Tunnel Stability under Different Conditions: Analysis by Numerical and Empirical Modeling

V. Rama Sastry, K. Ram Chandar, S. Madhumitha, T.G. Sruthy*
Department of Mining Engineering, National Institute of Technology Karnataka, Surathkal, Mangalore, India

Abstract
Stability of the tunnels under different geotechnical conditions can be effectively assessed using numerical modelling techniques. In this paper, the stability of tunnel was analyzed along with support requirement, based on stress distribution around tunnel, using the finite element modelling software – NISA. A case study was taken up of a tunnel proposed by Karnataka Netravari Nigam Limited near Siddapura, Udupi District, Karnataka, India. Results from finite element modeling were verified using empirical approach (Geomechanics Rock Mass Classification) and CMRR (Coal Mine Roof Rating) and ARBS (Analysis of Roof Bolts) softwares. Results from FEM and RMR classification system were in agreement with the other approaches.

Keywords: numerical modeling, RMR classification, rock bolting, tunnel stability

****Corresponding Author***
E-mail: vedala_sastry@yahoo.co.in

INTRODUCTION

For designing a tunnel a detailed investigation on ground conditions and planning are required. Depending upon the type of strata encountered the stability of tunnel and support System requirement can be assessed. The design methods available for analyzing the stability of tunnels are categorized as analytical methods, observational methods and empirical methods. Analytical methods utilize the mathematical formulation like closed form solutions, numerical methods (Finite Difference, Finite Element, Boundary Element, Distinct Element), analog simulations (electrical and photoelastic) and physical modeling. Distinct element method was the first to consider discontinuous rock mass as an assembly of quasi–rigid blocks interacting through deformable joints of definable stiffness (Cundall, 1976).[1] In observational methods, the field monitoring of ground movement and its interaction with support is analyzed, which includes the New Austrian Tunneling Method (NATM) and the Convergence Confinement method. Empirical methods are statistical methods, which are mainly Geomechanics Classification proposed by Beinaiwski (1976)[2] and the Q-system proposed by Barton *et al.* (1974).[3] In Geomechanics classification by considering six parameters, the RMR value and the corresponding stand-up time for a given span can be assessed.

Peng and Tang (1984)[4] studied the stress distribution around a coal seam before and after the excavation of opening. However, when an opening is made, the stress equilibrium is disturbed and it tries to achieve new equilibrium state by undergoing deformations resulting in sagging of roof. For the rock mass stabilization, rock bolts and wire mesh and/or shotcrete for external support or combination of these also can be used for both temporary as well as permanent support. They also proposed that the rock bolts are mainly used for binding together the bedding planes, natural joints, fractures and also cracks and fractures formed as a result of excavation.

The U.S. Bureau of Mines (USBM) carried out research on beam building, in which the strata were clamped together by tensioned bolts with enhanced bending strength. Panek (1956)[5] reported that for beam building to take place, the bolts must be in tension producing a normal force between layers such that the frictional forces can carry the horizontal shear stress. Fairhurst *et al.* (1974)[6] presented a two-dimensional plate buckling criterion, in which the effectiveness of beam building is a measure of the moment of inertia of the bolted beam, which depends on friction between layers, bolt density, and shear stiffness of bolt-grout-rock interface. In contrast, according to Krohn (1978)[7] the fully grouted, un-tensioned bolts are more effective in terms of the reduction of mid-span deflection. In fact, as Jeffrey *et al.* (1982)[8] pointed out, bolting provides additional force, which increases the interaction and complete bonding of the layers on either side of the interface. Their research showed that, bending takes place about neutral axis of the composite laminated beams with complete interaction. The location of maximum inter-laminar shear stress takes place at a location that is away from the end, towards the inflection point of the beam, instead of at the ends of beam with clamped ends. This is where shear failure is most likely to initiate. Peng *et al.* (1989)[9] developed a 2D boundary element model for fully grouted bolts and applied dimensional analysis to derive a series of equations to determine the bolt length, number of bolts, tensile fracture, shear fracture at mid span and shear fracture at the entry corners. Stankus *et al.* (1997)[10] simulated tensioned bolts using Ansys software and proposed the Optimum Beaming Effect. Their study indicated that, point anchor and fully tensioned resin-assisted roof bolts are very effective, installed at high tensions in highly bedded and laminated strata. JunLu Luo’s (1999)[11] study also showed that installing bolts immediately after excavation is essential for beam building effect to develop. Hoek *et al.* (1980)[12] studied the practical applications of numerical methods in different geological strata and the potential fracture zones around excavations. Their study showed that the tensile failure occurred first in the crown portion of the tunnel.

NISA (Numerically Integrated elements for System Analysis) is a finite element program, developed to analyze a wide spectrum of problems encountered in engineering mechanics, which can handle linear and nonlinear structural and shape optimization, fatigue and fracture analysis, fluid flow analysis and printed circuit board stress and heat transfer analysis etc. All analysis programs are directly interfaced with the Display program for pre- and postprocessing.

