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TUNNEL SUPPORT DESIGN BY COMPARISON OF EMPIRICAL AND FINITE ELEMENT ANALYSIS OF THE NAHAKKI TUNNEL IN MOHMAND AGENCY, PAKISTAN

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Abstract: The paper analyses the geological conditions of study area, rock mass strength parameters with suitable support structure propositions for the under construction Nahakki tunnel in Mohmand Agency. Geology of study area varies from mica schist to graphitic marble/phyllite to schist. The tunnel ground is classified and divided by the empisical classification systems like Rock mass rating (RMR), Q system (Q), and Geological strength index (GSI). Tunnel support measures are selected based on RMR and Q classification systems. Computer based finite element analysis (FEM) has given yet another dimension to design approach. FEM software Phase2 version 7.017 is used to calculate and compare deformations and stress concentrations around the tunnel, analyze interaction of support systems with excavated rock masses and verify and check the validity of empirically determined excavation and support systems.

Key words: *rock mass strength parameters, rock mass classification, FEM analysis, Phase2, tunnel support design*

1. INTRODUCTION

The aim of this research is to analyze the geological parameters of the rock mass and to obtain rock mass strength parameters, finally suggesting tunnel support measures of the under construction Nahakki Tunnel situated on the road Ghalanai–Mohmand gat from RD 15 + 010 at El. 827.00 m and RD 15 + 670 at El. 820.00 m in Mohmand Agency (Fig. 1).

Mohmand Agency is situated in a complicated terrain at the foothills of Himalayas. Geologically, the Mohmand Agency comprises Paleozoic rocks, largely unclassified. A great variety of metamorphic rocks is exposed in the region ranging from mica schist, schists and phyllites to quartzite and marbles (Technical feasibility report Nahakki road tunnel-NESPAK; National Engineering services Pakistan private limited).

The Tunnel is situated 10 km north of Ghalanai¸ on the road Ghalanai–Mohmand gat in northern Pakistan. Frontier Works Organization (FWO) is main contractor with NESPAK as consultant/designer. Nahakki Tunnel is a D shaped single tube bi-directional two lane road tunnel. Currently, the tunnel is under construction and it has achieved excavation of 400 meters approximately out of the total 660 meters. The Tunnel is 10.50 meters wide with semicircular roof arch of 5.1 meter radius resting on 3 meter high walls.

A detailed geological face mapping is being carried out during construction phase. To conclude the geotechnical properties of the surface/sub-surface rock mass along the tunnel alignment, field quantities (i.e., lithological documentation, determination of discontinuity characteristics, etc.), geological face mapping (determination of lithological units, rock discontinuities, rock quality designation (RQD) (Palmström [1]), joint conditions including roughness, persistency, aperture, weathering, etc.) were executed. The rock descriptions include both rock mass and rock material characteristics based on (Brown [5]). Additional geological face mapping practice was done for this research paper. The entrance section of Nahakki tunnel area is characterized by the presence of schists, dolomitic marble, quartzite and phyllites. It is observed during the site geological appraisal that the schist is predominant at the southern half of the proposed tunnel. Schist is very weak to moderately weak with moderately to highly weathering effect,

