

Ground and Lining Response Due to Construction of a Horseshoe-Shaped Tunnel Considering Staged Excavation

Asit Bayen* and Manojit Samanta

Geotechnical Engineering and Geo-Hazard Group, Council of Scientific and Industrial Research-Central Building Research Institute (CSIR – CBRI), Roorkee, Academy of Scientific and Innovative Research (AcSIR), Ghaziabad, 201002, India

Abstract: This study investigates the surface and lining responses that result from constructing a horseshoe-shaped tunnel in mixed ground conditions, considering both uniform and topographic variations in ground profiles through three-dimensional (3D) finite element analysis. The Hardening Soil (HS) model and Mohr-Coulomb (MC) model have been considered for evaluation of the ground and lining responses. Different parameters, including surface settlement, ground volume loss, lining deformations, tunnel convergence, and the zone of influence, are evaluated through numerical investigation and compared with field measurements. The results show that the MC model underestimates the surface settlement, surface volume loss, and tunnel convergence, while the HS model reasonably agrees with the field observations. The results also indicate that the variation of ground topography significantly impacts the ground settlement and tunnel lining responses compared to a uniform ground profile. The zone of influence, width of inflection points, and trough width are estimated from numerical analysis and compared with the available literature. The study shows that the use of an appropriate soil model, material properties, and numerical modeling procedure reasonably estimates the post-construction parameters of the tunnel and ground. Different details of tunnel construction, support systems, numerical modeling procedures, field, and numerical results are reported in the current study.

Keywords: Tunnel, Staged construction, Lining, Numerical modelling, Monitoring, Settlement.

1. INTRODUCTION

Rapid industrialization, population growth, and urbanization of cities have led to a greater demand for transportation facilities. To meet the increased traffic demand, underground tunnels have been constructed across various parts of the country. These tunnels play a significant role in the socio-economic growth of any country. However, the construction of tunnels leads to the development of ground settlement, overturning moment, and transverse shear, which causes additional deformation/distress in the existing structures. The performance of the existing structures is assessed by estimating the ground settlement (lateral, vertical, and distortional) caused by the construction of the tunnels. Hence, proper estimation of these parameters is necessary before constructing the tunnels at the urbanized locations. Proper alignment and ground settlement are usually predicted before a tunnel is constructed at any location. The following section provides a brief review of several case studies related to the settlement of ground, distress of tunnels and surface infrastructures, and probable reasons for distress reported in the literature.

Hoek and Guevera [1] investigated the ground control strategies for the Yacambu-Quibor Tunnel in Venezuela. The tunnel passes through a weak graphitic-phylite rock medium and encounters

squeezing problems due to the considerable height of overburden. To address the squeezing problem, 600 mm thick coatings with sliding joints, reinforced with steel shafts and steel ribs, were installed. Amorosi *et al.* [2] performed a two-dimensional finite element analysis to investigate the deformation and damage mechanisms induced by shallow tunneling on masonry structures. The study aims to provide insight into damage patterns of ancient masonry structures interacting with underground constructions through numerical investigation. The results show that the settlement troughs agree with the empirical Gaussian predictions at different volume losses under free-field conditions. Oke J *et al.* [3] illustrated the application of advanced numerical modeling tools in analyzing design parameters for utilizing forepole elements in tunneling through complex ground conditions. The complexity of ground-support interaction was analyzed using two-dimensional (2D) and three-dimensional (3D) finite element analysis, along with a homogeneous reinforced region. Additionally, the study further demonstrated the sequential optimization of two design parameters - spacing and overlap - through numerical tunnel modeling. Barla Giovanni [4] illustrated the cutting-edge in full-face tunneling in challenging conditions for the Saint-Martin-La-Porte Lyon-Turin Base Tunnel, which encountered severely squeezing conditions during excavation in the Carboniferous formation. An innovative tunnel excavation and construction method was established, combining full-face excavation with face reinforcement using fiberglass elements and a yield-control support system. Bilotta *et al.* [5] assessed the distress of the Chiesa di Santa Maria Della Vittoria historic church due to the

*Address correspondence to this author at the Geotechnical Engineering Group, Council of Scientific and Industrial Research-Central Building Research Institute (CSIR – CBRI), Roorkee, Academy of Scientific and Innovative Research (AcSIR), Ghaziabad, 201002, India; E-mail: asit.bayen@rmit.edu.au

construction of a tunnel beneath a corner of the church through a numerical approach. They considered several factors, such as the position of the tunnel, the stiffness, and the weight of the building, for the estimation of ground movements. Results show that the shear stress induced by the weight of the church can reduce the soil stiffness and increase the induced ground movements. Paternesi *et al.* [6] investigated through numerical investigation the surface settlement due to the construction of twin shallow tunnels in Southern Italy. Several factors influencing the numerical simulations of staged tunneling were identified in this study. Das *et al.* [7] examined the stability of shallow-depth Kainchi-Nerchowck twin tunnels in Himachal Pradesh, India, considering the topography variation of the ground. The maximum ground deformations are found to be 0.99 mm and 3.14 mm. They observed the occurrence of differential settlement in an equidimensional noncircular tunnel due to undulating topography variation of ground. Ziaei and Ahangari [8] investigated the effect of topography on the stability of the shallow diversion and conveyance tunnels of the Safa Dam, which is located in Kerman province, about 30 km northeast of Baft City, Iran. The back analysis approach was employed to determine the precise geomechanical parameters of the rock mass. A numerical investigation was conducted to assess the significance of topographic ground responses induced by shallow tunneling and compared with existing literature. It was suggested that ignoring the actual topography of the ground amplifies the error function obtained from monitoring and numerical results by 1.5 to 10 times, highlighting the importance of considering the actual topography. Aygar [9] evaluated the performance of the new Austrian tunneling method (NATM) used to construct the tunnels. A case study on the Bolu tunnel in Turkey has been considered to evaluate the efficacy of this technique. The findings indicate that using the NATM method is not always an appropriate or cost-effective approach. The probable causes of failure and the precautions to be adopted at the site are also discussed. Aygar and Gokceoglu [10] examined the problems encountered during the excavation of the T4 tunnel (Izmir high-speed shallow railway tunnel running between the Afyonkarahisar-Banaz section) through weak ground by employing analytical and 3D numerical analysis. The stability of the tunnel face was enhanced through strengthening measures such as rock bolting and grouting. Vitali *et al.* [11] analyzed the effects of constructing the São Paulo metro tunnel in Brazil, which passes through residual red porous clay, using the 3D finite element method. The study assessed factors such as partial face excavation, unsupported span length, support stiffness, and the implementation of a pipe roof umbrella. The findings revealed that

partial face excavation significantly minimized ground surface settlement in front of the tunnel face. Additionally, the pipe roof umbrella proved to be the most effective method for mitigating ground deformation around the tunnel. Nong *et al.* [12] examined the issue of excessive ground settlement in water-rich strata in Guangzhou, China, focusing on granite residual soil and fully weathered granite rock using the finite element method. Various approaches, including closed-form solutions, analytical methods, and coupled and non-coupled numerical analyses, were used to assess the maximum ground surface settlement. The findings revealed that excessive settlement was primarily caused by ground consolidation due to water loss. The study also detailed the back-analysis process and proposed mitigation measures to address excessive settlement, offering valuable guidance for tunnel construction in similar ground conditions in the future. Mao *et al.* [13] examined the displacement and stress distribution in the rock mass, as well as the structural stability of the Linpanshan tunnel lining located in Shaoxing, Zhejiang Province, China, passing through jointed layered strata by numerical simulations and field observations. A series of parametric analyses were conducted, considering varying joint inclinations and spacings to assess their effects on the surrounding rock and tunnel lining. The findings revealed that joint inclination had a more significant impact on the stability of the surrounding rock and tunnel structure than joint spacing. To enhance stability, additional anchors were installed in the weak zones of the tunnel lining. Hu *et al.* [14] investigated pavement settlement and deformation during the construction of the Liuye Avenue West Extension tunnel using the pipe jacking method beneath the Changzhang Highway. A three-dimensional numerical model, supported by field data, was used to analyze key factors influencing significant settlement and rebound. The results indicated that pavement settlement increases rapidly as the jacking distance nears 10 m, then stabilizes into a more consistent growth trend.

Conventional hazard assessments of tunnels and nearby infrastructure commonly assume a relatively uniform ground surface, often overlooking the effects of topographic irregularities on tunnelling-induced ground movements and structural responses. However, changes in ground elevation, overburden thickness, and subsurface geological conditions can alter stress redistribution and deformation patterns during tunnel excavation. To address this limitation, the present study investigates the effects of varying ground topography on ground surface settlement and tunnel lining deformation, with particular emphasis on their implications for tunnel performance under complex

geological conditions. The investigation considers key influencing parameters, including overburden thickness, the mechanical characteristics of the surrounding soil and rock strata, and tunnel depth, to evaluate their respective contributions to the observed deformation behavior. The numerical predictions are assessed against field monitoring measurements and established empirical methods to examine the model's capability to reproduce the observed ground and lining responses. The comparative assessment provides a basis for understanding the applicability and limitations of the numerical approach under the investigated conditions. The findings contribute to a better understanding of topography-dependent tunnelling behavior and can support more informed assessments of tunnel stability, ground movement, and potential impacts on surrounding infrastructure in areas with ground topography.

2. DESCRIPTION OF SITE

2.1. General

The site is located at the ridge of Chamba town, Uttarakhand, India. The Chamba tunnel, constructed in

Tehri District, Uttarakhand, India, is a part of the road network developed between Rishikesh-Dharasu (NH96). The tunnel is located at an elevation of about 1354 m above mean sea level. Figure 1 shows the location of the Chamba tunnel on the geological map of India.

2.2. Geological and Geotechnical Characteristics

This section presents the quantitative description of the physical, mechanical, and engineering properties of the existing strata obtained through borehole investigation and field measurement. The investigation of the subsurface includes detailed geological and geotechnical investigations. Figures 2 and 3 show the layout plan and the cross-sectional profile of overburden material along the tunnel alignment. The distance between the south (designated as 0 m) and the north portal (designated as 440 m) is 440 m. The rock mass rating (RMR) and rock mass quality (RMQ or Q) along the tunnel length have been investigated. The RMR values are found to be in the range of 18 -19 (rock class V), 33 -38 (rock class IV), 18-19 (rock class V), 33-38 (rock class IV), 42-45 (rock class III), and 33-38 (rock class IV) from 15 to 20 m, 20 to 75 m, 75

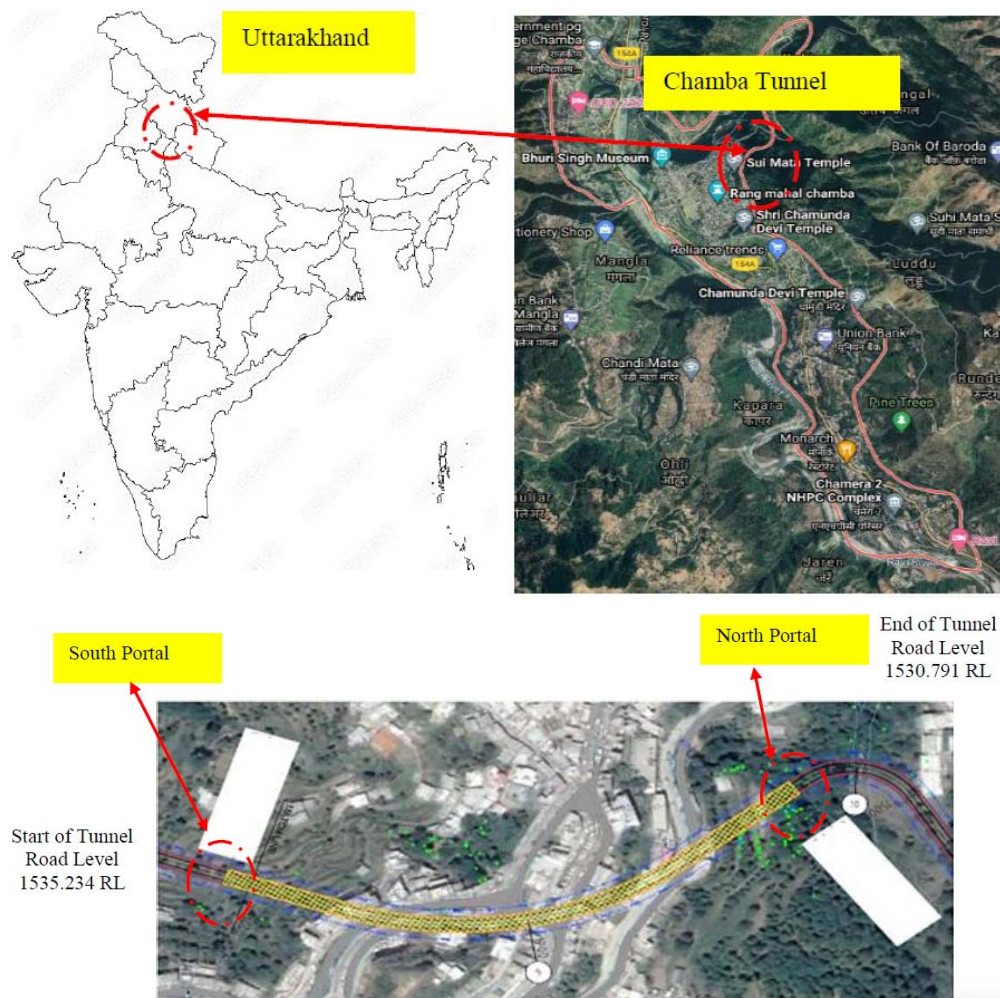


Figure 1: Location of Chamba tunnel

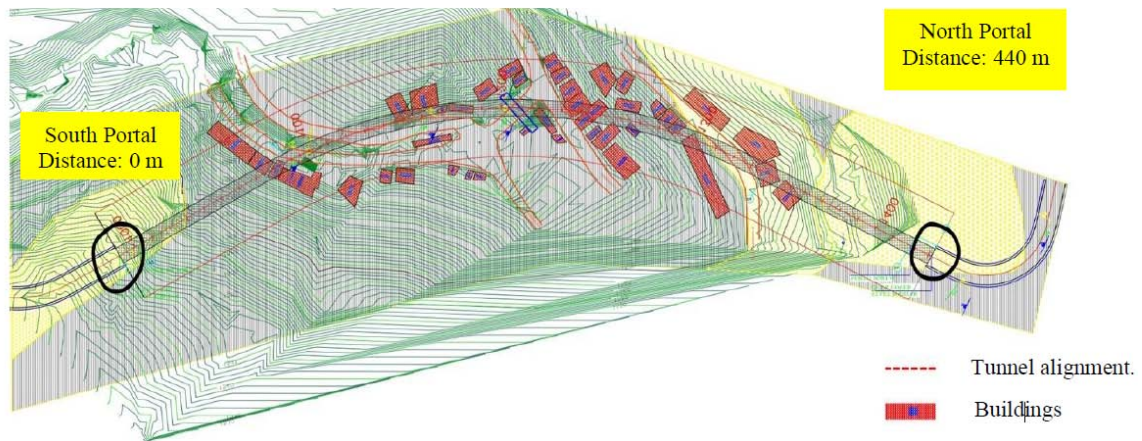


Figure 2: Layout plan of Chamba tunnel.

