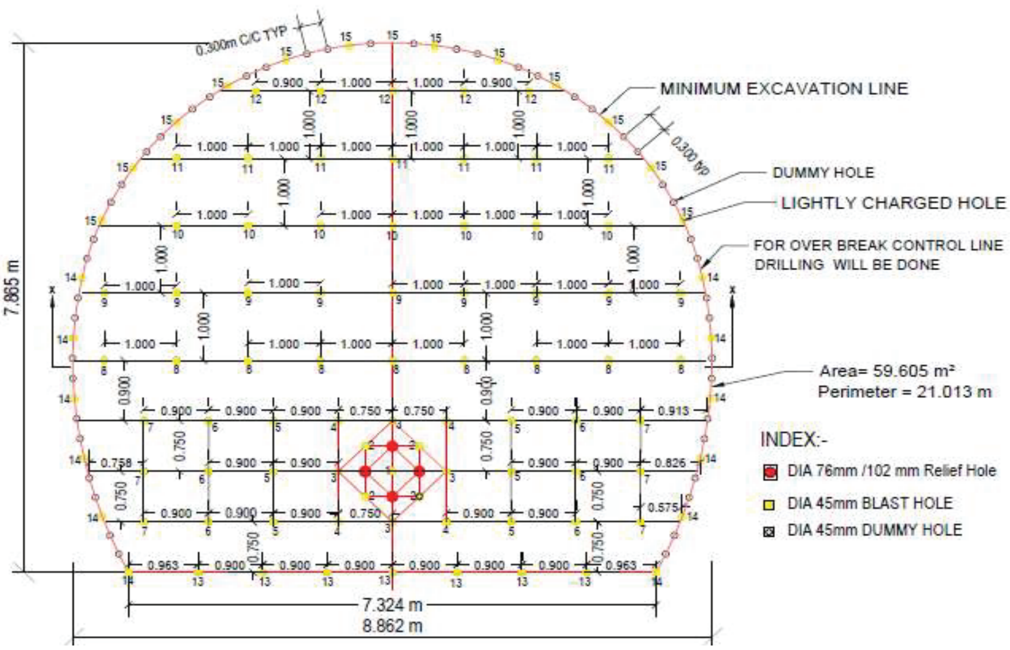


Fig. 7 : Parallel-hole or burn cut blasting pattern for RC III

Wedge or V-cut: In the Wedge or V-cut pattern, angled holes formed a wedge for the initial blast, aligned with the face's centerline. This displaced a rock wedge, with subsequent blasts widening it to the full drift width. The optimal apex angle was about 60°. This method was used in rock classes 4 and 5. (Figure 8)

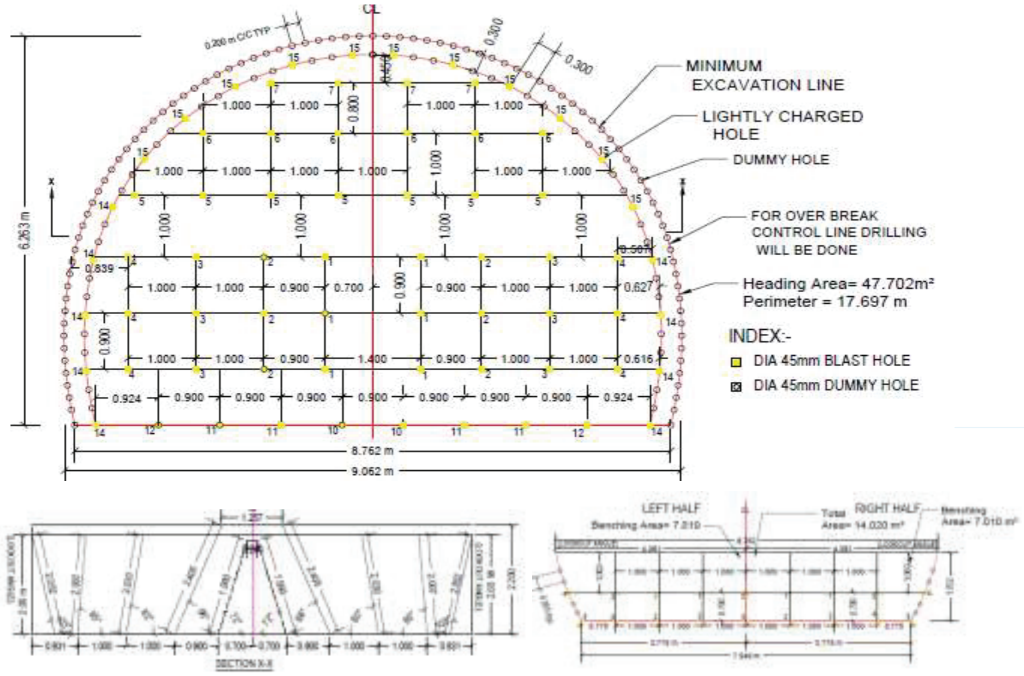


Fig. 8 : Wedge or V - cut blasting pattern for RC IV

4.2.1.3 *Cycle Time of Excavation in each Rock Class* (Table 3)

Table 3 : Advance Cycle Time

Advance Cycle Time For 3m Round Pull Length					
S. No.	**Description of Activities**	**Time (Hrs.)**			
		RC II	**RC III**	**RC IV**	**RC V**
1	Survey and Face Mapping	0.5	0.5	0.5	0.5
2	Drilling	2.8	2.8	1.9	1.6
3	Charging and Blasting	1.7	1.7	1.7	1.7
4	Bottom Cleaning & Loose Scaling	0.3	0.3	0.3	0.3
5	Mucking- Side Tilting Wheel Load-er	3.1	3.1	2.5	2.4
6	Shotcrete at crown, sides - 1st Layer- 30m3 Shotcrete Machine	0.7	0.7	0.6	0.5
7	Wiremesh 150mmx 150mm x 6mm	0	0.8	0.8	0.8
8	Shotcrete at crown, sides - 2nd Lay-er	0	0.7	0.6	0.7
9	Wire mesh 150mmx 150mm x 6mm - 2nd Layer	0	0	0	0.8
10	Shotcrete at crown, sides - 3rd Layer	0	0	0.8	0.7
11	Rock Bolts at Crown and Sides- Two boom Hydraulic Drill Jumbo & Mai Pump	1.3	1.3	1.6	2.7
12	Fore poles at Crown- Two boom Hydraulic Drill Jumbo & Mai Pump	0	0	2.9	4.8
13	Lattice Girder- 115/25/32 @ 15kg/m	0	0	2	2.5
	Sub Total	**10.3**	**11.7**	**16.2**	**19.8**
	Contingency @ 5%	**0.5**	**0.6**	**0.8**	**1**
	Total Cycle Time	**11**	**12**	**17**	**21**

4.2.1.4 *Water proofing and concrete lining*

After the completion of excavation, tunnel lining works began. Currently, lining works are being executed at Tunnel-5, Tunnel-6, and Tunnel-7 of the project. The lining of Tunnel-5 is complete. A hydraulically operated gantry was used to deploy a 300 mm thick concrete overt lining across all rock classes for the entire length excavated. A lining shutter system, consisting of two shutters (Kerb and Overt) and one traveler, was used to achieve a finished diameter of 8.181 meters. The lining of the tunnel comprises the following stages:

- Smoothening of the tunnel surface with a finishing layer of shotcrete to minimize the risk of puncturing the waterproofing membrane.
- Installation of a 500 GSM geotextile membrane, fastened to the surface, followed by fixing a 2mm PVC waterproofing membrane as recommended by the manufacturer.
- Casting of Kerb concrete.
- Installation of the hydraulic tunnel lining gantry on rail lines over the Kerb concrete and placing poly fibre reinforced overt concrete (PFRC).
- Casting of invert concrete.

4.2.2 *Construction of Cut & Cover Portions*

The Cut and Cover method is a tunneling technique used for constructing tunnels at shallow depths, where the cover is relatively low. This approach involves digging a trench or cut to the desired depth, building the tunnel structure within the trench, and then covering it back up to restore the surface. Currently, the Cut and Cover method is being executed at Tunnel-4 (initial stretch: 1.55 km), the Escape Tunnel (0.6 km) towards P-1 face, Tunnel-6 (0.03 km) in the middle portion, and Tunnel-7 (0.1 km) in the middle portion. Excavation is complete in Tunnel-7 and Tunnel-6, and progress is ongoing in Tunnel-4 (0.35 km completed) and the Escape Tunnel (0.10 km completed). (Figure 9)

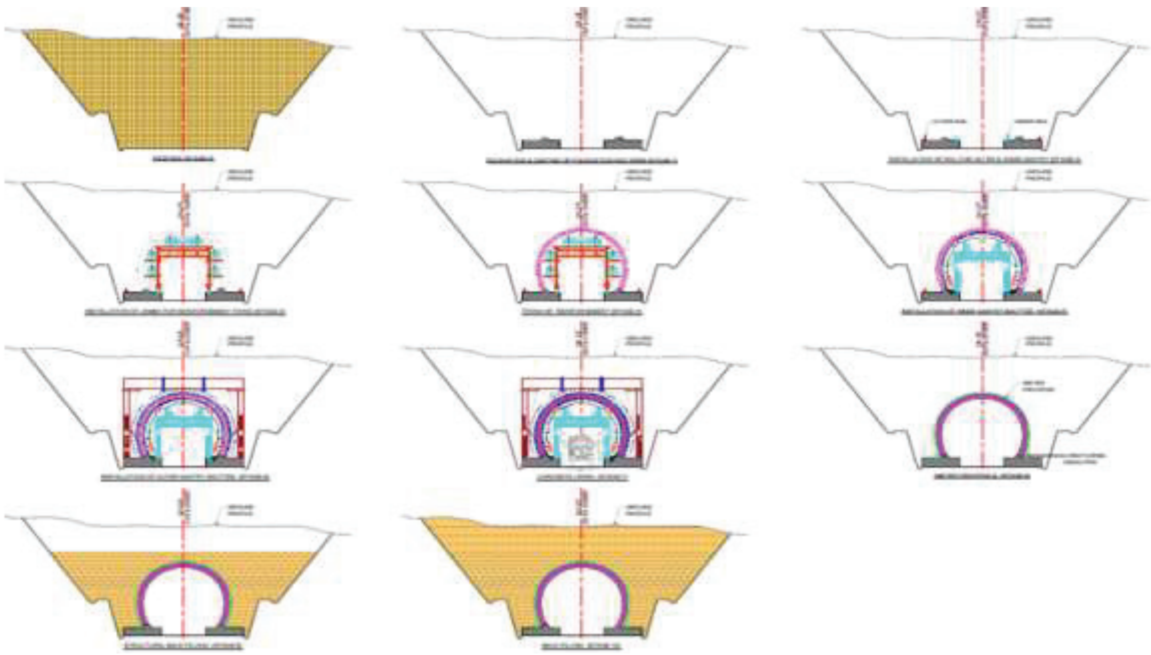


Fig. 9 : Construction sequences of Cut & Cover Tunnel

Construction of cut & cover tunnel was executed stretch wise, initial 100m stretch of C&C tunnel was excavated after providing suitable ramp from front side, after completion of 100m stretch including cast in situ concreting and back filling next 100m stretch was taken up for execution. The construction stages and sequences of each 100m stretch given below:

- Step-1: Excavation of Deep cut up to Formation level.
- Step-2: Construction of Foundation of Cast In-situ structure.
- Step-3: Construction of Kerb/ Kicker beam
- Step 4: Fixing of Rail Line for Inner and outer Gantry Shutter
- Step-5: Fixing of Reinforcement with the help of Jumbo.
- Step-6: Erection of Inner Gantry Shutter
- Step-7: Erection of Outer Gantry Shutter
- Step-8: Concreting with Boom Placer/ concrete pump
- Step-9: Fixing of Water Proofing Membrane

- Step-10: Backfilling in Layers up to required level
- Step-11: Installation of Tunnel Interior including Drains, utility, and Footpath
- Step-12: Installation of BLT system
- Step-13: Installation of MEP System

4.2.2.1 *Concreting and Backfill in Cut and Cover Tunnels*

The type of foundation for the Cast In-situ Cut & Cover tunnel structure depended on site-specific soil and rock conditions. Typically, cast-in-place spread footings were used, often as strip footings with a pedestal. These foundations were connected by reinforcement to form a continuous monolithic body, as per the approved drawings.