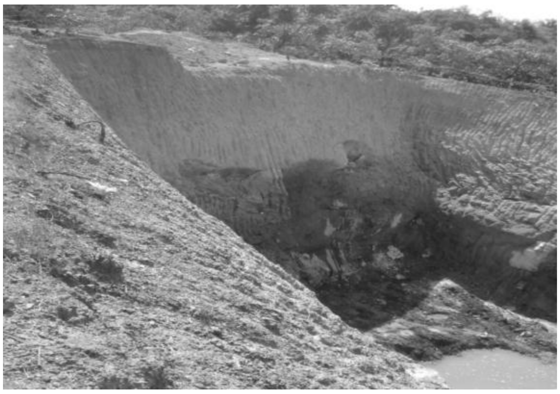
INVESTIGATIONS

A case study was taken up of a water tunnel of 1.3 km long of 5.3 m finished diameter, proposed by Karnataka Neeravari Nigam Limited, near Siddapura, Udupi District, Karnataka, for carrying water, as a part of Varahi Reservoir Project. Analysis was carried out using finite element modeling for the stability and stabilization of proposed tunnel. Results were verified using empirical approach proposed by Bieniawski (Geomechanics Rock Mass classification) and the CMRR (Coal Mine Roof Rating) and ARBS (Analysis of Roof Bolts) softwares.

The proposed tunnel commences at Chainage-6750 m and ends at Chainage-8050 m, connecting to the canal on both sides. The tunnel site in Varahi Right Bank Canal is approached by open cut canal at

Chainage 8100 m (Figure 1). The proposed tunnel is having different strata in the crown portion.

(a)



(b)

Fig. 1. *Tunnel Site (a) Canal Excavation Leading to Tunnel (b) Exposed Tunnel Portal Area.*

In the present study, two approaches were used. Initially, the most widely used RMR (Rock Mass Rating) Classification system proposed by Bieniawski (1984) was used for assessing the tunnel stability. The next step was to simulate the tunnel for given geo-technical conditions using finite element modeling. The Finite Element Modeling software NISA was used for modeling studies.

Field study was conducted at the canal excavation site to collect borehole logs and collect relevant information for carrying out further investigations. Exposed surface clearly indicated the presence of a minimum of 3 sets of major joints. One set of jointing is horizontal to semi-horizontal. The frequency of this set of discontinuity is around 1.0–1.2 m. This portion comes into crown portion of the tunnel, making the opening unstable. It was also observed that the portion above this bed was highly fractured (Figure 2).

Fig. 2. *Semi Horizontal Bed and Highly Jointed Rock Mass in Crown Portion of Tunnel.*

The other side of the tunnel portion also indicated presence of highly jointed formation, contrary to the initial geological report. In fact, this side of the proposed tunnel zone consists of more than three sets of joints as clearly seen in Figure 3.

Fig. 3. *Highly Jointed Rock Mass in Crown Portion of Tunnel (Left of Tunnel Area).*

A closer study of the face revealed the presence of semi vertical jointed rock mass from both sides of the tunnel excavation area (Figure 4). It also facilitates the ground water flow into the tunnel, causing some more instability.

Fig. 4. *Presence of Vertical Jointing in the Tunnel Zone.*

Observation of the rock mass in the floor exposed in canal excavation site revealed the presence of open joints running across the cross section of the tunnel (Figure 5). These are detrimental to the stability and also serve as channels for water flow.

Fig. 5. *Open Joints in the Floor Zone.*

In addition, the formation at many places (exposed) is weathered. Slopes of the approach canal clearly showed the weathering of rock mass which forms the crown portion of the proposed tunnel. Layers of laterite soil were found in between rock mass boulders, highly detrimental to the stability of tunnel.

In general, the strata observed in the field are complicated with 3 sets of joints, presence of open joints and boulders with soil layers at places. Water percolation is also high with approximately greater than 10 lt/min flow into tunnel cross section. During the rainy season the water make may further increase due to about 5000 mm rain fall experienced in this region. The situation, therefore, becomes more critical during rainy season.

The lithological data for different chainages of the tunnel also showed a wide variation in the presence of different layers of strata in the tunnel and crown portion also. At locations like Chainages 7000, 7025, 7050, and 7075 m, squeezing conditions exist in the tunnel alignment, posing serious problems to the excavation. Observation of the L-sections/sub-surface strata indicated wide variations in the formations. There is wide variation in the strata in crown, with as many as 8–10 different layers at many chainages. Between Chainage-7500 and 7700 m, there are 7–13 different formations in the superincumbent strata. Though the depth of tunnel is good at these chainages, the large variation in strata in the overburden is going to be a critical factor. This type of formation is not suitable for supporting by rock bolts.

Assessment of Tunnel Stability Using Geo-mechanics Classification

To apply the geo-mechanics classification, the rock mass along the entire tunnel alignment was divided into a number of structural regions, as per the borehole data provided at 25 m interval and as per the geological sections (L-sections) provided at different chainages from 6750 to 8100 m. Additional data/parameters were assessed through field study and laboratory testing. The RMR values for the entire tunnel length at different chainages are given in Table 1.

The RMR values indicated that except for a couple of sections, the entire length of tunnel is passing through Fair rock. The strata are not good or excellent. The natural stand-up time estimated as per Bieniawski's RMR rock mass classification for different chainages is given in Table 2.