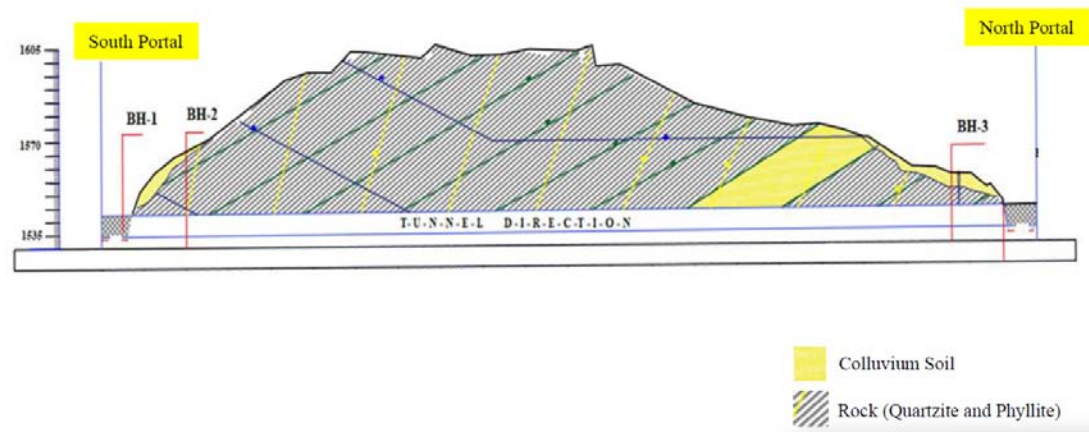


Figure 3: Cross-sectional profile of overburden materials.

-95 m, 95 -255 m, 255 -305 m and 305 to 425 m respectively. The Q values are observed to be in the range of 0.04, 0.04 - 0.132, 0.047-0.132, 0.047 - 0.056, 1.0 – 1.2, and 0.1 – 0.41 for 15 to 20 m, 20 to 75 m, 75 -95 m, 95 -255 m, 255 -305 m, and 305 to 425 m, respectively. The geotechnical investigation work includes drilling three boreholes up to 12.50 m, 38.50 m, and 40.00 m depth from the south portal of the tunnel at 10 m, 40 m, and 405 m to establish the subsoil profile along the tunnel length, respectively. The subsurface medium at the south and north portals mainly comprises colluvium soil, pyroclastic bedrock strata (pozzolana, pumices, tuff), and quartzite. The borehole drilling and geological mapping results reveal two geological mediums of varying depth for the entire stretch. At the south portal, the top 8.20 – 10.50 m is a colluvium deposit, followed by a layer of rock consisting of quartzite and phyllite deposits from the south portal entrance point (0 m) to 40 m. However, at the north portal, the top colluvium layer extends up to a depth of 13.50 m, followed by a layer of rock at a depth of 26.50 m from 40 m to 340 m. The top colluvium soil is very heterogeneous, and the presence of different fractions of particle size in various percentages is observed. The

core recovery is observed to be fair to the poor. The rock quality designation (RQD) is found to be extremely poor to nil. The results obtained also reveal that the groundwater table is at a great depth and does not interfere with the construction of the tunnel. Table 1 shows details of boreholes and the depth of the various strata.

2.3. Empirical and Analytical Approaches for Surface Settlement

The construction of tunnels through the earth medium results in surface and subsurface settlement. Prediction of possible damage and hazards is crucial to ensure the safety of nearby structures. Generally, the volume losses caused by tunnel construction led to surface settlements. The magnitude of settlements is primarily a function of volume loss and ground conditions. Martos [15] first reported surface settlement trough through a Gaussian distribution curve. Field observations, numerical and experimental investigation confirm the nature of the surface settlement, and the magnitude as proposed by Mair [16] and Peck [17] are given in Eq. 1 and 2, respectively.

Table 1: Details of Borehole

Br. No. (BH)	Position	Distance from south portal (m)	Depth of Bore Hole (m)	Depth of Colluvium (m)	Depth of Bedrock (m)	Compositions
1	South Portal	10	12.5	10.5	2.0	Colluvium, Phyllites and Quartzite
2		40	38.5	8.2	30.3	
3	North Portal	405	40.0	13.5	26.5	

$$S_{max} = 0.313V_L (\%) \frac{D^2}{i} \quad (1)$$

$$S = S_{max} e^{-\frac{x^2}{2i^2}} \quad (2)$$

Where S is the surface settlement in the vertical direction at a distance of x , S_{max} is the maximum vertical surface settlement at the tunnel centerline, x is the lateral distance from the tunnel centerline, and ' i ' is the horizontal distance from the tunnel centerline to the point of inflection of the surface settlement. The parameter ' i ' denotes the distance between the inflection point of the Gaussian curve and the tunnel axis, and it is equal to $i = kZ_0$ [18]. Where k (coefficient of earth pressure) is a parameter depending on the type of ground. k varies from 0.2 - 0.45 for sands and gravels, from 0.4 - 0.6 for stiff clayey soils, and from 0.6 - 0.75 for soft clays [19]. Z_0 is the depth of the tunnel's central line. In the study, the value of the coefficient of earth pressure (k_0) is in the range of 0.455 - 0.758. The detailed graphical explanation of the surface settlement as per the Gaussian curve is presented in Figure 4. The total volume of surface settlements equals the volume loss of the tunnel, as calculated by integrating the previous equation, as shown in equation 3.

$$V_L = \int_{-\infty}^{\infty} S_{max} \exp\left(-\frac{x^2}{2i^2}\right) dx, V_L = \sqrt{2\pi}iS_{max} \quad (3)$$

Different analytical methods are also used to predict the surface and subsurface settlement due to tunneling. Sagaseta [20] proposed a closed-form solution to investigate the strain field (surface ground loss) in an isotropic and homogeneous incompressible soil due to tunneling. Following Sagaseta's [20] solution, Verruijt and Booker [21] presented an analytical solution for vertical surface settlement (U_z) due to tunneling inhomogeneous elastic half-spaces. A comparison with the current investigation is not performed using this analytical formula due to the absence of various parameters for analytical approaches.

3. FIELD OBSERVATIONS

The tunneling construction results in ground movements, causing surface settlement and distress to the existing infrastructures adjacent to the tunnel locations. The observed distress appeared during the tunnel construction and was active almost 6-9 months after the completion of the tunnel. A detailed field investigation has been undertaken to characterize the

distress on the ground. Figure 5 shows the ground distress in the form of tension cracks due to the construction of the tunnel.

3.1. Deformation Monitoring

The surface settlement is monitored by marking monitoring points in the plinth level of a building at locations close to the south portal at different offset distances. The settlement of these points is monitored through total station against the previously established permanent benchmark (BM). The surface settlement and lining deformations are measured till there is no change observed with time in magnitude. Figure 6a shows the different positions of the monitoring points marked as S_1 , S_2 , S_3 , and S_4 . It is to be noted that the deformation measured in the present study does not represent the greenfield conditions. The recorded settlement at the plinth level of the building is influenced by the presence of the building and its characteristics. In the present study, the settlement measured at the plinth level of the building has been compared with the Greenfield settlement obtained from numerical analysis.

The deformation of the tunnel lining in a section at different points has been monitored and recorded at different chainage points at regular time intervals along the tunnel length. Figure 6b and 6c depicts the tunnel lining monitoring operation system in detail. Figure 6b shows the location of different points on the lining for monitoring the post-construction deformation. Figure 6c shows the position of monitoring points in the initial and deformed positions for a typical section. Point P_1 represents the initial position of the tunnel crown, and P_2 and P_3 indicate the initial left and right initial position of the tunnel lining at 1.5 m below the crown, respectively. P_4 and P_5 represent the left and right monitoring points at the lining bottom, respectively. The subsequent deformed position of the corresponding point is denoted as P_1' , P_2' , P_3' , P_4' , and P_5' , respectively. The deformation of tunnel lining in the vertical direction is represented by U_z .

4. DESCRIPTION OF TUNNEL

The Chamba tunnel is being constructed within the scope of the Char Dham roadway project. The tunnel is horseshoe-shaped and 440 m long, with a portal length

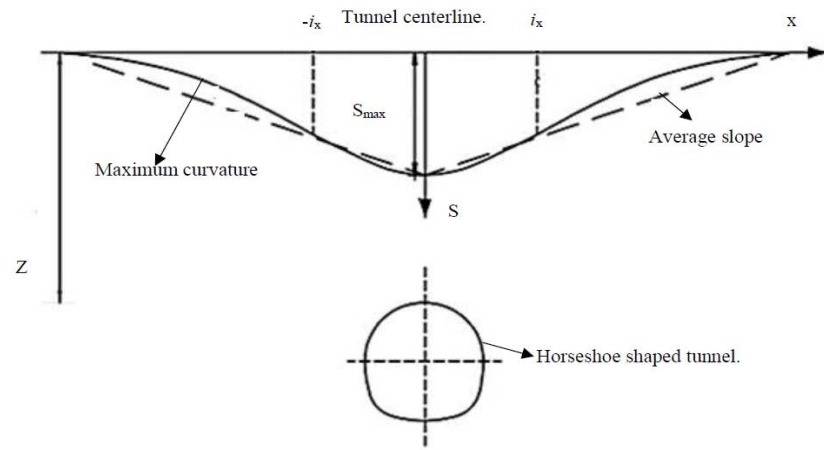


Figure 4: Typical surface settlement profile due to tunneling.



Figure 5: Ground distress due to tunneling.

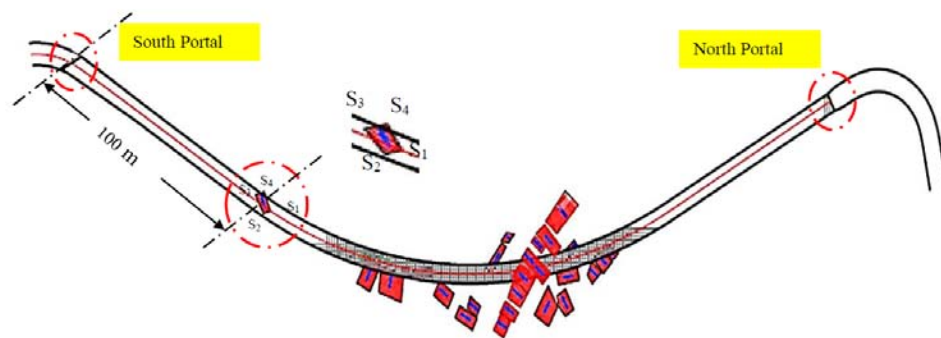


Figure 6a: Surface settlement monitoring points.