The cast-in-situ concrete structure was constructed in stages, starting with a 100-meter length using double shutters (inner and outer). The process began by installing a shutter at Chainage 156+195 and moving it towards Chainage 157+770, using a 12-meter gantry shutter to complete the length in a month. Kerb installation occurred concurrently over 36 meters with the concrete casting. After installing the kerb, reinforcement and rail lines were set up for the shutters, which were then erected. Concrete was poured using a boom placer or pump, with vibration provided by shutter and needle vibrators, and transported by transit mixers. Gantry shutters included cutoffs for trolley refuges to ensure proper openings. (Figure 10 & 11)

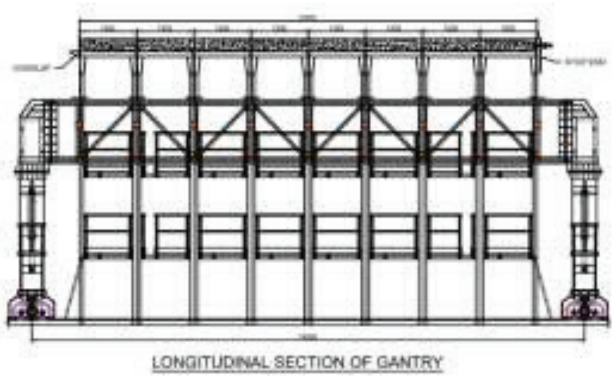


Fig. 10 : Gantry Typical L-section

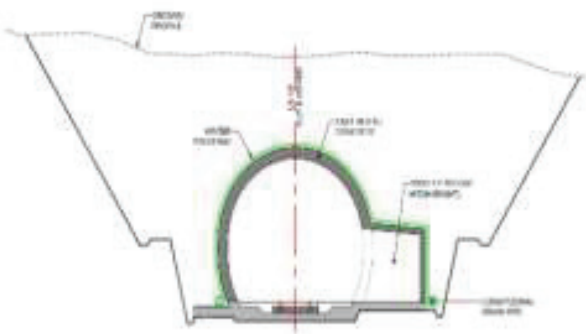


Fig. 11 : Typical Section of Trolley Refuges

The RCC cast-in-situ structure used a 2mm thick, heat-welded PVC waterproofing membrane applied to the crown and sidewalls, wrapped with a 150 mm diameter longitudinal drainage pipe. The membrane was protected during installation and backfilling. The installation length depended on the construction schedule, with careful handling required during the first backfill layer. Transverse drains (125mm diameter) were installed every 50 meters to channel seepage water from the longitudinal drains to the main drain. (Figure 12)

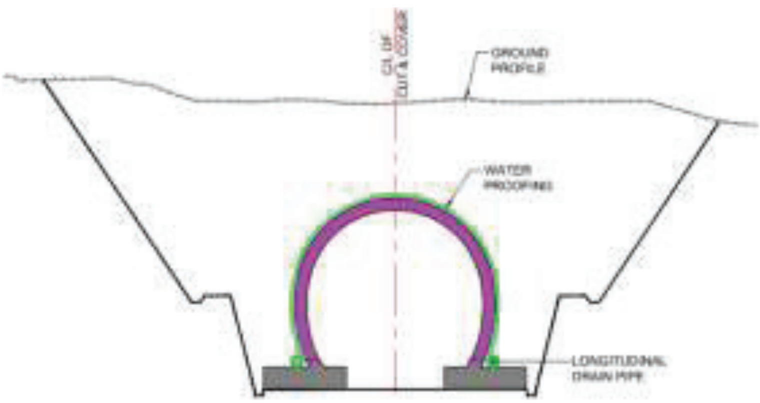


Fig. 12 : Waterproofing membrane

The backfilling operation imposed significant loads on the structure, as the backfill became an integral, load-carrying component. It had to permanently fulfill its purpose. The material used for backfilling was, whenever possible, sourced from the required excavation of the cut & cover portion and underground excavation works. The backfill material was placed and compacted in layers not exceeding 0.5m in loose thickness before compaction and was compacted to a dry density of no less than 95 percent of the maximum laboratory dry density, as per IS 2720, Part 8. (Figure 13)

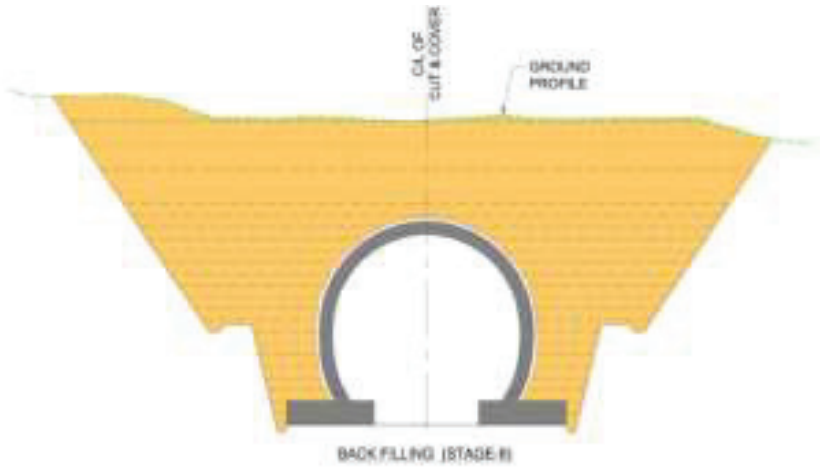


Fig. 13 : Backfilling in Layers

5. MECHANICAL, ELECTRICA AND PLUMBING (MEP) WORKS

The MEP works begins after completion of tunnel lining in the whole stretch of the tunnel. (Figure 14) MEP works in railway tunnels encompass essential systems that ensure the tunnel's safety, functionality, and operational efficiency. Mechanical systems include ventilation with jet fans, air quality sensors, and fire suppression installations to manage airflow and control fires. Electrical systems cover power distribution, lighting, communication setups like public address systems and emergency telephones, and fire detection through heat-sensing cables and CCTV monitoring. Plumbing works involve drainage systems to prevent flooding and water supply for firefighting. These integrated systems collectively maintain a secure and efficient tunnel environment. The MEP works associated with the project are outlined below:

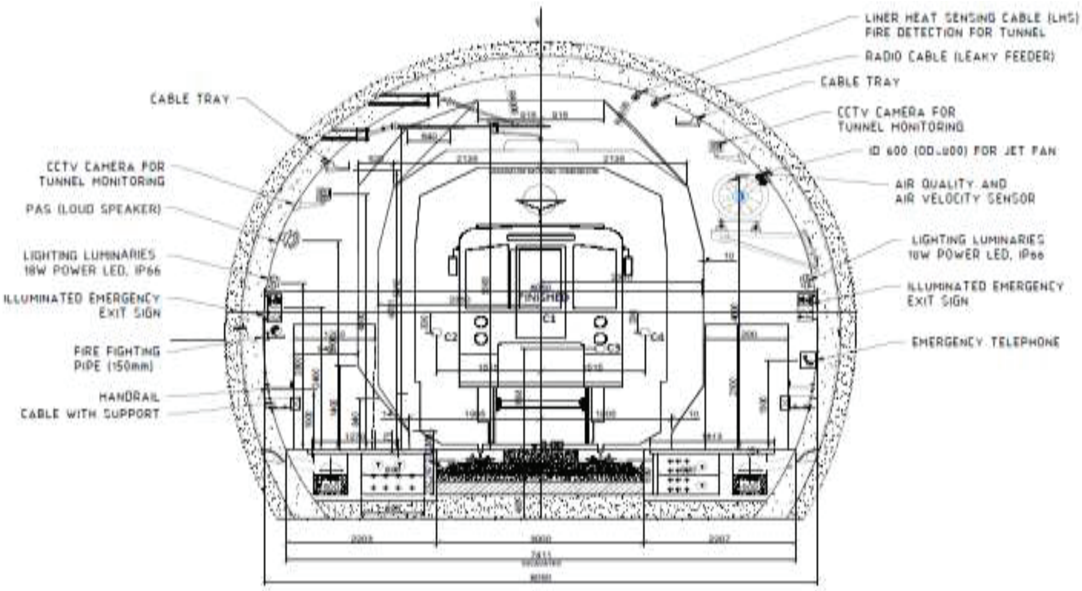


Fig. 14 : Cross section of tunnel showing arrangement of MEP Elements of the Project

Mechanical Works:

- Jet Fan (OD 800 mm): Provides ventilation, controlling airflow and maintaining air quality, especially during emergencies.
- Air Quality and Velocity Sensor: Monitors air quality and airflow to ensure a safe tunnel environment.
- Fire Fighting Pipe (150mm): Delivers water or fire retardants throughout the tunnel in case of fire.

Electrical Works:

- Cable Tray: Organizes and safely routes power, communication, and signal cables.
- Lighting Luminaires (18W Power LED, IP66): Illuminates the tunnel with durable, water-resistant LED lights.

- Illuminated Emergency Exit Sign: Guides passengers to the nearest exit during emergencies.
- Emergency Telephone: Facilitates communication with control canters during emergencies.
- PAS (Public Address System) Loudspeaker: Broadcasts announcements and emergency instructions.
- Liner Heat Sensing Cable: Detects heat along the tunnel lining to trigger fire alarms.
- Radio Cable (Leaky Feeder): Ensures continuous radio communication within the tunnel.

Plumbing Works:

- Drainage System: Manages water ingress, keeping the tunnel dry and safe.

Structural and Support Elements:

- Handrail with Cable Support: Provides safety and supports cable routing.
- CCTV Cameras for Tunnel Monitoring: Enables real-time monitoring for security and safety.

SCADA: SCADA stands for Supervisory Control and Data Acquisition, is a system used to monitor and control industrial processes and infrastructure. It integrates hardware and software to gather real-time data from sensors and devices across a network, allowing operators to supervise, analyze, and manage complex systems remotely. They typically include components like human-machine interfaces (HMIs), programmable logic controllers (PLCs), and communication networks that work together to monitor and control industrial processes from a centralized location. For monitoring of MEP works 2 SCADA Control Buildings are planned to be constructed for Tunnel-4 and Tunnel-7. The typical general arrangement drawsing for the works is show in Figure 15.

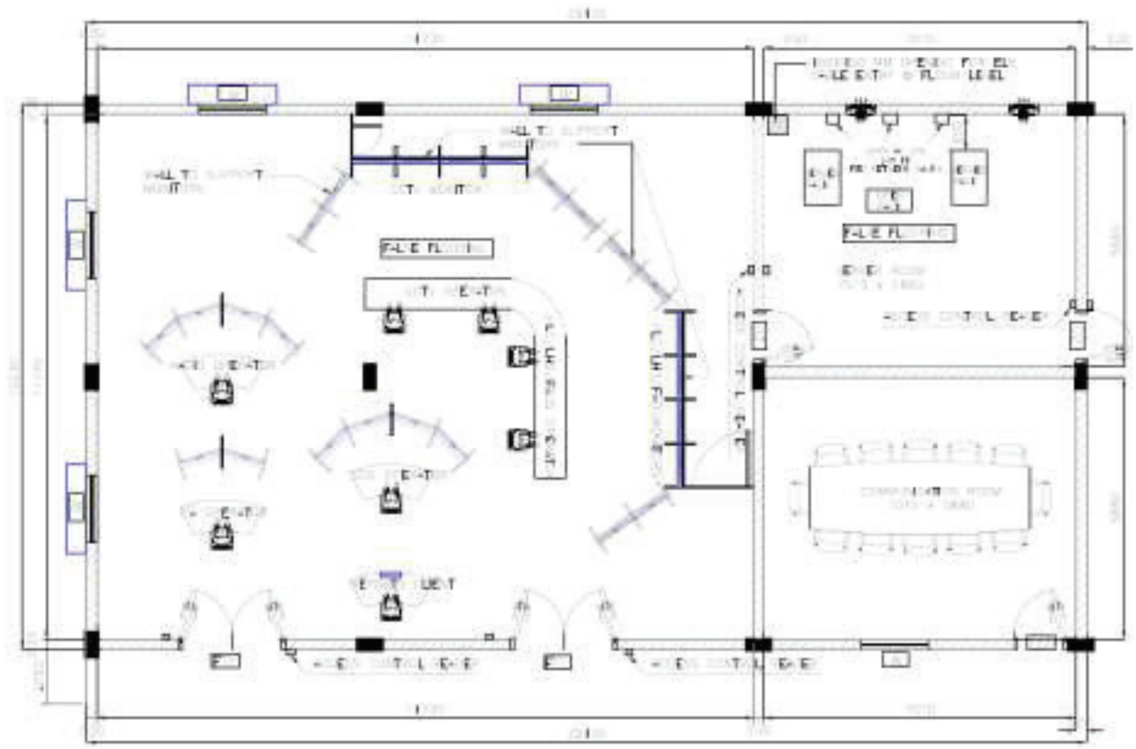


Fig. 15 : Cross section of tunnel showing arrangement of MEP Elements of the Project

6. INSTRUMENTATION AND MONITORING

The purposes of tunnel instrumentation and monitoring include ensuring safety during and after construction by providing early warnings of excessive ground movement, verifying design assumptions by collecting deformation and loading data for both initial and final tunnel supports, and optimizing excavation and support activities during construction based on real-time information about tunnel behavior.

For monitoring and measuring the deflection of open excavation work, piezometers and inclinometers were used at tunnel portals of T4, T5, T6 and T7.

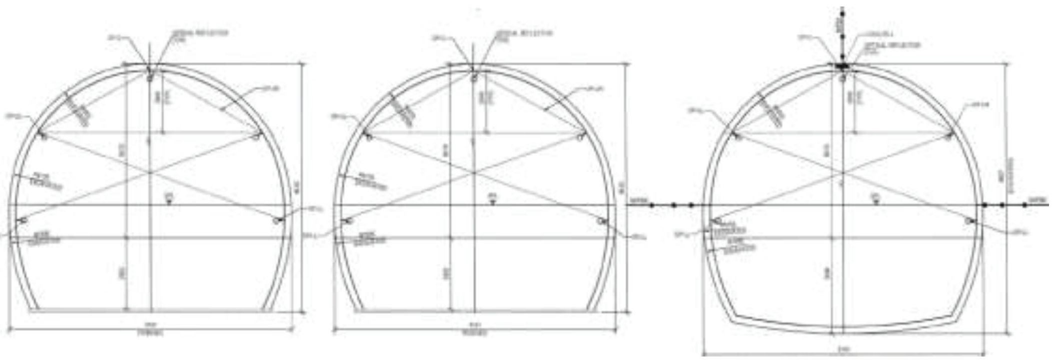
1. Piezometers were provided at the tunnel portal as per the issued drawings to measure changes in piezometric pressure at different depths.
2. Inclinometers were installed at the portal slopes to measure lateral displacements.