Table 1. *RMR Values at Different Sections.*

Chainage (m)	Rock mass rating	Classification
6750	59	Fair
6775	27	Poor
6800–6975	49–59	Fair
7000–7075	46	Poor
7100–7175	49-54	Fair
7200	64	Good
7225–7250	59	Fair
7275	64	Good
7300–8100	46–59	Fair

Table 2. *Natural Stand-Up Time of Tunnel.*

Chainage (m)	Stand-up time (days)
6750	14
6775	Immediate failure
6800	1
6825	Immediate failure
6850	1
6875–6900	3
6925	1
6950	3
6975	1
7000–7150	Immediate failure
7175	1
7200	14
7225–7250	3
7275	14
7300	3
7325–7375	1
7400-7450	Immediate failure
7475–7525	3
7550–7575	Immediate failure
7600–7675	3
7700	1
7725–7775	Immediate failure
7800	1
7825–7875	Immediate failure
7900–8000	1
8025	3
8050	1
8075–8100	Immediate failure

Analysis revealed that the tunnel is not going to stand even for a day without artificial support at as many as 20 sections. At many other sections, the stand-up time is just a day. Only a few places, the stand-up time of the tunnel is relatively good with two weeks. This analysis indicates that at a large number of sections, the tunnel will be unstable to stand on its own. There may be collapse as soon as the new face is exposed, posing threat to the safety of people. In such cases, the freshly exposed roof of tunnel should be supported properly before proceeding further.

It is initially proposed to use 2 m long rock bolts of 25 mm diameter at 2 m apart, 4 in a row, for supporting the tunnel, before RCC lining. However, observation of the strata details provided indicates that the crown portion of tunnel is having large variations, mostly within 2m intervals, almost at all sections of the tunnel. The basic principle of rock bolting, which is anchoring of layer/loose rock mass to the upper layers, will be missing in these strata. Subsequently, analysis was carried out using standard softwares for analyzing:

(1) Stability of tunnel by stress analysis.
(2) Effectiveness of rock bolts for the present strata.

Initially, the strata in the crown of the tunnel were analyzed using CMRR and ARBS softwares. These softwares facilitate analysis of the effectiveness of rock bolting in given geo-technical conditions for given operating conditions of the tunnel.

CMRR (Coal Mine Roof Rating) Software

Many research studies have proved that the structural discontinuities in the rock mass /strata influence the stability of openings very significantly. Like other classification systems, the CMRR begins with the premise that the structural competence of roof rock is determined primarily by the discontinuities that weaken the rock fabric.

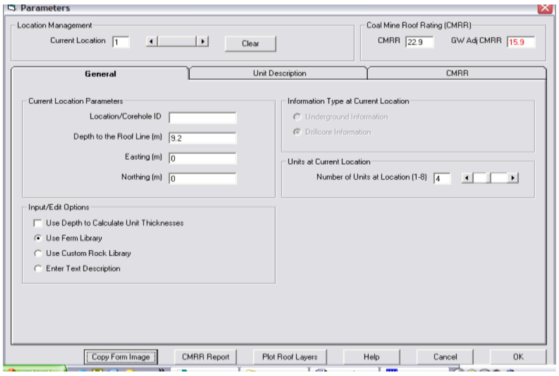
ARBS (Analysis of Roof Bolts) Software

ARBS were developed for more difficult conditions, where the roof is weaker and/or the stress is higher. It starts with the most important factors that determine the performance of a rock bolt system, which are the roof quality (measured by CMRR), the depth of cover (which correlates with stress), and the intersection span.

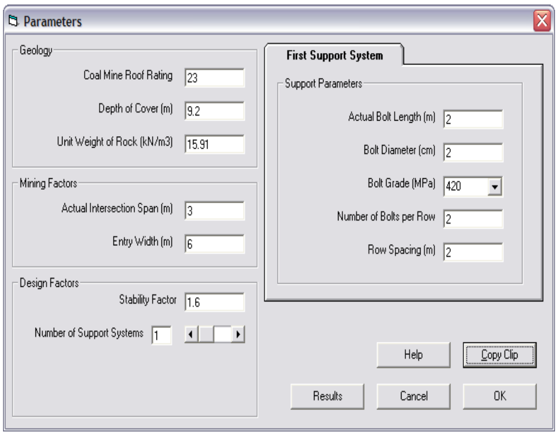
To cross check the results, at some important points given by RMR, analysis was carried out by CMRR and ARBS. Results suggested usage of roof bolts of 5 m length. Typical results of CMRR and ARBS analysis are shown in Figure 6.

Numerical Modeling by NISA

Modeling has been carried out with finite element modeling software NISA. The width of excavated tunnel was 6 m and height 6 m, before the lining. Conditions prevailing and the properties of the rock mass at different depths as per the geological sections were considered as major inputs for the model. Different tunnel sections were considered for generating the models, representing the entire length of tunnel. Vertical stress distribution was studied at the boundary of the tunnel. Initially, models were developed for analyzing the vertical stress and displacements without any support. Later, analysis was carried out with rock bolts of different lengths. Models were generated with 6 m × 6 m tunnel size for 10 chainages of Ch: 6825, 6850, 7050, 7100, 7350, 7375, 7375, 7800, 7900, 7925, and 7975 m. The vertical stress and vertical displacement from numerical modeling were studied at different points along the tunnel crown.



