Stabilization Measures for Shallow Tunnels with Ongoing Translational Movements Due to Slope Instability

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ABSTRACT

This paper describes the design methodology adopted for stabilization measures of a twin bored highway tunnel in Northern Greece, where severe cracking and translational displacements of the temporary support shell were observed. Cracks occurred due to an active landslide in the vicinity of the tunnel that moves both bores downhill.

A detailed numerical back-analysis based on actual geotechnical measurements data and crack observations on the support shell and the ground surface, was performed, in order to determine the possible sliding shear zone. Shear strength properties of the rockmass elements that failed in plastic yield were properly reduced, until the calculated support shell displacements reached the actual measured displacements.

The effectiveness of different types of stabilization measures was investigated by using excessive multi-staged FDM (Finite Difference Method) analysis. The most effective combination of stabilization measures was selected in order to prevent additional ground movements and to stabilize the tunnel. Stabilization measures consist of reinforced concrete piles, pre-stressed permanent anchors and consolidation cement grouting applied on the periphery of the tunnels as well as on the intermediate pillar. The effect of potential seismic loading was numerically investigated.

1. INTRODUCTION

Egnatia highway has a total length of 687 km. It links Igoumenitsa, the biggest port of North - West Greece, with Kipi, the eastern border of the country. Tunnels, 75 in total, comprise an important portion of Egnatia highway. Owner of the Egnatia Odos Highway Project, is the Egnatia Odos S.A (E.O.A.E.). The S3 tunnel is located in the section 5.2 of the highway in the Nothern Greece. The location, the exit portal and a view of the tunnel are shown in Figure 1. The tunnel axis is extended from NE to SW direction. It is a shallow twin bored slope tunnel with average length of 230 m. Tunnel width is about 12 m. The axial spacing between the two tunnels is 30 m. The right bore of the tunnel is located approximately 5 m higher than the left one. The tunnel is passing along a hill slope, with geological formations of poor quality.

During the construction process, much higher displacements than those initially expected, cracks and local failures on the temporary support shell as well as on the ground surface upstream of the tunnel, were observed. Geotechnical monitoring indicated that both tunnel bores were moving translationally downhill. This movement continued despite the fact that the construction process had already been suspended.

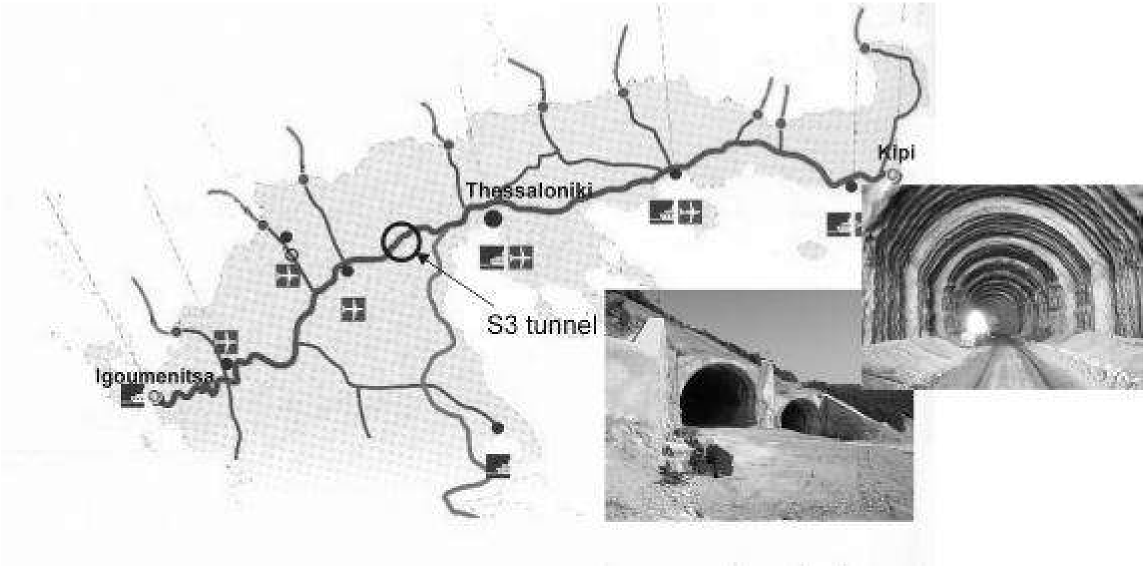


Figure 1 Tunnel location along Egnatia Highway.

