Underground Space – the 4th Dimension of Metropolises – Barták, Hrdina, Romancov & Zlámal (eds)
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Redesign and monitoring of a portal structure in built-up area

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ABSTRACT: In this work, the redesign studies and the construction of the ground anchorages of the portal structure of a shallow urban tunnel, which is located on the main transportation artery of Black Sea Coast Road, are given. This tunnel has been constructed near an old existing tunnel under heavy traffic conditions and surrounded by residential area. During the research studies of the design stage, restricted number of boreholes was opened because of the expropriation problem. In view of the site observations and information obtained from these boreholes, it was observed that the rock encountered at the portal sites is a weathered and weak volcanic rock composed of medium to fine-grained pyroclastic material. Due to this reason, a portal structure that consists of bored piles, ground anchorages and cut and cover was designed. The original project of the tunnel was approved by General Directorate of Highways in December 2000. The construction of the tunnel was started at June 2003. After completion of the first test anchorage, it was observed that it could be pulled out only one third of the design load. After solving the expropriation problem, additional boreholes could be opened and the existing project was redesigned using the new data. During the construction of anchorages, special care was given and the monitoring of the portal structure was performed using load cells, extensometers and inclinometers. This study contains two main parts: Firstly, the redesign of the pre-stressed anchorages and construction techniques of them for improving the capacity will be given and then the evaluation of the monitoring data during construction of portal structure will be explained.

1 INTRODUCTION

Kalepark Tunnel, which is the scope of this study, is located on the main transportation artery of Black Sea Coast Road, city crossing of Trabzon. It is a shallow urban tunnel that is surrounded by residential area and located under a historical reputational area and military zone. This tunnel has been constructed near an old tunnel under heavy traffic conditions and surrounded by residential area (Photograph 1). The length of new tunnel was 144 m including 15.5 m and 28.5 m cut and cover sections at entrance and exit sections respectively. The horizontal distance between the old and new tunnel at the entrance and exit portals are 11 m and 20 m, respectively. The overburden of the new tunnel is varied between 2.5–3.5 m at portal areas and the maximum overburden through the tunnel is 11.5 m.

In the entrance zone there was a permanent excavation from 6.5 to 13.5 m depth. In 1997, during the research studies of the design stage, restricted numbers of boreholes were opened because of expropriation problem. Only four boreholes were drilled during the tunnel research studies at that time. As it was shown in Figure 1, ESK-1 borehole, quite far from deep permanent excavation, was representative for the portal structure. In view of the site observations and information obtained from these boreholes, it was observed that the rock encountered at the portal sites is a volcanic rock composed of medium to fine-grained pyroclastic material. Due to this reason, a portal structure that consists of bored piles, ground anchorages and cut and cover were designed. Besides, at the design stage it is advised that the confirmation of the geotechnical values and the assumptions of the project should be checked. The original project of the tunnel was approved by General Directorate of Highways in December, 2000.

During the design stage, the geotechnical model for the right section of the entrance portal, which is the scope of this study, was described as weathered tuff and

Photograph 1. A view from entrance portal of Kalepark Tunnel.