CMRR Result



ARBS Result

Fig. 6. *CMRR and ARBS Results at Chainage: 6825 m.*

The investigations were carried in stages to study the influence of jointing and rock bolting on the stability of the proposed tunnel. Granitic formation was considered at each stage. The depth of overburden was 30 m, with an average density of overlying strata as 2750 kg/m^3. The physico-mechanical properties of different materials used in the model are given in Table 3. Details of the study carried out are given below.

Table 3. *Input Properties of the Rock Mass Considered in the Model.*

Rocktype	Young's modulus (N/m^2)	Poisson's ratio	Cohesion (N/m^2)	Angle of internal friction	Mass density (kg/m^3)
Granite	48×10^9	0.23	80×10^6	32°	2750
Steel	2×10^{11}	0.30	–	–	7850
Cement grout	60×10^6	0.20	–	–	1910

The rock mass was modeled with 2D elements (quadrilateral and triangular) and the joints in the rock mass were modeled with contact surface elements, which are connected together to form a continuum around the tunnel. Plain strain condition was assumed to prevail. The boundary elements were modeled as roller supports.

Since the rock mass behaves non-linearly with the loading conditions, an elasto-plastic model was considered with Mohr-Coloumb criterion.

Observation points considered in the study are shown in Figure 7. Typical stress contours and vertical displacement obtained for Chainage 6,825 m without support are shown in Figure 8.

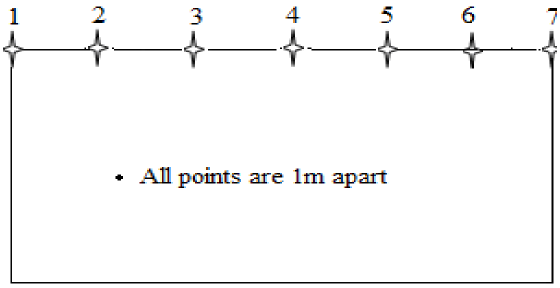


Fig. 7. Observation Points for the Study.

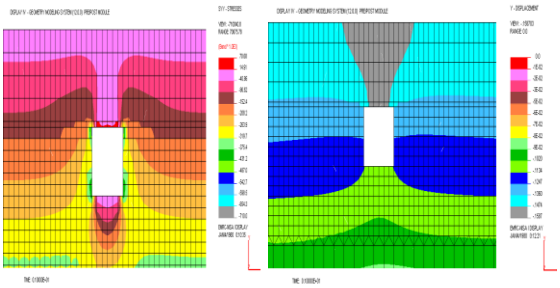


Fig. 8. Vertical Stress and Vertical Displacement Contours at Chainage: 6825 m (Without Bolting).

RESULTS AND ANALYSIS

Study revealed a displacement of 5–38 cm at different sections, excluding the ones having squeezing conditions, where there will be plastic flow of material. Most of the tunnel sections showed displacements of more than 10 cm. These are significant displacements from stability point of view.

In the next stage, models were developed by installing 2, 3, and 4 m long rock bolts of 25 mm diameter, as per the specifications provided by the Nigam. Observation points considered in the study are shown in Figure 9. Typical vertical stress contours and vertical displacement contours obtained for Chainage: 6825 m are shown in Figures 10 and 11.

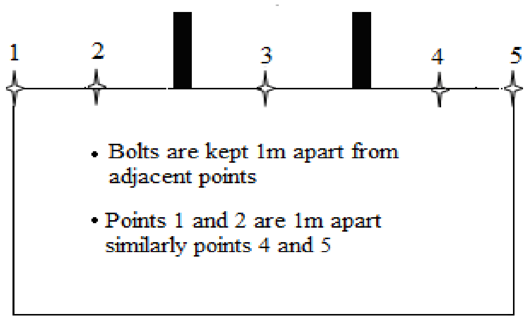
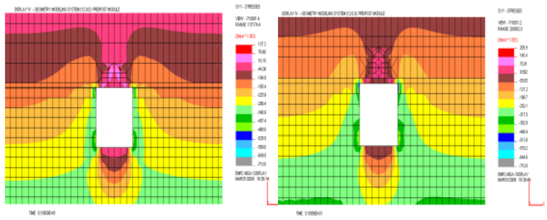
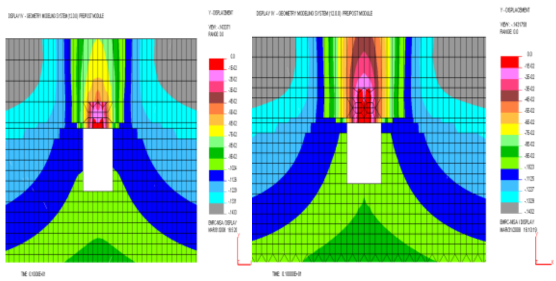


Fig. 9. Observation Points Along Crown.



a. 2 m bolt b. 3 m bolt

Fig. 10. Vertical Stress Contours at Chainage: 6825 m with Different Lengths of Rock Bolts.



a. 2m bolt b. 3m bolt

Fig. 11. Vertical Displacement Contours at Chainage: 6825m with Different Lengths of Bolts.