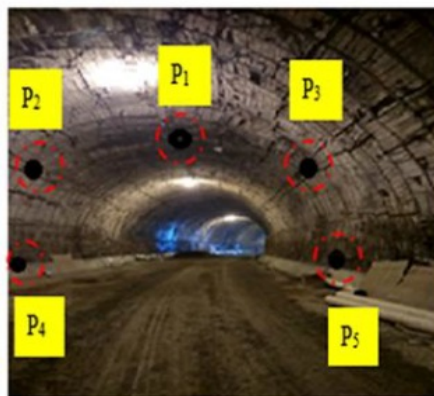
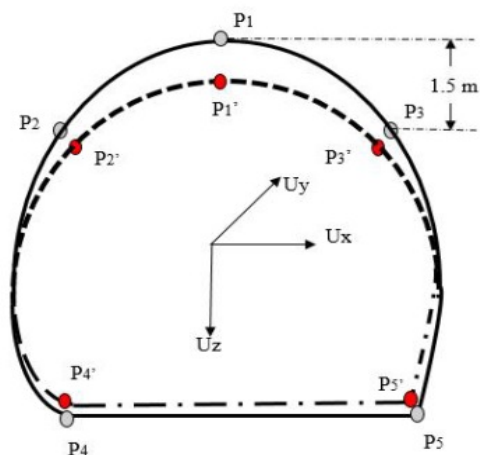


Figure 6b and 6c: Tunnel lining deformation monitoring points.

of 30 m. The diameter and height of the tunnel are 12 m and 8.78 m, respectively. The minimum and maximum overburden thickness between the two portals are 18 m ($C/D = 1.5$) and 60 m ($C/D = 5$). An inverted slope of $\pm 2\%$ has been provided at the bottom of the tunnel in both directions to provide proper drainage facilities. An angle of 4° to 6° has been provided along the length of the tunnel to prevent the sharp lighting transition effect. The primary and final lining of the tunnel is made of reinforced cement concrete (RCC). Figure 7a shows the cross-sectional view of the tunnel lining support system.

5. CONSTRUCTION STAGES AND SUPPORT SYSTEMS

The construction of the Chamba tunnel was completed in 4 years using the staged excavation method. The Chamba tunnel has been constructed in stages using a mechanical excavator following the methodology of the New Austrian Tunnelling Method (NATM). The length of each excavation stage is 2 m. Different support systems consisting of primary lining, final lining, rock bolts, and composite lattice girder (if required) are provided to ensure the stability of the tunnel in its service life. Depending on the quality of the rock, three types of support systems have been used for the entire stretch. For rock class III (RMR value of 42 - 45) and IV (RMR value of 33 - 38), primary lining, final lining, and rock bolting have been used as a support system. Similarly, for rock class V (RMR value of 18 - 19), primary lining, final lining, rock bolts, and composite lattice girder have been used as tunnel support systems. The primary lining is made of M35-grade concrete having a thickness of 100 mm for rock class III. The thickness of the primary lining is 150 mm and 200 mm for rock classes IV and V, respectively. The final lining is made of M40-grade concrete and is 250 mm thick throughout the length of the tunnel. A two-layered 4 mm-thick geomembrane (t_{wpm}) has been placed around the periphery of the tunnel. Figure 7a shows the typical cross-sectional view of the tunnel lining and support system. In the case of rock class III, rock bolts of 5 m length, 25 mm diameter, and 2.5 m c/c spacing in the width and longitudinal direction of the tunnel are provided. For rock classes IV and V, the spacing of the rock anchors is reduced to 2 m in both directions. The minimum design capacity of the rock bolt is 16 tons. The confirmatory rock bolt pull-out test is conducted at the site to verify the actual capacity of the rock bolt system [22]. Figure 7b shows the typical pullout load vs. displacement behavior of rock bolts conducted at the site. The results show that the design pullout capacity is achieved in the field. For rock class V, the lattice girders are comprised of two 25 mm dia. steel bars at the bottom and one 32 mm dia. steel bar at the crown, which are connected by 12 mm dia. steel bars @ 650

mm c/c spacing. Figure 7c shows the different stages of tunnel construction.

6. NUMERICAL MODELLING

This section describes the different details of the 3D finite element model developed to investigate the different post-construction parameters, such as surface settlement, lining deformation, and volume loss due to tunneling. Finite element modeling offers advantages for evaluating the complex tunnel-soil interaction. The finite element-based Plaxis 3D has been used in the present investigation. The numerical modeling procedure, soil layering, material modeling, parameters, and loading conditions are presented in detail in the following sections.

6.1. Model Geometry and Boundary Conditions

A 3D finite element-based numerical model has been developed in the present study. Due to symmetry, only half of the geometry is considered in the present study. The model consists of the soil continuum, geometry of the tunnel, contact interface, and boundary conditions. The numerical domain has been fixed using trial calculations, during which the boundaries are increased till the stresses and deformations due to the construction of the tunnel are insignificantly influenced by further increase in the domain. The numerical domain of size 250 m (L) $\times$ 90 m (W) $\times$ 60 m (H) is used for the tunnel cover depth up to 29 m in the case of a uniform ground profile. The model dimensions are 250 m (L) $\times$ 90 m (W) $\times$ 100 m (H) for a tunnel cover depth of 60 m with a uniform ground profile. The numerical domain of size 410 m (L) $\times$ 140 m (W) $\times$ 50 - 100 m (H) is considered in the case of topographic variation of the ground profile, with the tunnel cover varying from 18 - 60 m. The depth of tunnel cover is considered as 18 m ($C/D = 1.5$), 29 m ($C/D = 2.42$), and 60 m ($C/D = 5$) for uniform ground profile conditions. The tunnel cover with topographic variation is taken as 18 m ($C/D = 1.5$) for both ends of the tunnel portal and 60 m ($C/D = 5$) for the middle portion. To avoid the modeling complexity, the tunnel is considered straight along its length. The tunnel is assumed to have a perfectly horizontal alignment, but in reality, it typically inclines to maintain the elevation between the two portals. Figure 8a and 8b show the numerical domain of uniform and topographic variation of ground profile layering in the model. The depth of the water table is considered at the bottom level of the domain. The horizontal boundary of the soil domain is extended to 7.5 times the tunnel diameter to eliminate the possible boundary effect [23]. The bottom boundary is fixed at a depth within 2-3 times the tunnel diameter from the inverted level to simulate the floating tunnel conditions [6]. The length of the tunnel is chosen in such a way that

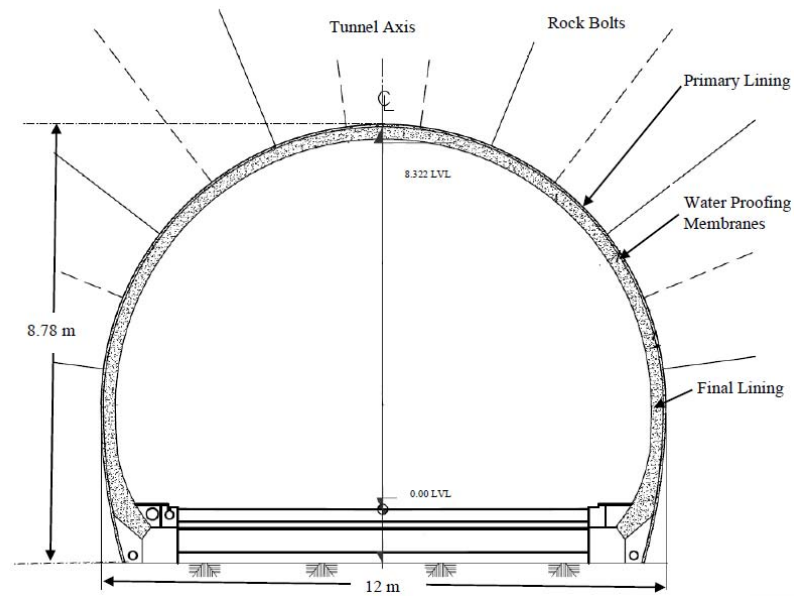


Figure 7a: Cross-sectional view with tunnel lining support system.

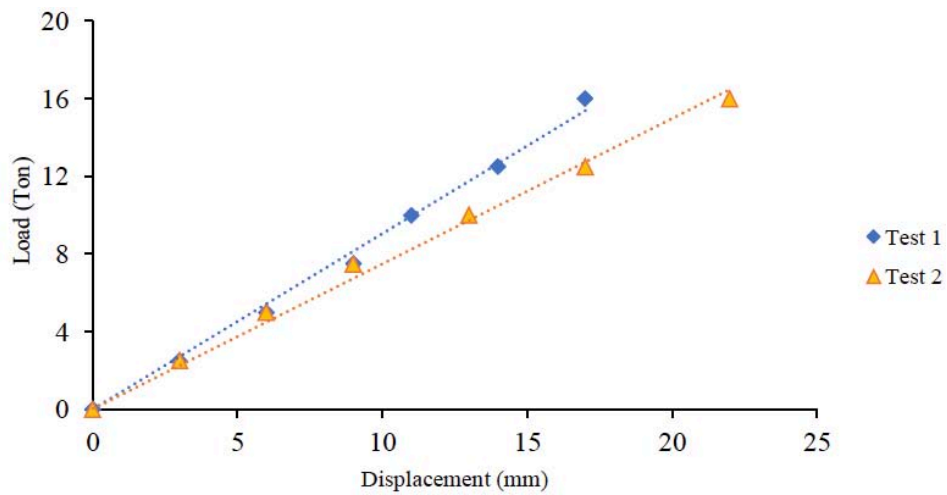


Figure 7b: Pullout test of rock bolt system.

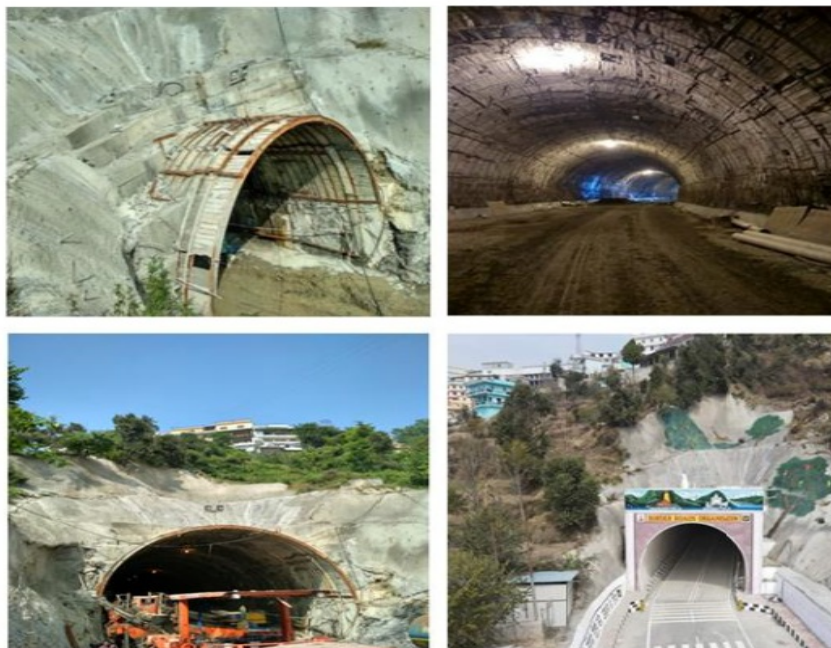


Figure 7c: Staged construction of Chamba tunnel.

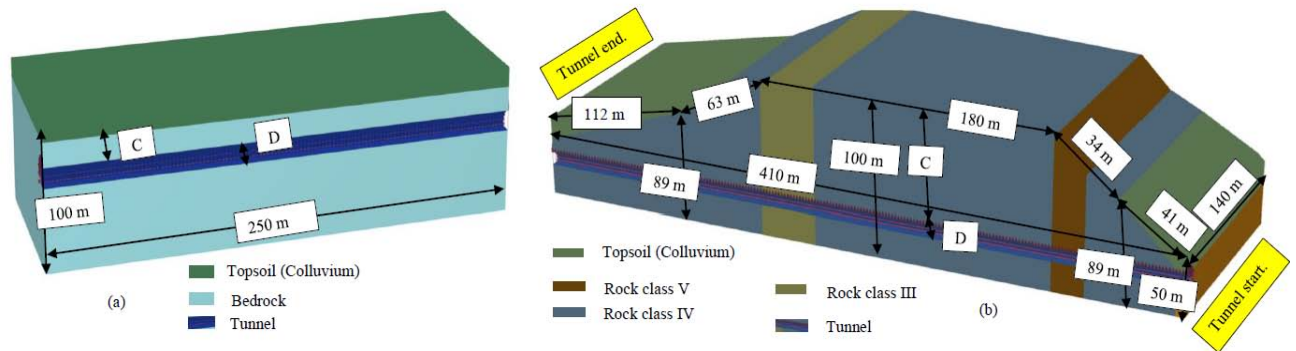


Figure 8: Numerical domain (a) uniform ground profile and (b) topographic variation of ground profile

excavation of full length creates a steady state settlement condition at the mid-length [6, 23- 24]. Steady-state conditions refer to the settlement at the measured section that does not increase with further progress in the tunnel excavation. In this present study, steady-state settlement is estimated for a range of tunnel cover depth obtained in the field investigation, considering it is to be maximum and on the conservative side. The vertical boundaries of the numerical model have been fixed in perpendicular directions and allowed to move in-plane only. The bottom boundaries are fixed in all directions, and the top surface is free to move in all directions. The tunnel face is free to rotate and move in any direction.

