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Engineering Geological Assessment and Preliminary Support Design for Cankurtaran Tunnel, NE Turkey

Ayberk Kaya [1]*, Fikri Bulut [2]
[1] Recep Tayyip Erdogan University, Department of Geological Engineering, Rize
[2] Karadeniz Technical University, Department of Geological Engineering, Trabzon
**Corresponding Author: ayberk.kaya@erdogan.edu.tr*

Abstract
The main purpose of this study is to determine the geotechnical properties of the rock masses and to suggest the convenient support design for the Cankurtaran (Hopa-Artvin) tunnel, NE Turkey. The detailed geotechnical investigations were carried to define the rock masses that mainly consist of sedimentary, pyroclastic, and volcanic units. Empirical and numerical methods were utilized and results were compared for safe tunnel design. The RMR, Q, and NATM systems were used as empirical methods to characterize the rock masses and to determine the support design. The performance of suggested support design, induced stress distributions and deformations were controlled using the finite element method (FEM). It was concluded that the empirical support design was not successful to prevent deformations developed around the tunnel in very poor rock masses under great in-situ rock stresses. Therefore, a new support design was suggested and, its performance was also checked using the FEM analyses. Finally, it is concluded that the empirical and numerical methods should be combined for more realistic and safe support design.

Keywords: Tunneling, Rock mass classification, Support design, FEM.

1. Introduction

In this study, the Cankurtaran tunnel (Figure 1) was selected as an application site for empirical and numerical tunnel support procedure. The detailed engineering geological investigations include field and laboratory studies followed by classification of rock masses and numerical modeling to determine the preliminary support design have been carried out. The field studies consisted of geological mapping, drillings, water pressure tests, scan-line surveys, and seismic investigations. The necessary support designs were estimated by means of the RMR, Q, and NATM systems. In addition to the empirical methods, the FEM based numerical analyses were also undertaken in order to define the stress distributions and deformations developed around the tunnel and to control the performance of empirical support design. Further, a new support design was recommended for unstable sections of the tunnel by the help of numerical analyses because designing a safer and economical support system is important in tunneling projects.