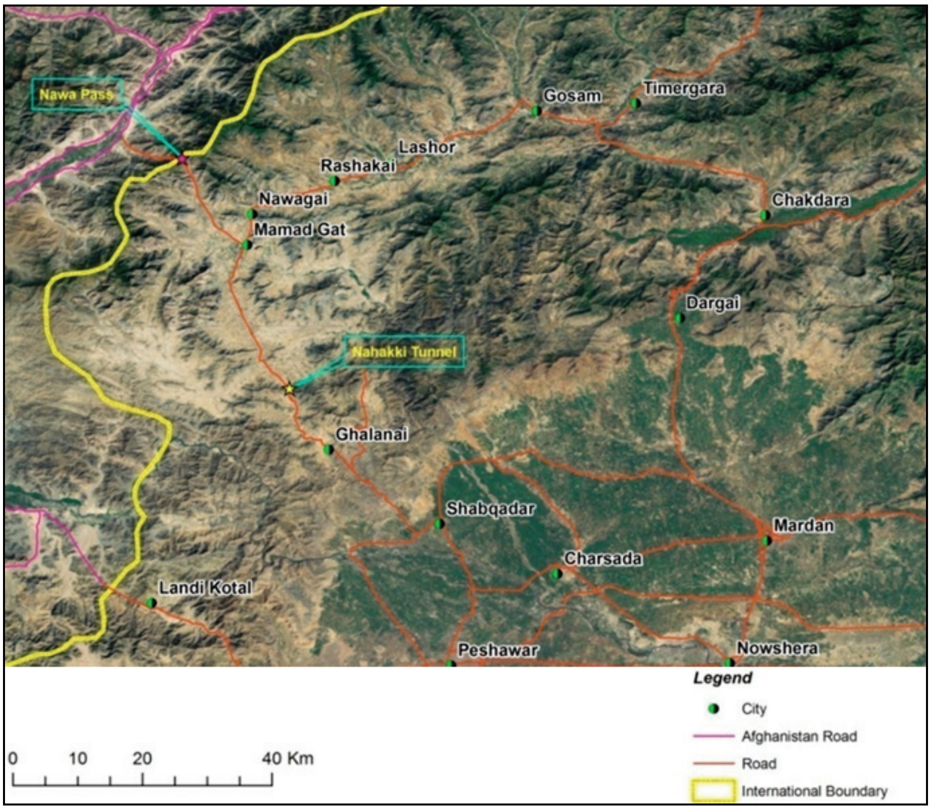


Fig. 1. Project location map

infrequently strong metamorphic rock. This formation is effortlessly disjointed along the foliation planes, with high persistency. The rocks across the proposed tunnel alignment are characterized by the presence of well developed, three to three plus random joint sets. The major controlling joint set is the one parallel to schistosity, which is generally dipping 40° or more with an orientation in the space 290° (Bieniawski [3]). Joint roughness forms important basis of the rock mass classification and also for the in-situ shear strength estimation of the joints. The joints particularly in the schists are smooth-planar with a few exceptions of smooth-undulating. Schist has 0.05 to 1.5 m average joint spacing. The geotechnical units in the northern half of the proposed tunnel are characterized by tight shear folding, which classify the joints mostly as smooth-undulating to occasionally rough-undulating. The joint walls as observed in the field are generally fresh and slightly to highly weathered. Subsurface water cannot be predicted with high certainty levels. It can be anticipated in view of the observations made for seepages during rainy season. However, as per the construction history of the tunnel, much less water had ingress into the tunnel during rains. The strikes of joints are perpendicular or nearly perpendicular to tunnel axis. Therefore, the locations of these discontinuities are evaluated as "favorable" (drive with the dip 20°–45°) in accordance with tunnel excavation from southern side, and "fair" (drive against the dip 45°–90°) in accordance with tunnel excavation from northern side (Bieniawski [3]).

2. TUNNEL SUPPORT DESIGN BY EMPIRICAL ROCK MASS CLASSIFICATION

The Tunnel is divided into five geological classes (GC-1 to GC-5) as shown in Fig. 2a according to the predominant geological strata as revealed in Fig. 2b having geological section of tunnel alignment. RMR (Bieniawski [3]), Q system (Barton et al. [2]), and GSI (Sonmez and Ulusay [12]) rock mass classification system were used for rock mass classification and the tunnel support system was selected according to RMR and Q only. The rock mass classification (RMR, GSI and Q) is shown in Table 2, rock mass geomechanical properties as listed in Tables 1 and 3 are selected from physical face mapping (face mapping sheet of GC-5 is shown in Fig. 3) and the data bank of RocLab (RockLab [11]) based on the generalized Hoek–Brown failure criterion (Hoek et al. [4]) subsequently support system based on Q method is tabulated in Table 4.

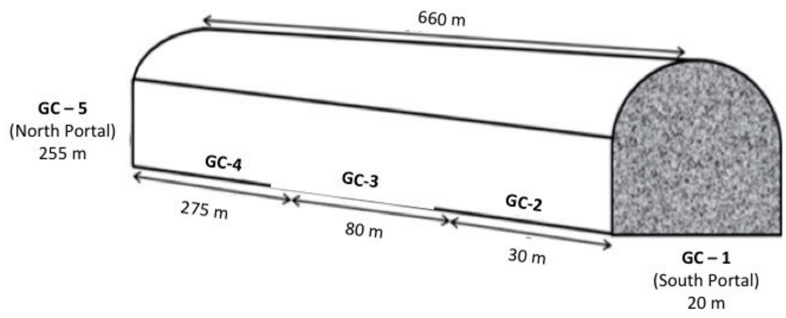


Fig. 2a. Schematic layout of tunnel with geological class division (not to scale)

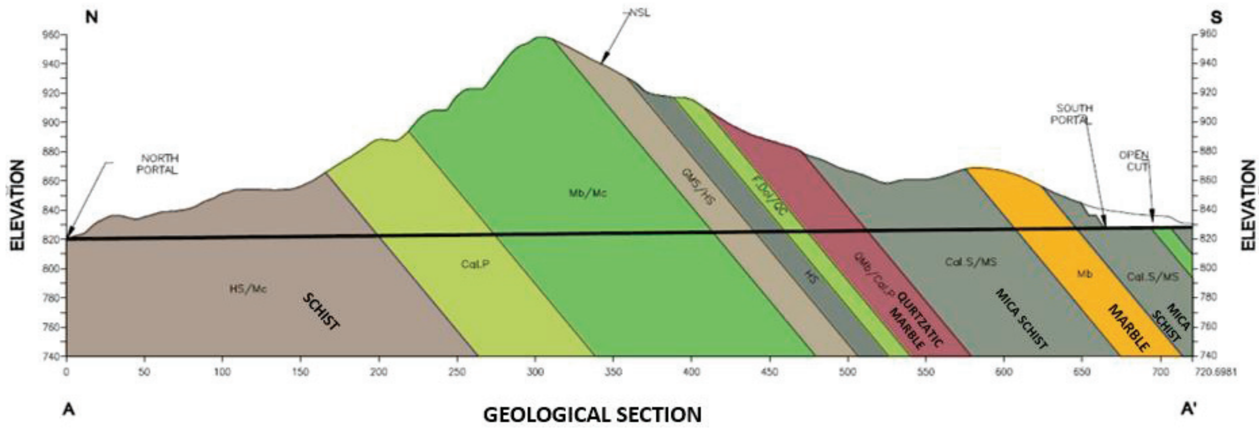
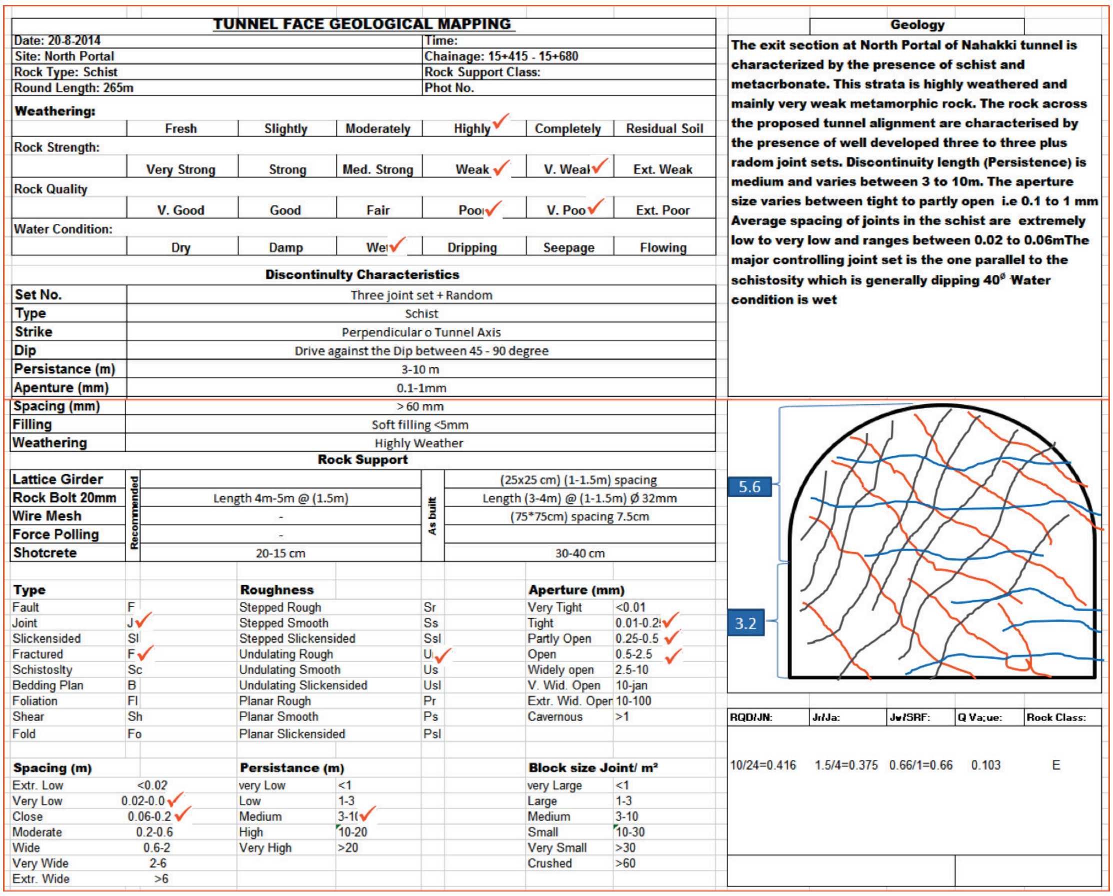