6.2. Material Modelling and Mesh Generation

The soil and rock behavior are simulated using the Mohr-Coulomb (MC) and Hardening Soil (HS) models, and drained analysis has been considered [25]. The geological survey conducted at the site revealed that the subsurface strata primarily comprise Class III, IV, and V rock formations. The uppermost layers predominantly consist of soft colluvium soil, characterized by loose compaction and a minimal degree of susceptibility to deformation when subjected to loading. Due to its unconsolidated nature, this soil exhibits significant strain-dependent behavior, necessitating advanced modeling techniques to accurately simulate its response under varying stress conditions. To effectively capture the complex mechanical behavior of the ground, both the Mohr-Coulomb (MC) and Hardening Soil (HS) constitutive models were employed. The MC model provides a simplified yet widely accepted framework for simulating soil failure based on shear strength parameters, while the HS model offers enhanced capabilities for modeling stress-dependent stiffness and plastic deformation, making it particularly suitable for representing the behavior of soft soils and intermediate geomaterials underloading. These modeling approaches have been validated through prior studies conducted under similar geotechnical

conditions, notably by Shin *et al.* [26] and Aygar and Gokceoglu [10,27], who demonstrated their applicability and reliability in simulating subsurface responses. Furthermore, no bedding plane or joint was observed in the field geological observations. Therefore, a continuum modeling approach is adopted here. The HS model is an advanced soil constitutive model to simulate the behavior of various types of geomaterials. The stress dependency of soil stiffness is a fundamental component of the HS model. The HS model has several advantages over the MC model, including the ability to manage stress level dependence and the use of a hyperbolic stress-strain curve rather than a bilinear curve. In general, this model assumes two types of yield surface plasticity, including soil dilatancy and a yield cap. Hardening is assumed to be isotropic in this model, depending on both the plastic shear strain and the plastic volumetric strain. Elastic modulus (E , E_{ur}^{ref} , E_{oed}^{ref} , E_{50}^{ref}) for soil/rock elasticity and plasticity, Poisson's ratio, cohesion, friction angle, and dilatancy angle are the input parameters used in this model. A 10-noded, second-order tetrahedral element with three translational degrees of freedom in three perpendicular directions is used to discretize the soil continuum [28]. The engineering properties of bedrock are estimated from the relationship $E_{mass} = 10^{(RMR-10)/40}$ GPa for $RMR < 50$ [29,30]. Where, E_{mass} and RMR are the deformation modulus of rock mass and the rock mass rating value, respectively. The possible range of cohesion, angle of internal friction, and elastic modulus for the colluvium soil are obtained from Bowles [31]. In the present analysis, two different types of top colluvium soil (A and B), having different engineering properties, are used to consider the possible variation in the material properties in the field. In the present study, two types of soil models, Mohr-Coulomb (MC) and Hardening Soil (HS) are used. The relevant properties of the soil and bedrock, i.e., E_{ur}^{ref} , E_{oed}^{ref} , E_{50}^{ref} are taken as $E_{oed}^{ref} = E(1-\mu)/((1-2\mu)(1+\mu))$ (kPa), $E_{50}^{ref} = E_{oed}^{ref}$ (kPa), and $E_{ur}^{ref} = 3E_{50}^{ref} = 3E_{oed}^{ref}$ (kPa), respectively. Where μ is the poisson's ratio, E = Elastic modulus (kPa), E_{ode}^{ref} = Odometer

loading/reloading stiffness (kPa), E_{50}^{ref} = Triaxial loading stiffness (kPa), and E_{ur}^{ref} = Triaxial unloading stiffness (kPa). The properties of the primary lining, final lining, rock bolts, and lattice girder are obtained from their respective grade and field tests. The lattice girder for rock class V has been modeled with primary lining by assigning the equivalent material properties of the composite materials (primary lining and lattice girder) estimated through the relationship between respective volume and elastic modulus. Tables 2 to 7 show the physical and mechanical properties of topsoil and bedrock, primary and final lining, rock bolt, and lattice girder. To accommodate the possible variation of material properties, two different cohesion values representing the maximum and minimum range in each rock class have been considered in the present analysis. Figure 9 shows the primary lining, rock bolting, interface, and final lining in the numerical model. The tunnel structure (primary and final lining) is considered to be a nonporous and linear elastic material. The primary and final lining is modeled using plate elements composed of 3-nodes with six degrees of freedom at each node. The rock bolt is modeled as an embedded beam element having an axial capacity of 32 kN/m. The soil-tunnel interface is simulated through quadrilateral interface elements having 16 nodes. The eight pairs of nodes of the interface elements are compatible with the quadrilateral side of the soil and tunnel elements in the vertical direction. Each node has three translational degrees of freedom, allowing simulation of slipping and gapping between the soil and the tunnel. This interface element has no thickness, but a virtual thickness is used to calculate the stiffness properties of the interface. The parameter R_{inter} (considered as 1) determines the stiffness and strength of this interface. The support system in a particular excavation stage has been activated in the following sequence: initial condition, primary lining and rock bolting, followed by the final lining.

A mesh convergence study was conducted before finalizing the mesh size. Since negligible differences in surface settlement were observed between the fine and very fine meshes, the fine mesh was adopted to ensure adequate accuracy while maintaining computational efficiency. The mesh convergence study shows that increasing the number of elements beyond 97,047 and 1,57,870 nodes has no significant effect on the magnitude of stresses and displacement for uniform ground profiles. For the ground profile with topographic variation, the number of elements and nodes are 1,53,983 and 2,48,224, respectively. In the current study, a relatively finer mesh (average size of 0.035 m) is used around the tunnel, and a larger mesh size (0.075 m) is used towards the remote boundaries of the numerical domain.

The staged construction for the initial stress conditions is generated within the medium through K_0 and gravity loading conditions for uniform ground and ground profiles having topographic variation, respectively. Subsequently, phased excavation (each 2 m) of the tunnel has been simulated. The total number of stages for the present study consists of 125 and 205 phases for uniform and topographic variation of ground profile, respectively. Every single phase includes removing the soil/rock of a specified length, installing the primary lining (shotcrete), rock bolts, and lattice girder (if required), and installing the final lining after the completion of the entire tunnel length. In the present study, stress relaxation in the excavated medium due to the delay in the application of the support system is not considered, and primary support (consisting of shotcrete, rock bolt, and lattice girder if required) is applied simultaneously after the excavation. This assumption is in agreement with NATM construction as the pressure from the surrounding medium is taken by the primary support/shotcrete, and a similar methodology is followed by various previous researchers [26, 32-37]. The considered colluvium depth in the present analysis covers the maximum and minimum range observed in the field. Table 8a and 8b shows the depth of colluvium soil, rock strata, and tunnel crown position for the section and C/D ratios considered in the present analysis. It is to be noted here that the ground surface in the longitudinal direction of the tunnel is not uniform, and depth increases towards the center. In the present investigation, two types of numerical models have been developed. In the first type of model, the uniform ground profile of different layer thicknesses has been considered. The layer thickness adopted in the present study covers the maximum and minimum range in the field for each rock class. The length of the tunnel in each numerical model is chosen in such a way that excavation of full length creates a steady state settlement condition at the mid-length [6, 23, 24]. Steady-state conditions refer to the settlement at the measured section that does not increase with further progress in tunnel excavation. In this present study, steady-state settlement is estimated for a range of tunnel cover depth obtained in the field investigation, considering it is to be maximum and on the conservative side.

Also, to investigate the effect of topographic variation of the ground on the surface and tunnel response, the numerical model with variable soil layer thickness along the tunnel length has been modeled. In this model, the entire tunnel length has been considered. It is to be noted that topographic variation of the ground surface is incorporated in the model

Table 2: Material Properties of Bedrock

Rock Class	Unit Wt. (kN/m ³)	Cohesion (kPa)	Friction angle, ϕ (°)	Poisson's Ratio (μ)	E (kPa)	E_{oed}^{ref} (kPa)	E_{50}^{ref} (kPa)	E_{ur}^{ref} (kPa)
V	27	50, 90	14	0.23	15.84×10^5	18.37×10^5	18.37×10^5	55.12×10^5
IV		100, 150	20		42.16×10^5	48.88×10^5	48.88×10^5	14.66×10^6
III		200, 250	30		70.79×10^5	82.07×10^5	82.07×10^5	24.62×10^6

Table 3: Material Properties of Colluvium Soil

Topsoil Type	Unit Wt. (kN/m ³)	Cohesion (kPa)	Friction angle, ϕ (°)	Poisson's ratio (μ)	E (kPa)	E_{oed}^{ref} (kPa)	E_{50}^{ref} (kPa)	E_{ur}^{ref} (kPa)
A	19.31	10	33	0.3	15×10^3	20.19×10^3	20.19×10^3	60.57×10^3
B					10×10^3	13.46×10^3	13.46×10^3	40.38×10^3

Table 4: Material Properties of Primary Lining/Shotcrete (M35)

Rock Class	Unit Wt. (kN/m ³)	Thickness (mm)	Material type	Elastic modulus (kPa)	t_{wpm} (mm)	Poisson's Ratio (μ)
III	24	100	Linear Elastic (LE)	29.58×10^6	4	0.15
IV		150				
V		200				

Table 5: Material Properties of Final Lining (M40)

Rock Class	Unit Wt. (kN/m ³)	Thickness (mm)	Material Type	Poisson's Ratio (μ)	Elastic Modulus (kPa)
III	25	250	Linear Elastic	0.15	31.63×10^6
IV					
V					

Table 6: Material Properties of Rock Bolts (Fe500)

Rock class	Spacing of rock bolt along length direction (m)	Spacing of rock bolt along width direction (m)	Length of rock bolt (m)	Elastic modulus (kPa)	Unit Wt. (kN/m ³)	Dia. of rock bolt (mm)
III	2.5	2.5	5	2 x10 ⁸	78	25
IV	2.0	2.0				
V						

Table 7: Material Properties of Lattice Girder

Rock mass	Lattice girder spacing (mm)	Material grade	Elastic modulus (kPa)
III	-	Fe 550	-
IV	-		-
V	650		2×10^8

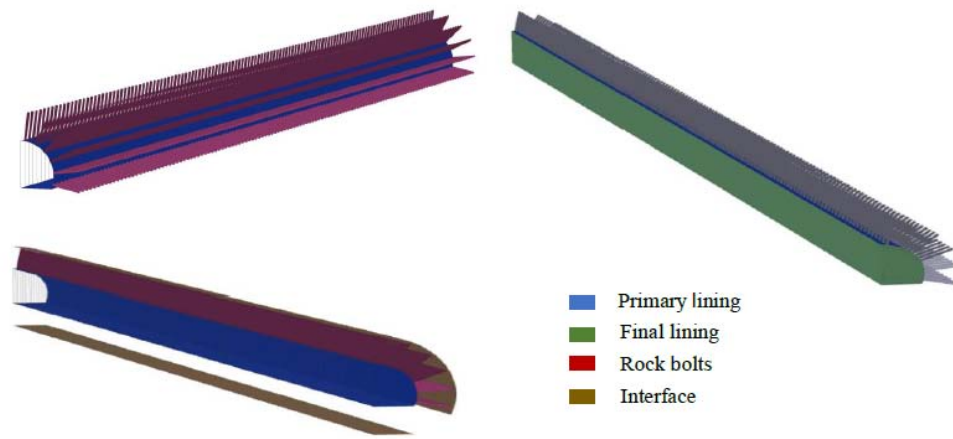


Figure 9: 3D Numerical model of tunnel.

Table 8a: Analysis Schemes

S. No.	Topsoil and rock type	Type of ground profile	Colluvium soil height (m)	Position of tunnel crown from the ground surface (m)	C/D ratio
1	A, Rock Class IV, V	Uniform	14	18	1.50
2			8	29	2.42
3	B, Rock Class IV, V		14	18	1.50
4			8	29	2.42
5	Rock Class III		-	29	2.42
6			-	60	5.00

Table 8b: Analysis Schemes

S. No.	Topsoil and rock type	Type of ground profile	Topsoil height (m)	Position of tunnel crown from the ground surface (m)	C/D ratio
1	A Rock Class III, IV and V	Topographic variation	0 - 14	18 - 60	1.5 - 5
2	B, Rock Class III, IV and V				

through average layer thickness. The microscopic undulation of the ground surface is not considered in the present study. The thickness of the different layers and ground surface topography is taken from the average slope angle as estimated in the field. Das *et al.* [7] investigated the ground subsidence through a two-dimensional numerical analysis for two asymmetrical tunnels. The variation of the topography is considered through the average thickness of each layer in the analysis in the transverse direction only. The stress relaxation method is used to simulate the application of the tunnel support system at the face and wall. The response of the tunnel at the crown, middle, and invert levels is investigated for various support systems. Ziaei and Ahangari [8] investigated the influence of surface topography on the response of tunnel walls and roofs through three-dimensional numerical investigations. The topographic undulation of

the ground significantly influences the tunnel response. The tunnel walls and roof response are observed to vary up to 10 times when the actual surface topography is considered in the analysis. The results show that the symmetric deformation of the tunnel walls and roof is significantly influenced by the consideration of the real topography. The above-mentioned study does not include the zone of influence during staged tunneling due to consideration of the surface topography. The depth of the different layers varies in the longitudinal direction of the tunnel while it is constant in the transverse direction of the tunnel. In the topographic variation of the ground profile, only the HS material model is considered in the numerical model. The numerical results are obtained at the mid-length of the model in the longitudinal direction. The tunnel alignment is considered to be straight in the present analysis for both cases.

7. RESULTS AND DISCUSSIONS

This section provides details of different results obtained from numerical analysis. The surface settlement, zone of influence in the transverse and longitudinal direction, volume losses, lining deformations, etc., are estimated from the numerical analysis and reported here. The results of surface settlement, volume loss, and tunnel lining deformation obtained from numerical analysis are compared with the field observations.