For monitoring tunnel convergence and deformation below given instruments were installed at Tunnel T4, T5, T6 and T7.

1. Extensometers and convergence bolts were provided to measure convergence at tunnel openings or at any other critical sections.
2. Multipoint borehole extensometers (MPBX) were installed in poor and very poor rocks or soft grounds to monitor deformations with depth.
3. Targets (reflectors) were used to determine 3D coordinates and monitor 3D absolute displacement, tracking target movements in space to allow a realistic assessment of the deformation behaviour of the tunnel.
4. Rock bolt load cells were installed at the anchor plates to obtain information on the maximum anchor load and the degree of utilization of the anchor.

The frequency of monitoring deployed was up to one month after the installation of the primary support system and weekly thereafter.

After construction, the frequency remained weekly; however, if deformations exceeded the Attention Limit, the monitoring frequency was increased from weekly to daily. When nearing the Alarm Limit from the Attention Limit (5mm below the Alarm Limit), the frequency was increased from daily to every 12 hours. The absolute displacement measurement intervals in each rock class were adopted as per the issued/approved drawings. The convergence limits, including the Alert limit, Attention Limit, and Alarm limit, were followed as per the issued/approved drawings for each class. (Figure 16)



DETAILS		
S NO	SIGN	INSTRUMENT
1		DEFORMATION MONITORING POINT OPTICAL REFLECTOR
2	MPBX	MULTI-POINT BOREHOLE EXTENSOMETER ANCHOR DEPTH - 2.0m, 4.0m & 6.0m
3	□	LOAD CELL

CONVERGENCE LIMIT (3DMS)-MM			
CONVERGENCE	ALERT LIMIT (MM)-50%	ATTENTION LIMIT (MM)-80%	ALARM LIMIT (MM)-100%
CLASS-I & II	10	16	20
CLASS-III	15	24	30
CLASS-IV	25	40	50
CLASS-V	30	48	60

Fig. 16 : Cross-section showing Instrumentation in each rock class with limits

7. PROBLEM ENCOUNTERED AND INNOVATION DONE

I. Problem encountered and innovation done at Tunnel T4 cut and cover portion:

At the tender stage, there was no provision for the cut-and-cover method in the low-cover zone. The contract documents primarily specified the use of pipe roofing and lattice girders as additional support, resulting in increased cycle time and cost. As an innovative solution, the cut-and-cover method was implemented for the 1.55 km low-cover zone in tunnel T4, proving to be more economical than the method specified in the tender stage drawings. Additionally, the pioneering use of a double gantry was introduced by 'GRIL' for the first time in India to cater the placing of 750 mm thick lining concrete.

II. Problem encountered at Tunnel T7 and cover portion:

Although the cut-and-cover method was initially selected for 100 m of the low-cover zone, the entire stretch was completed using the NATM tunneling method due to the lack of forest clearance and inaccessibility of the area to execute the job as per the methodology for cut and cover. The stretch of 100 m was successfully completed within 2 months by NATM method.

III. Land acquisition problem for Escape tunnel:

There was a land acquisition issue due to local complications. These complications prevented the client from timely transferring the land required for the construction of the escape tunnel. As a result, the construction work for the escape tunnel was delayed, impacting the overall project schedule, and potentially affecting subsequent phases of construction. Further in the best interest of the project progress GRIL vide its internal Liaoning with the local authorities and locals acquired the requisite land for construction of escape tunnel.

VI. Problem encountered for access road:

According to the contract conditions, the responsibility for providing access to the site was within our scope. All possible scenarios for the shortest route from the National Highway to the portal location were thoroughly explored. Extensive liaison with local residents and authorities was carried out to secure the necessary land. The selected shortest route enabled the project team to construct the access efficiently and reduced the cost of land acquisition. This optimization not only streamlined the logistics to the site but also contributed to timely start of project works.

V. Introduction of Poly fiber Reinforced Shotcrete (PFRS) instead of Plain shot-crete and wire mesh:

At the tender stage, the drawings specified the installation of wire mesh and the application of plain shotcrete for the underground tunnel works. The project design team proposed an innovative alternative to these conventional support measures: the use of PFRS instead of plain shotcrete and wire mesh. After a comprehensive review, the client approved this proposal. The adoption of PFRS not only expedited the installation process by eliminating the need for wire mesh but also reduced the overall execution costs in RC II, III, and IV.

VI. Innovation in concrete lining

At the tender stage, the client initially specified RCC (Reinforced Cement Concrete) lining for the project. Instead, polyfibre concrete lining was proposed by the project team and executed. This alternative was chosen for its cost-effectiveness, enhanced safety features, and ability to significantly reduce construction time. Polyfibre concrete lining provided superior performance characteristics, including greater durability, fire resistance and reduced maintenance requirements. The advantages of this approach were presented to the client, who readily approved the change, demonstrating flexibility and a commitment to optimizing project outcomes.

8. QUALITY ASSURANCE & QUALITY CONTROL

Need of QA/QC: The Quality Assurance (QA) System aims to provide confidence that a structure or system will perform satisfactorily in service. It clearly defines responsibilities and authority, ensuring effective cooperation. An efficient QA/QC system is crucial for fostering teamwork and achieving good workmanship, while acknowledging natural variations in craftsmanship and maintaining realistic standards.

Quality Assurance System: The Quality Assurance System establishes procedures for defining, developing, and implementing quality standards for design, construction, and operation. It includes monitoring, testing, inspecting, documenting, and reviewing to ensure compliance. It also outlines administrative procedures, responsibilities, and communication patterns. Common quality control methods are inspection (checking physical characteristics), testing (verifying performance), and sampling (examining portions of large batches to infer overall quality).

9. HEALTH, SAFETY, AND ENVIRONMENTAL (HSE)

Ensuring the highest standards of health, safety, and environmental protection has been a top priority for the Adenigharh to Puranakatak project. A comprehensive HSE management plan was implemented to safeguard the well-being of all personnel and minimize environmental impact throughout the project's duration. Rigorous safety protocols, regular training sessions, and proactive risk assessments were conducted to mitigate potential hazards and ensure a secure working environment. One of the project's notable achievements is the attainment of 1.5 million safe working hours without any major incidents, demonstrating a robust commitment to minimizing risks to workers' lives. Additionally, by adopting the NATM in Tunnel-7 for 100 m reach instead of the cut-and-cover method, the project effectively avoided the deforestation of nearly 3,000 trees, highlighting its dedication to environmental stewardship and sustainable practices.

10. CONCLUSION AND SUCCESS STORY

The Adenigharh to Puranakatak New BG Rail Line Project showcases exemplary innovation and efficiency in large-scale infrastructure development. During the project, significant challenges were met with innovative solutions. Initially, the cut-and-cover method was implemented in Tunnel T4, replacing the conventional NATM method in low cover zone. This approach proved more economical and effective, while the introduction of a double gantry by GRIL for placing 750 mm thick lining concrete demonstrated cutting-edge innovation.

For Tunnel T7, the project team successfully adapted the NATM method due to forest clearance issues and accessibility constraints, completing the 100-meter stretch in less than 2 months. Land acquisition problems for the escape tunnel were resolved through proactive engagement with local authorities, minimizing delays and keeping the project on track. The access road was optimized by identifying a more efficient route, reducing land acquisition costs, and enhancing logistics. Additionally, the project adopted Polyfibre Reinforced Shotcrete (PFRS) instead of plain shotcrete and wire mesh, streamlining installation and reducing costs. The transition to polyfibre concrete lining, approved by the client, provided enhanced safety, durability, and cost-efficiency. The project achieved 1.5 million safe working hours and preserved nearly 3,000 trees, reflecting a strong commitment to safety and environmental sustainability.

POLICY, FINANCING & PROJECT DELIVERY FRAMEWORK FOR TUNNELLING-BASED INFRASTRUCTURE PROJECTS

RAVINDRA KUMAR PATHAK
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***Introduction*-Tunnelling Asia 2025** serves as a vital platform for advancing the goals of fostering innovation, sharing global best practices, enhancing institutional capacity, and strengthening industry collaboration. Today, tunnelling is no longer viewed solely as an engineering activity; it is recognized as a strategic enabler of sustainable urban development, regional integration, and climate resilience. To thrive in India's ambitious public infrastructure landscape, stakeholders must embrace digital technologies (including use of AI and ML, adopt risk-sharing contractual models, invest in skilled manpower, and prioritize safety, quality, and lifecycle performance of assets.

Tunneling is now a strategic enabler of sustainable, resilient, and space-efficient infrastructure in India's fast-growing urban and mountainous regions. To fully utilize its potential, India must adopt:

- A modernised and harmonised policy framework
- Innovative PPP and financing mechanisms
- Robust contractual and risk management systems
- Advanced institutional capacity and skill development

With these reforms, tunnelling will drive the next era of green mobility, energy security, urban efficiency, and climate-resilient national connectivity.

Now India experiencing rising urban density, geographical constraints, and increasing environmental sensitivities, tunnelling has emerged as a transformative solution for next-generation infrastructure development. Whether in metro rail systems, strategic road tunnels, hydropower conduits, railways, city utilities, or multimodal transport, underground infrastructure offers unmatched benefits in:

- Land optimisation in congested urban areas.
- Reduced ecological footprint compared to surface alignments.
- Enhanced safety and climate resilience
- Improved travel efficiency and reduced congestion
- Long life-cycle performance with lower social disruption
- Ultimate increase in Public transport system

- Creation of world class infrastructure

To unlock these advantages at scale, we need a coherent framework integrating policy, financing, PPP structures, risk governance, and institutional capacity, ease of doing business, skill development.

Evolving Policy & Regulatory Frameworks National Alignment.

Development in Tunnelling Infrastructure to be aligned with national planning and sectoral strategies including:

- National Infrastructure Pipeline (NIP)
- Gati Shakti PM–National Master Plan
- National Transit Oriented Development (TOD) Policy
- Green Mobility & Net-Zero commitments
- Hydropower/clean energy missions

PPP Models & Innovative Financing Mechanisms

Tunnelling projects require high CAPEX, long gestation, and technically sophisticated delivery, making financing structures critical.

Suitable PPP Structures for Tunnels

(A) Hybrid Annuity Model (HAM) – suitable for mountain/road tunnels with uncertain traffic
(B) DBFOT (Toll-Based) – only where traffic/revenue is predictable (city connectors)
(C) Availability-Based PPP – ideal for metros, utility tunnels, strategic tunnels
(D) EPC + Long-Term O&M Hybrid – useful in large scale projects
(E) Annuity + Revenue Share – balances fiscal and market risks

In India, the PPP mode has been successful in Road construction. National Highways Authority of India is doing good job. Metros are yet to take lead in PPP structure.

Innovative Financing Instruments

- Viability Gap Funding (VGF) for low-revenue tunnels
- JICA/ADB/World Bank long-tenure loans for strategic & trans-hill tunnels
- Land Value Capture & TOD around tunnel stations/portals.

Financial Appraisal Metrics

- Financial Internal Rate of Return (FIRR): Typically 8–12% for transport tunnels; lower for metros (requiring VGF)
- Economic Internal Rate of Return (EIRR): National approval threshold 12–18%; tunnelling often exceeds due to time savings, safety, emissions reduction

Project Delivery & Contractual Risk Management

Risk Category	Description	Impact
Environmental	EC delays, vibration restrictions	Medium
Land/R&R	Delay in land acquisition	High
Utility Shifting	Challenges in shifting of utilities in urban areas	High
Equipment	TBM breakdown	High
Financial	Escalation in materials, Foreign currency fluctuation	Medium
Time and cost over run	Avoidable and Unavoidable-at the end of contractor/Principal	High
Delay in receipts of project fund	DPR related	Medium
Legal/Contractual, contractors variation extra items	Differences and Disputes	High

Risk Allocation Principles

Tunnelling risks are unique and must be handled through:

- GBR-based geological risk sharing
- Differing Site Condition (DSC) clauses
- Clear definitions of expected/abnormal strata
- Time and cost compensation for validated DSC events
- Import/currency fluctuation clauses for TBM spares

Land Acquisition, R&R & Utility Management

Land Acquisition Optimisation

Tunnels reduce land needs, but acquisition is needed for:

- Entry/exit portals
- TBM launch & retrieval shafts
- Ventilation & emergency shafts
- Muck disposal areas

- Construction yards

Policies should:

- Use Government/Right of Way land
- Adopt subsurface rights / underground easements
- Pre-acquisition of land tender as part of project readiness

Rehabilitation & Resettlement

- Comply with LARR Act, 2013
- Compensation for structures, livelihood, temporary relocation
- Community engagement in vibration/blasting zones

A major cause of delays in urban tunnelling

- Underground Utility Shifting
- Mandatory GPR surveys + GIS mapping
- Utility coordination cell with all agencies
- Trial pits at conflict points
- Consider future-use multi-utility tunnels

Environmental, Forest & Wildlife Clearances

Tunnelling requires specialised environmental assessment including:

- Groundwater modelling
- Vibration/noise impact
- Muck disposal & circular reuse
- Subsidence prediction
- Wildlife/eco sensitive mitigation

Clearances must be streamlined through:

- Parallel processing
- Pre-approved muck disposal sites
- Tunnel-specific EIA guidelines

Time & Cost Overrun Control Mechanisms

Major Time & cost Overrun Causes

- Land acquisition
- Litigation and LA/R&R bottlenecks
- Forest/environmental clearance delays
- TBM breakdown/import delays
- Slow utility shifting
- Availability of funds
- Force majure
- Political reasons
- Geological risk contingencies
- TBM part cost escalation
- Steel/cement inflation
- Improper investigation or DPR errors

Control Strategies

- GBR-based contract management
- BIM-linked scheduling
- DAAB-led monthly risk review
- Price indexation for key inputs

Capacity Building, Training & Skill Development

Institutional Capacity

Skills for India's Growing Tunnel Sector

Knowledge Ecosystem

- Annual Tunnel Technology Roadmap
- National tunnel data repository
- Training mandatory for contractors/consultants

Dispute Prevention & Resolution

Given the scale, complexity, and long gestation period of major public infrastructure projects in India, setting up a robust, transparent, and time-bound dispute resolution mechanism is essential to ensure uninterrupted project delivery. A well-designed system reduces delays, avoids cost escalation and fosters a healthier contractor ecosystem. The following multi-tiered framework is recommended for large Government contracts

- *Tiered Dispute Resolution Mechanism*
- *A layered system ensures disagreements are resolved at the earliest possible stage:*
- *On-Site Dispute Resolution Committee Including independent technical expert.*
- *Meets at fixed intervals (e.g., monthly) to address emerging issues.*
- *Targeted resolution timeline 28 days.*
- *Prevents small disagreements from escalating.*
- *Standing Dispute Resolution Board (DRB)*
- *A three-member independent board appointed at project inception.*
- *High-Level Contract Review Committee (HLCRC)*

Technology-Enabled Dispute Management

- *AI/ML and digital tools can strengthen dispute prevention and resolution:*
- *Digital claim management platforms to track variations, delays, payments, and correspondence transparently.*
- *AI-driven early risk identification to flag potential disputes.*
- *Centralized document management to eliminate ambiguity and loss of records by way of robust software tools such as SAP*

Many disputes originate from unclear contracts

- *Use of standardized contract templates (DOE, CVC, CPWD/Metro guidelines).*
- *Clear provisions for force majeure, price adjustment, geological surprises, and unforeseen conditions—especially critical for tunnelling.*
- *Balanced risk allocation between owner and contractor.*
- *Independent engineer/authority engineer empowered to make prompt decisions.*

- *Accountability and Timelines in Government Decision-Making*
- *Statutory timelines for approvals, certifications, and claim decisions.*
- *Defined escalation ladder to prevent file stagnation.*
- *Regular monitoring of dispute pendency by the ministry or nodal agency.*
- *Mediation as a Parallel Recourse*
- *Post-Project Review and Learning System*
- *Periodical review of recurring dispute patterns.*
- *Updates to contract structures and guidelines based on lessons learned.*

Aligned with the Ministry of Finance Office Memorandum no. F-.1/2/2024-PPD dated 03.06.2024, the framework should ensure for domestic procurement by the Government and its entities and agencies(including CPSEs, PSBs etc and Government Companies

- Dispute Avoidance First – via DAAB (Dispute Avoidance & Adjudication Board)
- Structured Mediation – mandatory before arbitration
- Internal Review Committee (IRC) – approves only essential arbitration cases
- Institutional Arbitration – at Delhi, Mumbai and Hyderabad

This reduces cost, time, adversarial escalation, result in speedier and finality of the award.

As per guidelines in the said OM, the Arbitration (if included in contracts) may be restricted to disputes with a value less than Rs.10 crores. The figure is with reference to the value of the dispute (not the value of the contract, which may be much higher).It may be specifically mentioned in the bid conditions that in all other cases, the arbitration will not be a method of dispute resolution in the contract.

Integration of AI, machine learning, and advanced digital technologies

As the country is transforming the way tunnelling and large-scale public infrastructure projects are conceived, executed, and managed. Digital tools such as Building Information Modelling (BIM), digital twins, automated monitoring systems, and data-driven predictive analytics are enabling faster decision-making, enhanced design optimization, and real-time risk mitigation. AI- and ML-based models can accurately forecast geological behaviours, optimize TBM performance, improve schedule reliability, and reduce cost overruns—challenges that have historically affected underground works in India. These technologies also play a pivotal role in strengthening worker safety through continuous equipment health monitoring, hazard prediction, and remote operations. By embracing these digital advancements, public infrastructure agencies and project stakeholders can significantly improve productivity, transparency, accountability, and lifecycle asset performance, thereby ensuring more prompt, efficient, and sustainable project delivery across India's rapidly evolving infrastructure landscape.

Conclusions

As India embarks on an unprecedented phase of infrastructure expansion, tunnelling has appeared as a cornerstone of progress across metro systems, highways, railways, hydropower, and water management projects. With urban centres expanding and environmental pressures intensifying, the demand for sustainable, resilient, and future-ready underground solutions has never been more critical. Sustaining success in India's public infrastructure ecosystem requires not only technical excellence but also a deep commitment to long-term planning, robust project governance, community engagement, and environmentally responsible construction practices.

The Asian region—with its rapidly growing economies, diverse terrains, and massive infrastructure requirements—offers immense opportunity for collective learning, technology exchange, and growth in underground space development. As we move forward, I encourage active participation from all ITA member nations, especially our Asian counterparts, to share their experiences, demonstrate their progress, and build enduring partnerships through this prestigious platform. Together, we can shape a more sustainable, resilient, and innovative future for tunnelling and underground infrastructure across India and the region.

Tunnelling is now is a strategic enabler of sustainable, resilient, and space-efficient infrastructure in India's fast-growing urban and mountainous regions. To fully utilize its potential, we must adopt:

- A modernised and harmonised policy framework
- Innovative PPP and financing mechanisms
- Robust contractual and risk management systems
- Advanced institutional capacity and skill development

With these reforms, tunnelling will drive the next era of green mobility, energy security, urban efficiency, and climate-resilient national connectivity.

COMPARATIVE ANALYSIS OF TBM RETRIEVAL, TRANSPORTATION AND LOWERING: STRAND JACK GANTRY SYSTEM VS CONVENTIONAL METHODS

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Mumbai Metro Rail Corporation Limited

ABSTRACT

Mumbai Metro Rail Corporation Ltd is implementing a 33.5 Km fully underground metro rail project, Colaba-Bandra-SEEPZ (Line 3) having 26 UG stations and one At-Grade. The project is substantially completed, and trials and testing are going on in the first reach. Tunnelling work was completed using 17 Tunnel Boring Machines (TBMs) with multiple drives through an array of launching and retrieval shafts. While there were Drive-Through and Drag-Through at some stations, many times the TBMs had to be retrieved, transported and lowered between re-trieval and launch shafts for successive drives. Retrieval, transportation, and re-deployment pose significant logistical challenges. Traditionally, TBMs are dismantled into components for trans-portation and lowering, which is time consuming and messy. The advent of the Strand Jack Gantry System offers a novel approach, potentially trans-forming TBM handling efficiency by enabling the transport and lowering of an intact TBM. Both these methods of retrieval, transportation and lowering were adopted by Package 3 contractor to relaunch the TBMs. This paper aims to com-pare the effectiveness, cost, time efficiency, and safety of using the Strand Jack Gantry System versus conventional dismantling methods for TBM retrieval, transportation, and lowering and seeks to provide a comprehensive analysis to the decision-makers in tunnel construction projects

1 INTRODUCTION

Mumbai Metro Rail Corporation Ltd (MMRCL) is undertaking the construction of the 33.5 km fully underground metro rail project, Colaba-Bandra-SEEPZ (Line 3), which includes 26 underground stations and one at-grade station. As one of the most ambitious infrastructure projects in Mumbai, this line aims to address the city's increasing transportation needs by providing a high-capacity metro system. With most of the project substantially completed and testing underway, the tunneling phase was accomplished using 17 Tunnel Boring Machines (TBMs) across multiple drives, utilizing various launching and retrieval shafts.

Mumbai Metro Line-3 is divided into seven contract packages over a length of 33.5 km. Package 03 of Mumbai Metro Line-3 involves the construction of five underground stations Mumbai Central, Mahalaxmi, Science Museum, Acharya Atre Chowk, and Worli, along with associated tunnels. The work has been awarded to a joint venture between Doğuş İnşaat ve Ticaret A.S. (Turkey) and Soma Enterprise Limited (India). As part of Package 03, approximately 7573 meters of tunnelling was executed using a Slurry Tunnel Boring Machine (TBM), which are named as TANSA-1 and TANSA-2 manufactured by Robbins.

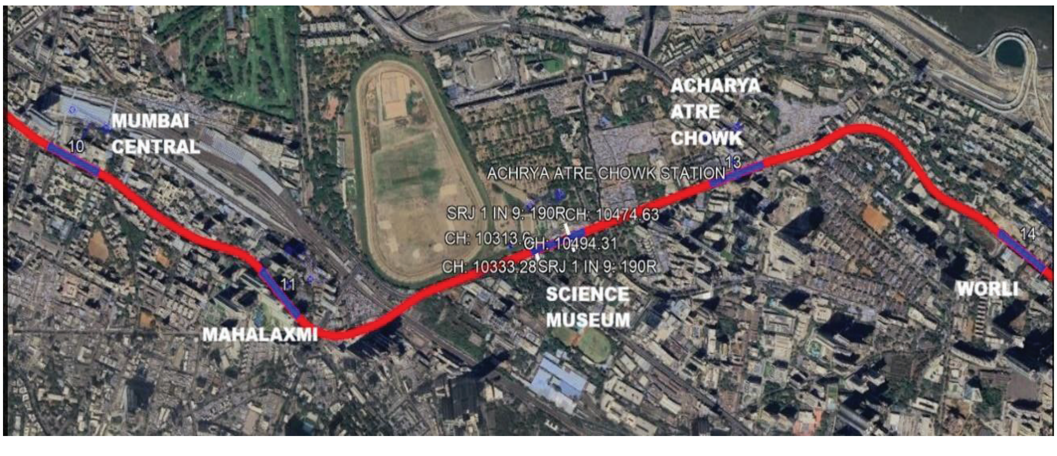

Figure 1: Mumbai Metro Line 3 Route Map for UGC 03

The contractor successfully launched and retrieved the TBM from designated cut-and-cover station boxes, facilitating precise excavation and ensuring alignment within the challenging urban environment. While some stations allowed Drive-Through and Drag-Through operations, mostly TBMs had to be retrieved, transported, and re-deployed be-tween shafts for successive drives with conventional method, which are time-consuming and labor-intensive along with substantial logistical challenges. And at some locations MMRC have introduced the Strand Jack Gantry System for TBM retrieval, transport, lowering and relaunch for successive drives. This Stand Jack System offers a novel approach to TBM handling, allowing the intact machine to be lifted, transported, and lowered with effectiveness, cost, time efficiency, and safety.

In this paper, it is aimed to provide a comparative analysis of the effectiveness, cost, time efficiency, and safety of the Strand Jack Gantry System versus conventional TBM dis-mantling, lifting and transportation methods by exploring both approaches. The analysis is intended to inform decision-makers in tunnel construction projects and contribute to the development of more efficient TBM handling techniques, and to provide insights for future tunnel construction projects, focusing on potential improvements in logistics and overall project management.

2 BACKGROUND

2.1 *Tunnel Boring Machines (TBMs) and their challenges*

Tunnel Boring Machines (TBMs) are highly specialized and massive pieces of equipment designed to excavate tunnels through different soil and rock strata. They provide the precision and speed required to construct large-scale underground projects while minimizing surface disruption. However, their sheer size and complexity pose challenges once tunnel-ling work is complete. The retrieval of TBMs often requires the dismantling of the machine into smaller, transportable components, followed by reassembly at new locations.

The conventional method of TBM retrieval, transportation, and re-deployment involves disassembling the machine in the underground shaft, which requires significant labor and time. Then lifting to the surface and transporting the disassembled components to a new location and lowering the components and reassembling them is both costly and logistically complex. The dismantling process also introduces safety risks, as working with large, heavy machine components in confined spaces is inherently dangerous to cities like Mumbai.

2.2 *Strand Jack Gantry System*

The Strand Jack Gantry System offers a new approach to TBM retrieval and transportation. It is an advanced hydraulic system that allows for the lifting, moving, and lowering of entire TBMs without requiring extensive disassembly. The system employs hydraulic jacks, wire strand technology, and gantry cranes to move heavy loads, including fully assembled TBMs, from one location to another. By bypassing the dismantling process, this system has the potential to reduce time, costs, and risks associated with TBM retrieval and re-deployment.

3 METHODOLOGY

The following parameters are considered to compare the effectiveness, cost, time efficiency, and safety of using the Strand Jack Gantry System with conventional methods for TBM retrieval, transportation, and lowering. The comparison is based on four key factors:

- Time efficiency: The duration required for retrieval, transportation, and re-deployment.
- Cost implications: The overall cost of the operation, including labor, equipment, and logistics.
- Safety considerations: The safety of workers and the minimization of hazards during the process.
- Operational effectiveness: The ease of operation and coordination in the project workflow.