Fig. 2b. Geological section of tunnel alignment



TUNNEL FACE GEOLOGICAL MAPPING

Date: 20-8-2014	Time:
Site: North Portal	Chainage: 15+415 - 15+680
Rock Type: Schist	Rock Support Class:
Round Length: 265m	Phot No.

Weathering:	Fresh	Slightly	Moderately	Highly ✓	Completely	Residual Soil
Rock Strength:	Very Strong	Strong	Med. Strong	Weak ✓	V. Weal ✓	Ext. Weak
Rock Quality	V. Good	Good	Fair	Poo ✓	V. Poo ✓	Ext. Poor
Water Condition:	Dry	Damp	We ✓	Dripping	Seepage	Flowing

Discontinuity Characteristics

Set No.	Three joint set + Random
Type	Schist
Strike	Perpendicular o Tunnel Axis
Dip	Drive against the Dip between 45 - 90 degree
Persistance (m)	3-10 m
Apenture (mm)	0.1-1mm
Spacing (mm)	> 60 mm
Filling	Soft filling <5mm
Weathering	Highly Weather

Rock Support

	Recommended	As built
Lattice Girder		(25x25 cm) (1-1.5m) spacing
Rock Bolt 20mm	Length 4m-5m @ (1.5m)	Length (3-4m) @ (1-1.5m) Ø 32mm
Wire Mesh	-	(75*75cm) spacing 7.5cm
Force Polling	-	
Shotcrete	20-15 cm	30-40 cm

Type		Roughness		Aperture (mm)	
Fault	F	Stepped Rough	Sr	Very Tight	<0.01
Joint	J ✓	Stepped Smooth	Ss	Tight	0.01-0.2 ✓
Slickensided	Sl	Stepped Slickensided	Ssl	Partly Open	0.25-0.5 ✓
Fractured	F ✓	Undulating Rough	U ✓	Open	0.5-2.5 ✓
Schistoslty	Sc	Undulating Smooth	Us	Widely open	2.5-10
Bedding Plan	B	Undulating Slickensided	Usl	V. Wid. Open	10-jan
Foliation	Fl	Planar Rough	Pr	Extr. Wid. Oper	10-100
Shear	Sh	Planar Smooth	Ps	Cavernous	>1
Fold	Fo	Planar Slickensided	Psl		

Spacing (m)		Persistance (m)		Block size Joint/ m²	
Extr. Low	<0.02	very Low	<1	very Large	<1
Very Low	0.02-0.0 ✓	Low	1-3	Large	1-3
Close	0.06-0.2 ✓	Medium	3-1(✓	Medium	3-10
Moderate	0.2-0.6	High	10-20	Small	10-30
Wide	0.6-2	Very High	>20	Very Small	>30
Very Wide	2-6			Crushed	>60
Extr. Wide	>6				

Geology

The exit section at North Portal of Nahakki tunnel is characterized by the presence of schist and metacrbonate. This strata is highly weathered and mainly very weak metamorphic rock. The rock across the proposed tunnel alignment are characterised by the presence of well developed three to three plus radom joint sets. Discontinuity length (Persistence) is medium and varies between 3 to 10m. The aperture size varies between tight to partly open i.e 0.1 to 1 mm Average spacing of joints in the schist are extremely low to very low and ranges between 0.02 to 0.06mThe major controlling joint set is the one parallel to the schistosity which is generally dipping 40ø Water condition is wet

RQD/JN:	Jr/Ja:	Jw/SRF:	Q Va;ue:	Rock Class:
10/24=0.416	1.5/4=0.375	0.66/1=0.66	0.103	E

Fig. 3. Geological face mapping sheet for GC-5

Table 1. Geological analysis data (rock mass/joint properties)