2. GEOLOGY

The terrain in the vicinity of the tunnel is characterized of mild inclination approximately 20^o parallel to the tunnel axis. The maximum height of the overburden of the tunnel is 25-30 m. Most of the tunnel is excavated in overburden of about 15 to 20 m. Tunnel axis runs along the slope of a hill, between two very deep creeks.

The rocky background of the area is mainly consisted of alterations of thin-bedded to mid-bedded limestones and argillaceous phyllites, including all the intermediate relevant lithological types. The geotechnical formations are characterized by a high tectonic and most likely slope disturbance and looseness (even at the end of the drilling depth of 20.0m – 45.0m). Due to the above-mentioned regime, the geological formations are intensively folded, faulted and loosened. The RQD index is almost close to zero, except from specific parts, in which it varies between 10 –20 %. The geological formations in their majority are characterized by a high permeability (> 10^{-2} cm/sec). In the close area of the NE (entrance) portal, the rockmass is comprised of phyllites, with a high degree of weathering, that are converted locally to soil like material. Small to medium size fragments of stronger rock are present. The overall mass has been extensively relaxed with open structural discontinuities in places. In the SW (exit) portal, the rock mass is of better quality but still with open structure. However, in most cases, the layered structure of the phyllitic material is retained.

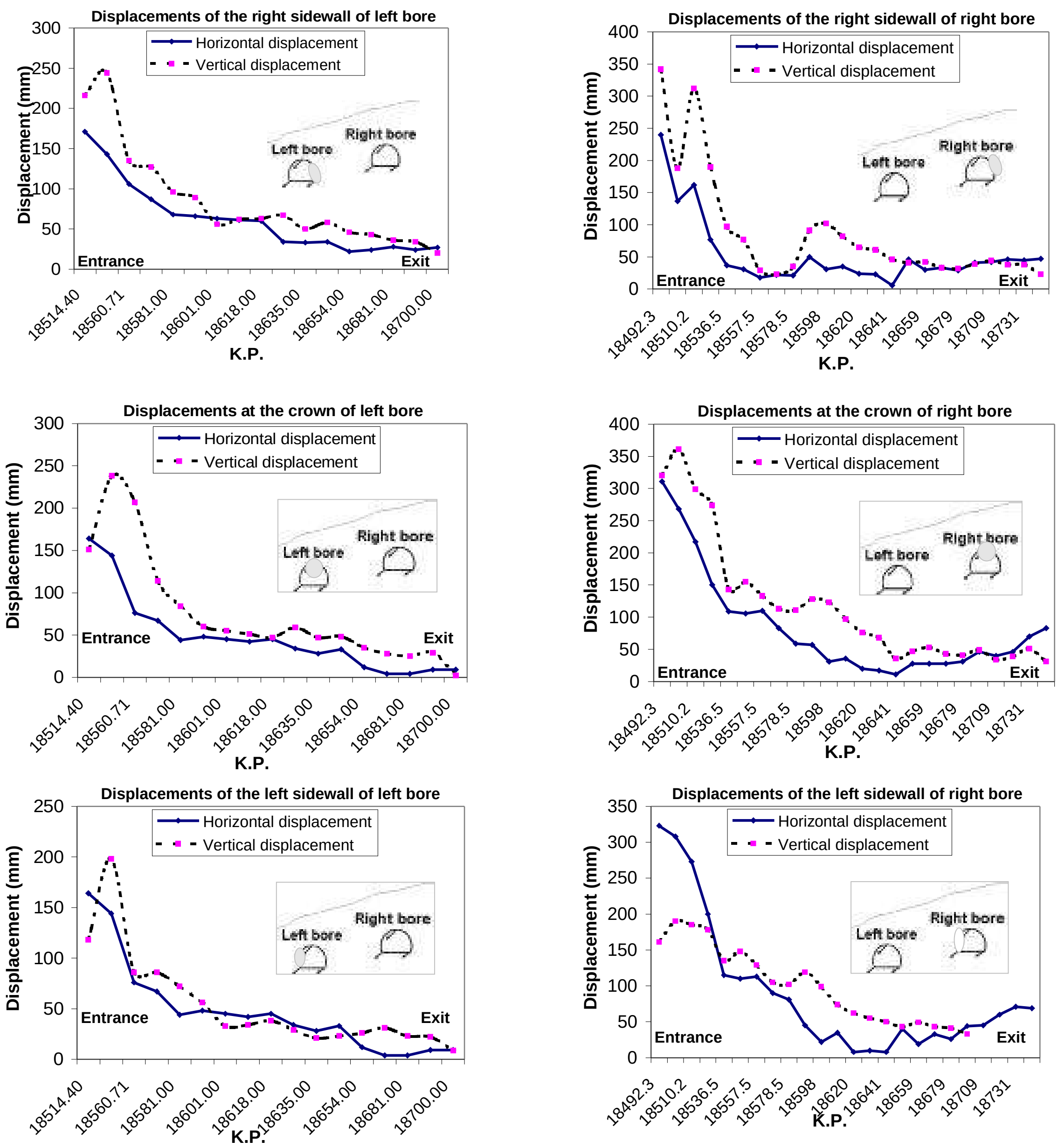
It is very difficult to adopt a strictly confirmed geotechnical model, in terms of detecting the specific generative causes, for the situation of the geological formations in the area of the tunnel. It must be emphasized that the geologic surface formations in the vicinity of the tunnel are in limit post-equilibrium situation, characterized by a significant slope relaxation process in the geologic history and meta-stability marginal conditions.