The data for this study is drawn from UGC 03 Contract of the Mumbai Metro Line 3 project (Colaba-Bandra-SEEPZ), where both conventional method and strand jack gantry system were used for TBM retrieval, transport and lowering for completing the tunneling project scope, and serves as the main case study for this analysis.

4 CONVENTIONAL TBM RETRIEVAL, TRANSPORTATION AND LOWERING METHOD

The conventional method for TBM retrieval involves dismantling the assembled TBM machine along auxiliary gantries into transportable components. This typically includes separating the cutterhead, shield, backup gantry, and other components. Each part must be transported individually, usually by heavy-duty trucks, gantry cranes/conventional cranes, or barges, depending on the location. The machine is reassembled at the new site before it can be deployed again. Conventional methods involve using cranes, skidding systems, and slide rails to retrieve, transport, and lower TBMs. Cranes are often used for lifting, while skidding systems or slide rails are used for horizontal movement. The major advantage of the Conventional method is that all equipment like cranes and skidding systems are easily available in nearby areas of construction sites, lower initial set up cost due to readily available cranes, these setups do not require any special skills. Therefore, there is no need for significant customization in the use of conventional methods to adapt them to different sites. Conventional methods have some disadvantages such as, significant areas required for crane operations and setup due to tandem operation for lifting/lowering extremely heavy TBM components, in this method where cranes have been used can pose higher risk due to possibility of load sway, uneven lifting, or failure of mechanical components hence vulnerable for their risk against stability, as cranes especially working with extremely heavy loads may pose higher risk of instability. The setup time for conventional systems is usually longer, and the process might be slower, particularly if space is constrained.

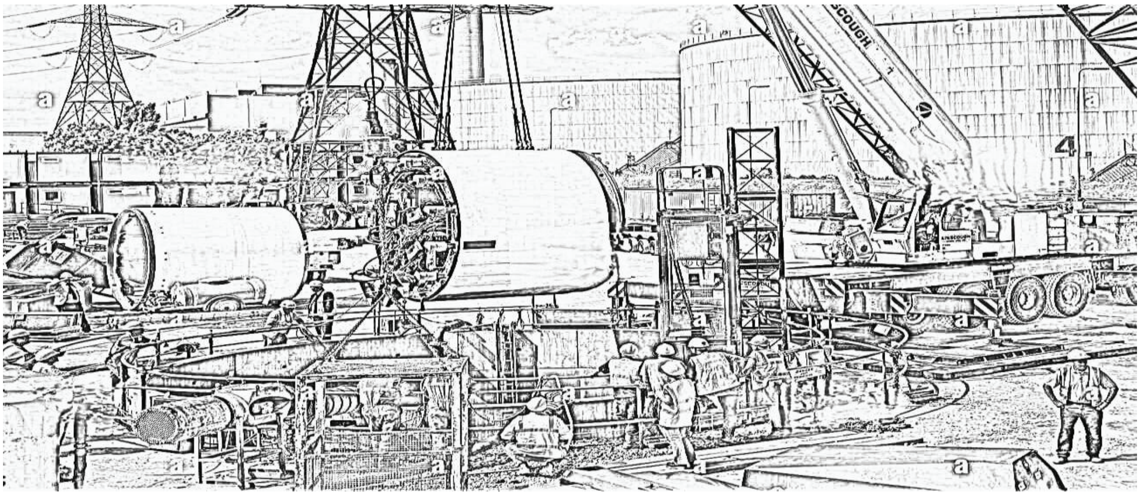

Figure 2: Typical conventional method lifting/lowering operations

5 STRAND JACK GANTRY SYSTEM

The Strand Jack Gantry System offers an innovative approach to Tunnel Boring Machine (TBM) retrieval, transport and lower at launching/retrieval shaft by eliminating the need for extensive disassembly. Instead of breaking down the TBM into smaller components, hydraulic jacks and a gantry crane lift the intact machine from the shafts for tunnel. The TBM is then transported using specially designed vehicles and lowered into the new site without major disassembly. This streamlined process significantly reduces the time required for TBM retrieval and re-deployment thus saving almost an extensive amount of time for the retrieval, transport and launch of the TBM and completing all the tasks. Though the initial cost of the Strand Jack system is higher due to specialized equipment, overall project costs are reduced by minimizing labor hours, project downtime, and logistical challenges. The Strand Jack system also improves safety by reducing manual interventions and handling TBM components, lowering the risk of accidents. Its hydraulic operation allows for precise, controlled movement, further enhancing safety. The system's modular design adapts to different site conditions, making it ideal for large, complex tunneling projects where efficiency, safety, and precision are paramount. While requiring a higher initial investment, long-term savings and reduced operational risks make it a superior alternative for TBM retrieval and transportation in modern tunneling projects.

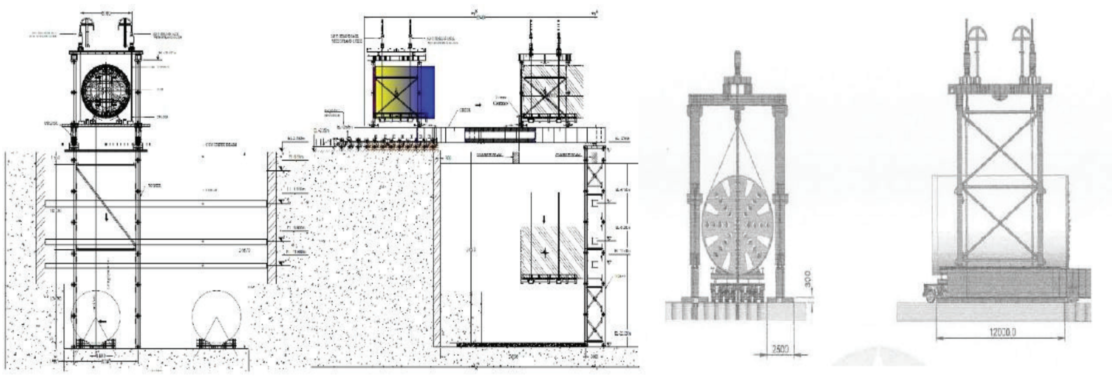

Figure 3: Typical Strand Jack System lifting/lowering operations

6 COMPARITIVE ANALYSIS AT UGC 03 OF MUMBAI METRO LINE – 3

- First Drive: TBM-1 and TBM-2 were lowered into shaft with conventional method and launched from the south end of Science Museum station, with bore through Acharya Atre Chowk, and finished TBM tunneling up to the north end of Worli station. The TBMs were then lifted and relocated back to the south end of the Science Museum by strand jack method.
- Second Drive: After retrieval of TBM from Worli station with strand jack method and transported back up to south end of the Science Museum station by SPMT (self-propelled modular transporter) and lowered both TBMs by strand jack method for re-launch from the Science Museum towards Mahalaxmi station continued the drive to the north end of Mahalaxmi station. The machines were then dragged to the south end of Mahalaxmi station and relaunched to complete the drive up to North end of Mumbai Central Station.
- Final Drive: The TBMs tunneled from Mahalaxmi to Mumbai Central station, where they were retrieved by conventional method.

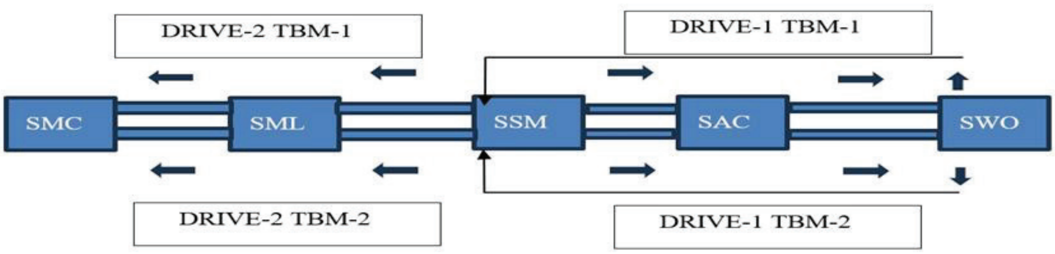

Figure 4: Schematic representation of TBM Drive details of UGC03 at MML3

6.1 *Conventional Method*

In the Mumbai Metro Line 3 UGC 03 project, a 500-ton capacity crane was used for the lifting, transportation, and lowering of the 450-ton Tunnel Boring Machine (TBM). The operation involved breaking down the TBM into smaller, modular parts for easier handling. The total TBM weight is 450 tons, with the cutter head weighing 65 tons. The intermediate shield is split into two parts, weighing 84 and 54 tons. The drive unit weighs 54 tons, and the front and rear shields weigh 57, 48, and 31 tons, respectively. Gantries, critical for auxiliary operations, range from 19 to 38 tons, and the connecting beam weighs 39 tons. The lightest component is the erector, weighing 6.6 tons, and these components were transported through urban area of Mumbai city multiple transport vehicles and using specialized equipment and permits to transport them through urban environments.

Process Breakdown of TBM Handling:

- Disassembly: In the conventional method, the TBM is entirely disassembled within the underground shaft. Each part is carefully removed, requiring the use of skilled labor and special tools to separate components such as the cutter head, shield, and backup gantries.
- Cranes for Lifting: Cranes are deployed to lift individual TBM components out of the shaft. Since the space within the shaft is usually confined, the crane operations must be carefully orchestrated, often requiring cranes to be mobilized for extended periods.
- Transportation: Once the components are retrieved, they are transported to the new tunneling site. Since TBM components vary in size and shape, their transportation requires multiple trips using different transport vehicles, adding to the complexity.
- Re-assembly: At the new site, the disassembled TBM components are reassembled, which involves rebuilding the entire TBM piece by piece. This process is time-consuming and requires the same level of precision and labor as the disassembly phase.

Challenges:

- Space Constraints: The underground shaft's limited space makes the disassembly process difficult. Workers must navigate within confined areas, which leads to slower progress and increased complexity in crane operations.
- Time-Consuming: Disassembly and re-assembly are labor-intensive and slow. The transportation of parts also requires precise coordination and multiple trips. Each stage adds to the project timeline, creating delays.
- Costly Operations: The need for continuous crane operations, skilled labor for both disassembly and re-assembly, and the logistical requirements for transporting multiple TBM parts results in high costs.
- Logistical Complexities: Transporting disassembled TBM parts through busy urban areas involves coordination with city authorities to minimize disruption. Traffic management, road closures, and nighttime operations add logistical difficulties, often requiring additional resources.
- Special Permits: Oversized components need permits to move through urban streets, necessitating careful planning.

Advantages:

- Proven Track Record: The conventional method is a well-established process globally, with a proven track record. It's a familiar approach that tunnel engineers and construction crews are comfortable with, reducing the risk of surprises.
- Modular Flexibility: Since the TBM is disassembled, the parts are modular, allowing for customized transport solutions. Smaller pieces are easier to manage individually, which gives flexibility in transportation and storage.

Disadvantages:

- Time-Intensive: Disassembling and reassembling TBMs takes a significant amount of time, causing delays in project schedules.
- Higher Costs: The requirement for continuous use of cranes, skilled labor for each phase, and logistical planning leads to higher overall costs.
- Urban Disruption: The longer timeline and continuous operations in urban environments disrupt local traffic and businesses, especially in densely populated areas like Mumbai.

6.2 *Strand Jack Gantry System*

In the Mumbai Metro Line 3 UGC 03 project, the COVID-19 pandemic caused delays in completing the tunneling work. To expedite progress and meet the project timeline, the Mumbai Metro Rail Corporation (MMRC) and the contractor introduced the strand jack method through Freight Wings logistic solutions through the strand jack method. This method allowed for the lifting of the entire assembled Tunnel Boring Machine (TBM) from the 25-meter-deep Worli station shaft, transporting it in one go and lowering it at Science Museum station to complete the remaining tunneling activities.

Process Breakdown of TBM Handling:

- No Disassembly: TBM crew proposed a bold, unconventional solution: retrieving the TBMs without disassembling them. This approach eliminated the need to take apart the machines in the underground shaft, bypassing the time and labor-intensive disassembly phase entirely.
- Mega Lift and Strand Jacks: To lift the entire TBMs as single units in the strand jack system, TBM lifting crew used 4 synchronized strand jacks, each with a capacity of 325 MT, mounted on a 2500 MT Mega Lift system. The TBMs were skidded along with heavy-duty skidding girders, a critical component that allowed precise and synchronized lifting.
- Transportation by SPMT: Once the TBMs were lifted, they were transported as whole units using a Self-Propelled Modular Transporter (SPMT) with 20 axle lines. This specialized equipment allowed the massive machines to be moved smoothly, even though narrow city streets, without disassembly.
- Minimal Urban Disruption: TBM crew completed the entire transportation process in 6 hours, carefully timing the operation to be finished before early morning traffic began at Worli's Dr. Moses Road. This minimized disruption to the city's daily activities.

Challenges:

- Engineering Complexity: Transporting TBMs as whole units posed complex engineering challenges. Special attention was required to ensure the ground could bear the weight of the TBMs, and regulatory compliance had to be maintained throughout the process.
- Risk of Damage: Moving such massive machines without disassembly carried potential risks. If not executed with precision, the entire TBM could be damaged, leading to costly repairs and project delays.