7.1. Surface Settlement

The magnitude and zone of influence of surface settlement must be assessed properly to predict the potential hazard caused by tunneling. The magnitude and influence zone help analyze and organize the safety measurement process during tunnel construction. Figure 10a and 10b shows the development of a transverse surface settlement profile for progressive tunnel construction for topsoil B, rock class V, and tunnel crown position at a depth of 18 m for HS and MC models, respectively, for the uniform ground profile. It is to be noted that the transverse surface settlement is computed at the middle ($x/D = 10.42$) of the longitudinal axis of the tunnel. The development and extent of surface settlement in transverse and longitudinal directions are found to be similar for the MC and HS models. The surface settlement extends in the transverse direction up to a

distance of $x/D = 5$ to 5.5 from the longitudinal axis of the tunnel. Results from the other cover depths of the tunnel (C/D of 2.42 and 5) show that the development of the surface settlement in the transverse direction is significantly influenced by the tunnel cover depth. The zone of influence increases with an increase in the tunnel cover depth (C/D ratio). For a tunnel cover depth of 60 m ($C/D = 5$), the observed influence zone is $7D - 7.5D$. The zone of influence along the transverse direction is found to be $5D - 7.5D$ from the longitudinal axis for the various cover depths considered in the analysis. The zone of influence for the transverse and longitudinal direction of the tunnel has been investigated and reported by various researchers through extensive numerical, experimental, empirical, and field observations. The zone of influence in clay soil in the transverse direction from the tunnel longitudinal axis is observed to be in the range of $3.5D$ to $8.6D$ for a C/D ratio of 2 to 3.5 [32, 38- 41]. For sandy soil, the transverse influence zone is observed to be in the range of $4D$ to $7D$ for a C/D ratio of 1.5 to 7 [42, 43]. The reported influence zone for the mixed ground condition is observed to be in the range of $4D$ to $10D$ for a C/D ratio of 1 to 7 [5, 24, 44]. The estimated influence zone in the present study is observed to be in the range mentioned in the previous study for mixed ground conditions.

Figures 11a and 11b show the longitudinal surface settlement profile for topsoil B, rock class IV, and

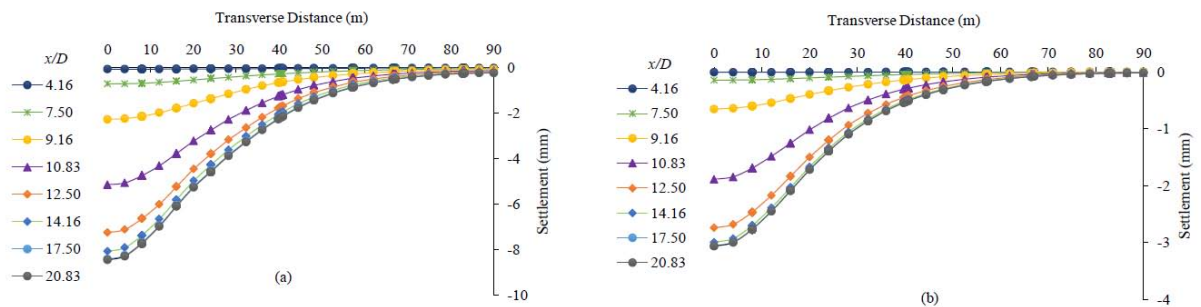


Figure 10: Typical transverse surface settlement of uniform ground profile, topsoil B, rock class V, tunnel crown position at 18 m, $c = 90$ kPa (a) HS model and (b) MC Model.

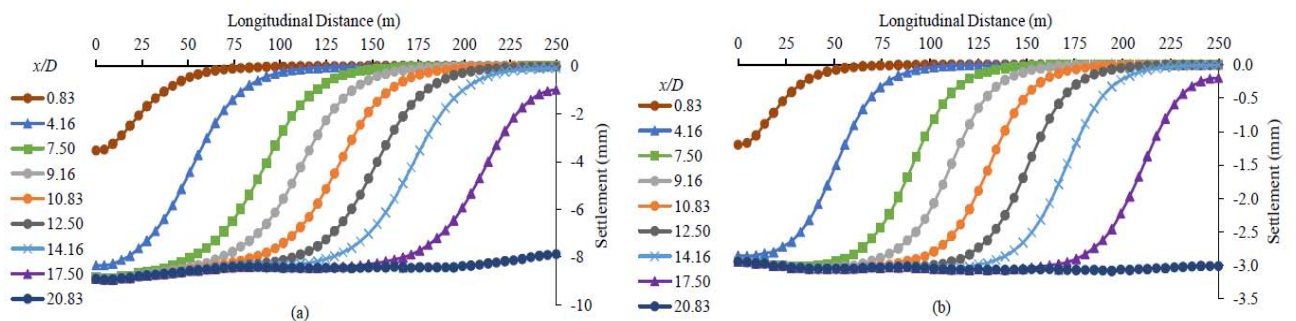


Figure 11: Typical longitudinal surface settlement for uniform ground profile, topsoil B, rock class V, tunnel crown position at 18 m, $c = 90$ kPa (a) HS model and (b) MC model.

tunnel crown position at 18 m for HS and MC models. The settlement at the monitoring section started and reached a maximum for tunnel excavation face at $x/D = 7.50$ and 17.50 , respectively. The zone of influence in the longitudinal direction is observed to be approximately $3.5D$ and $7D$ ahead and beyond the monitoring section. For different tunnel cover depths (i.e., 29 and 60 m, corresponding C/D ratio of 2.42 and 5 respectively), the influence zone is observed to be in the range of $6D$ and $10D$ ahead and beyond the monitoring section, respectively. The influence zone in the longitudinal direction is reported to be in the range of $3D$ to $7D$ for a C/D ratio of 2 to 6 in sandy soil from numerical, experimental, and theoretical investigation [32,41,43,45].

Figures 12 and 13 show the development of the vertical deformation (U_z) contour for topsoil B, rock class IV, crown position at 18 m, $c = 150$ kPa for HS and MC soil model for a C/D of 1.5 and 2.42 respectively in the transverse direction. The contour shows the formation of an arch-shaped zone above the tunnel crown and extends vertically and laterally toward the ground surface. It is observed that the settlement contour does not extend to the lateral boundary of the numerical model. The deformation contour establishes the sufficiency of the model dimension considered in the present analysis. The pattern of the settlement contour is found to be almost similar for both models, though the magnitude of deformation is observed to be more for HS soil. Furthermore, for lower tunnel cover depths ($C/D = 1.5$), the extension of the influence zone

in the lateral direction is observed to be less compared to higher tunnel cover depths ($C/D = 2.42$). The primary settlement zone is observed to be up to a distance of 1-2 times the tunnel diameter above the tunnel crown. For the low cover depth of the tunnel, the formation of a distress zone in the shape of an arch starts from the tunnel crown and reaches the ground surface. The soil above the tunnel mostly moves vertically within this arch, resulting in a smaller surface settlement trough. The extent of the distress zone may not reach the ground for a larger cover depth of the tunnel. Ground movement in this zone propagates vertically and radially, resulting in a wider surface settlement trough. A similar observation has also been made in the previous experimental and numerical analysis [43, 46-48].

Table 9 summarizes the maximum settlement of the surface from numerical, field investigation, and different previous approaches. Figure 14 shows the comparison of surface settlement profiles for different approaches considered in the present and previous studies. It is to be noted that most of the analytical and empirical approaches overpredict the maximum surface settlement (19 - 117 %) for mixed ground conditions considered in the current study. The empirical methods developed by Peck [17], Mair [16], and Attewell [49] show strong agreement with the findings of the present study. Figure 14 shows that the empirical results exhibit a relatively narrow zone of influence curve, whereas the numerical results show a wider zone. This observation aligns with previous studies by Aswathy *et*

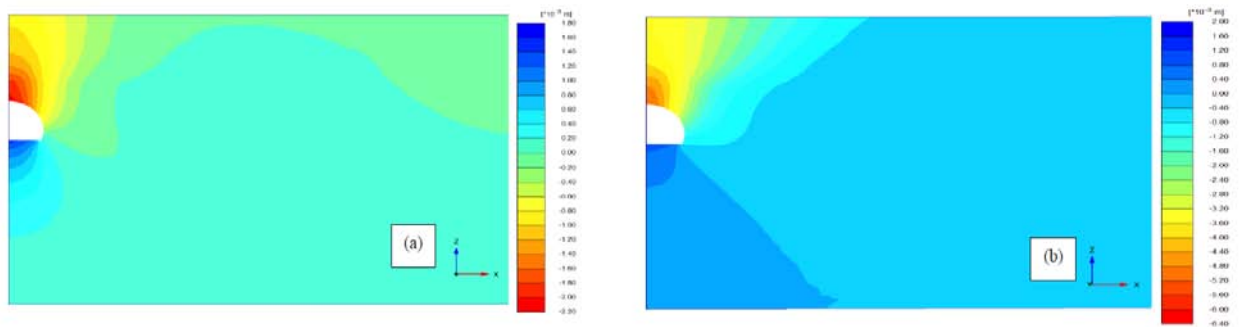


Figure 12: Ground deformation contour for uniform ground profile, topsoil B, rock class IV, crown position at 18 m, $c = 150$ kPa (a) HS model and (b) MC model.

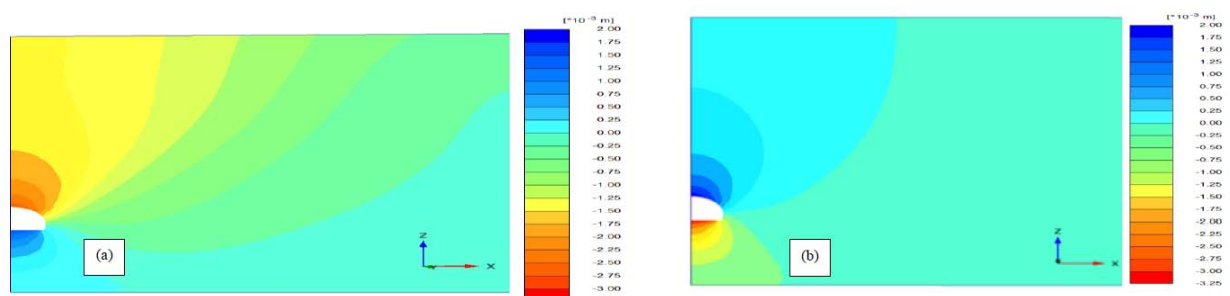
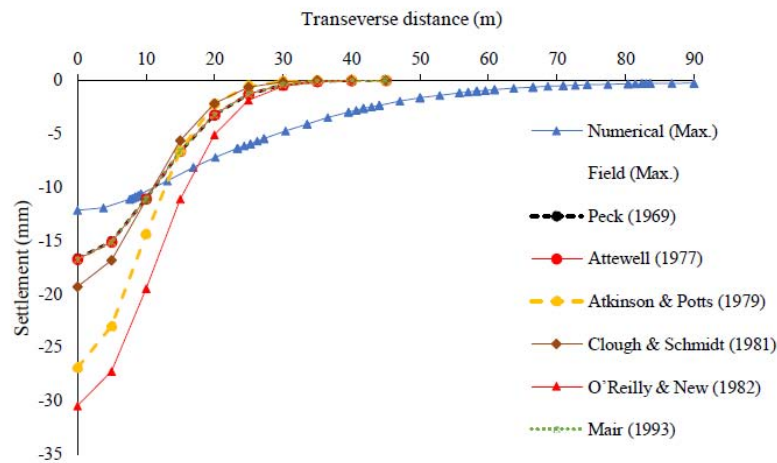


Figure 13: Ground deformation contour for uniform ground profile, topsoil B, rock class IV, crown position at 29 m, $c = 150$ kPa (a) HS model and (b) MC model.

Table 9: Maximum Surface Settlement for Different Approaches

Different methods	Trough width (i) value	Remarks	Max. surface settlement (mm)	
			Uniform ground	Topographic ground
Field observation (present study)	-	-	14	14
Numerical (present study)	-	-	12.16	8.78 - 4.22
Peck (1969)	$i/R = (Z_0/2R)^n : n = 0.8 \text{ to } 1$	Based on field observations	16.69 – 19.33	-
Atkinson and Potts (1979)	$i = 0.25 (Z_0 + R) : \text{For loose sand}$ $i = 0.25 (1.5 Z_0 + 0.5R) : \text{For dense sand and over consolidated clay}$	Based on field observations and model tests	20.67 – 26.92	-
O'Reilly and New (1982)	$i = 0.43 Z_0 + 1.1 : \text{Cohesive soil}$ $i = 0.28 Z_0 - 0.1 : \text{Granular soil}$	Based on field observations of UK tunnels	17.48 – 30.46	-
Mair (1993)	$i = 0.5 Z_0$	Based on field observations worldwide and centrifuge test	16.78	-
Attewell (1977)	$i/R = \alpha (Z_0/2R)^n : \alpha = 1 \text{ and } n = 1$	Based on field observations of UK tunnels	16.78	-
Clough and Schmidt (1981)	$i/R = \alpha (Z_0/2R)^n : \alpha = 1 \text{ and } n = 0.8$	Based on field observations of US tunnels	19.33	-

**Figure 14:** Comparison of surface settlement with different approaches.

al. [50], which similarly highlight the influence of young alluvium soil conditions on tunnel behavior. However, it is important to note that all the empirical methods referenced are primarily based on observations of soft-stiff soils and granular ground conditions. These conditions typically involve soils with varying stiffness and cohesion, which are common in many tunneling projects. In contrast, the subsurface conditions in the current study are more complex, consisting of mixed ground, including a top layer of colluvium soil (which is loosely deposited, often through weathering and erosion) followed by hard rock layers. This unique combination of materials introduces additional challenges in predicting ground behavior during tunneling, as the properties of the colluvium soil and hard rock differ significantly from those of soft and granular soils typically considered in empirical models.

