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Monitoring during the construction of a shallow urban tunnel and countermeasures

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ABSTRACT: A shallow urban tunnel has been constructed with conventional method at a residential area in Black Sea Region of Turkey. The excavation and support measures of the tunnel were designed considering soft rock conditions (highly weathered tuff-agglomerate) from site explorations. An extensive monitoring plan was performed to control the design considerations, interaction of the tunnel construction with surrounding structures and influence of the new tunnel excavation on the old tunnel under traffic. During the excavation near to the west portal of the tunnel, deformations and cracks were recorded at the ground surface, fortress masonry walls and one storey buildings inside the fortress area. By evaluating geological documentation, monitoring data and other records such as water discharge measurement, the mechanism of the instability problem was analyzed and required measures were determined. The measures comprised mainly grouting application in limited areas to prevent developing of the instability problems. This paper contains analyzing of data during the tunnel excavation and countermeasures to avoid adverse influence of the tunnel excavation.

1 INTRODUCTION

Design and construction of a shallow urban tunnel requires special considerations due to several reasons. During the determination of the alignment and site investigations local factors limit extent and quality of site investigation data. Changes in stratification and properties of layers in short distances are not unexpected facts. Empirical and analytical solutions or numerical modeling provide tools to determine support measures, predict deformations due to tunneling induced ground movement; surface, subsurface settlement, lateral deformation of the ground, longitudinal movement of the ground ahead of the tunnel face. However the prediction of those values depends on the quality of input data. Due to the nature of shallow tunneling, change in situ stress and hydrogeological conditions bring higher influence on surface buildings.

Design considerations should be checked during the construction with monitoring of both subsurface and surface movements, impact on nearby buildings and services. Geotechnical monitoring capabilities both at surface and subsurface provide early detection of negative effect of tunneling on nearby structures and taking measures on time. Improvements in evaluation methods suggested by Sellner et al 2004 and measuring devices such as chain inclinometer installed to the pipe roof systems explained by Volkman & Schubert 2006 increase those capabilities.

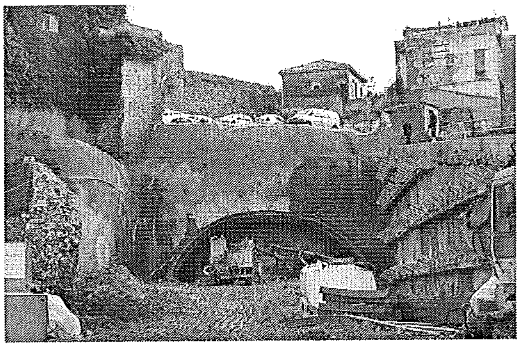

Figure 1. View from the west portal.

Kalepark Tunnel is a typical shallow urban tunnel with a maximum overburden of 21 m in a residential area. Tunnel with three lanes was planned to construct at a distance of 30 m from the axis of the old tunnel due to the traffic requirements (Figure 1).

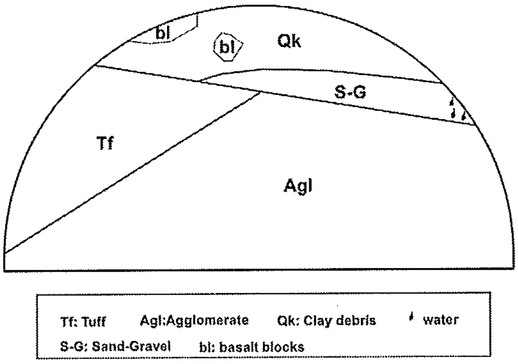

Figure 2. Face mapping from km: 0 + 549.

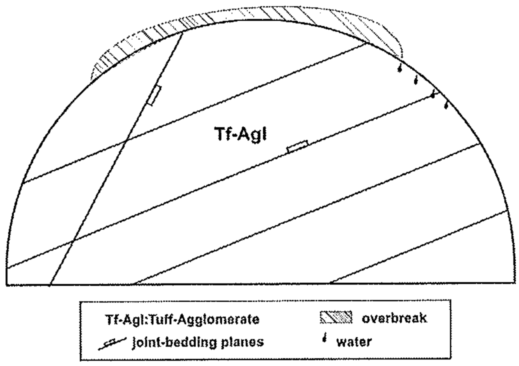

Figure 3. Face mapping from km: 0 + 567.

Site investigation works consisting limited boreholes and geological mapping of outcrops could be realized and tunnel design was performed accordingly. Tunnel excavation was designed as sequential excavation; top heading, bench and invert. Support system includes fibre reinforced shotcrete, steel ribs, rock bolt, forepoling, shotcrete and rock bolting at the face if required Due to the high expenses for nationalizing works, structures with historical importance and varying types of buildings around the tunnel area, geotechnical monitoring and evaluation gained importance.