According to values obtained for Chainages 6825 and 7975 m, there is considerable reduction in displacement along the crown of tunnel with bolting. All the three bolt lengths showed almost similar reduction in displacement, but results found were not indicating prevention of crown failure even with bolts of 4 m length.

FEM modeling results at different tunnel sections indicated no change in vertical stress condition upto a bolt length of 4 m and also this bolt length was not sufficient to prevent the vertical displacement along the crown. Analysis with 2 m long bolts of 25 mm diameter using FEM indicated insignificant effect of rock bolting in the present strata. There was no change in vertical deformation even after bolting. Vertical displacements continued to be the same even with increased bolt lengths of 3 m and 4 m. Roof displacement at Chainage 6825 m without bolts varied from 13.7–15.8 cm, with maximum displacement at middle point. With bolt length of 4 m, the displacement decreased to 6.6 cm, but the displacement did not minimize to zero, which is essential for excavating the tunnel further. Similar trend was observed at different chainages of the tunnel. This point is of significance for providing the permanent support (concrete lining) in the tunnel. The tunnel should stand on its own or with rock bolts and arch supports till permanent lining is provided with concrete. The 2 m long bolt suggested initially is not going to stabilize the strata, as variation in roof strata is very frequent. Study has clearly indicated the requirement of bolt lengths more than 4 m. This is due to the presence of large number of variations in the rock mass encountered in crown zone of tunnel, continuously for the entire length.

Analysis with RMR also indicated the requirement of rock bolts of more than 4m long. Results of CMRR and ARBS analysis also suggested roof bolts of 5 m length. All the three analyses almost indicated similar results.

Effect of Joints on the Tunnel Stability

Tunnel with horizontal joints, spaced at 1 m, was considered for analysis at this stage. Depth of overburden was taken as 30 m. Results from numerical modeling were considered at different locations around the tunnel as shown in Figure 12. Vertical stress contours obtained are shown in Figure 13.

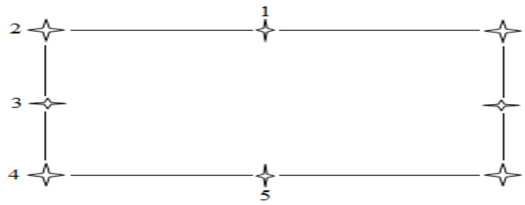


Fig. 12. *Observation Points Considered for Analysis.*

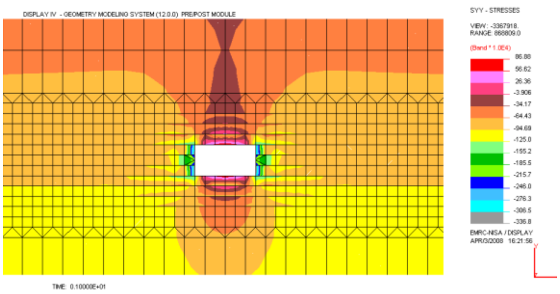


Fig. 13. *Vertical Stress Contours Around Tunnel.*

According to the results obtained for the tunnel with horizontal joints spaced at 1m, the vertical stress at the center of tunnel crown changed from –0.809 MPa (virgin compressive stress) to 0.071 MPa after tunnel opening was made and also on incorporating the joints, the stress further increased to 0.836 MPa (Figure 14). At the side walls of tunnel, the stresses remained compressive with considerable increase in stress, after excavation and when joints are incorporated (Figure 14). The vertical stress at the center of the tunnel floor changed from –0.907 MPa (virgin compressive stress) to 0.048 MPa after tunnel opening was made, and with joints the stress further increased to 0.869 MPa (Figure 14).

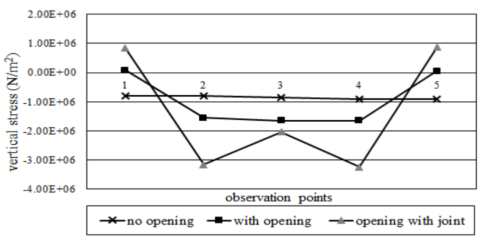


Fig. 14. *Vertical Stress Distribution Around Tunnel.*

Presence of horizontal jointing in the strata resulted in significant change in the vertical stress condition at all points of observation. Vertical displacements were considerable after incorporating horizontal joints, at all observation points (Figure 15). In general, the introduction of horizontal joints with a frequency of 1 m resulted in significant increase in vertical stress.

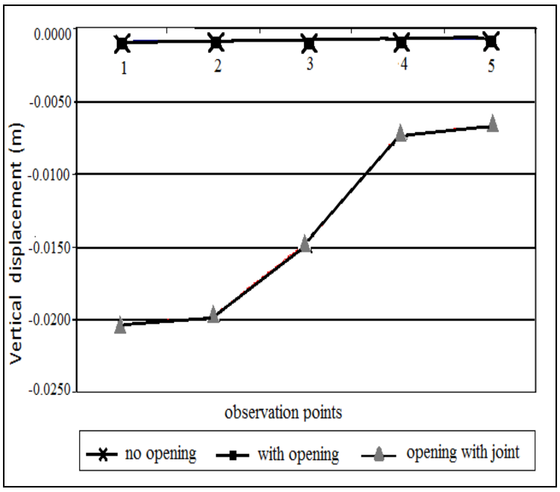


Fig. 15. *Vertical Displacement Around the Tunnel.*

Effect of Rock Bolting Along the Crown of Tunnel

Rock bolts were modeled along the crown of tunnel by varying the length, density and spacing.

