Evaluation of collapse mechanism and portal interaction of a High-Speed Railway Tunnel (T29 Tunnel, Turkey)

E B Aygar and C Gokceoglu

1 Fugro Sial Geosciences Consulting Engineering, Ankara, Turkey
2 Hacettepe University, Department of Geological Engineering, 06800, Beytepe, Ankara, Turkey

e.aygar@fugro.com

Abstract: Within the scope of the Ankara-Istanbul High Speed Train Project, the T29 tunnel was opened to traffic after completing excavation and support implementation. The tunnel was excavated in weak to very weak graphitic schists causing severe face stability problems and deformations on the tunnel supports during excavation stage. In the progress of the excavation stage, a serious face failure occurred at Km: 225 + 784, and the effects of this face failure were observed on the surface. Since the cover thickness is approximately 35 m in this section of the tunnel, it was decided to pass this zone with a cut and cover structure. The portal excavation was carried out from the failure zone, and it was planned to provide stability with the closure of the surface by constructing a cut-and-cover structure immediately afterwards. However, slope failures occurred during the portal excavations. This section was reworked, and it was decided to support this area with a pile system. Consequently, in this study, the reasons for the face failure and slope failures are investigated, and the construction stages are explained and discussed.

Keywords: Tunnel, High-Speed Railway, Failure, Slope Stability

1. Introduction

T29 tunnel, which is located between Bilecik and Bozüyük within the scope of Ankara-Istanbul High Speed Train Project, is located between Km: 225 + 600 - 225 + 120. The total length of the tunnel is 520 m and the maximum overburden thickness is 40 m. Location map and the satellite image are given in Figures 1 and 2, respectively. During the tunnel excavation, on November 01, 2019, at Km: 225 + 784, the tunnel excavation was interrupted as the collapse in the tunnel face spread to inside of the tunnel. This collapse caused an effect of about 6 x 4 meters on the surface and on the tunnel face (Figures 3 and 4).

The impact of the collapse in the tunnels is observed at the surface, especially in shallow tunnels. Special solutions are required, especially in cohesionless or low cohesion soils (Aygar and Gokceoglu, 2020a). In addition, the most important factor in this cohesionless soil is the stability of the tunnel face and tunnel ceiling (Aygar and Gokceoglu, 2020b, c and d). In order to pass the collapsed section, the main support systems are ground improvement and umbrella system from the surface or in the tunnel (Aygar and Gokceoğlu, 2020e). In addition, tunnel and portal relationship is always an important situation in tunnelling. Because a collapse in the portal significantly affects the tunnel section and causes unexpected damages and time losses. For this reason, tunnel face and ceiling stability should be provided together with tunnel excavation, especially in shallow tunnels.
Figure 1. Location map of the study area

Figure 2. Google earth view of the study area.
2. Geological Characteristics of the Tunnel Route and Geotechnical Parameters

The unit outcropping along the tunnel route consists of green-greenish gray, slightly-moderately weathered, quartz veined, medium-strength chlorite schist (Pzpş) and alluvium (Qal). Discontinuities comprised slightly rough, clay-silica filled, schistocyte surfaces are slickensided. Granite intrusions are observed in two different locations within this formation. The joints in which intrusion developed were subsequently faulted by re-activation. Mylonitic zones are encountered along the fault planes that form the boundaries of the granite intrusion and cut the tunnel route. According to the basic drilling data, the groundwater levels in this section varies between 8.15-33.35 meters and is above the excavation altitude. As can be seen from Figure 5, the tunnel was completely excavated in the schist unit.

![Figure 5. Geological cross section of the tunnel route](image_url)
From the drillings on the route, the boring numbered EKS-41 at Km: 226 + 009 was completely drilled in schists and RQD values were obtained between 0–14%. In the borehole numbered EKS-42, which was opened at Km: 226 + 655, graphite schist - schist - chlorite schist alternation was passed and RQD values were obtained between 0 and 100%. Chlorite schists within the Pazarek complex (Pzpș); light gray - green - greenish gray, medium - very weathered, weakly resistant schists with calcite - quartz veins and lenses have a distinct foliation and the effect of tectonism is obvious. However, the graphite schists are black - dark gray - greenish dark gray color, distinct schistosity, weak strength. The schistosity surfaces in the graphite schists are easily separated, 10.00 meters of marble blocks and 2.00 meters of quartz veins can be observed. Rocks belonging to the unit generally have low permeability - impermeable properties in terms of groundwater.

2.1 Geotechnical parameters

Two boreholes were drilled on the tunnel route by Sial (2010), and the necessary rock mechanics tests were carried out on the core specimens taken from these drillings and the summary table is given in Table 1.

| Borehole No | Start Depth (m) | Finish Depth (m) | Unit Weight (kN/m³) | Point Load Index | UCS (MPa) | Ei (GPa) | Poisson Ratio | m_0 | s | a |
|-------------|----------------|------------------|--------------------|------------------|-----------|---------|----------------|-----|---|---|
| EKS – 41    | 28,83          | 29,00            | 2,57               | 1,35             |
| EKS – 41    | 34,20          | 34,40            | 2,61               | 7,45             | 7,10      | 0,20    |
| EKS – 42    | 18,70          | 18,90            | 2,94               | 2,01             |
| EKS – 42    | 23,00          | 23,30            | 2,61               | 3,35             | 60,0      | 0,20    |
| EKS – 42    | 23,50          | 23,75            | 2,95               | 1,67             |
| EKS – 42    | 24,30          | 24,70            | 2,76               | 1,40             |
| EKS – 42    | 29,60          | 29,75            | 2,76               | 3,74             |