Figures 15a, 15b, and 15c show the development of a transverse surface settlement profile for the ground

surface having topographic variations with progressive tunneling. The transverse surface settlement is computed at three sections i.e., 30 m away from the tunnel starting point (i.e., $x/D = 2.5$), at the middle (205 m away from the tunnel starting end, $x/D = 17.08$) and 30 m before the end of the tunnel (i.e., $x/D = 31.66$) after the completion of entire tunnel length. The magnitude of surface settlement is observed to vary in all three cases. This is due to the presence of different geological mediums along the tunnel length. The presence of rock medium (Class - III, IV and V) at the mid-section having maximum cover distance ($C/D = 5$) without any colluvium material results in no further increase in the surface settlement with progressive tunneling beyond $x/D = 2.5$. The influence zone in the transverse direction is observed to be significantly influenced by the cover depth of the tunnel. For the monitoring section at $x/D = 2.5$ having $C/D = 2.42$, the observed influence zone in the transverse direction is $10.5D$ from the tunnel longitudinal axis. For the monitoring section at $x/D = 17.08$ having $C/D = 5$, the

observed influence zone is 11.66D from the tunnel longitudinal axis. For the monitoring section at $x/D = 31.66$ having $C/D = 2.42$, the observed influence zone is 9.5D from the tunnel longitudinal axis. The observed influence zone in the transverse direction is in the range of 9.5D to 11.66D for the different cover depths of the tunnel. The zone of influence along the transverse distance of the tunnel is observed to increase as compared to the uniform ground profile. The present analysis highlights the importance of considering topography in hazard estimation. The influence zone in the transverse direction is observed to increase by 1- 2.16D as compared to a uniform ground profile having the same cover-to-depth ratio. Figure 15d shows the development of a longitudinal surface settlement profile for a ground surface having topographic variations with progressive tunneling. The zone of influence from the longitudinal axis of the tunnel is observed to vary as compared to a uniform ground profile. In the present case, the monitoring section is considered at $x/D = 17.08$. The settlement at the monitoring section started and reached the maximum for tunnel excavation face at $x/D = 9.16$ and 34.16, respectively. The zone of influence in the longitudinal direction is observed to be approximately

11.75D and 9.25D ahead and beyond the monitoring section. It is also observed that the shape of the transverse and longitudinal surface settlement profile is significantly influenced by the consideration of the surface topography. The shape of the transverse and longitudinal surface settlement is observed to follow the Gaussian distribution curve for the uniform ground profile. The shape of the transverse and longitudinal surface settlement does not follow the Gaussian distribution curve, and several kinks are observed (15 a, b, and d) for the ground surface having topographic variation. The asymmetry in geometry results in the surface settlement in a non-Gaussian pattern. The findings of the present study are important mainly due to the fact that hazard estimation due to tunneling is computed considering the settlement profile to be Gaussian in nature. Usually, the ground surface in urban and hilly regions has significant topographic variation, and the non-Gaussian nature of the settlement profile and increased influence zone may significantly influence the zone of hazard around tunneling.

Figure 16 shows the development of the vertical deformation (U_z) contour for ground having topographic

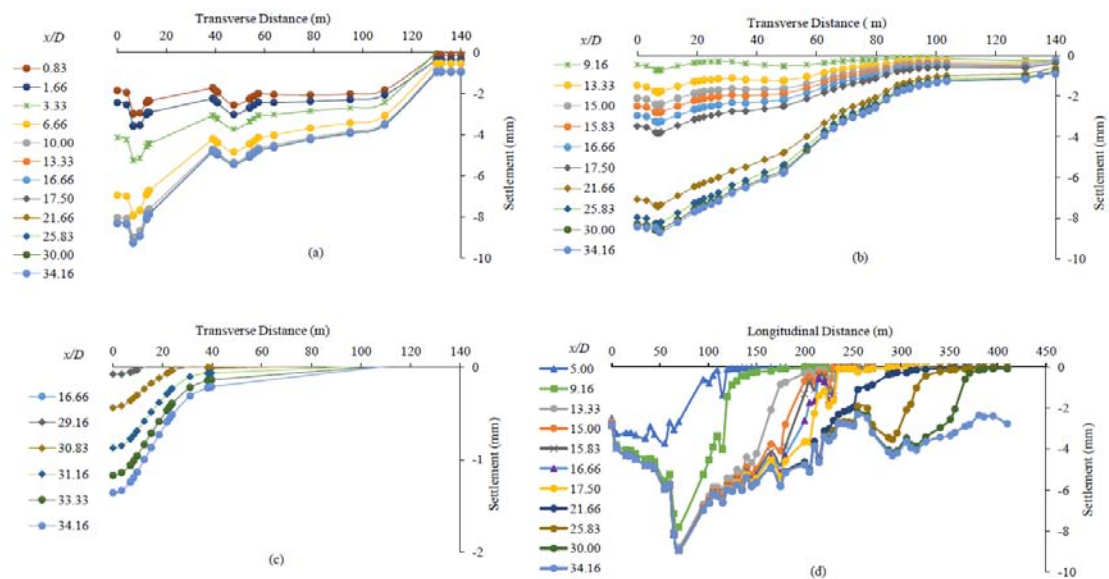


Figure 15: Typical surface settlement of topographic variation of ground profile, topsoil B, HS model transverse direction (a) section at 30 m away from tunnel face ($x/D = 2.5$). (b) section at middle of tunnel ($x/D = 17.08$) (c) section at 30 m before the tunnel end ($x/D = 31.66$), and (d) longitudinal surface settlement profile for progressive tunneling.

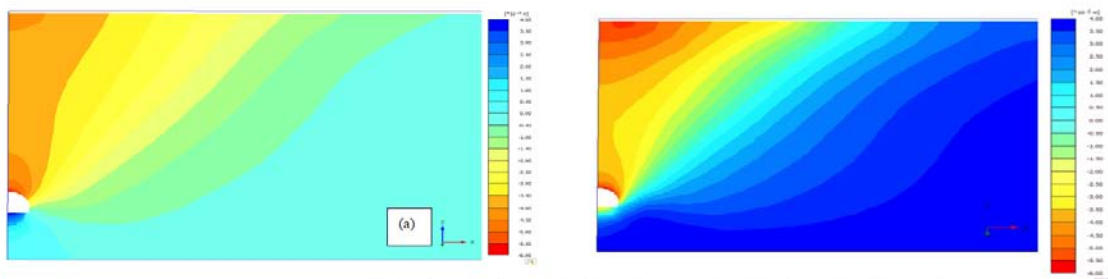


Figure 16: Ground deformation contour for ground profile having topographic variation (a) topsoil A and (b) topsoil B.

variation for topsoil A and B for variable tunnel cover depth (i.e., $C/D = 1.5 - 5$). The contour represents the formation of an arch-shaped zone above the tunnel crown that extends laterally and towards the ground surface. The figure shows that the settlement contour does not extend to the lateral boundary, confirming that consideration of the lateral boundary is sufficient. The pattern of settlement contour is found to be nearly similar for both topsoil A and B models. The results also indicate that the stress contour extends further in the case of a topographically varying ground profile compared to the uniform ground profile considered in the current numerical study. A similar observation was made in previous numerical analyses by Das *et al.* [7].

Table 10a shows the maximum surface settlement for the different cases and material properties having uniform ground profiles. The Mohr–Coulomb (MC) model underpredicts the surface settlement by a factor of 1.81–3.56 compared with the Hardening Soil (HS) model for the investigated parameters and geometric profiles. This discrepancy can primarily be attributed to the differences in the constitutive formulations and their representation of soil stiffness. The MC model assumes a constant elastic stiffness, whereas the HS model incorporates stress-dependent stiffness and nonlinear soil deformation through separate parameters for primary loading, oedometer loading, and unloading–reloading. During tunnel excavation, stress redistribution and unloading around the tunnel

induce nonlinear soil deformation, which can be represented more realistically by the HS model. Consequently, the HS model predicts greater surface settlement than the MC model for the adopted soil parameters and modelling conditions. However, the magnitude of this difference is also influenced by the selected stiffness parameters, initial stress conditions, tunnel geometry, and cover depth. Therefore, the observed underprediction by the MC model should be interpreted in the context of the investigated cases rather than as a universal limitation of the MC constitutive model.

The study also shows the sensitivity of the cohesion parameter of the rock layer on the settlement magnitude. For each rock class, two cohesion values have been used, representing lower and average ranges for the specific rock class. For rock class V, topsoil B, tunnel cover depth of 29 m, and HS soil model, the settlement increases in the range of 30–35% when cohesion is changed from 90 kPa to 50 kPa. The adopted properties of the topsoil do not significantly influence the surface settlement in the present analysis, and the variation is observed to be less than 10% for the various rock classes and the cover depth of the tunnel. The surface settlement measured at the plinth level of the building during tunnel construction ranges from 2 mm to 14 mm in the field. The maximum settlement in the field is observed to be at a distance of 100 m from the start position of the tunnel, having rock

Table 10a: Surface Settlement and Tunnel Lining Deformation

Topsoil Type	Rock Class	Type of ground profile	Cohesion (kPa)	Crown Position (m)	Surface Settlement (mm)		Tunnel Lining Deformation (mm), U _z			
					Numerical Analysis		Field Measured (Max. -Min.)	Numerical Analysis		Field Measured (Max. -Min.)
					MC model (Max.)	HS model (Max.)		MC model (Max.)	HS model (Max.)	
A	IV	Uniform	150	18	1.04	3.61	14 - 2	2.20	4.53	24 - 6
B					1.04	3.63		2.21	4.60	
B					-	3.72		-	5.93	
A	IV		150	29	0.95	3.06		3.36	5.94	
B					0.95	3.06		3.38	6.22	
A	V		90	18	3.06	8.46		5.59	11.52	
B					3.06	8.93		5.64	11.68	
B					6.70	12.16		9.25	15.02	
A	V		90	29	2.17	7.71		5.79	12.48	
B					2.18	7.74		5.81	12.65	
B					4.25	10.01		18.99	22.12	
-	III		250	29	0.71	1.56		1.19	1.95	
				60	0.42	1.29		2.69	3.12	
			200		-	1.54		-	3.15	

class V. Maximum surface settlement for the uniform ground profile estimated from the numerical analysis is approximately 12.16 mm for rock class V having cohesion 50 kPa, topsoil B for tunnel crown position at 18 m. The maximum surface settlement for rock class III in a uniform ground profile with various tunnel cover depths ranges from 0.42 mm to 1.54 mm. For rock class IV, the surface settlement for different tunnel cover depths ranges from 1.04 mm to 3.72 mm in the present numerical analysis. It is important to note that the present study considers the Greenfield condition for numerical analysis, while ground surface settlement measurements were taken at the basement level of an existing building. Consequently, it should be mentioned that the self-weight of the existing building influences the ground surface settlement. The present study shows that the appropriate numerical modeling procedure and soil model can predict surface deformation in the field with reasonable accuracy.

Table 10b shows the maximum surface settlement for the ground surface having topographic variation and different material properties. The surface settlement estimated from the numerical analysis ranges from 4.22 mm to 8.76 mm, depending on the different tunnel cover depths and the properties of the topsoil and rock used in the analysis. The observed variation in the surface settlement is less compared to the uniform ground profile and was found to be approximately 50% when the lower range of cohesion values is used for

each rock class. It is to be noted that the estimated surface settlement for the ground surface having topographic variation underpredicts the measured settlement values in the field. It is to be noted that the field-measured settlement is at the plinth level of the building. The loading and stiffness of the building also influence the magnitude of the settlement and are expected to increase when compared with the Greenfield settlement. Also, the local variation in the material properties and macro topography has not been considered in the present study. The overburden pressure on the tunnel support system is higher for the ground surface having a uniform profile, resulting in a higher settlement at the surface. All of the mentioned reasons result in the lower surface settlement prediction when topography is considered in the analysis.