Advantages:

- Significant Time Savings: By eliminating the need for disassembly and reassembly, the project saved one month. This reduction in time was crucial for the UGC 03 project where deadlines were critical.
- Cost Efficiency: Without the need for cranes and extensive labor for disassembly, reassembly, and transport, this method resulted in substantial cost savings for the TBM crew. Fewer workers, equipment, and time meant lower operational costs.
- Minimal Urban Disruption: By completing the transportation process in a few hours and before peak traffic, the impact on local traffic and daily activities was kept to a minimum, making this approach ideal for urban projects.
- Urban Adaptation: This method was designed specifically to overcome the constraints of working in a dense urban environment. The method accounted for narrow roads, ground-bearing capacities, and the need to operate within strict timelines.

Disadvantages:

- High Initial Setup Cost: The innovative method required significant initial investment in specialized equipment and planning. While the overall cost savings were high, the initial outlay for customized tools and processes was substantial.
- Risk in Unconventional Method: As this was a relatively unconventional approach, it carried a level of risk. Any failure in the precise synchronization of the Mega Lift system or SPMT could lead to costly delays or equipment damage.

Table 1. Key Comparative points

Aspect	Conventional Method	Strand Jack Method
Disassembly	Full disassembly required	No disassembly, retrieved as whole units
Time Taken	Time-consuming	Significant time savings (1 month saved)
Cost	Higher due to continuous crane use	Lower due to less equipment and labor needs
Transportation	Requires disassembled parts transport	Single-unit transport with SPMT
Crane Requirements	Continuous crane operations needed	Minimal crane use, Mega Lift system utilized
Urban Impact	Disruptive due to longer timeline	Minimal disruption, fast transport
Space Constraints	Disassembly limited by space	Overcame space constraints by avoiding disassembly
Re-assembly	Re-assembly needed at new location	No re-assembly required, saved time
Logistical Complexity	High, with continuous crane movement	Reduced due to faster, whole-unit movement

7 PHOTOGRAPHS OF STRAND JACK GANTRY SYSTEM AT MUMBAI METRO LINE3

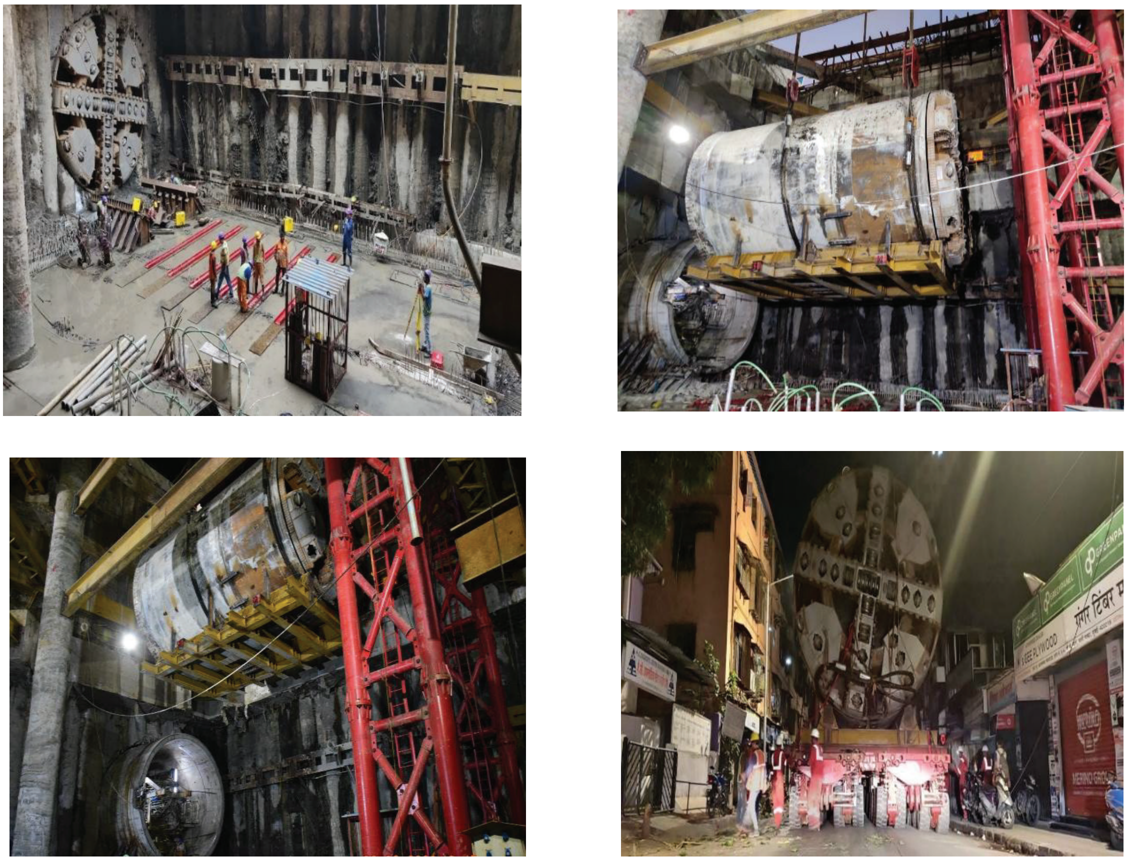

Figure 5: Preparatory works and TBM lifting operations using Strand Jack Gantry system and TBM movement using SPMT at UGC03 alignment

Figure 7: TBM lowering operations using Strand Jack Gantry system at UGC03

8 CONCLUSION

The Strand Jack Gantry System represents a major advancement in TBM handling, offering superior efficiency, cost-effectiveness, and safety compared to conventional methods. For large-scale projects like the Mumbai Metro Line 3, the system's ability to streamline the TBM retrieval and transportation process provides substantial benefits. While higher upfront investment may be a deterrent, the long-term savings and operational advantages make it a valuable tool for modern tunnelling projects. Future projects should consider adopting the Strand Jack Gantry System, particularly in scenarios where time, safety, and logistical efficiency are of the utmost importance.

9 REFERENCES

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NEW APPROACH TO IMPROVE THE USE OF SPRAYED CONCRETE FOR FINAL SUPPORT AND LINING

BENOIT DE RIVAZ
Bekaert

Introduction

Underground space plays a vital role in sustainable urban planning as it provides a solution to the limited surface area available. In terms of transportation, underground networks enable efficient and rapid mass transit, helping alleviate congestion and reduce carbon emissions. Moreover, underground space is essential for the infrastructure required to sustain cities, such as dense networks of pipes for delivering fresh water and sewers for wastewater treatment. Additionally, it accommodates the extensive network of cables and service stations necessary for modern communication systems. By utilizing underground space, cities can optimize their resource management and minimize their environmental impact, contributing to a more sustainable urban future. For a sustainable use of structural concrete, environmental and mechanical performances of concrete structures must have the same im-portance. By means of sufficiently high mechanical performances, the structural safety of a construction is ensured.

In a tunnelling project, it is generally considered that 60% to 70% of embodied carbon is contained in the concrete linings of the shafts and tunnels. It is paramount, therefore that the tunnelling industry does its utmost to significantly reduce or eliminate its use of cement in all applications – segmental linings, in-situ linings, sprayed concrete, and annulus grouts.

This is the reason why a great challenge for the coming years will be develop solution for low carbon lining.

Mechanical excavated tunnels (tunnels excavated with a TBM – Tunnel Boring Machine) are more and more used in Civil Engineering. In these tunnels, the lining is made assembling precast segments used by the TBM as reacting elements in the excavation process.

The use of Fibre-Reinforced Concrete (FRC) allows to reduce or eliminate the traditional reinforcement in the precast segment production. Over the last twenty years, the use of this technology has increased. The use of Fibre Reinforced Concrete (FRC) allows several advantages, compared with traditional steel mesh or steel bar reinforcement according to fib bulletin 83 and all main recommendation published as:

- Higher impact resistance
- Durability advantages at final stage
- Reduction of costs
- Sustainability advantages
- Cracking control during construction phases
- Boosting of the production process

Recent project has demonstrated that structural ductility, durability, and sustainability are going hand in hand.

This holistic approach will be clearly a new booster for FRC tunnel lining. This paper will provide the start of the art on this issue, the key design principal and detail recent cases studies showing impact in carbon calculation saving in France, Middle East, and Australia.

1 Sustainability & Structural Requirement

For a sustainable use of structural concrete, environmental and mechanical performances of concrete structures must have the same importance. By means of sufficiently high mechanical performances, the structural safety of a construction is ensured. At the same time, a low environmental impact guarantees a sustainable development, which is, in accordance with the definition by the Brundtland Commission of the United Nations, a "development that "meets the needs of the present without compromising the ability of future generations to meet their own needs" (Brundtland Commission, 1987).

FRC acts on the tensile behaviour of cracked concrete and imparts ductility to a fragile material. FRC's excellent properties which overcome cracking as well as its improved durability over reinforced concrete are why we continue to develop that material and explain its economic success. The Life Cycle Assessment (LCA) is a methodology for assessing environmental im-pacts associated with all the stages of the life cycle of a product or process. It quantifies a material impact on the environment over its entire existence, from extraction of the raw materials required for its production up to its end of life. This approach, combined with research into a low-carbon solution, will give new momentum to FRC.

2 Basic FRC Behaviour

A minimum tensile (strength) strength > 2200 MPa is recommended for final lining application considering the performance required and concrete classes.

The hooked ends ensure the desired fiber pull-out. This is the mechanism that generates the renowned concrete ductility and post-crack strength.

The tensile strength of a steel fiber has to increase in parallel with the strength of its anchorage. Only in this way can the fiber resist the forces acting upon it. Otherwise, it would snap, causing the concrete to become brittle. On the other hand, a stronger wire cannot be fully utilized with an ordinary anchor design. Therefore, the tensile strength of a fiber has to be perfectly aligned with its anchorage system and its diameter.

Wire ductility and concrete ductility are two different aspects. Dramix® 4D steel fibers create concrete ductility by the slow deformation of the hook during the pull-out process, and not by the ductility of the wire itself. The network provides by the fibre get a fundamental importance. Recommended diameter is 0.75mm and l/D= 80 to ensure a network > 10km/m3 with 40kg/m3 . This network will play a key role in cracking control, structural ductility and allow low dispersion in the result.

Table 1. Influence of the l/D ratio on the network effect.

l/D	80/60	65/60	45/50
Length (mm)	60	60	50
Diameter (mm)	0.75	0.90	1.05
Aspect Ratio	80	65	45
Network (m/kg)	276	200	147

3 Design Principle

Model Code 2010 is the most comprehensive code on concrete structures. It covers their complete life cycle from conceptual design, dimensioning, construction, and conservation through to dismantlement. It is edited by fib Model Code 2010 was produced through the exceptional efforts of participants in 44 countries from five continents. The fib bulletin 83 document aims to support designers, contractors and clients with guidance for the use of steel fibre-reinforced concrete, known as FRC, in precast segmental lining tunnels constructed using tunnel boring machines (TBMs) The document is intended to complement the fib Model Code 2010 (MC2010), which presents a section on the design of FRC, with the Model Code 2010 being considered as the reference basis for the design of FRC segmental lining.

Figure 1. Design process.

The tensile behavior of the materials was characterized by performing bending tests on a notched beam. The tests were performed according to the EN 14651 European code (ref 5), which is the reference standard for the CE label of steel and for ISO certification.

The compressive strength of the materials was measured by a testing cube with a side of 150 mm. For every cast made to produce every single segment, three beams were produced. In agreement with EN 14651, nominal strengths corresponding to four different crack mouth opening displacement (CMOD), namely 0.5, 1.5, 2.5 and 3.5 mm, were evaluated.

Figure 2 and 3 shows a typical result of the beam tests considering 40KG fibre type Dramix® 4D 80/60BGP with significant strength values. FL is peak force, fR1 and fR3 are the stresses related to CMODs equal to 0.5 and 2.5 mm respectively. These values are the reference ones for final lining design performed according to the fib Model Code 2010 prescriptions.

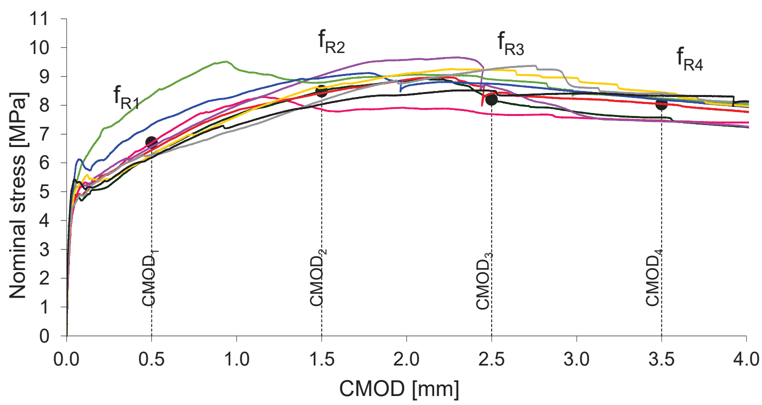

Figure 2. Curve Load -deflection of the beam bending tests according to EN 14651. (Roma University Report).

	f_L [Mpa]	f_{R1} [MPa]	f_{R2} [MPa]	f_{R3} [MPa]	f_{R4} [MPa]
Beam_01	4.68	6.70	7.86	7.69	7.47
Beam_02	4.90	6.28	8.49	8.20	7.58
Beam_03	4.78	6.45	8.41	8.42	8.04
Beam_04	5.15	6.56	9.04	8.64	7.44
Beam_05	5.72	7.33	8.95	8.75	8.19
Beam_06	5.03	6.27	8.60	9.23	8.45
Beam_07	5.63	7.75	10.2	8.99	8.54
Beam_08	4.60	6.28	8.16	9.25	8.40
Beam_09	5.43	6.18	8.03	8.50	8.33
Average	5.10	6.64	8.64	8.63	8.05
Characteristic	4.30	5.58	7.26	7.65	7.19

Figure 3. Results of the beam bending tests according to EN 14651 mean and characteristic value (Roma University Report).