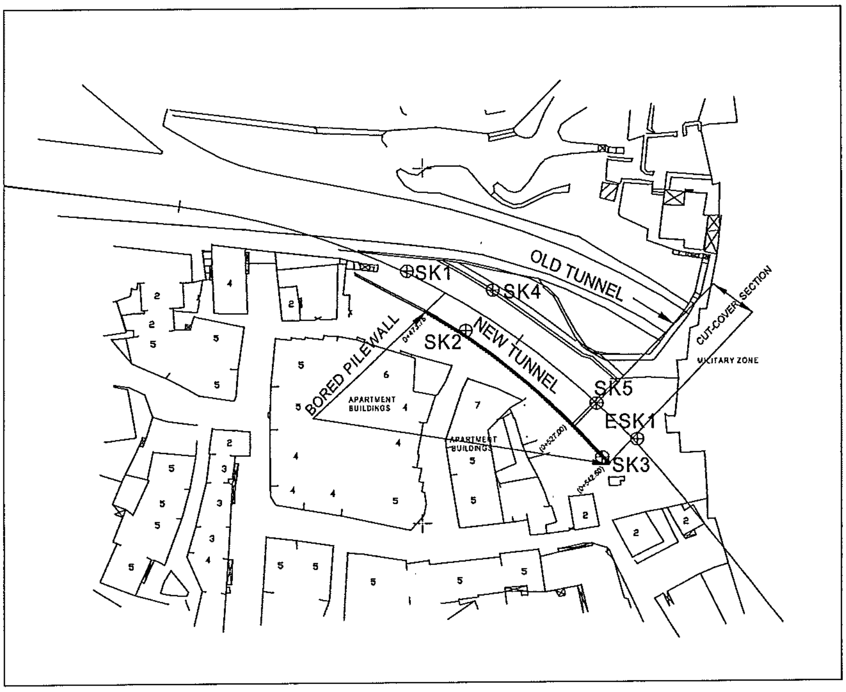
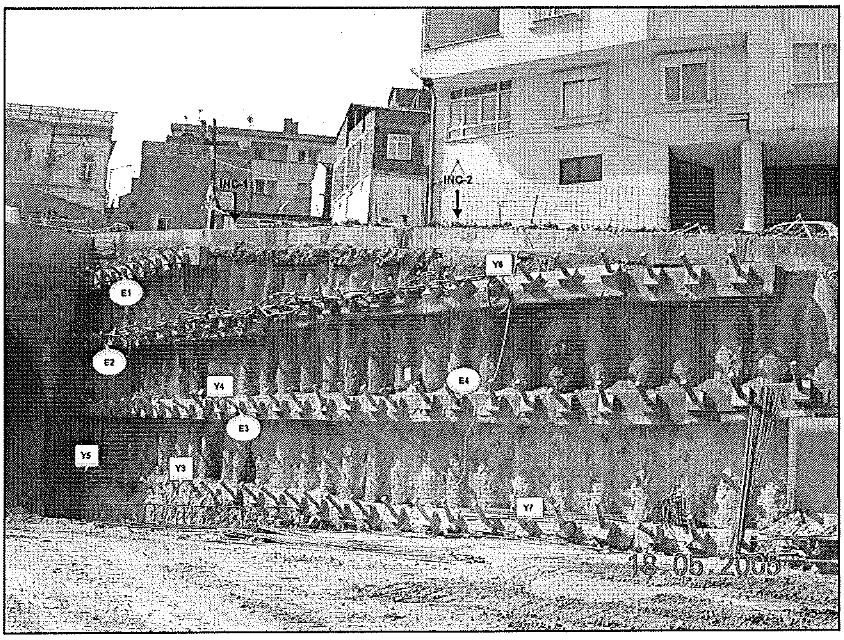


Figure 1. Plan view of Kalepark Tunnel entrance portal and the location of boreholes.



Photograph 2. Monitoring system and retaining structure of Kalepark Tunnel.

agglomerate that contains clay layers. The retaining structure of the entrance portal at the design stage was determined as bored pile wall with pre-stressed ground anchorages (Photograph 2 and Figure 1).

The construction of the tunnel was started at June, 2003. Construction and research studies for comparison were performed simultaneously. After solving the expropriation problem, additional boreholes were drilled. Bored piles were constructed. Three pre-stressed anchorages at the first row were constructed and anchor tests were performed. During the drilling process of anchorages, it was seen that soil around the anchor bond length was containing water. The test results were not in consistent with the project values as it was expected. During anchor tests, some of the anchorages could be pulled out only one third of the design load. In view of the evaluation of the new borehole data, the results of anchor tests and the observations made during the pile drilling, it was concluded that the foreseen geotechnical evaluation was different from the actual site. Due to this fact, the existing project was redesigned using the new data.