7.2. Lining Deformation

Tunnel lining deformation is one of the important factors that affect the surface and sub-surface settlement, resulting in the risk to the ground surface and adjacent structures. In the present study, tunnel lining deformation in the vertical direction is computed and reported only. Figure 17a shows the typical tunnel lining deformation profile obtained from the numerical analysis. Figure 17b shows the actual and deformed profile of the tunnel lining schematically. Table 10a shows the maximum lining deformation (U_z) at the

Table 10b: Surface Settlement and Tunnel Lining Deformations

Topsoil Type	Rock Class	Type of ground profile	Cohesion (kPa)	Crown Position (m)	Surface Settlement (mm)		Tunnel Lining Deformation (mm), U_z	
					Numerical Analysis	Field Measured (Max. - Min)	Numerical Analysis	Field Measured (Max. - Min.)
					HS model (Max)		HS model (Max)	
A	III, IV and V	Topographic variation	250, 150 and 90	18 - 60	4.22	14 - 2	11.74	24 - 6
B	III, IV and V		250, 150 and 90		4.32		11.82	
B	III, IV and V		200, 100 and 50		8.76		21.61	

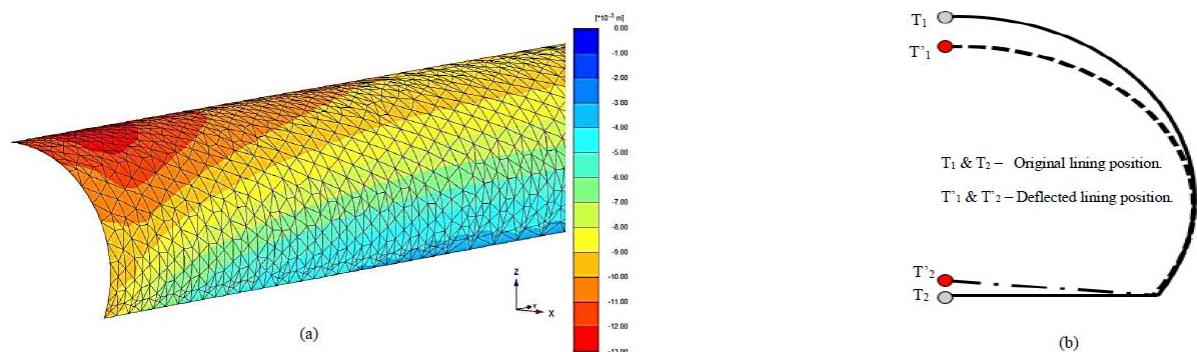


Figure 17a and 17b: Tunnel lining deformation profile

crown position (P_1) for the MC and HS models for different tunnel cover depths having uniform ground profiles. The result shows that the lining deformation for rock class IV is in the range of 2.2 – 3.38 mm and 4.53 – 6.22 mm for the MC and HS models, respectively, for different material properties and cover depths of the tunnel considered in the present analysis. Similarly, for class V, the lining deformation is in the range of 5.59 – 18.99 mm and 11.52 – 22.12 mm for the MC and HS models, respectively. The observed tunnel lining deformation for rock class III is in the range of 1.19 – 2.69 mm and 1.95 – 3.15 mm for the MC and HS models, respectively. The lining deformation is found to be 2.2 – 22.12 mm for the various tunnel cover depths and soil/ rock classes considered in the present analysis. In all cases, the lining deformation is lower for the MC model than for the HS soil model. For various material properties and tunnel cover depths, the lining deformation in the MC model is found to be 14 – 52% lower than the HS soil model. For rock class IV, topsoil B, and crown at 18 m, the lining deformation increases from 4.60 mm to 5.93 mm when rock cohesion decreases from 150 kPa to 100 kPa. All of the other cases in the current study show similar trends. Tunnel lining deformation is measured in the field through a tunnel monitoring system, varying from 6 – 24 mm. The maximum lining deformation in the field is observed at the tunnel line passing through rock class V. The maximum lining deformation in the present study is found to be 22.12 mm for rock class V, HS model at tunnel cover of depth 29 m ($C/D = 2.42$). It has been observed that the lining deformation increases with increasing tunnel cover depth. Also, the present analysis shows that the lining deformation obtained from the numerical investigation is up to 8% less than the field measurement value for rock class V, case B, with a tunnel cover depth of 18 m. Additionally, it can be stated that a decrease in the soil and rock cohesion parameter causes an increase in lining deformation. The lining deformation spreads vertically and laterally along the settlement stress contour, resulting in a higher tunnel convergence. A similar observation has also been made in previous literature [51].

Table 10b shows the tunnel lining deformation estimated for the topographic variation of the ground profile for variable tunnel cover depth ($C/D = 1.5$ to 5). The tunnel lining deformation for the HS model varies between 11.74 – 21.62 mm for two different top colluvium soil properties (i.e., A and B) with different rock cohesion and tunnel cover depth. The results indicate that the numerical investigation of tunnel lining deformation provides values up to 10% lower than those observed in field monitoring. It also shows that the lining deformation is greater for uniform ground profiles than topographic ground profiles. The

overburden pressure on the tunnel support system is higher for the ground surface having a uniform profile, resulting in a higher deformation of the tunnel lining. The maximum lining deformation is observed to be at the highest cover-to-diameter ratio of the tunnel, indicating less influence of surface structures. Also, the deformation mainly depends on the overburden pressure around the tunnel lining and support system. A close match in both analyses indicates that the estimation of material properties of the support system, the simulation of the support system, and the overburden pressure is correctly simulated.

7.3. Ground Volume Loss and Tunnel Convergence

In the present study, ground volume loss refers to the surface volume loss calculated through numerical investigations, which captures the displacement and compaction of the surrounding ground due to tunneling. On the other hand, tunnel convergence quantifies the reduction in tunnel cross-sectional area caused by the deformation of the tunnel lining during construction. Both parameters are critical for assessing the impact of tunneling on the surrounding environment and the tunnel structure itself. In the present study, tunnel convergence and ground volume loss have also been estimated from the field measurement and numerical analysis results. Table 11a shows a comparison of ground volume loss and trough width obtained from numerical and field investigation in uniform ground profile. For rock class IV, topsoil A, crown position at 18 m, the ground volume loss for the HS model and MC model is 0.05% and 0.03%, respectively. For tunnel cover depth of 29 m, rock class IV with topsoil A, the ground volume loss is found to be 0.03% and 0.01% for HS and MC soil models, respectively. The ground volume loss for rock class V, topsoil B, and crown positions at 18 m are 0.06% and 0.04% for the HS and MC models, respectively. Ground volume loss is found to be 0.04% and 0.03% for rock class III, crown position at 29 m for HS and MC model analysis. The surface volume loss is observed at 0.10% in the case of rock class III, crown position at 60 m for HS model analysis. The surface volume loss is found to be in the range of 0.01 – 0.10 % for the different parameters considered in the present study. The results show that the surface volume loss decreases with increasing tunnel cover depth ($C/D = 1.5$ to 5). It also found that the volume loss increases with a decrease in the cohesion properties of soil and rock. A direct comparison of the results on the ground volume loss among the field measurement and numerical analysis has not been made in the present numerical analysis. A limited number of monitoring points at the ground surface during the construction of the tunnel makes it difficult to

Table 11a: Volume Loss and Trough Width (*I*) for Uniform Ground Profile

Rock/Topsoil Class	Crown Position (m)	Type of ground profile	Cohesio n (kPa)	Ground Volume Loss (%)		Tunnel Convergence (%)			Trough Width, <i>i</i> (m)	
				Numerical Analysis		Numerical Analysis		Field Measured	Numerical Analysis	
				MC model (Max.)	HS model (Max.)	MC model	HS model		MC model	HS model
IV, A	18	Uniform	150	0.03	0.05	0.14 - 0.03	0.22 - 0.07	0.27 - 0.02	10	14
IV, B				0.03	0.05				10	14
IV, B				100	-				0.05	-
IV, A	29		150	0.01	0.03				16	18
IV, B				0.01	0.03				16	18
V, A	18		90	0.04	0.06				10	14
V, B				0.04	0.06				10	14
V, B			50	0.05	0.10				10	14
V, A	29		90	0.02	0.03				16	20
V, B				0.02	0.03				16	20
V, B			50	0.03	0.05				16	20
III	29		250	0.03	0.04				17	18
III	60			0.05	0.06				26	30
III			200	-	0.10				-	30

make a direct comparison with field observations and numerical estimation. Various researchers reported a typical range of ground volume loss for various soil conditions. The volume loss due to NATM tunneling in Frankfurt clay [52], Singapore Boulder Bed Clay [53], London Clay [54], stiff clay [19], London clay [55,56] has been observed to be in the range of 1 – 2 %, 0.27 – 0.48%, 1%, 0.5 – 1.5 %, 1.1 – 1.5 %, and 1.1 – 1.8 %, respectively. Similarly, Kavvadas *et al.* [57] observed a volume loss of up to 0.2 % due to tunneling in weak rock ground. The range of ground volume loss found in this study is within the ranges observed by other researchers and refers to stable ground conditions. In the present study, the MC soil model significantly underpredicts the ground volume loss than the HS soil model. Table 11b shows the ground volume loss estimated from the numerical analysis and tunnel convergence from field observations for the topographic variation of the ground profile. The estimated ground volume loss is in the range of 0.05 – 0.10 %. The results show that the topographic variation of the ground profile is approximately identical to the ground volume loss when compared to the maximum volume loss for the uniform ground profile. The zone of settlement in the transverse direction is larger for the topographic variation as compared to the uniform ground profile. The ground volume loss is a function of both the magnitude of the settlement and the width of

the settlement zone. This results in identical surface volume loss in topographic variation when compared with uniform ground variation, though less settlement is observed in the topographic variation of the ground profile.

The estimated tunnel convergence from the field observation is in the range of 0.02 – 0.27%. The numerical analysis using MC and HS soil models with uniform ground profiles shows tunnel convergence in the range of 0.03 - 0.14% and 0.07- 0.22%, respectively. Tunnel convergence has been observed to increase with a decrease in soil and rock cohesion and with an increase in tunnel cover depth. A decrease in soil and rock cohesion reduces the shear strength of the surrounding ground, thereby weakening its resistance to deformation and resulting in increased tunnel convergence. Similarly, an increase in tunnel cover depth leads to higher overburden stress, which increases the stress acting on the tunnel boundary and may induce greater inward deformation. However, the extent of convergence also depends on the ground stiffness, geological conditions, excavation sequence, and support system. In the case of topographic variation of the ground profile, the tunnel convergence is found to be in the range of 0.18 – 0.22%. The numerical analysis slightly underpredicts the tunnel convergence as compared to the observation in the field. Usually, the tunnel convergence and ground

Table 11b: Volume Loss for Topographic Variation of Ground Profile

Topsoil Type	Rock Class	Type of ground profile	Crown position (m)	Cohesion (kPa)	Ground Volume Loss (%)	Tunnel Convergence (%)	
					Numerical Analysis	Numerical Analysis	Field Measured (Max. - Min.)
					HS model (Max)	HS model (Max. - Min.)	
A	III, IV and V	Topographic variation	18 - 60	250, 150 and 90	0.05	0.22 – 0.18	0.27 – 0.02
B	III, IV and V			250, 150 and 90	0.05		
B	III, IV and V			200, 100 and 50	0.10		

volume loss are observed to be equal for clay in undrained loading. The ground volume loss for the cohesionless soil significantly depends on many factors, such as cover depth, confining pressure, and dilatancy behavior. Assumptions regarding the uniform layer thickness, horizontal ground surface, and straight alignment of the tunnel in the numerical analysis result in a slight underprediction of the lining deformations and, subsequently, lower tunnel convergence estimated in the numerical analysis. Also, it is to be noted that the tunnel convergence is significantly higher than the ground volume loss. A significant depth of the subsurface above the tunnel is rock medium, above which soil is observed at the top surface. Less contractive behavior of the rock medium results in less ground volume loss in the mixed ground condition when compared with tunnel convergence. Based on the findings of the present analysis, the post-construction parameters (maximum surface settlement and tunnel convergence) of the tunnel may be predicted with sufficient accuracy with proper consideration of the soil and rock model and parameters. The study also highlights the importance of proper consideration of soil and rock profile and tunnel alignment for a more accurate prediction of post-construction parameters of the tunnel.

7.4. Trough Width (i)

The trough width, i.e., the width of inflection point (i) of the settlement trough, is estimated from the surface settlement obtained through numerical analysis and compared with the various literature. Detailed information on the trough width parameter from the field is not available due to the limited number of measurement points. Table 11a shows the trough width obtained from the numerical analysis for various geometric conditions considered in the present analysis for uniform ground profile. The results show that the i for uniform ground profile for various tunnel cover depths ranges between 10 - 30 m ($0.83 - 2.5D$). The variation in tunnel cover depth affects the trough width, as observed in the current analysis. It has been noted that the trough width increases with increasing tunnel cover depth. Various researchers, through field

observations and experimental, numerical, and analytical investigations, proposed different relationships of trough widths. Peck[17], Atkinson and Potts [58], Selby [59], and Arioglu [60] proposed relationships to evaluate trough width, i for stiff clay, loose sand, dense sand with over consolidated clay, clay overlain by sand and mixed ground conditions as $i = (D/2) \times (Z_0 / D)^{0.1 \text{ to } 0.8}$, $i = 0.25 (Z_0 + R)$ and $i = 0.25 (1.5 Z_0 + 0.5 R)$, and $i = 0.386 Z_0 + 2.84$ respectively. The estimated trough width from the above-mentioned literature is found to be in the range of $0.5 - 2.5D$ for different cover depths of the tunnel with uniform ground profile. The trough width parameter (i) obtained from the numerical analysis shows a close match with the existing literature for uniform ground profile model analysis. The trough width (i) obtained from the numerical analysis for the topographic variation of the ground profile is not reported here. As reported earlier, the surface settlement does not follow the Gaussian distribution pattern, and earlier reported values of i in literature usually indicate the trough width for the Gaussian distribution of the settlement pattern.

Figure 18 compares the trough width (i) obtained by the present numerical analysis with other approaches. Results show that the maximum trough widths (i) for $C/D = 1.5, 2.42$, and 5 are 14 m , 20 m , and 30 m , respectively, in the present study. It is also observed that the estimated trough width (i) is larger for $C/D = 1.5$ and 2.42 when compared with the previous studies. The observed trough width is shorter when compared with Peck [17], Attewell [49], and Mair [16] for $C/D = 5$. It is observed that the variation in trough width (i) ranges between $6 - 31\%$ for the present numerical analysis and previous studies.