Rock mass Properties	GC1	GC2	GC3	GC4	GC 5
RQD	16	21.5	43.5	19	10
UCS (σ_c) (MPa)	20	50	20	40	20
Spacing of discontinuities	0.05–1.00 m	0.05–1.00 m	0.05–1.00 m	0.05–2.00 m	> 60 mm
Persistence (m)	1–3	1–3	1–3	3–10	3–10
Aperture (mm)	0.1–1	0.1–1	0.1–1	0.1–1	0.1–1
Roughness	slightly rough	slightly rough	slightly rough	rough	slightly rough
Infilling	hard filling <5 mm	hard filling <5 mm	hard filling <5 mm	hard filling >5 mm	soft filling <5 mm
Weathering	moderately weathered	slightly weathered	slightly weathered	slightly weathered	highly weathered
Groundwater condition	dry	damp	damp	wet	wet
Discontinuity orientation	favorable	favorable	favorable	favorable	fair
Joint set number	3 joint set	3 joint set	2 joint set + random	3 joint set + random	3 joint set + random

Table 2. RMR, Q and GSI values along Nahakki Tunnel

Geological class (GC)	Lithology	RMR	Q	GSI
GC-1: South portal (20 m)	Mica schist	46	0.425	38
GC-2 (30 m)	Marble	43	1.183	40
GC-3 (80 m)	Mica schist	48	1.435	42
GC-4 (275 m)	Quartzatic marble/phyllite	33	0.392	32
GC-5: North portal (255 m)	Schist	23	0.103	26

Table 3. Summary of geomechanical properties of rock mass sections along Nahakki Tunnel

South portal ⇩	Rock mass properties			Overburden (m)	Hoek–Brown parameters		Rock mass parameters (undisturbed rock)			Rock mass parameters (disturbed rock)		
	σ_c (MPa)	E_i (MPa)	γ (MPa)		m_i	D	E_m (MPa)	m	s	E_m (MPa)	m	S
GC-1	20	13500	0.026	35	15	0.6	1880	1.638	0.0010	802	0.634	0.0002
GC-2	50	42500	0.026	40	9.3	0.6	6785	1.091	0.0013	2838	0.436	0.0002
GC-3	20	13500	0.026	45	15	0.6	2470	1.89	0.0016	1018	0.778	0.0003
GC-4	40	22000	0.026	50	13	0.6	2040	1.146	0.0005	1172	0.405	0.0001
GC-5	20	13500	0.026	40	12	0.4	857	0.854	0.0003	547	0.441	0.00007

Table 4. Summary of support systems of Nahakki Tunnel according to Q System

Support systems		GC-1	GC-2	GC-3	GC-4	GC-5
Q Value		0.46	1.18	1.44	0.39	0.10
Rock class		E	D	D	E	E
Support cat		27	23	23	31	32
Rock bolt	Dia (mm)	20	20	20	20	20
	Spacing (m)	1	1–1.5	1–1.5	1	1
	Length (m)	4	4	4	4	4
Shotcrete (cm)		7.5–10	5–10	5–10	5–12.5	20–40
Unsupported span (m)		1.42	2.2	2.3	1.4	0.8

3. VERIFICATION OF TUNNEL SUPPORT BY FINITE ELEMENT MODELING SOFTWARE PHASE2

The aim of the finite element method analysis is to verify the empirically (Nghia and Kristina [7]) evaluated tunnel support design given in Table 4. For this purpose, the FEM software module Phase2 was applied to evaluate induced stresses and maximum deformation for excavated portion of the tunnel and to examine the proposed tunnel support. Excavation section will be D shaped 12 m × 8.5 m. Complete tunnel has been divided into sections according to the rock mass geotechnical properties assessed through geological face mapping. Q system classifies the tunnel with dominating E class, i.e., very poor (Barton et al. [2]) towards North portal. Class E is also expected to be faced for weak zone of mica schist. The Tunnel was divided into five geological classes (GC-1 to GC-5) to be used as Phase2 models to compute and interpret the stresses and deformations leading to proposed support elements. The worst geological conditions are encountered by GC-5 as a result of Q classification shown in Table 4 and geological face mapping of GC-5 is shown in Fig. 3. For this paper, numerical analysis is performed only for the worst geological portion, i.e., GC-5. Actual dimensions of the tunnel opening are drawn in Autodesk and then imported into Phase2 as excavation opening. Finite element mesh is built around the opening with extension factor as per requirement. For Nahakki tunnel, excavation opening with finite element mesh is shown in Fig. 4.

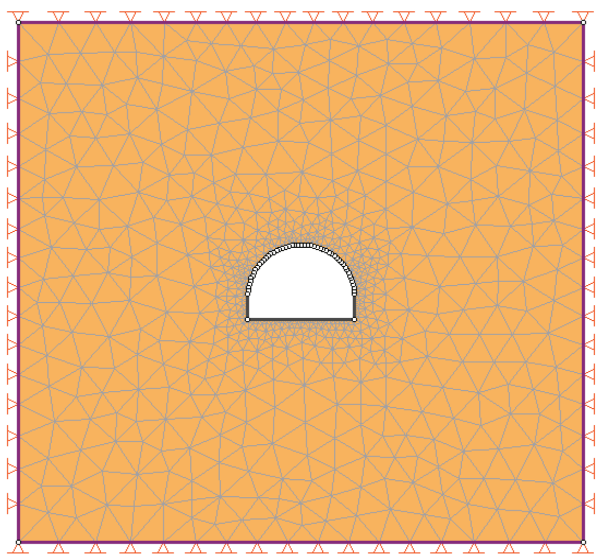


Fig. 4. Excavation section modeled in finite element mesh

Stresses in field conclude the preliminary conditions of in-situ stress as pre excavation state. As a practice, σ_1 and σ_3 are taken as in plane in-situ major and minor principal field stresses, respectively, and σ_2 as out of plane stress. At Nahakki tunnel, σ_1 and σ_3 are acting in horizontal direction. Vertical stresses have been calculated related to the depth below the surface and unit weight of the rock. Plate tectonic movements, which produce tectonic stresses, generate horizontal stresses, and these can be much higher than vertical stresses. This anisotropy in the stress field is represented by initial stress ratio K ($\sigma_h = K\,\sigma_v$). Two dimensional tunnel modeling is assumed to have in-situ stress ratio (k) near to unity, so as to achieve stable tunnel closure at the end of model (Vlachopoulos and Diederichs [13]), however, for this paper value for in-situ stress ratio has been taken from the stress ratio chart formulated by Hoek and Brown in 1978. Calculation of in-situ field stresses for this study has been shown in Table 5. Snap window as shown in Fig. 5 reflects the input stresses used for finite element modeling.