According to the results obtained from laboratory tests, the unit volume weight of the rock mass was chosen as an average of 23 kN/m³ and the uniaxial compressive strength of the rock was chosen as an average of 5.0 MPa and GSI value of this part was predicted as 25, and the material constant as m_i = 10. For the 3 m section around the portal slopes, the disturbance factor was chosen as 0.7, considering that the ground was damaged. Hoek-Brown relative failure criterion constants "m_0", "s" and "a" were calculated to determine the strength of the rock mass.

| Disturbance factor, D=0 | m_0 | s | a |
|-------------------------|-----|---|---|
| 0.687                   | 0.0002 | 0.531 |
| Disturbance factor, D=0.7 | 0.162 | 1.9e-006 | 0.531 |

After this stage, total failure analyses were carried out to pass the remaining part of the tunnel with open excavation. In the analyses, the safety factor was considered as 1.3 for the short term and 1.5 for the long term.
3. Slope Stability Analysis

Analyses were performed with Slide v6.0 program. In these analyses, both the front and side slopes are evaluated. In the stability of the front slope of the tunnel entrance portal, it was examined by considering the static condition (Figures 6-9).

Rock bolts with the length of 12 meters and the diameter of Ø28 mm were used during the slope stability analyses. The pattern of rock bolts was designed as 1.5 x 1.5 meters. Here, it is observed that the safety factor is calculated as 1.33 in the short term. In the side slopes, the critical right-side slope is analysed. In the analyses, short, long term and dynamic conditions were considered in the analyses. The safety factor in the static conditions was calculated as 1.5, the safety factor in the static conditions after the cut-and-cover structure was found to be 2.3 and in the dynamic conditions, it was found as 1.77. Since the area where the T29 tunnel is located is the 1st degree earthquake zone, the maximum horizontal ground acceleration value is 0.4g. In the analysis, the horizontal ground acceleration value was considered as 0.2g while the vertical ground acceleration value was taken into consideration as 0.13.

Figure 6. T29 Entry portal face slope stability analysis (long term, Static)

Figure 7. T29 Inlet portal side slope stability analysis (Static), FS = 1.504
The support systems applied in the T29 tunnel portal slopes are summarized in Table 3. Here, the front slope is taken as 1H: 3V and the side slopes as 1H: 2V. In addition, 5 m of berms are left on each slope. Final excavation plan and support details given in Figures 10 and 11.

**Table 3.** Support details (Sial, 2010)

| Rock Bolts       |                |
|------------------|----------------|
| Hole diameter    | Ø101 mm        |
| Bolt diameter    | Ø28 mm         |
| Bolt Length      | 12 m           |
| Pattern          | 1.5 x 1.5 metre|

| Shotcrete and wire mesh |                  |
|-------------------------|------------------|
| Shotcrete type          | C20/25           |
| Shotcrete thickness     | 150 mm           |
| Wire mesh type          | Q 221/221        |
Figure 11. Support details of portal slopes

4. Problems encountered during the portal excavations

As the portal slope excavations were on-going, excessive precipitation occurred and severe displacements on the slopes observed. While the excavation and supports of the last stage of the portal slopes were maintained, construction was interrupted and in order to prevent the development of the existing movement and the stability of the slopes completely, backfilling was made up to 664 m level. In the first stage, a cut and cover structure was planned to construct after the completion of the portal slopes. However, in the final stage the excavation of the last slope of the tunnel was not completed due to the severe deformations. The final system was designed with a piled system and the cut and cover structure (Figure 12).
5. Conclusions
A high-speed railway tunnel excavation was stopped because the collapse occurred at Km: 225 + 784 during the T29 tunnel excavation. The failure was observed at the surface. In the re-design studies, it was specified appropriate to proceed with an open excavation for the rest of the tunnel due to the low overburden thickness.

In the present study, the re-design studies are presented. During the studies, the slope stability analyses were performed for the portal slopes, and the proper support systems were designed. During the portal excavation phase, the project was revised due to severe deformations. Consequently, the tunnel and portal excavation support operations have been completed successfully, and the tunnel has been opened to traffic.

References
[1] Aygar, E.B, Gökçekoğlu C., 2020a. Problems encountered during a railway tunnel excavations in squeezing and swelling materials and possible engineering measures: A case study from Turkey Sustainability. 12, 1166; doi:10.3390/su12031166 www.mdpi.com/journal/sustainability
[2] Aygar, E.B, Gökçekoğlu C., 2020b. Zayıf Zeminlerde Açılan Büyük Çaplı Bir Tünelin Destek Sistemi Tasarımı (Çukurçayır-2 Tüneli, Trabzon) Design of a Wide Tunnel Excavated In Weak Grounds (Çukurçayır-2 Tunnel, Trabzon), Yer Altı Kaynakları Dergisi, 8 (18), 97-118.
[3] Aygar, E.B, Gökçekoğlu C., 2020c. A Support System For Oceli Highway Tunnel Excavated In Fault Zone (Ordu Ring Road Project, Turkey) Eurock 2020, Trondheim
[4] Aygar, E.B, Gökçekoğlu C., 2020d. Assessment of collapse mechanism of a High-Speed Railway Tunnel (T5 Tunnel of Ankara-Sivas Railway Project, Turkey), Eurock 2020, Trondheim
[5] Aygar, E.B, Gökçekoğlu C., 2020e. Portal and tunnel stability relations in a tunnel (Ankara Sivas High Speed Railway Project, T3 tunnel, Bilimsel Madencilik Dergisi 59(3):157-168, DOI: 10.30797/madencilik.792389
[6] Sial Yerbilimleri Müşavirlik ve Mühendislik Ltd. Şti., 2010. T29 Tüneli Giriş Portalı Kazı Planı ve Destek Sistemi İçin Gerekçe Raporu.