2 GEOTECHNICAL EVALUATION

2.1 *Geotechnical evaluation at the first design stage*

The site was described as weathered, weak and water susceptible tuff and agglomerates containing thin clay layers. The rock quality designation (RQD) values are zero and the core recovery ratio (TCR) values were differing from 5–43%. The water level was approximately 10 m below the ground surface. The strength parameters for the weathered tuff and agglomerate and clay layers were decided as c = 100 kPa, Ø = 25° and c = 0.5 kPa, Ø = 9–10°, respectively. Using these parameters, intermittant bored piles (Ø80) with 15 cm clear space and pre-stressed ground anchorages were designed.

2.2 *Geotechnical evaluation at the construction stage and new project studies*

At the entrance portal four additional boreholes were drilled. Two of these boreholes (SK2 and 3) could be representative for the right section of the portal. Pressuremeter tests were performed in these boreholes.

Unfortunately, no additional boreholes that could give information about the fixed anchor zone were drilled because of residential buildings surrounding the site. In order to overcome this problem, results of anchor tests were evaluated. From this investigation, it was seen that, approximately the first 4 m of the geotechnical profile was formed of road fill. Then, highly weathered and water susceptible clayey tuff was encountered up to approximately 10th meter. After this layer, weathered tuff with 0–60% RQD value and 30–80% TCR value is laid down. The water level was 2.70–5.45 m below the ground surface. The water level was different from the former boreholes, the main

Table 1. Soil parameters used in the redesign stage.

Layer	c′ (kPa)	Ø′ (°)	E (kPa)	ν
Fill	0.5	27	7500	0.33
Tuff 1	2.0	27	15000	0.30
Tuff 2	10.0	28	25000	0.30

reason for that was thought as the percolation of the surface runoffs and the percolation of water due to surrounding buildings.

The pressuremeter test results were used in determining the elasticity modulus and friction angle of the layers. Menard and Calhoon Methods were used and it was seen that friction angle values obtained from Calhoon method gives more conservative values than Menard Method. As it was a permanent excavation, without any boreholes drilled at the anchor fixed zones and since it was a residential area, lower values were chosen as design friction values. Moreover, lower cohesion values were chosen as shown in Table 1 due to reasons mentioned above.

From the results of the evaluation of new borehole data, the results of anchorage tests and the observation made during the pile drillings, the parameters used in the redesigned project was given in Table 1.

3 PROJECT DESCRIPTION

3.1 *New design project details*

The safety of permanent retaining structures in such residential areas is very important. In the entrance portal structure, the depth of the excavation was varied between 6.5 m to 13.5 m. As expected, horizontal and vertical displacements are normally occurred in every deep excavations. But it is important to keep these deformations in permissible limits in this project due to surrounding built-up area.

At the redesign stage, the construction of the intermittant bored piles (Ø80) with 15 cm clear space was constructed. Firstly, the existing bored pile wall was taken into consideration and redesign was performed by considering new parameters obtained from new investigations. The location of pre-stressed ground anchorages is another important factor since the construction was located in a residential area. All the buildings near the portal structure were reevaluated and the anchorage patterns and inclination of anchorages were decided.

On the other hand, the pre-stressed anchorages and pile wall of the system were modeled by finite element program PLAXIS. In this program all the construction stages can be modeled with any loading conditions. Forces acting on the ground anchorages and pile wall can be evaluated for all stages of the construction.

The design of retaining wall was made based on six typical types of solution depending on the depth of excavation and surrounding buildings near to the excavation. The working loads of the anchorages were calculated from the program. Additional analyses were performed in order to see the displacements occurred at prescribed locking loads. By evaluating the results of the analyses, locking loads for the ground anchorages were adjusted. As a result of these analyses, it was seen that the anchorage working loads (service load) were varied between 20–30 tons.

Horizontal and vertical displacements just near and around the retaining walls were restricted between 5–15 mm depending on the neighboring conditions. As a result, no any visible cracks were observed around the permanent excavations.