Table 5. Field stresses

Section	Overburden Z (m)	γ (MPa)	σ_v (σ_3) (MPa)	K (100/Z + 0.3)	σ_h (σ_1) (MPa)	σ_2 (MPa)
GC-1	35	0.026	0.91	3.16	2.87	2.35
GC-2	40	0.026	1.04	2.8	2.91	2.4
GC-3	45	0.026	1.17	2.52	2.95	2.45
GC-4	50	0.026	1.3	2.3	3	2.5
GC-5	40	0.026	1.04	2.8	2.91	2.4

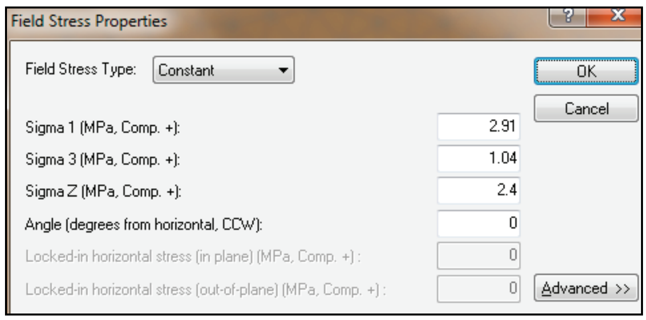


Fig. 5. In-situ field stresses input properties to software

In-situ stresses are taken as the category of constant loading for evaluation with average overburden height of 55 meters. The Hoek–Brown failure criterion was selected for FEM analysis (Hoek et al. [4]). The class E excavation is simulated through ten stages. Internal pressure equal to in-situ stresses was applied. Internal pressure factor 1 at Stage 1 means the magnitude of internal pressure will be the same as the field stress, while factor 0 means no load will be applied at that stage (Rocscience Inc. 2006). Other values of factor will be decreased gradually between stage 2 to stage 9 as shown in Fig. 6 and model at stage 1 will look as shown in Fig. 7.

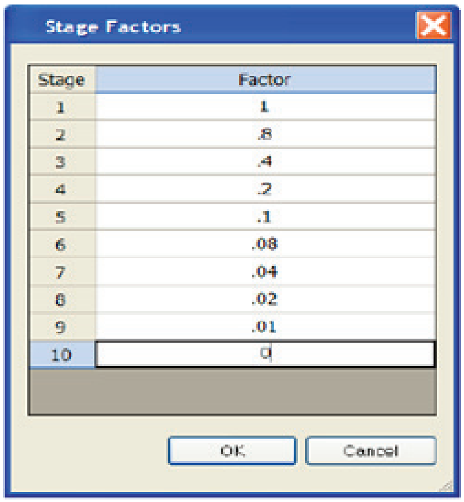


Fig. 6. Internal pressure stage factor

Phase2 computation engine will evaluate total displacement at each stage and maximum displacement will be at stage 10, as the internal pressure is zero. In this study, the maximum displacement was 0.101 m at stage 10 as shown in Fig. 8. The plot in Fig. 9 was created using the Vlachopoulos and Diederichs equations (Kersten [6]). Using this plot, it can easily estimate the amount of closure prior to support installation if the plastic radius and displacement far from the tunnel face are known.

To estimate the amount of closure prior to support installation, distance from the tunnel face is 1 m. The radius of plastic zone R_{pz} is 19 m, the tunnel radius R_t is 5.6 m (Fig. 9) and maximum displacement u_{max} = 0.101 m (Fig. 8).

The distance from tunnel face/tunnel radius (D_{ft}/R_t) = 1/5.6 = 0.178. The plastic zone radius/tunnel radius (R_{pz}/R_t) = 19/5.6 = 3.39. Figure 10 depicts maximum closure of 0.33, here closure equals C_p = (0.33)*(0.101) = 0.033 m prior to support installation. This shows the tunnel displaces 0.033 m before the support is installed.