Cement grouted bolt of 20 mm diameter in a borehole of 40 mm diameter was modeled for the study. Bolting was studied keeping depth of overburden same as 30 m, with a density of 2750 kg/m^3.

Bolts were inserted at the center of the tunnel crown. Length of the bolts modeled were 2.5, 3.5, and 4 m.

Effect of these bolts was studied on the tunnel with horizontal joints spaced at 1m and also without joints. Observation points considered in the study are shown in Figure 16.

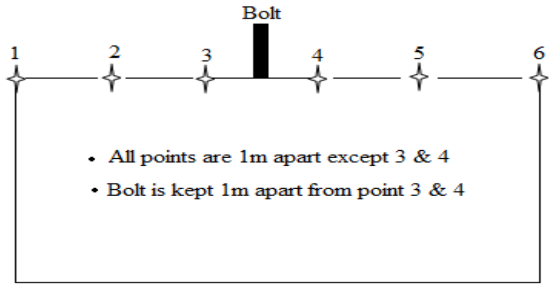


Fig. 16. *Observation Points Considered for Analysis with one Bolt at Center.*

The study conducted without joints and with one bolt of varying length revealed that the two points near to bolt on both sides were subjected to compressive stress, whereas all other points were under tension (Figures 17 and 18). The vertical stress and displacement contours are given in Figure 17. All the three bolt lengths showed almost similar change in vertical stress at all points. It was observed that with increase in bolt length, there was slight increase in stress (Figure 19). Though there was not much vertical displacement along the crown with the insertion of bolt, there was reduction in displacement which was almost similar for all the three bolt lengths (Figure 19).

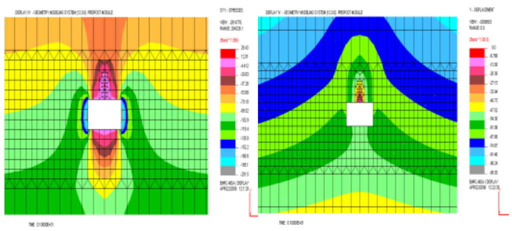


Fig. 17. *Vertical Stress and Displacement Contours with 3.5 m Long Bolt, Without Joints.*

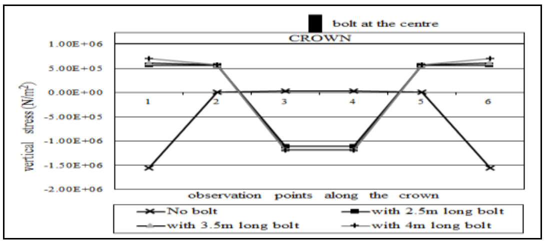


Fig. 18. *Vertical Stress Along Crown of Tunnel With Bolt at Centre, Without Joints.*

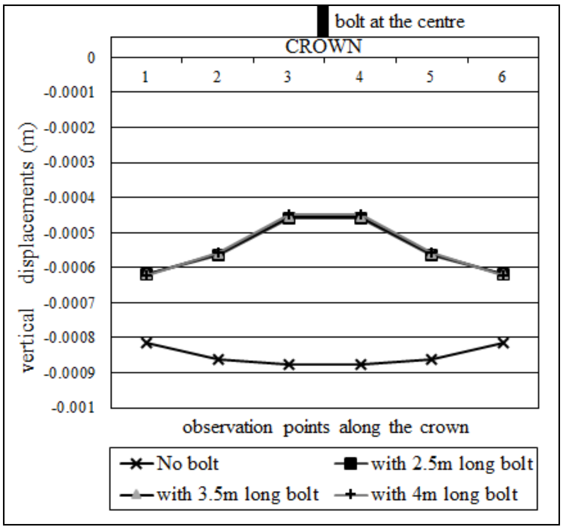


Fig. 19. *Vertical Displacement Along crown of Tunnel With Bolt at Centre, Without Joints.*

Analysis also carried out in case of the tunnel with horizontal joints spaced at 1m and with one bolt of varying length (Figures 20–22).

Vertical stress at all the points remained almost equal with all the three length of bolts (Figure 20). There was significant increase in stress at two points at a distance of 1m from the two ends, when bolt is inserted (Figure 21).

All other points were subjected to compressive stress with bolting (Figure 21). Displacement at all the points remained equal with all the three bolt length. At two points on either sides of the bolt, displacement reduced from 1.7 to 0.50 cm with bolting (Figure 22).