To dimension a steel fiber-reinforced concrete segment, a reference test methodology needs to be adopted for the characterization of performance. In addition to the mechanical performance, various properties of the FRC can be specified.

Since brittleness must be avoided in structural behavior, fiber reinforcement can be used as substitution (even partially) of conventional reinforcement (at ULS), only if both the following relationships are fulfilled:

$f_{R1k}/ f_{Lk} > 0.4$

$f_{R3k} / f_{R1k} > 0.5$

Where fLk is the characteristic value of the nominal strength, corresponding to the peak load (or the highest load value in the interval 0 – 0.05 mm), determined from the EN 14651 beam test.

It is recommended to realize 12 beams per dosage and concrete mix formula.

If fibres are used as the only reinforcement for final lining, hardening post-crack behavior at section level (beam test) allow immediately:

- Cracking control at SLS
- Structural ductility (ULS)

The figure 2 how the typical expected result considering FRC only as reinforcement. The performance class according to Model Code 2010 in this example is C40/50 5e FRC material, which means,

Fr1k> 5MPA

Fr3k/fr1k> 1,3

Indeed, materials with *fR1k* ranging from 4.0 MPa mini to 6.0 MPa are commonly used for precast tunnel segments without any bar reinforcement, combine with a *fR3k* / *fR1k* ratio in the ranges 1.1 < *fR3k* / *fR1k* < 1.3 or1.3 < *fR3k* / *fR1k* < (class *d and e* respectively, according to the Model Code 2010 definition).

4 Carbon Counting

Among various construction processes, tunnel construction results in a significant amount of CO2 emissions because almost all tunnels are lined with reinforced concrete and utilize various high energy consuming equipment for excavation. Embodied carbon and high energy consumption can be minimized through three distinctive ways. Two main complementary approaches can be adopted to mitigate embodied carbon and reduce high energy consumption. The first method includes decreasing the overall quantity of reinforced concrete utilized through design optimization. The second approach is lowering the embodied carbon within each unit volume of the reinforced concrete by reducing the usage of Portland cement and steel rebar. This can be established by substituting Portland cement in the concrete mix low carbon binder and rebars by steel fibre. Indeed, the total mass of Co2eq is what we want to minimize from environmental product declaration and by decreasing the total mass of material.

Mix Design Components	Portland Cement	Slag (GGBS)	Fly Ash	Silica Fume	Admixture	Aggregate	Rebar	Steel Fiber
CO_{2eq} Factor (kg CO_{2eq})	0.92	0.1466	0.093	0.014	1.67	0.06	1.85	0.7

Figure 4. Co2 eq Factors from EPDs For Each Mix Design Component.

An Environmental Product Declaration (EPD) is a document that transparently communicates the key environmental performance indicators of a product over its lifetime.

A third-party verification ensures that data relating to environmental aspects of Dramix® has been validated by an external organization.

This declaration is the Type III Environmental Product Declaration (EPD) based on EN 15804:2012+A1 and verified according to ISO 14025 by an external auditor. It contains the in-formation on the impacts of the declared construction materials on the environment. Their aspects were verified by the independent body according to ISO 14025. Basically, a comparison or evaluation of EPD data is possible only if all the compared data were created according to EN 15804:2012+A1.

The environmental impact of Dramix® product (cradle to gate with options) is largely dependent on the energy intensive production of steel (half product) on which the manufacturer has only a limited influence. The carbon impact of steel production (Wire Rods) in the product stage A1 is as high as 85%. The impact of the production line largely depends on the amount of electricity consumed by manufacturing plant (0.34 kWh/kg of product). There are no significant emissions or environmental impacts in the A3 production processes alone (partly gas combustion). The production process itself does not have significant environmental impacts in the life cycle.

5 Low Carbon Precast Segments Case Studies The Montreal Metro Blue Line Extension Project

The Montreal Metro Blue Line Extension Project consists of construction of 6 kilometers of tunnel, as well as five new underground stations. This represents a good example as well showing than durability and sustainability goes hand in hand. Indeed, the reduction in segment thickness achieved with fibers can be primarily attributed to the concrete cover requirements of 60-75 mm on both intrados and extrados rebar to ensure the durability against corrosion when designing according to Canadian code CSA A23.1:19 (2019). In contrast, when subjected to chloride exposure, corrosion in steel fiber reinforced concrete is limited to just a few millimeters from the surface, and nonetheless, does not lead to spalling cracks and is not regarded as a durability issue. CO2 savings in the segments is realized by replacing rebar with steel fibers as the quantity of steel required is 50% less per m3 of concrete with fibers (40 kg/m3 vs 80 kg/m3). Additionally, the CO2 equivalent factor for rebar is reported to be 1.85 vs 0.88 for fibers. The fiber reinforced segments can be reduced in thickness due to no requirement for cover like rebar. This quantity of concrete savings also lowers the carbon footprint.

		Baseline Concrete Mixture (OPC) with Rebar			Optimized SCM Concrete Mixture with Steel Fiber		
Mix Design Component	CO_{2eq} Factor	Mass (kg/m^3)	CO_{2eq} (kg/m^3)	% Replacement by Mass	Mass (kg/m^3)	CO_{2eq} (kg/m^3)	% Replacement by Mass
Portland Cement	0.92	475	437		346.8	319.056	
Slag	0.1466	0	-	0%	104.5	15.3	22%
Fly Ash	0.093	0	-	0%	0	0	0%
Silica Fume	0.014	0	-	0%	23.8	0.3	5%
Admixtures	1.67	4.5	7.5	1%	4.5	7.5	1%
Aggregate	0.006	1430	8.6		1430	8.6	
Steel bar	1.85	80	148		-	-	
Steel Fiber	0.92	-	-		40	36.8	
Total		Total	601.1		Total	387.6	

Figure 5. Embodied carbon in unit volume for the baseline and the optimized final mix designs.

The owners' design engineer, AECOM, as part of a commitment to integrating sustainability best practices, performed a study utilizing the Envision framework to evaluate alternatives to achieve a most sustainable infrastructure project. Based on the results of this study the TBM bored tunnel sections will be lined with steel fiber reinforced precast concrete segments using low-carbon Supplementary Cementitious Materials (SCM) concrete.

In the TBM tunnel sections lined with pre-cast concrete segments, high performance Dramix® steel fiber 4D80/60BGP with a dosage of 40 kg/m3 is designed as standalone reinforcement.

See below table summarizing the results of the evaluation showing a reduction in total CO2 equivalent by nearly 50% using SFRC with an optimized SCM concrete mix design:

	Ring width (m)	Tunnel length (m)	D_{ex} (m)	D_{in} (m)	Ring Volume (m^3)	Total concrete volume (m^3)	CO_{2eq}/m^3 (kg)	CO_{2eq}/1 m tunnel (ton)	Total CO_{2eq} (ton)
40 cm Thick Segments	1.8	6000	9.4	8.6	20.4	67858	601.1	6.8	40,79
35 cm Thick Segments SCM w/ Fiber	1.8	6000	9.3	8.6	17.7	59046	387.6	3.8	22,886

Figure 6. Calculation of total embodied carbon footprint of the PLB tunnel segmental lining for the baseline and the optimized final designs.

7 Cracking Control&durability

As regards durability, the requirement for conventional reinforcement cages was 100 years. However, comparative checks on the segments installed have shown that the fiber reinforced segments have a better crack control behavior.

The use of fibers is perfectly suited to this type of geometry, especially since the cracking process generates finer cracks than the cracking process of a beam on two supports. In the case of tunnel lining segments, the final coating constitutes a hyperstatic mechanical system. This is a situation in which the fibers work perfectly.

Indeed, as only micro-cracks (<=0.2 mm) are observed and the segments work in compression when the ring is formed, they close up automatically.

When the cracks are very fine, i.e. with crack openings not exceeding 0.5 mm, the fibers are much more efficient than reinforcement bars in acting on this cracking. This is simply because the diameter of the fibers is mechanically better suited to these cracks than the diameter of concrete reinforcement bars. It is a problem of coherence of scale, as Pierre Rossi reminds us (international expert on fiber concretes/Fiber Concretes Martialis Edition)

In effect, the majority of steel fiber concretes are mechanically efficient up to crack openings not exceeding about 2 mm. Crack openings of between 1 and 2 mm, correspond, for the vast majority of cases, to the ultimate behavior of steel fiber concrete structures. Therefore, studying the durability of steel fiber concretes for crack openings around 1 mm can be considered as meaningless and seems unnecessary for the practice.

Also noteworthy is the excellent corrosion behavior of the fiber reinforced segments, linked to the small diameter of the fibers and their distribution.

This can be valuable when high external water pressures are combined with saline conditions (high chloride concentrations), which can lead to severe corrosion scaling in reinforced concrete segments.

In summary, a SFRC segment does not crack more than a reinforced concrete segment, with a corresponding smaller crack size, the cracks closing more easily as well, providing a much better risk/benefit balance in terms of corrosion.

A SFRC segment ensures greater durability than a reinforced concrete segment.

8 Quality Control

To provide additional peace of mind for dosage in situ on spray concrete lining, ,new inductive test to determine the content and orientation of steel fibers in fiber reinforced concrete (FRC) has been developed In collaboration with UPC – Polytechnic University of Catalonia –
According to fib bulletin 83 The procedures for the control of Fibre-Reinforced Concrete performance should be defined in the design process.

Usually, a quality control procedure considers two steps:

• initial qualification of the material (trials testing);

• tests during the segment production (production testing).

Before starting the segment production, compressive and bending tests in accordance with EN14651 have to be performed in order to control the fulfilment of the characteristic values defined in the design. In addition, tests should be carried out in order to verify the fibre content or the fibre orientation. In order to check the compressive properties of the concrete, the same procedure adopted for ordinary concrete should be followed. For the definition of the tensile properties of the FRC, tests according to EN 14651 should be performed. The material should be classified according to the Model Code 2010: characteristic values of the FRC residual strengths (fLk, fR1k and fR3k) have to be determined. In this phase, it is suggested to perform at least 12 beam tests according to EN14651 at 28 days of curing for each fibre dosage/fibre type and concrete mix that is to be considered. The test results can be considered positive if: • fLk fulfils specific requirement provided by the designer; • the characteristic value of fR1k is higher than the design one; • the ratio between fR3k and fR1k fulfils the design requirement; if a higher strength ratio is obtained, the material can be accepted (if no specific requirements are present in the design); • the fulfilment of the Model Code 2010 requirement for substituting the traditional reinforcement with fibre is verified (fR1k / fLk > 0.4 and fR3k / fR1k > 0.5

Some tests can be made with the aim to determine the correct fibre content. The fibre content can be measured at fresh or hardened state according to EN 14721. Tests in the hardened state can be made on segments by drilling cores and evaluating the fibre content in each core. This allows to verify the homogenous distribution of fibres in different parts of the segments. Furthermore, by sawing each core in a different layer, it is possible to control the correct distribution of the fibres in the segment thickness (controlling possible fibre segregation). Usually, a fibre content measured according to EN 14721 can be accepted if this differs less than 20% from the nominal value. A possible alternative solution is the use of inductive test method

The equipment **allows to determine the content and orientation of the fibers** present in the concrete from the variation produced by the fibers in the magnetic field generated by the equipment. By giving you a clear view of how fibers are distributed, it helps you verify that your project meets design requirements before you commit to large-scale pours. With eyeD® Inspector, you can reduce risks, optimize performance, and save time on every job