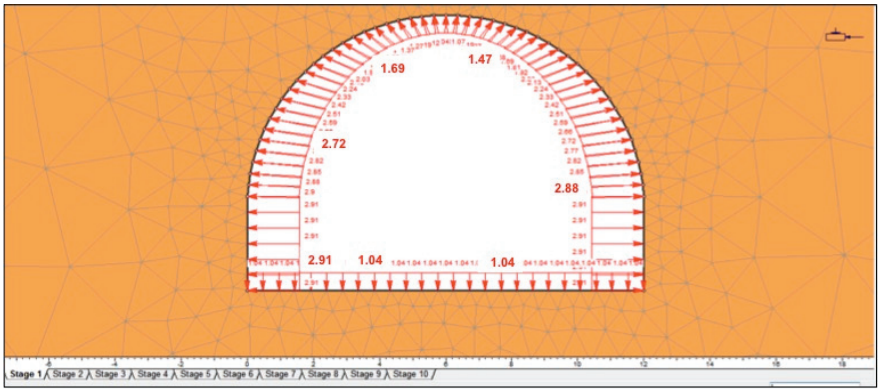


Fig. 7. Internal pressure distribution at stage 1

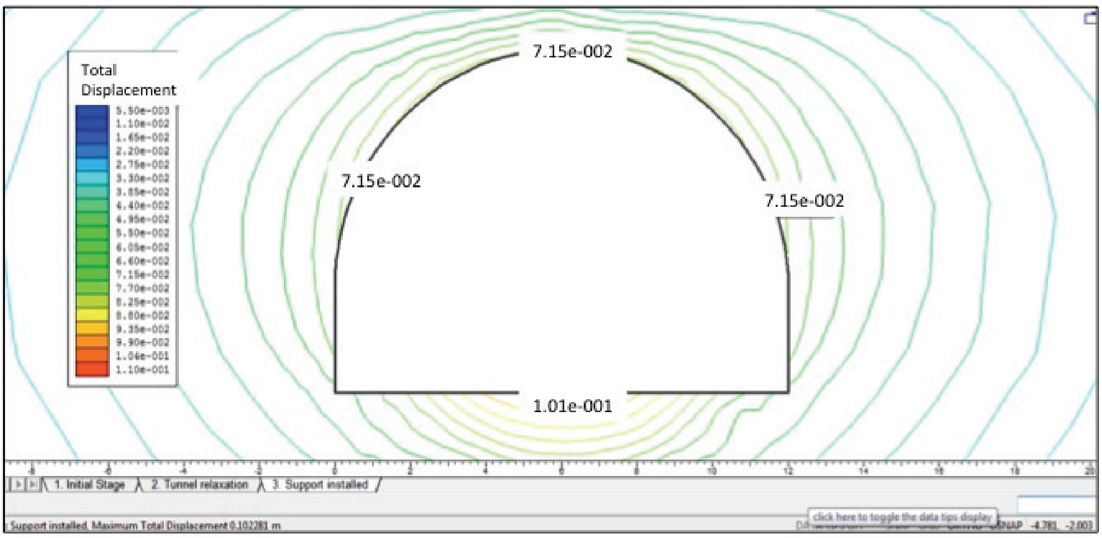


Fig. 8. Total displacements at the 10th stage

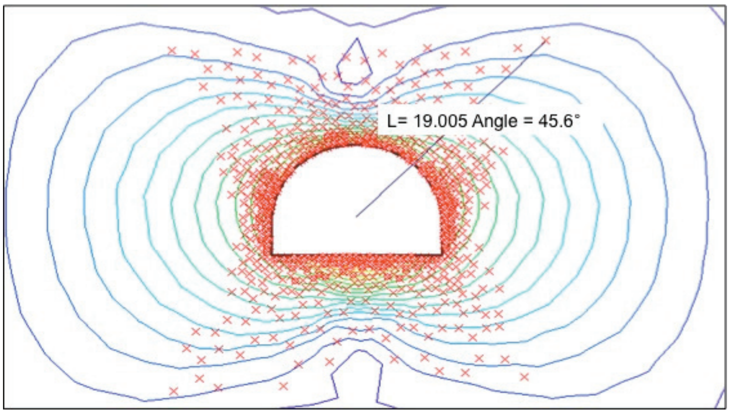


Fig. 9. Yielded zone (radius of plastic zone R_{pz} – 19 m)