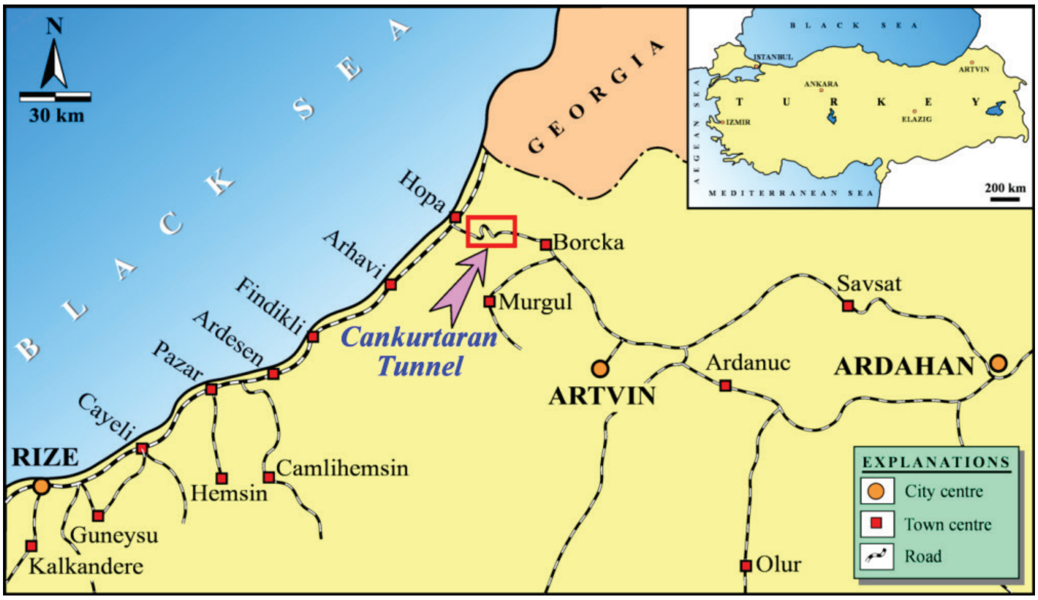

Figure 1. Location map of the study area

2. Geology of the Tunnel Alignment

In the study and surrounding area, geological units vary from old to young are the Late Cretaceous aged Subasi Ridge Formation, Late Cretaceous-Paleocene aged Cankurtaran Formation, Paleocene aged Senkaya Ridge Formation, Eocene aged Kabakoy Formation, and Quaternary aged alluvium (Capkinoglu, 1981; Guven, 1993; Kaya, 2012, Kaya and Bulut, 2013). The Subasi Ridge Formation is a volcano-sedimentary deposit characterized by the andesitic pyroclastites and intercalations of limestone, marl, sandstone, tuff, and siltstone. This formation is commonly exposed at the entrance section of the Cankurtaran tunnel comprising the study area. The Cankurtaran Formation consists of limestone and marly-limestone conformably overlies the Subasi Ridge Formation. This unit crops out in the inner section of the tunnel. Conformably overlying Senkaya Ridge Formation is composed of marl with various colors, limestone with partly red and gray laminate and intercalation of the claystone with thin and medium layered. This unit generally crops out towards the end of inner section of the tunnel. The Kabakoy Formation unconformably overlies the Senkaya Ridge Formation and lithologically consists of basalt, andesite, and their pyroclastites. The Quaternary aged alluvium is the youngest unit in the area. Each geological unit is shown in Figure 2 illustrates the geological cross-section.

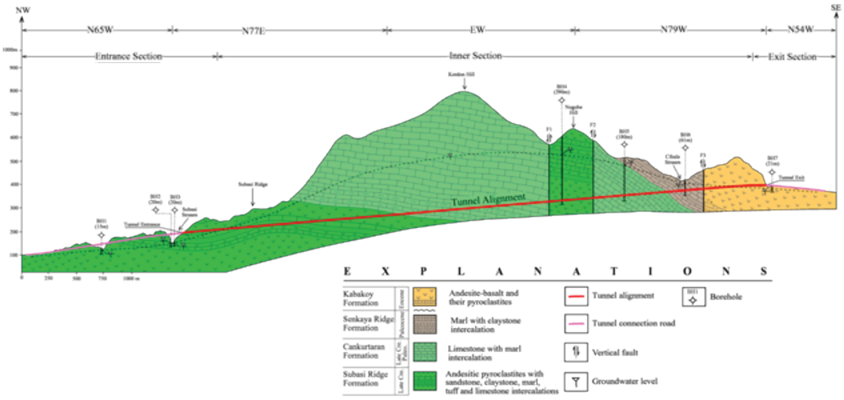

Figure 2. Geological cross-section showing the geological units along the tunnel alignment

3. Engineering Geological Investigations

Engineering geological properties of the rock masses crop out along the Cankurtaran tunnel alignment were determined in three stages comprising surface, subsurface, and laboratory studies. The field studies include detailed geological mapping, scan-line surveys, geophysical, and borehole investigations.

Seven investigation boreholes (Figure 2) having a total length of 607 m were drilled by Turkish General Directorate of Highways (KGM) in order to observe the rock mass characteristics at the tunnel level, to identify the discontinuity properties and groundwater level, sampling for the laboratory tests, and water pressure tests.

Laboratory tests were performed on the core samples taken from boreholes and rock blocks in accordance with the methods suggested by ISRM (2007) to determine the physico-mechanical and elastic properties of rock materials. Furthermore, the Rock Quality Designation (RQD) values were identified from boreholes and scan-line surveys using the techniques suggested by Deere (1964) and Priest and Hudson (1976).

The quantitative description of the discontinuities in the geotechnical units such as orientation, spacing, persistence, infilling, roughness, aperture, and weathering degree were defined by analyzing the cores and scan-line surveys according to ISRM (2007) suggested method.

The geophysical studies were performed by using the seismic refraction method in six lines to determine the dynamic Poisson's ratio (ν) of each geological unit. The dynamic Poisson's ratio values were calculated using the method proposed by Bowles (1988).

In order to determine rock mass parameters such as Hoek-Brown constants (mb, s, a), deformation modulus (Em), and uniaxial compressive strength (σcm), Hoek-Brown failure criterion suggested by Hoek et al. (2002) was used. The post-peak behavior of geological units upon tunnel excavation was determined with the help of method recommended by Cai et al. (2007). The most commonly utilized rock mass classification systems such as RMR, Q, and NATM were employed to characterize the geological units along the tunnel alignment and to conduct empirical support design. In this study, the latest version of the RMR system (Bieniawski, 1989) was considered. According to the RMR system, the geological units along the tunnel route were classified as poor and very poor rock masses. In terms of the Q system (Barton et al., 1974), the geological units were classified as extremely poor and very poor rock masses. The main rock mass classes based on the NATM (Ö-NORM B2203, 1994) for the geological units were classified as B3/rolling and C1/rock bursting (Table 1).