2 GEOTECHNICAL DESCRIPTION

The rock at the tunnel area is highly weathered tuff and agglomerate. Tuff and agglomerate is overlaid by clay debris containing basalt boulders. A layer of loose sand-gravel is encountered between tuff-agglomerate and clay debris at the first 25 m from the west portal. Highly weathered and water susceptible tuff-agglomerate has low strength. After 25 m advance from the west portal, tuff and agglomerate shows bedded structure. The bedding planes contain clay and calcite filling and rock mass is highly fractured. The water level was encountered at 10.5 m below the ground surface in the boreholes at the design level. During the tunnel construction the water income from the loose sand-gravel layer at the shoulders was observed as leakage in the first meters of the tunnel excavation. Water discharge increased with the tunnel advance. The rock mass conditions encountered during the top heading excavation are shown in two representative face mappings from km: 0 + 549 and 0 + 567. At km 0 + 549 clay debris and loose sand-gravel layer were encountered at the upper part of the top heading. Water inflow from the sand-gravel layer resulted in overbreak at the shoulders (Figure 2).

At km: 0 + 567 tuff and agglomerate at the face is highly fractured. Overbreak was observed at the crown. Water inflow from the right part of the face has a discharge rate of 0.4 lt/min (Figure 3).

3 MONITORING AND EVALUATION

An extensive monitoring program was performed both at the surface and inside of the tunnel. Measuring station in tunnel was established with a distance of 5 m and absolute measurements at five points in cross section were recorded during the construction. Before the construction, structures were examined; their type, actual condition and position relative to the tunnel were recorded.

The structures in a determined zone in portal area were nationalized. Topographic measuring stations at the ground and buildings were established. Strict acceptable limits for settlement values (absolute, differential settlement, rotation) were not determined due to varying characteristic of buildings such as reinforced concrete multistory buildings, masonry fortress walls etc. Extensometers were planned to set according to topographical measurement results.

When top heading excavation arrived km: 0 + 570, cracks were recorded at the ground surface, fortress walls and one storey buildings inside the fortress area. Figure 4 shows measuring points at the ground and fortress wall, position of the structures and fortress walls relative to the tunnel and recorded cracks.

To evaluate the mechanism bringing failures at the ground surface and structures, the displacement measurements inside the tunnel, crack development and deformation measurements from surface stations were evaluated with combination of tunnel advance data and water discharge rates. The displacement amounts in the tunnel are less than 1 cm and no failure was observed at the support elements; no cracks at the shotcrete or sign of overloading in rock bolts.

The measurement result showed that the deformations at the ground surface and fortress wall increased with increasing discharge rate. Considering the

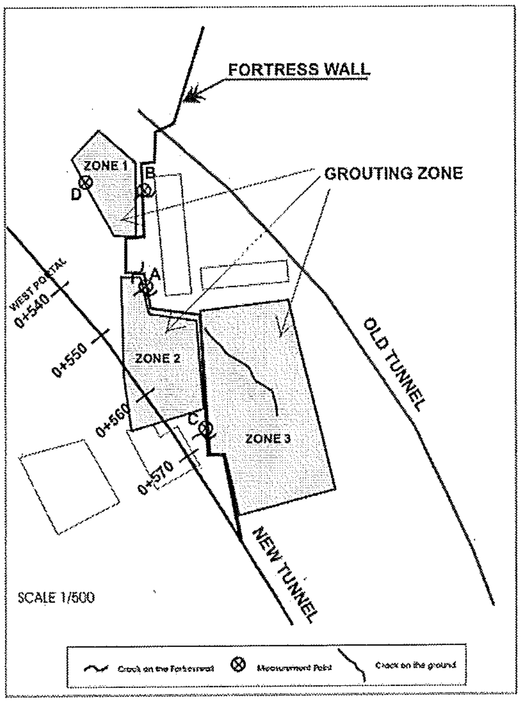

Figure 4. Plan view from west portal area showing measuring points, cracks and grouting areas.

deformation graphs and geological structure from face and surface mappings it is concluded that high water discharge due to tunnel excavation cause immediate settlement at the granular material. Tunnel support had required capacity to carry the additional load due to this fact. However masonry structures are sensitive to these movements and cracks occurred.

Damage in the masonry walls and buildings were evaluated according to the classification of visible damage of structures suggested by Burland 1995. The crack widths at four measuring stations were measured as 4–20 mm (Figure 6, Figure 7, Figure 8, and Figure 9). Typical crack widths of the masonry walls were 15–25 mm and classified as category of damage- 4, normal degree of severity-severe.

Because no other failure is observed in nearby structures and inside the tunnel, required measures were determined to rehabilite the structure and strengthen the foundation by cement grouting. This solution is also to prevent development of the failure with bench and invert excavation.

4 REMEDIAL MEASURES

Grouting application with vertical and inclined boreholes was planned in three zones (Figure 4). Location, inclination and extend of the grout holes were determined according to the crack measurements, position of the old tunnel, bolted area around the new tunnel, fortress wall and the buildings (Figure 5).