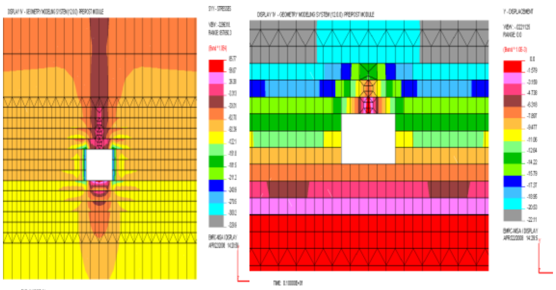


Fig. 20. *Vertical Stress and Displacement Contours with 3.5 m Long Bolt, With Joints.*

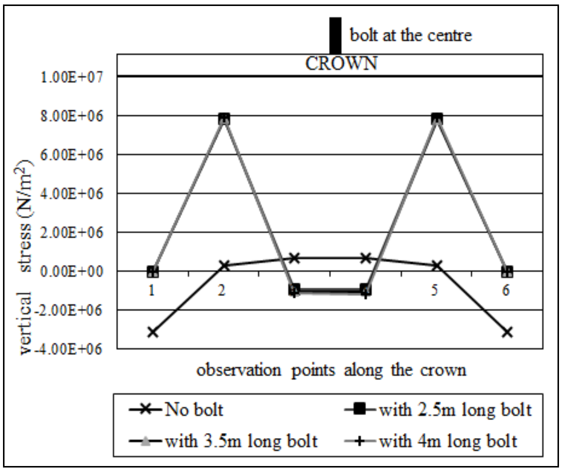


Fig. 21. *Vertical Stress along Crown of Tunnel with Bolt at the Centre, With Joints.*

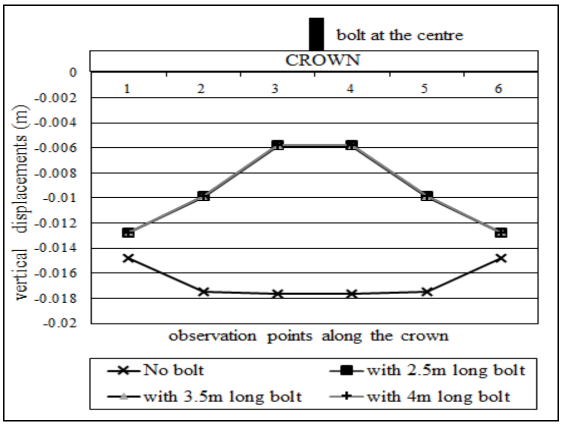


Fig. 22. *Vertical Displacement along Crown of Tunnel with Bolt at Centre, With Joints.*

Study was extended by providing two bolts of 2.5 m length, placed 2 m apart in the crown of tunnel. The observation points considered in the study are shown in Figure 23. The following observations were made for tunnel with horizontal joints spaced at 1m and also with no joints (Figures 23–29).

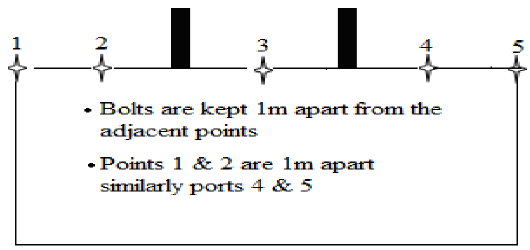


Fig. 23. *Observation Points Considered.*

The vertical stresses at two points, at a distance of 1m from the ends, before and after bolting, without joints were 0.013 and –0.847 MPa, respectively. At the center point, vertical stresses before and after bolting were 0.071 and –1.5 MPa, respectively, without joints. The vertical stresses with joints, before and after bolting were 0.310, 0.316 and –0.817, –0.819 MPa, respectively, for the two points at a distance of 1m from the ends. And at the center point, the vertical stresses with joints, before and after bolting was 0.836 and –3.71 MPa, respectively. There is not much displacement along the crown, when there are no joints and with joints, the displacement is about 1.7 cm at points other than end points, which reduced to 0.50, 0.07, and 0.5 cm, respectively, with bolting.

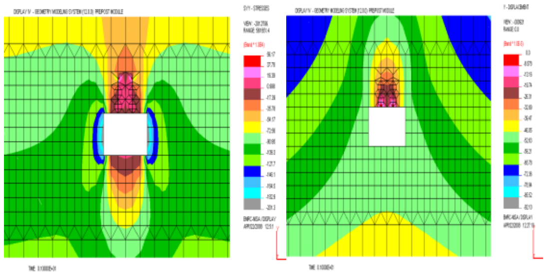


Fig. 24. *Vertical Stress and Displacement Contours with Two Bolts, Without Joints.*