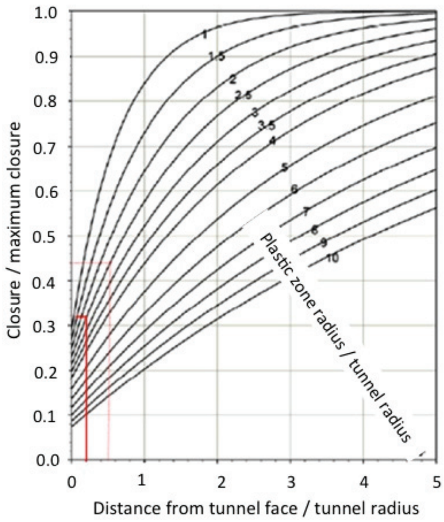


Fig. 10. Maximum closure of 0.33 m

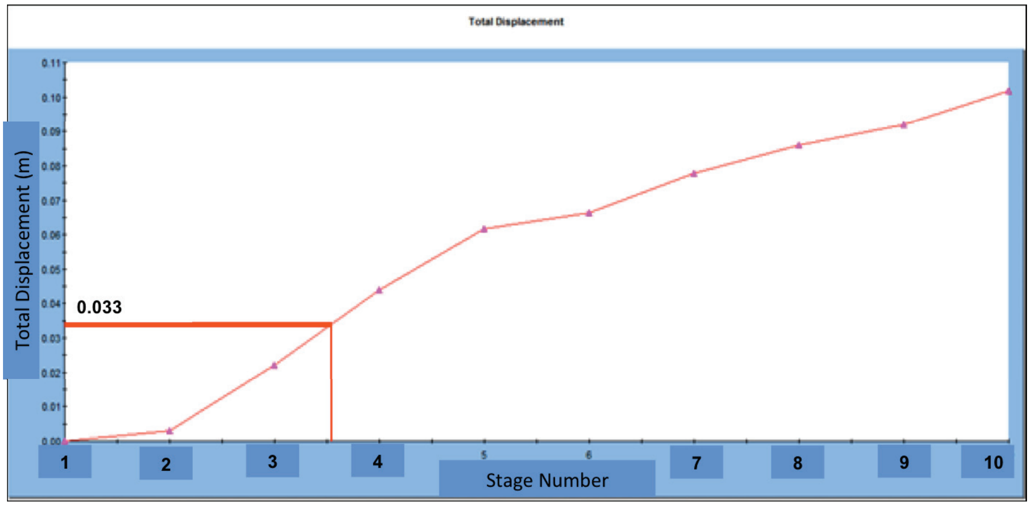


Fig. 11. Total displacement vs. stage number

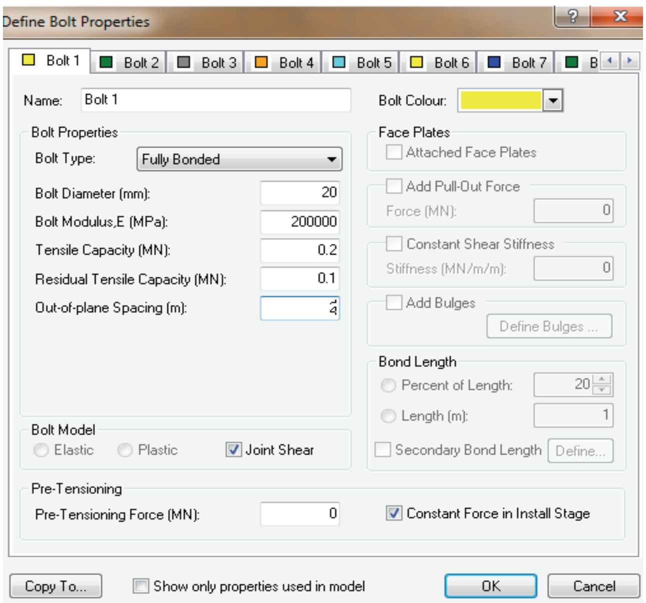


Fig. 12. Rock bolt properties

To determine the internal pressure that yields 0.033 m displacement, Fig. 11 shows the plot of displacement vs. stage excavation generated by the program. From this plot, at stage 4, the wall displacement is equal to 0.033 m, therefore support will be installed at stage 4.

Addition of rock bolts (4 meters in length each with grid spacing of 1 × 1 meter having load capacity of 0.2 MN as illustrated in Fig. 12 depicting input rock bolt properties to Phase2 model) normal to the boundary reduces displacement to 0.0638 m from 0.101 m is shown in Fig. 13.

To check the capacity of rock bolt, the maximum axial force on bolt # 15 is 0.195 MN as shown in Figs. 14 and 15, which is very close to the capacity of rock bolt, i.e., 0.2 MN.

Next category in support element is shotcrete. Phase2 takes this as liner with the properties of 30 cm thickness with peak compressive strength of 35 MPa and Young's modulus of 30000 MPa. Snapshots of window showing shotcrete liner parameters for Phase2 input are shown in Fig. 16.

By adding liner of 30 cm thickness, the displacement is further reduced to 0.0578 m as shown in Fig. 17.

Rock bolt number 33 is taking maximum axial force of 0.131 MN at floor of the tunnel as shown in Fig. 18. It is evident that loads taken by support elements are well within the maximum capacity of rock bolt (0.2 MN).

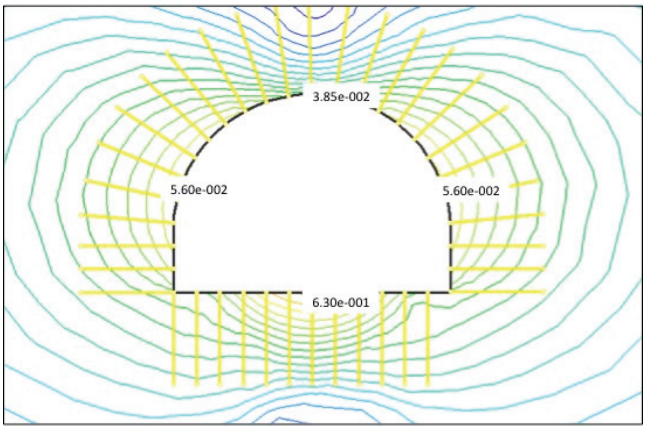


Fig. 13. Distribution of the maximum displacements (0.0638 m) with addition of rock bolts