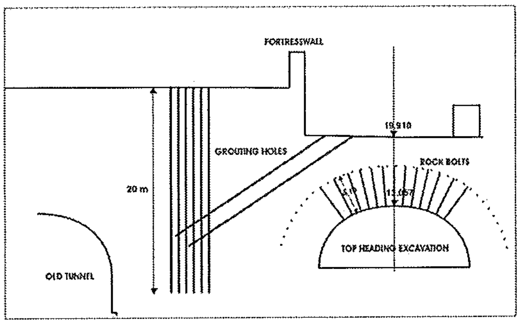

Figure 5. Grouting application at km: 0 + 560.

The benefits of the grouting were:

- to fill voids in formation,
- to increase resistance against deformation,
- to supply strength,
- to reduce conductivity,
- to control ground water table and seepage.

0.5 m*0.5 m square pattern is designed. But during construction according to monitoring records this pattern could be revised.

In cement grouting, the following principles are considered:

- Grouting is applied in 2 m levels
- Applied pressure (bar) is calculated from the following equation:

$$P=0.23*H \quad (1)$$

where H: distance between top of borehole and middle of each level (meter)

- Water cement ratio was decreased from 3:1 to 5:7 in stages. At the last stage sand (5%–20%) was added to the grout mix.

At Station A in Zone 2, crack width on the fortress wall was measured 20 mm before the grouting application within a month. Grouting at that region was completed in the next month. The deformation rate decreased during ground improvement and 5 mm deformation was recorded. After the completion of the grouting at that area no further deformation was detected (Figure 6).

At Station B in Zone 1, crack width on the fortress wall was measured 4 mm before the grouting application within 13 day period. During grouting lasting 19 days around Station B the deformation rate increased and deformations arrived 11 mm. After the completion of the grouting at that area deformations at Zone 1 stabilized (Figure 7).

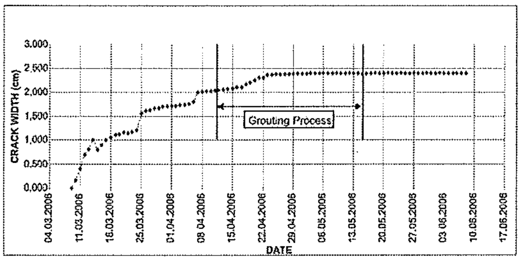

Figure 6. Crack width measurements from Station A.

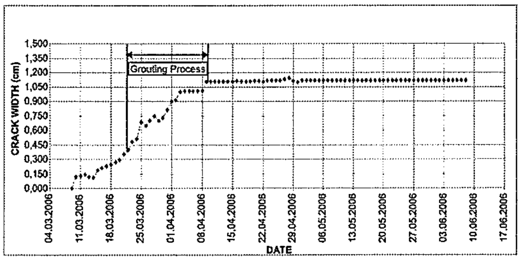

Figure 7. Crack width measurements from Station B.

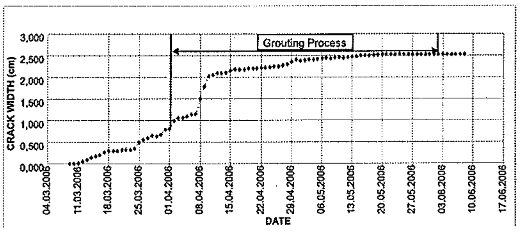

Figure 8. Crack width measurements from Station C.

At Station C in Zone 3, crack width on the fortress wall was measured 8 mm before the grouting application within 3 weeks. Due to the large size of that area grouting took 2 months. A sudden increase of 10 mm was recorded after start of the grouting. At the end of the application deformations was 25 mm. Stabilization of the deformation was achieved with grouting (Figure 8).

At Station D in Zone 1, crack width on ground surface was measured 4 mm before ground improvement within 2 week period. Grouting took 10 days near this Station. Similar deformation rate was observed during the grouting process. At the end of the grouting deformations reached to a final magnitude of 7 mm (Figure 9).

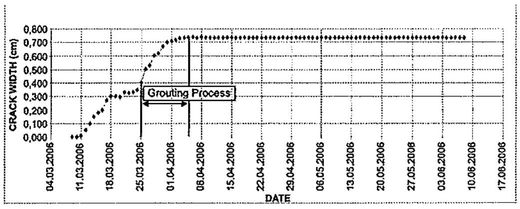

Figure 9. Crack width measurements from Station D.

Grouting was applied in three distinct zones to stabilize deformations on the fortress wall and ground. A close pattern of grout holes were applied. The success of the ground improvement was checked with the measurement of the crack widths at the measurement stations.

5 CONCLUSION

Due to characteristic of urban tunnels (soft rock/soil conditions, low overburden, groundwater level, nearby structures and services), monitoring of tunnel induced movements at the ground surface and buildings, change in ground water level are necessary to control negative influence of tunnel construction. Monitoring results should be analyzed with tunnel advance data and geological documentation to evaluate the mechanism of tunnel induced movements and failures at the surface. The measures to stop or limit deformations at the surface can be determined as stages. The effectiveness of the measures can be evaluated by continuous monitoring during all stages. The measures can be taken inside the tunnel or at the surface. For an economical solution type and extend of the measures should be determined considering failure mechanism.

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