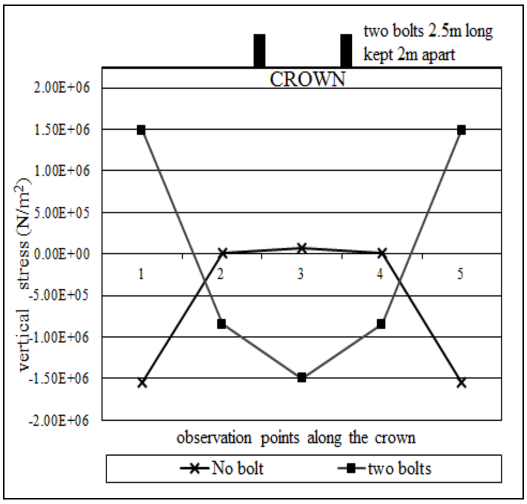


Fig. 25. *Vertical Stress along Crown of Tunnel with Two Bolts, Without Joints.*

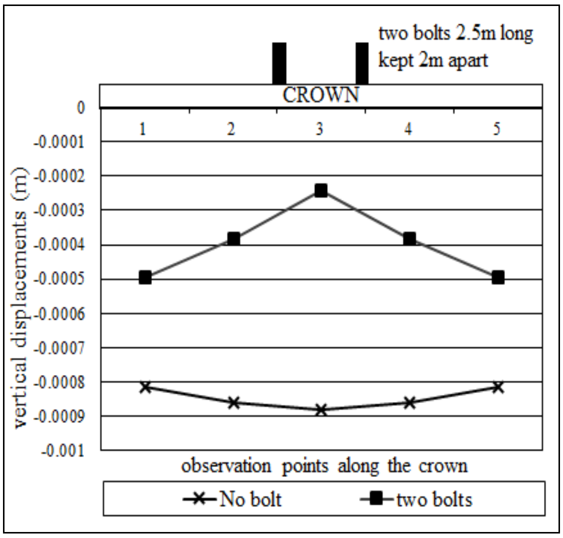


Fig. 26. *Vertical Displacement along Crown of Tunnel with Two Bolts, Without Joints.*

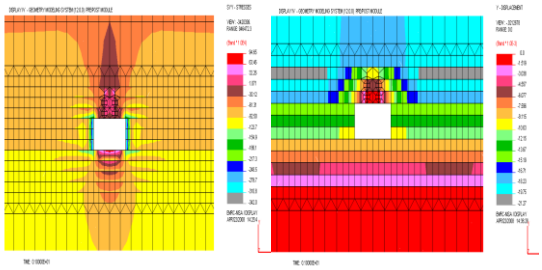


Fig. 27. *Vertical Stress and Displacement Contours with Two Bolts, With Joints.*

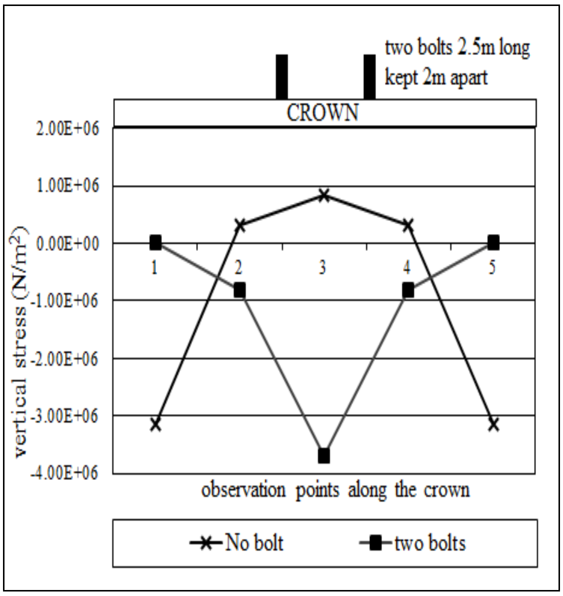


Fig. 28. *Variation of Vertical Stress along Crown of Tunnel with Two Bolts, With Joints.*

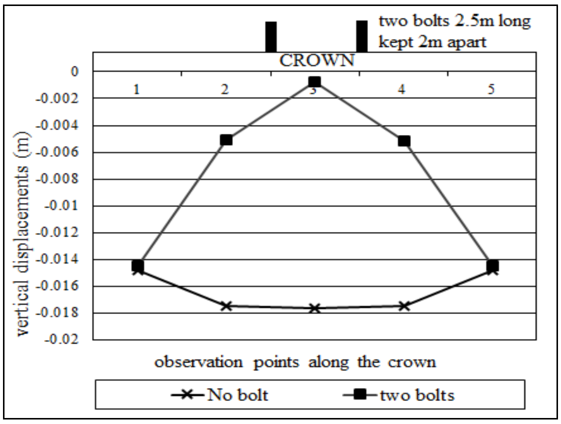


Fig. 29. *Variation of Vertical Displacement along Crown of Tunnel with Two Bolts, With Joints.*

CONCLUSIONS

Study carried out on the influence of the size of tunnel, jointing and rock bolting on the stability of tunnels lead to draw the following conclusions:

1. Tensile stresses are formed at the center of crown and floor of tunnel, whereas side walls are subjected to compressive stresses.
2. Points close to rock bolt along the crown of tunnel indicated compressive stress, whereas other points were subjected to tensile stress. With increase in length and number of bolts, more area in crown is subjected to compressive stress, resulting in effective consolidation of roof strata.
3. Vertical displacement along the crown considerably decreased with rock bolting, in case of models with joints. This clearly indicates the effectiveness of rock bolts in jointed/stratified formation. Rock bolts along the side walls of the tunnel showed reduction in vertical stress and increase in bolt length showed almost similar change in stress.
4. Case study demonstrated the significance of the geological variations in the overburden on the stability of the tunnel using RMR classification of rock mass for the design of tunnels, in the absence of numerical modeling techniques.
5. The results from FEM study and RMR analysis are almost in agreement and indicated that the proposed tunnel is unsafe and the proposed 2 m long rock bolts of 25 mm diameter are insufficient to provide stability to the tunnel.
6. FEM analysis indicated requirement of 5 m long rock bolts to make the tunnel stable.
7. Results from CMRR and ARBS approaches coincided with FEM analysis and RMR based results.

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