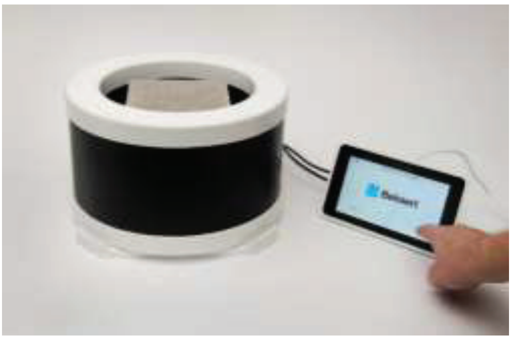

Figure 8 Dramix EyeD Inspector

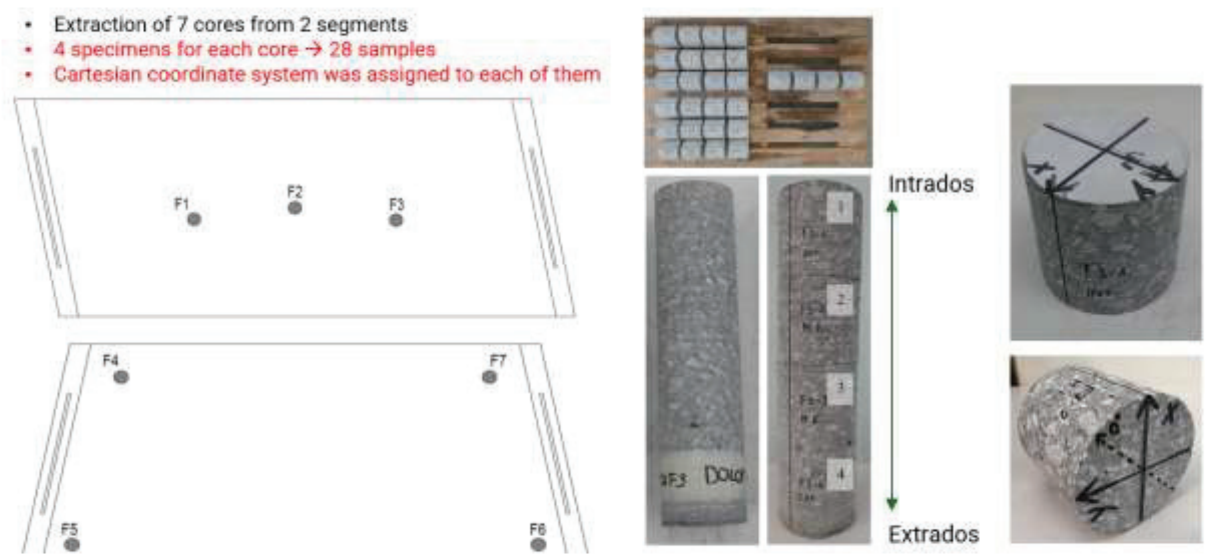

Figure 9 Example of specimen preparation (Roma University)

- Dramix Eye D inspector device shows good sensitivity and can be considered a valid screening tool, but its accuracy strongly depends on the acceptance threshold adopted. Fib Bulletin 83 acceptance criteria (20%) risk failing to detect non-compliant specimens due to minor overestimations, while more strictly margins (15%) increase the number of false positives, allowing for the identification of all truly non-compliant specimens.
- The Dramix EyeD inspector device could be used with a lower margin than that indicated in Bulletin 83, and, if necessary, supported by destructive tests on specimens that exceed this threshold, in order to ensure compliance more reliably.

In addition to the technical aspects, the use of the inductive method offers significant economic, time-related advantages. The ability to perform checks without crushing the specimens allows for reduced costs, shorter verification times

Conclusion

There has been a trend the last years that concrete tunnel linings have increased material consumption, cost, and environmental loads. Nowadays develop and/or improve tunnel construction methodology to choose the optimal tunnel lining, including environmental footprint and cost-effectiveness. Create required knowledge to produce final lining to meet new large infra-structure projects with modern demands to functionality incl. 100-year service life and environmental impact.

The use of steel fibre reinforced concrete will highly participate to meet low carbon lining by concrete consumption and steel reinforcement saving. If ductility and durability have been the key words the last 40 years, the sustainability will be the key driver for further FRC lining development in the coming years.

Indeed, new generation of binder combined with FRC allow new achievement:

- Provide excellent long-term durability performance exceeding that of Portland cement-based concretes.
- Excellent long-term durability performance exceeding that of Portland cement-based concretes. Extremely low embodied carbon footprint compared to conventional concretes on Port-land cements.
- Compared to reinforced concrete, fiber-reinforced concrete notably represents savings of around 5,000 tons of steel for 10 kilometers of tunnels (Typical Metro Tunnel).

We all believe that tunnels should use smart and sustainable construction materials. The future of tunnelling is choosing these materials today. High Performance steel fibre could play an important role in this final lining sustainable journey.

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ITB (2015) EPD – Environmental Product Declaration Type III.

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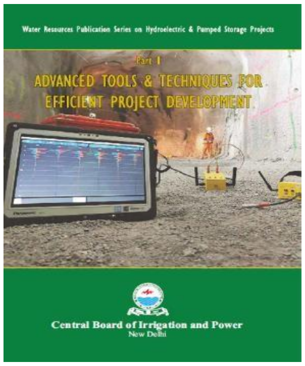

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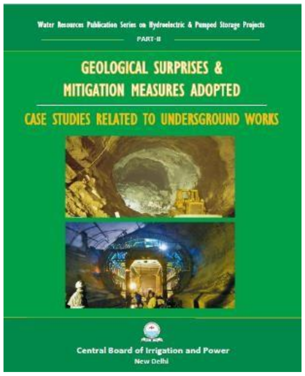

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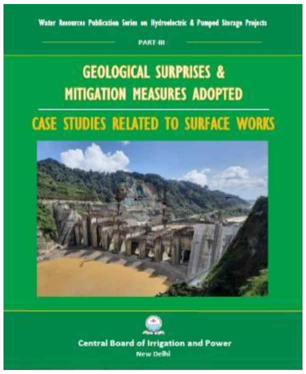

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Sensitive Heritage
Precinct

Extensive use of
temporary traffic deckings

TBM and NATM:
Tunnelling beneath Mithi River

Dense maze of civic utilities
and lack of space for diversion forced for in-situ support during construction

Alignment passing along:
Densely Populated areas | Very old dilapidated buildings | Skyscrapers Heritage precincts | Vital city infrastructures such as flyovers, metro line, etc

Transportation of fully assembled TBM
- a first for the country

Use of Steel Fibre Reinforced Spayed Concrete (SFRSC) permanent lining for a NATM cavern
- an innovative first for any railway tunnel in the country

Very impressive safety records
- achieved incident rate better than any similar project of the same magnitude elsewhere in the world

Seven hybrid stations:
Taken up 7 NATM Hybrid Stations simultaneously for the first time ever in the country

Very extensive use of
NATM technology
in cross-over caverns, stabling lines, stations platforms, pedestrian subway and cross-passages

Simultaneous deployment of 17 TBMs
- a record for the country

Complex logistical issues
due to congested roads and huge generation of muck

Length:
33.5 km
fully underground corridor

Safe Man Hours:
338
million
safe man hours

Total TBM Tunnel Rings
37,173

Total Floor Area Created below ground- Approx.
15.96 lakh
sq.m.

42
BreakThroughs

5,200
Temporary Traffic Decking

21,83,890
cum.
of muck

26 lakh
cum.
of concrete

3.56 lakh
metric tonne
Reinforcement Steel

3,300
millimetre
Intense Monsoon Averaging

92,300
metric tonne
Structural Steel

06
Casting Yards

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- Bridge over the Cadiz Bay, Spain
- Bridge over the River Lérez, Spain
- Gibraltar strait crossing, Gibraltar
- A4 Marão Highway, Portugal
- Sotra Link Road Tunnels, Norway
- Vestkorridoren E18 Road Tunnel, Norway
- Gubrist Road Tunnel, Switzerland
- Martigny High-Voltage Underground Link, Valais
- Limmern Pumped Storage Plant 1520 MW, Glarus
- Lötschberg Base Tunnel, Bern-Wallis CH Switzerland
- Ceneri Base Railway Tunnel, Switzerland
- Second Gotthard Road Tunnel, Ticino/Uri Switzerland
- North East Link, Australia
- Metro of Sydney, Australia
- Snowy 2.0 Hydro, Australia
- Forrestfield Airport Link, Australia
- HSR Zhengzhou-Zhoukou-Fuyang Railway (PMC), China
- HSR Beijing-Shenyang Railway (PMC), China
- HSR Shangqiu-Hefei-Hangzhou Railway (PMC), China
- Siddhababa Road Tunnel, Nepal
- HPP Bheri Babai Diversion Multipurpose 48MW, Nepal

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ADVANTAGES

- Improves Resistance to plastic and drying shrinkage crack.
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- Reduces Water Permeability.
- Cohesive mix.
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Fibre Tuff, Macro synthetic polypropylene fibre are heavy duty synthetic fibre specially engineered for use as an secondary reinforcement, providing excellent resistance to the post cracking capacity of concrete and are replacing steel fibre in a range of applications, including Tunneling and Mining, Marine structure, commercial & Industrial flooring, and precast concrete units.

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The properties of fibre are covered by BS EN 14889, fibre of concrete part 2, Polymer Fibres- Defination, specification and conformity.

BENEFITS OF USING FIBRE TUFF

- Significantly improves shrinkage and temperature crack control.
- Safer and lighter to handle than steel
- Reduces plastic settlement crack
- Reduces permeability
- Unlike steel, doesn't stain concrete with rust marks
- Potential for increase in joint spacing
- Increases flexural strength
- Increases impact resistant
- Increases residual strength
- Increases fatigue resistance
- Increases tensile strength
- Increases energy absorption
- Increases ductility
- Increases toughness
- Increases post crack load capacity

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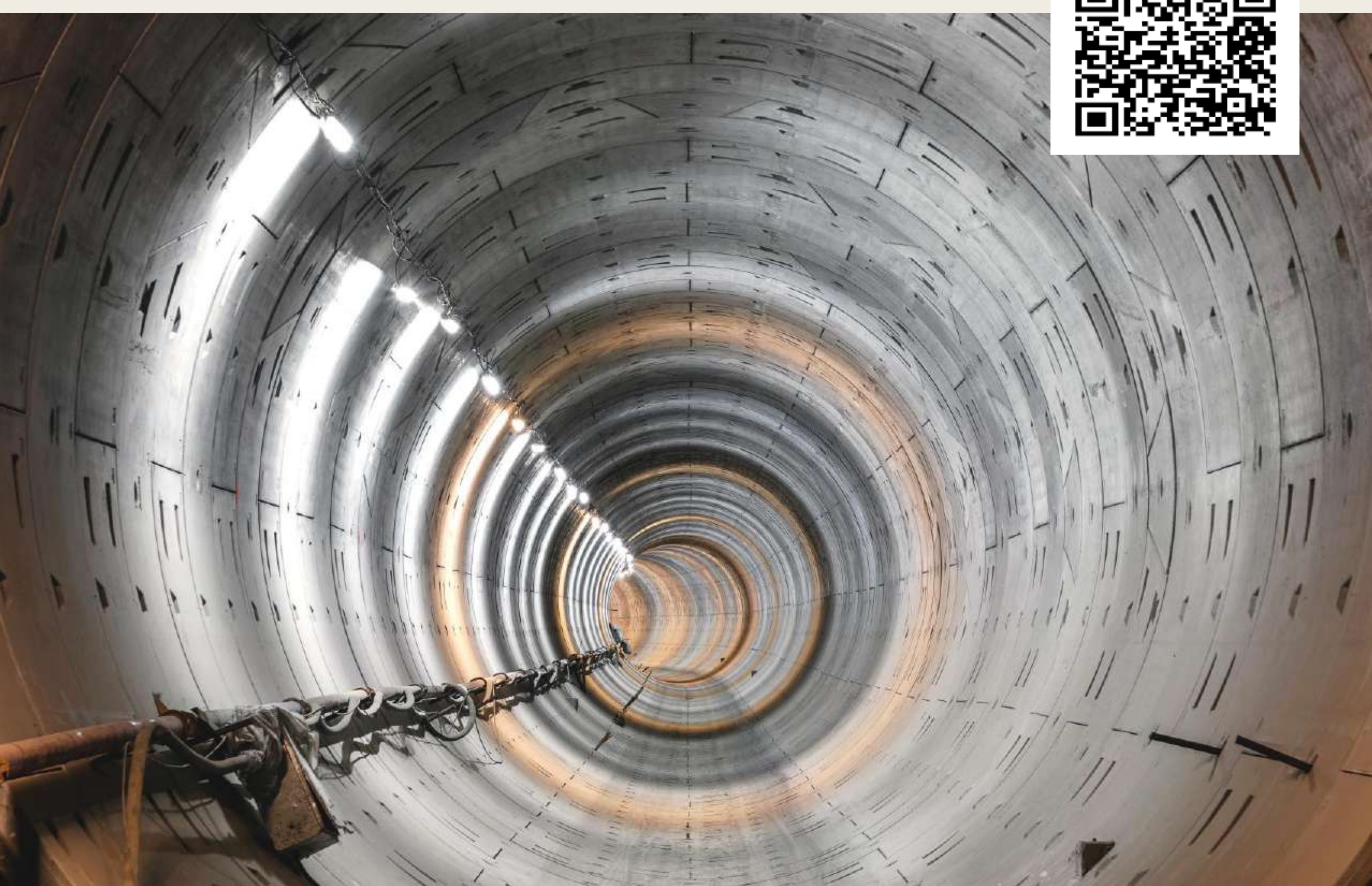

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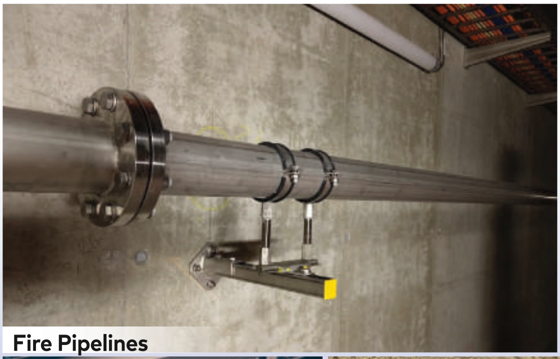
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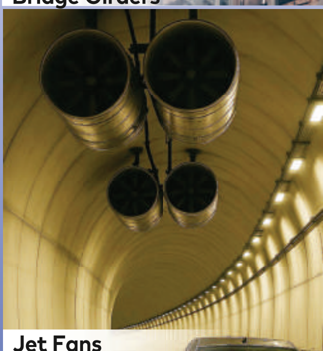
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