The moment and shear capacities of the piles were high. Depending on that fact, pre-stressed ground anchorages were designed as they located at every space between piles in the new design project. As a result of this pattern, the dimensions of breasting beam would be smaller and the construction rate would be fast. The anchorage pattern in the design project was determined as 0.95 m × 2.5 m (horizontal × vertical). Because of the adjacent buildings, the inclination of the anchorages was 20°, 25° and 35° from the horizontal. These inclinations were applied variably ±2 degrees in horizontal direction of the breasting beams, in order to prevent the fixed anchor zone interactions. After the construction of anchorages, the anchorage tests were made and it was observed that the working loads can be carried by these anchorages.

3.2 *Practical aspects in ground anchorage constructions*

Drilling and grouting methods are the main practical aspects that influence the anchorage capacities. Suitable diameters of augers (minimum 15 cm) were used in drilling operation in this project. After drilling without using any water, hole was filled with grouting material immediately in order to prevent the collapsing of the hole-walls. The ground anchorage assembly then was located through the grouted hole before setting time of grout. Some grout material was come back from the hole and some of it was percolated to the surrounding soil as was expected. Therefore, a post grouting process was found necessary to get desired capacities. Special packers were designed just between the free and fixed parts of the ground anchorages. A grout pipe entering into this packer was used to give grouting material. Thus a barrier was formed between free and fixed zones. After 1 to 3 hours, a post grouting was performed under 2 to 4 bars through another grout pipe. Thus, the final fixed zone of the anchorage does not contain any voids that may be produced after primary grouting which was performed by tremie method. The diameter of the fixed zone is also greater

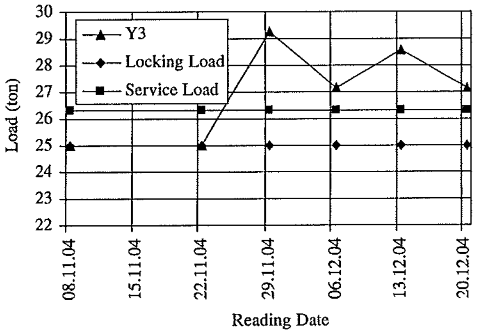


Figure 2. Data measured from load cell Y3.

after this post grouting process. All of these increase the capacity of the ground anchorages.

4 ANCHORAGE TESTS, MONITORING AND EVALUATION

Anchor tests were done during construction according to SIA 191 Standard. The anchor tests were performed in order to check the design assumptions.

4.1 *Anchor tests*

The working loads of the ground anchorages were about 60 tons in the original project. These capacities were found quite high after testing first anchorages. Additional site investigations confirmed these results. The working loads of the anchorages were decreased in the redesigned project. The results of anchor tests were satisfactory in the redesigned project. Thus, the permanent excavations were finished based on new site investigation and design studies with special measures in ground anchorage constructions as mentioned above.

4.2 *Load cell readings at anchorages*

Five load cells were located at the anchorages in order to monitor the load distribution (Photograph 2). The load variation during the construction process was followed with periodical readings.

As it can be seen from the Figures 2,3,4,5,6 during the intermediate construction stages, some data measured from Y3, Y5, Y7 load cells are 3, 4, 3 tons higher than service loads respectively. But it was observed that, the final readings taken after completion of the retaining wall showed that, the loads acting on the anchorages were all lower than the service loads.

4.3 *Extensometer results constructed on pile wall*

Four extensometers were located at the pile wall in order to monitor the deformations during construction. (Photograph 2). The values measured were followed

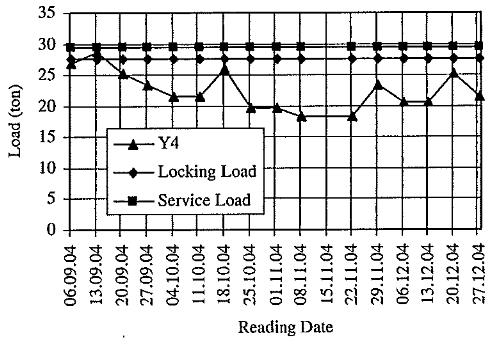


Figure 3. Data measured from load cell Y4.