In addition to this, the empirical tunnel support design (Table 1) was determined using the bolt length, bolt spacing and shotcrete thickness determination charts proposed by Lowson and Bieniawski (2013) and Barton (2002).

Table 1. RMR, Q, and NATM rock mass classifications and empirical tunnel support categories

Classification system	Subasi Ridge Formation	Cankurtaran Formation	Senkaya Ridge Formation	Kabakoy Formation
Basic RMR Adj. RMR Support	46.4 28.0/Poor Systematic rock bolts 6m long, spaced 0.7-0.9m and 250-300mm thick steel fibre/ wiremesh reinforced shotcrete (Sfr+B) in wall and crown	45.2 25.3/Poor Systematic rock bolts 6m long, spaced 0.7m and 250-300mm thick steel fibre/wiremesh reinforced shotcrete (Sfr+B) in wall and crown	26.1 14.8/Very poor Cast concrete arch (CCA) or systematic rock bolts 6m long, spaced 0.5m and ≥300mm thick steel fibre/ wiremesh reinforced shotcrete (Sfr+B) and steel ribs (RRS) in crown and wall	46.9 28.3/Poor Systematic rock bolts 6m long, spaced 0.9m and 250-300mm thick steel fibre reinforced shotcrete (Sfr+B) in wall and crown
Q Support	0.35/Very poor Crown: Systematic rock bolts 4m long, spaced 1.3-1.5m and 120-150mm thick steel fibre/wiremesh reinforced shotcrete (Sfr+B) Wall: Systematic rock bolts 3.5m long, spaced 1.5-1.7m and 90-120mm thick steel fibre reinforced shotcrete (Sfr+B)	0.49/Very poor Crown: Systematic rock bolts 4m long, spaced 1.5-1.7m and 90-120mm thick steel fibre reinforced shotcrete (Sfr+B) Wall: Systematic rock bolts 3.5m long, spaced 1.7-2.1m and 90-120mm thick steel fibre reinforced shotcrete (Sfr+B)	0.015/Extremely poor Cast concrete lining (CCA) or systematic rock bolts 4m long, spaced 1.0-1.2m and ≥250mm thick steel fibre/wiremesh reinforced shotcrete (Sfr+B) and steel ribs (RRS) in crown and wall	0.30/Very poor Crown: Systematic rock bolts 4m long, spaced 1.3-1.5m and 120-150mm thick steel fibre reinforced shotcrete (Sfr+B) Wall: Systematic rock bolts 3.5m long, spaced 1.5-1.7m and 90-120mm thick steel fibre reinforced shotcrete (Sfr+B)

NATM Support	B3/Rolling Utilization of systematic support and local forepoles	B3/Rolling Utilization of systematic support and local forepoles	C1/Rock bursting Utilization of systematic support in roof and wall	B3/Rolling Utilization of systematic support and local forepoles

4. Numerical Analyses and Proposed Tunnel Support Design

In order to determine the induced stresses, deformations, and developed plastic zones around the tunnel and to verify the results of the empirical methods, the FEM based computer software Phase2 v8.0 developed by Rocscience (2011) was used in the numerical analyses. An automatic mesh around the tunnel was generated and based on the elasto-plastic analysis, stresses and deformations were computed in this program. In order to analyze the deformations and tunnel stability and, to explore the concept of rock support interaction, a very simple model was used. Six-noded triangular finite elements were chosen in the mesh and finer zoning was applied around the excavation. The planned length of the double tube tunnel is 5.288 m and the excavation section is modified horse shoe-shaped with 12 m width and 9 m height. The excavation boundary of tunnel was generated considering its width and height in two stages as top heading and bench for the Subasi Ridge Formation, Cankurtaran Formation and Kabakoy Formation and, top heading, bench and invert for the Senkaya Ridge Formation according to construction procedure of the NATM (KGM, 2013). To simulate the tunnel excavation in all geotechnical units, four finite element models were generated using same tunnel geometry, mesh and different material properties. Hoek-Brown failure criterion was used to determine the plastic zones and yielded elements in the vicinity of tunnel. Material properties of the geological units used in the numerical analyses are given in Table 2.

The numerical analyses were performed in two steps as unsupported and supported cases for each geological unit considering groundwater condition and seismic effect. Taking into account the distance of the Black Sea Fault, the peak ground acceleration (PGA) value was determined to be 0.14 g using the Ulusay et al. (2004) attenuation relationship suggested for Turkey. Thus, a seismic loading of 0.14 g was applied into the analyze model. In the first step, following the examination of in-situ stress distributions, the yielded points, principal stress distributions, and induced displacements developed around the tunnel excavations were determined. Further, the maximum thickness of the plastic zones and total displacements were examined. In the second step, the performance of the empirical support design obtained from the RMR and Q classification systems were investigated using the same analyze model (Figure 3).