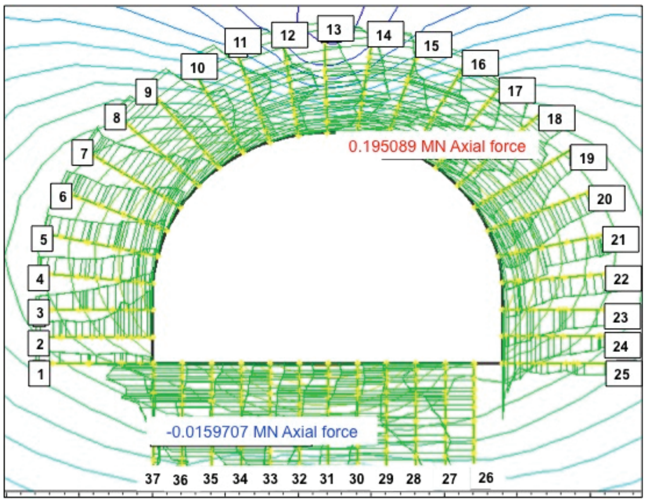


Fig. 14. Maximum and minimum axial force on rock bolt

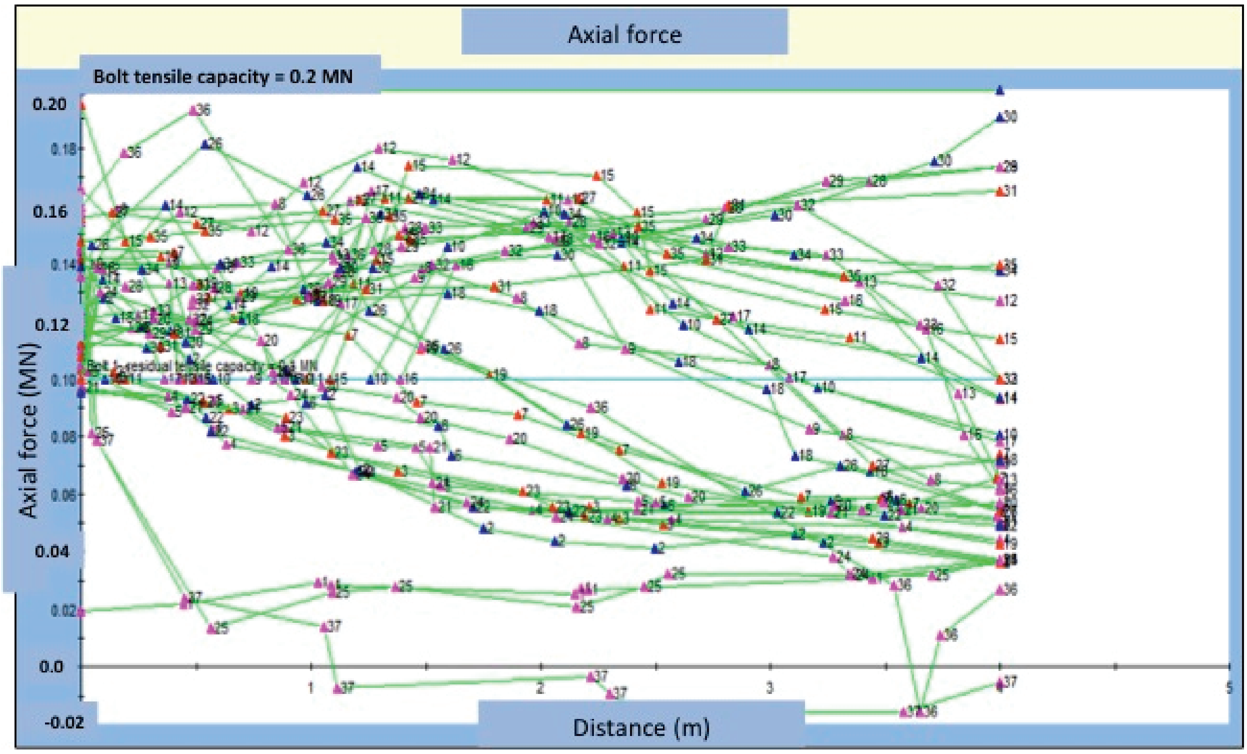


Fig. 15. Axial force on rock bolt with respect to its length inside rockmass

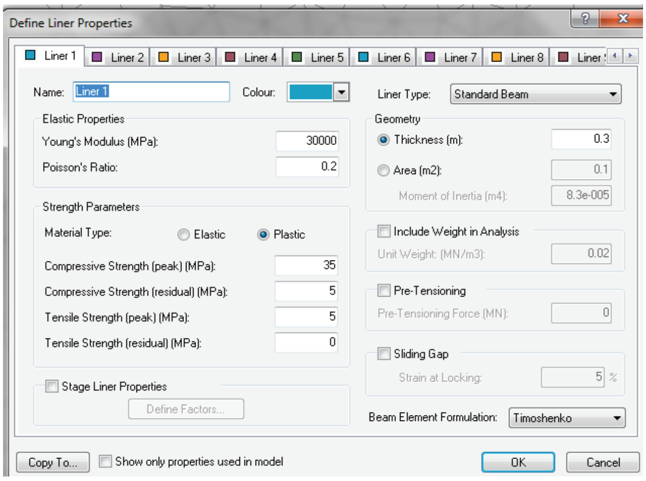


Fig. 16. Liner (shotcrete properties)

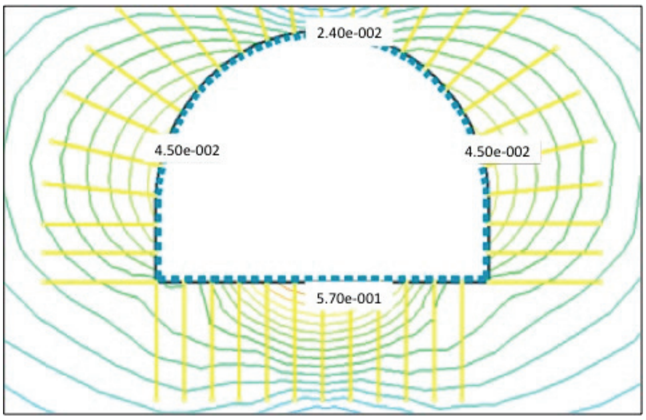


Fig. 17. Distribution of the maximum displacements (0.0578 m) with addition of shotcrete

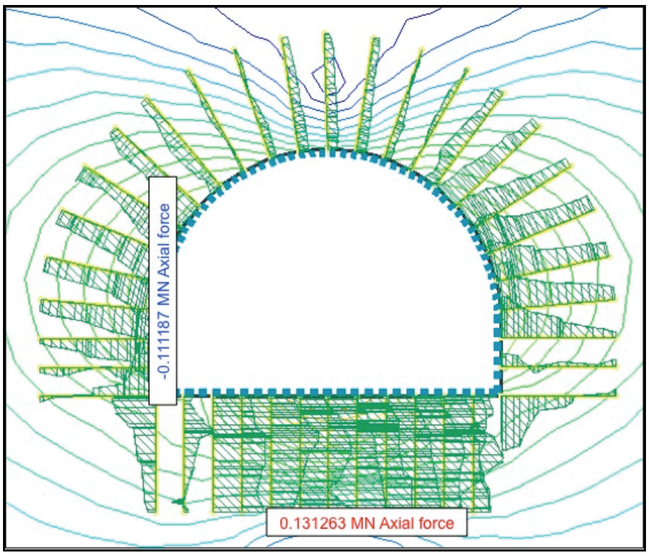


Fig. 18. Maximum and minimum axial force on rock bolt

4. CONCLUSION

This paper takes into account the geological characteristics of rock mass pitched with its geomechanical characteristics and suggests the suitable tunnel support systems. RMR, GSI and the Q rock mass classification systems were used. RocLab database was benefited to evaluate Hoek and Brown constants *m* and *s* and elastic modulus of rock masses. Five geological classes of Nahakki tunnel demonstrate poor to very poor quality rock mass. FEM software package, Phase2 is used to determine the induced stresses, deformations with proposal for support elements for the worst rock class E. Taking into consideration the geomechanical conditions of rock class E, load coming onto the excavated section and load carrying capacity of suggested support, it is determined that the suggested support for E Rock Class of Nahakki Tunnel is adequate.

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