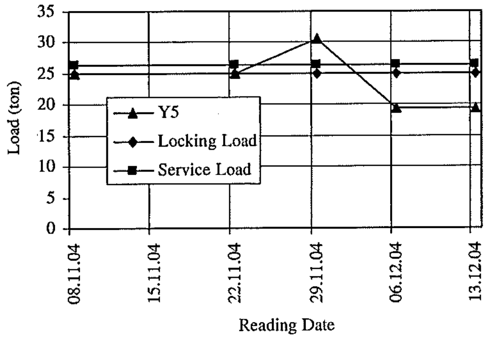


Figure 4. Data measured from load cell Y5.

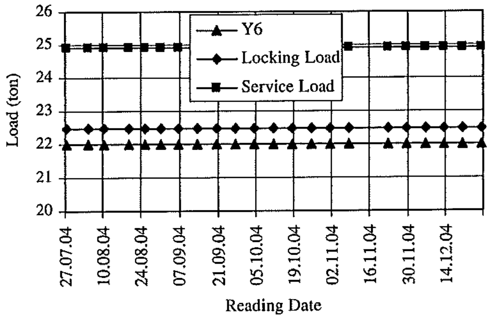


Figure 5. Data measured from load cell Y6.

with periodical readings and is given in Figure 7. The results were showed that, maximum measured deformations were about 80% of the calculated ones.

4.4 *Inclinometer measurements constructed on behind the pile wall*

Two inclinometers were located in order to control the displacements during the construction of the portal structure (Photograph 2). If the measurements taken

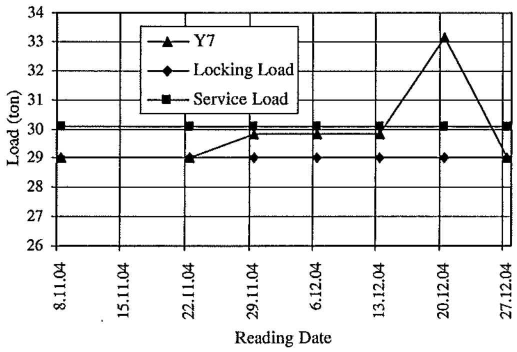


Figure 6. Data measured from load cell Y7.

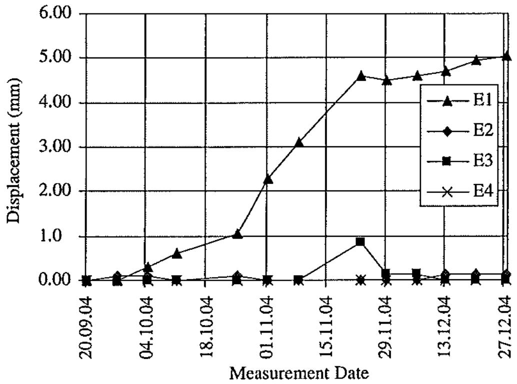


Figure 7. Data obtained from extensometer E1, E2, E3, E4.

from inclinometers and the extensometer data (E3 and E4) are compared, it is seen that the maximum displacement values are in 5–8 mm and convenient with each other.

5 CONCLUSION

In this case study, the importance of the confirmation of the assumptions at the project stage due to compulsory incomplete research studies was seen. Loads measured from the load cells at intermediate stages may show higher values than assumed ones in project due to improper construction sequence. Excavation, breasting beam and anchor construction activities should be adjusted carefully so that no any horizontal and vertical excavations are to be performed without locking the neighboring ground anchors. But on the other hand, by cross checking the inclinometer and extensometer readings it was concluded that no additional support was needed. As a result, detailed research studies, anchor tests and confirmation of project assumptions and also monitoring have vital importance on the safety of retaining structures especially in built up areas.

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