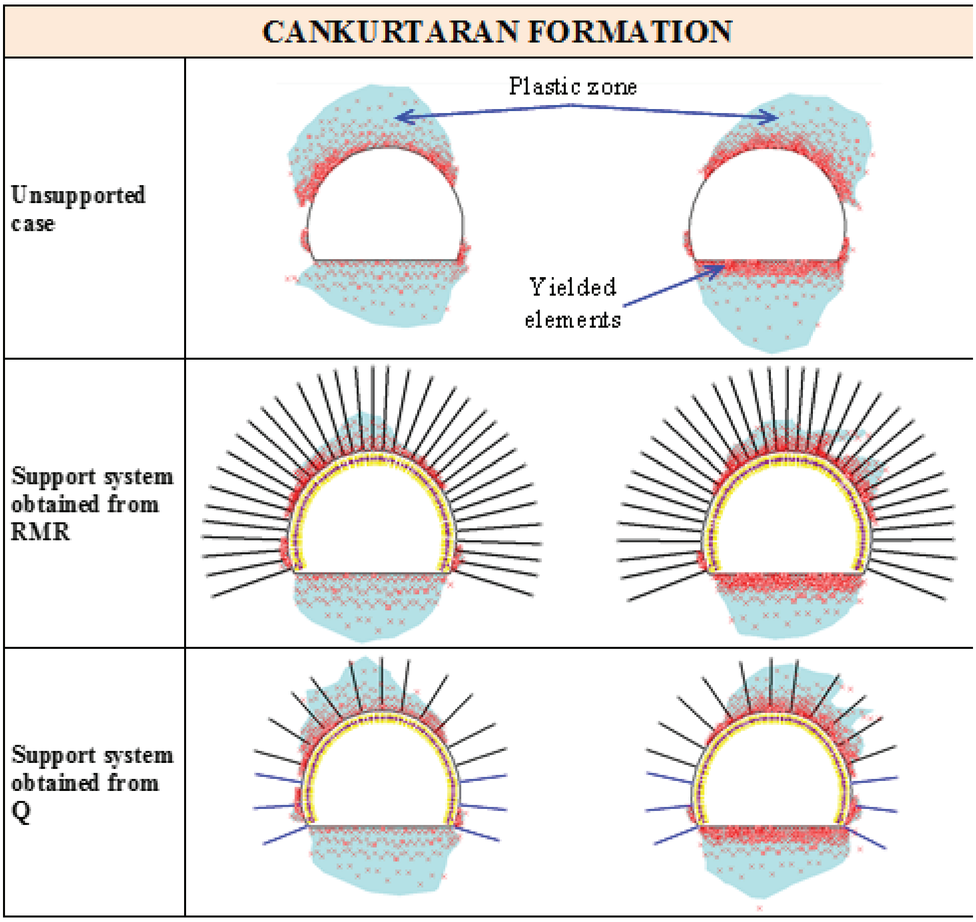

Figure 3. Numerical analyses showing the plastic zones developed in Cankurtaran Formation for unsupported and supported cases

It was concluded that the suggested empirical support designs are sufficient to provide the stability in the Subası Ridge Formation. It was observed that the support design suggested for the Cankurtaran Formation are not enough to reduce the maximum thickness of the plastic zones occurred in both roof and wall (Figure 3). On the other hand, the empirical support designs suggested by Q system were determined to be insufficient to reduce the plastic zones developed around the excavation boundary in the Senkaya Ridge Formation whereas all support designs were found to be applicable to prove the stability in the Kabakoy Formation.

Table 2. Material properties of the geological units for numerical analyses

Property	Subasi Ridge Formation	Cankurtaran Formation	Senkaya Ridge Formation	Kabakoy Formation
Elastic type	Isotropic	Isotropic	Isotropic	Isotropic
Rock mass strength (σ_{cm}, MPa)	14.00	11.66	1.44	19.99
Deformation modulus (E_m, GPa)	6.93	15.41	0.44	13.69
Poisson's ratio, υ	0.40	0.26	0.31	0.39
Material type	Plastic	Plastic	Plastic	Plastic
mi constant	7	8	7	25

m_b constant	0.93	2.06	0.34	5.78
s constant	0.0019	0.0147	0.0001	0.0105
a constant	0.509	0.502	0.561	0.503
m_{br} residual constant	0.47	0.59	0.31	1.83
s_r residual constant	0.00022	0.00030	0.00006	0.00029
a_r residual constant	0.533	0.527	0.573	0.528
Disturbance factor (D)	0	0	0	0
Dilation parameter	0	0	0	0
Vertical stress (σ_v, MPa)	2.30	12.80	1.82	2.97
Horizontal stress (σ_h, MPa)	1.53	4.50	0.82	1.90