8. FIELD OBSERVATIONS VS. NUMERICAL ESTIMATION

This section compares the tunnel lining deformation and surface settlement estimated from the numerical investigation with the field observations. Figures 19a and 19b show the comparison of the surface settlement obtained from the field observation (points S_1 to S_4) with numerical results. The settlement of

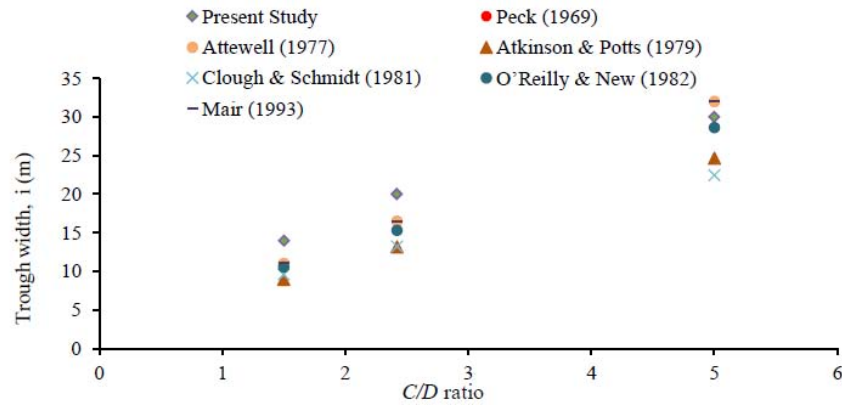


Figure 18: Comparison of trough width (i) for different approaches.

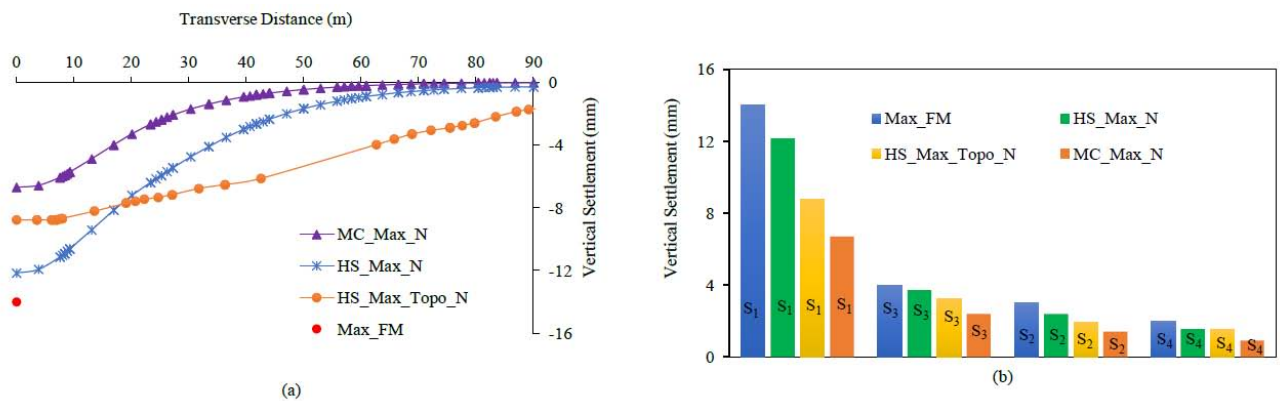


Figure 19a and 9b: Comparison of ground surface settlement (numerical vs. field measured).

monitoring points obtained from the field observation is in the range of 2 - 12 mm. Figure 19a shows the comparison of maximum surface settlement from field observation and the corresponding transverse surface settlement profile from numerical estimation for the MC and HS soil models for uniform and topographic ground profiles. Figure 19b shows the comparison of surface settlement at monitoring points (S_1 to S_4) for MC and HS soil models for uniform and topographic ground profiles. Considering the uniform and topographic variation of the ground profile, the maximum and minimum ground settlement are 0.42 - 6.70 mm and 1.29 - 12.16 mm for the MC and HS soil models, respectively. The results show that the variation of surface settlement for the MC and HS soil models with different cohesion and tunnel cover depth varies from 44 - 52% and 13 - 37%, respectively.

Figure 20 compares tunnel lining deformation (U_z) along the tunnel length obtained from field observation and numerical investigation. The lining deformation in the field is observed to be in the range of 6 - 24 mm. The numerical findings for various rock classes and crown positions with uniform ground profile model analysis show that the tunnel lining deformation ranges from 1.19 to 22.12 mm. In addition, consideration of the topographic variation of the ground profile shows the

tunnel lining deformation in the range of 11.74 to 21.61 mm. A close match in the deformation values has been observed at measurement points along the tunnel length at 60 m, 70 m, and 405 m for a uniform ground profile, and 6 m, 30 m, 41.50 m, 60 m, 61.50 m, 71.50 m, and 370 m for a topographic ground profile for the HS soil model. A reasonable match has been observed for vertical deformations among numerical analysis and field observations. The comparison shows a good match. The discrepancies in the range of 18 - 25 % when compared with the HS soil model at other measurement points may be due to material parameter estimation, local variation of material properties, approximation in construction procedure, and layering system, etc., in the numerical modeling. Considering the variability and uncertainty involved in the field, the present numerical model closely simulates the surface settlement and tunnel liner deformation response. Furthermore, the influence of existing buildings on ground surface settlement and tunnel lining deformation, compared with greenfield conditions, is acknowledged as a limitation of the present study.

9. CONCLUSIONS

The present study investigates the responses of the ground and tunnel lining resulting from staged

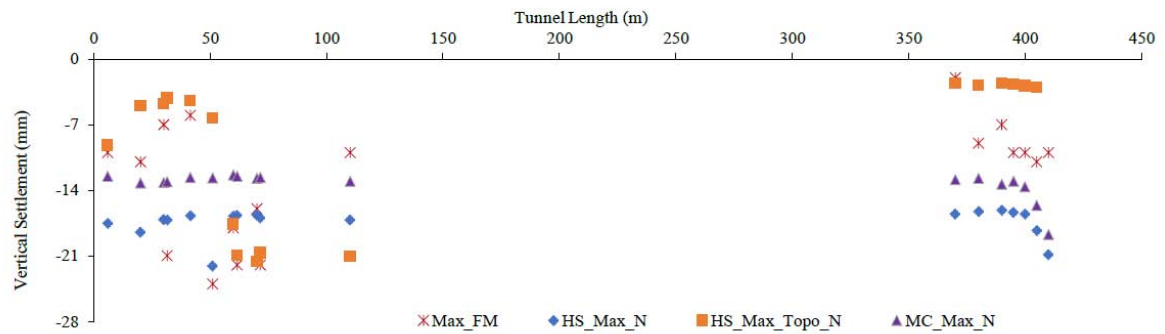


Figure 20: Comparison of vertical deformation (U_z) of tunnel lining (numerical vs. field measured).

tunneling through numerical analysis. Comprehensive geological and geotechnical investigations have been conducted to define the subsurface layering and identify key features. The geotechnical analysis involved borehole drilling, rock mass rating (RMR), and rock mass quality assessments (Q value) to determine the engineering properties of the existing strata. The study evaluates surface settlement, lining deformations, and volume loss caused by staged tunneling and compares these with field observations. Typically, the hazard assessment of tunnel linings and surrounding infrastructure is determined based on factors such as ground surface settlement, lining deformation, and the strain induced on the tunnel lining. However, there is limited literature on surface settlement and the lining deformation caused by variations in ground topography. To address this gap, the present study evaluated the response of the ground and lining due to changes in the ground topography and compared the results with field monitoring reports. The following conclusions may be drawn from the present study.

a) The present study shows that the magnitude and extent of surface settlement are significantly influenced by the tunnel and soil characteristics and ground profile. Mohr-Coulomb soil model significantly underpredicts the tunnel and ground response in all cases. The magnitude and zone of influence of surface settlement are observed to decrease and increase, respectively, with an increase in the cover-to-diameter ratio of the tunnel. The zone of influence in the transverse direction for the mixed ground condition and uniform soil profile is found to be in the range of $5D$ - $7.5D$ from the longitudinal axis of the tunnel. The influence zone in the longitudinal direction is observed to be in the range of $6D$ and $10D$ ahead of and beyond the monitoring section, respectively. The observed influence zone is compared well with the published literature for similar C/D ratios of tunnel and mixed ground conditions.

b) The topography of the ground surface significantly influences the magnitude, pattern, and extent of the surface settlement. An increase in the extent of the influence zone in both the transverse and longitudinal directions has been observed when the topography of the ground surface is accounted for using the average slope angle. The zone of influence along the transverse and longitudinal directions is observed to be in the range of $9.5D$ to $11.66D$ for the different cover depths of the tunnel. The zone of influence along the transverse distance of the tunnel is observed to increase as compared to the uniform ground profile. The influence zone in the transverse direction is observed to increase by $4.16D$ to $5.5D$ as compared to a uniform ground profile having the same cover-to-depth ratio. The influence zone in the longitudinal direction is observed to increase by $1.75D$ to $4.75D$ as compared to the uniform ground profile. The present analysis highlights the importance of considering the topography in hazard estimation with an increased deformation/influence zone.

c) It is also observed that the shape of the transverse and longitudinal surface settlement profile is significantly influenced by consideration of the surface topography. The shape of the transverse and longitudinal surface settlement is observed to follow the Gaussian distribution curve for the uniform ground profile. The shape of the transverse and longitudinal surface settlement does not follow the Gaussian distribution curve, and several kinks are observed for the ground surface having topographic variation. The asymmetry in geometry results in the surface settlement in a non-Gaussian pattern. The findings of this study are significant primarily because the hazard estimation due to tunneling is calculated by assuming the settlement profile follows a Gaussian distribution. Usually, the ground

surface in urban and hilly regions has significant topographic variation and a non-Gaussian settlement profile, and the increased influence zone may significantly influence the zone of hazard around tunneling.

- d) The surface volume loss is found to be in the range of 0.01 - 0.10 % for the different parameters considered in the present study. It was also found that the volume loss increases with a decrease in the cohesion properties of soil and rock. The results show that the ground volume loss with topographic variation of the ground profile is approximately identical when compared to the maximum volume loss for the uniform ground profile. The numerical analysis using the HS soil model having a uniform ground profile shows tunnel convergence in the range of 0.07- 0.22% and 0.18 – 0.22%, respectively, for uniform and topographic ground profiles. The maximum tunnel convergence in topographic variation is observed to same as compared to the uniform ground profile. The numerical estimation of tunnel convergence slightly underestimates the tunnel convergence observed in the field. Additionally, in this study, ground volume loss and tunnel convergence do not correspond to each other.
- e) A reasonable match has been observed for the vertical deformation of the lining at the crown between field observation and numerical estimation. The lining deformation in the field is observed to be in the range of 6 - 24 mm. The numerical findings for various rock classes and crown positions show that the tunnel lining deformation ranges from 1.19 – 22.12 mm and 11.74 - 21.61 mm for uniform and topographic ground profiles, respectively.

The limitations of the present study are associated with the idealization of geological conditions, groundwater effects, tunnel excavation procedures, and ground topography. The model does not explicitly account for variations in soil and rock properties, groundwater levels, or the progressive development of volume loss during staged excavation. Furthermore, a uniform annular gap and a straight, horizontal tunnel alignment were assumed, which may differ from actual field conditions. The ground surface was represented by a uniform slope; therefore, the effects of irregular topography, such as ridges and valleys, were not considered. These simplifications may influence the predicted ground settlement and tunnel lining deformation, and the findings should be interpreted within the scope of the assumptions adopted. Future studies should incorporate more realistic geological

profiles, groundwater conditions, construction sequences, and complex tunnel alignments and topographies.

ACKNOWLEDGEMENTS

The authors are grateful to the Director, CSIR–CBRI, for providing support and facilities to complete this study. The authors are thankful to CSIR-HRDG for sponsoring the fellowship to the first author. The authors are also thankful to Bharat Construction (India) Pvt. Ltd for providing all the necessary support during the study.

FUNDING DECLARATION

The first author is thankful to the Council of Scientific and Industrial Research (CSIR), India, for financially supporting him in his work (GATE - SRF, File No: 31/GATE/24(12)/2020-EMR-I).

CREDIT AUTHORSHIP CONTRIBUTION STATEMENT

Asit Bayen: Writing – original draft, Visualization, Methodology, Formal analysis, Data curation, Funding acquisition, Conceptualization. **Manojit Samanta:** Writing – review and editing, Supervision, Conceptualization.

SUPPLEMENTARY DATA

Some or all data, models, or codes that support the findings of this study are available from the corresponding author upon reasonable request.

DISCLOSURE STATEMENT

The authors reported no known potential conflict of interest.

NOTATIONS

E_{mass}	= Deformation modulus of rock mass (kPa)
RMR	= Rock mass rating value
E	= Elastic modulus (kPa)
$E_{\text{ode}}^{\text{ref}}$	= Odometer loading/reloading stiffness (kPa)
E_{50}^{ref}	= Triaxial loading stiffness (kPa)
$E_{\text{ur}}^{\text{ref}}$	= Triaxial unloading stiffness (kPa)
D	= Outer diameter of tunnel (m)

C	= Tunnel cover depth (m)
Z_0	= Distance from the surface to the center of the tunnel (m)
Z_1	= depth of sub-surface level from ground (m)
Z_2	= Distance from sub-surface level to the center of the tunnel (m)
x	= Distance of tunnel cross-section along the length of the tunnel (m)
i	= trough width of a Gaussian settlement profile (m)
MC_Max_N	= Maximum numerical value for MC model analysis (mm)
HS_Max_N	= Maximum numerical value for HS model analysis (mm)
HS_Max_Topo_N	= Maximum numerical value of topographic variation for HS model analysis (mm)
Max_FM	= Maximum field measured value (mm)

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<https://doi.org/10.65904/3083-3590.2026.02.10>

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