The numerical analyses results indicate that the support designs suggested by the RMR and Q systems are not used safely to provide the stability of tunnel roofs in the Cankurtaran Formation having very poor quality under great field loading. Therefore, the support designs have to be revised for this formation. In order to determine the optimal support design, all support scenarios including different bolt and shotcrete patterns and adding steel ribs were analyzed. Finally, providing the RMR support design given in Table 1 remain same; when the uniaxial compressive strength of the shotcrete was increased from 20 MPa to 45 MPa and tensile strength from 3.1 MPa to 4.7 MPa considering the tunnel specification of the KGM (2013), the extent of the plastic zones reduced to zero (Figure 4 and Table 3).

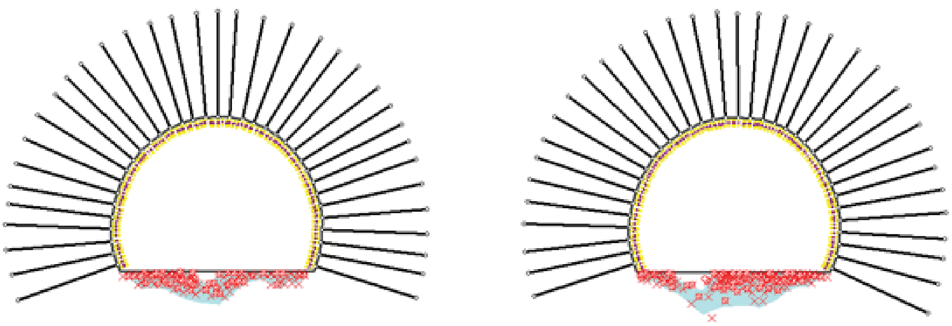

Figure 4. Plastic zones and yielded elements developed after application of the suggested support designs in the Cankurtaran Formation

Table 3. Stresses, displacements and the maximum thickness of plastic zones after revised support installation in the Cankurtaran Formation

		Left tube	Right tube
Roof	σ_1 (MPa)	7.35	6.30
	σ_3 (MPa)	1.70	0.90
Floor	σ_1 (MPa)	1.05	0.00
	σ_3 (MPa)	0.10	0.10
Left wall	σ_1 (MPa)	3.15	4.20
	σ_3 (MPa)	0.50	0.50
Right wall	σ_1 (MPa)	4.20	3.15
	σ_3 (MPa)	0.50	0.50
	T_{pl} (m)	0.00	0.00
	U_t (m)	0.0147	

σ_1 : Maximum principle stress (MPa)
σ_3 : Minimum principle stress (MPa)
U_t : Maximum total displacement (m)
T_{pl} : Maximum thickness of plastic zone (m)

5. Results

In this study, preliminary support design of the Cankurtaran tunnel was investigated. The rock masses along the tunnel alignment were characterized by means of the RMR, Q and NATM rock mass classification systems. The rock masses were classified as ranging from extremely poor quality rock mass to poor quality rock mass. In order to estimate the appropriate support requirements related to geotechnical rock mass parameters for the tunnel, these classification systems were also employed. FEM method was undertaken to check the validity of the empirical preliminary tunnel support requirements. Phase2 software was used to determine the induced stresses, maximum total displacements, and thickness of plastic zones developed around the rock masses surrounding the tunnel. According to results, the empirical support design recommended for the Cankurtaran Formation was not enough to reduce the maximum thickness of the plastic zones occurred in roofs. Therefore, a new support design was recommended using the numerical models considering the all possible support scenarios. When the strengthened shotcrete having an optimum strength value of 45 MPa was applied into the shotcrete liner pattern suggested by the RMR system, the deformations were stopped and the maximum thickness of the plastic zones were decreased to zero.

Consequently, the numerical analyses proved that the empirical support designs did not give the realistic solutions in very poor rock masses under great in-situ rock stresses. Therefore, it is suggested that the validity of the support design recommended by the empirical methods should be controlled prior to excavation phase using the numerical analyses and numerical models should be checked with the monitoring systems during the construction of a tunnel. Further, the rock masses in portals and fault zones should be reinforced with heavier supports in the form of steel sets or cast concrete arches.

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