3. IN SITU GEOTECHNICAL MEASUREMENTS

The main available geotechnical measurements include:

- Surface displacements in the ground surface above the tunnel and the portals' area.
- Displacements of the primary support shell.
- Displacements measured with inclinometers.

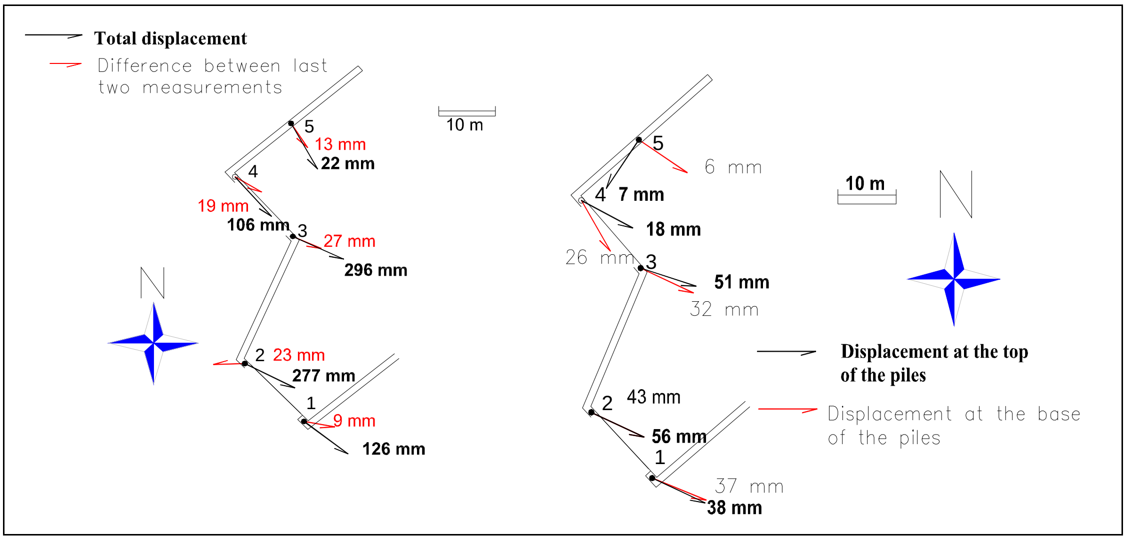
Tunnel movement continued at this time (July 2003) at a slow but continuous rate. Figure 2 shows the measured displacements as a function of the tunnel axis, while in Figure 3 the piles' displacements at the entrance portal are presented.



(a) (b)

Figure 2 Temporary support shell displacements along the tunnel axis (a) Left bore (b) Right bore.

From the above figures it is evident that displacements are much higher in the entrance portal area of the tunnel, where serious damage of the primary support shell in the invert and the sidewalls had occurred, as well as cracks in the upstream surface area. Deformations measured at the exit portal are significantly lower. From the interpretation of the recorded measurements, it is obtained that the tunnel is moving translationally downhill. Because of the large movements of the tunnel, severe failures of the temporary support shell at the entrance portal area had occurred as shown in Figure 4.



(a) (b)

Figure 3 Displacements of the piles at the NE (entrance) portal (a) cap-piles (b) top and base of the piles.

4. MODELING METHODOLOGY-RESULTS

Following the interpretation of the above data, urgent efforts were taken in order to design and apply the appropriate stabilization measures. These efforts comprised at first the determination of the sliding zone and its shear strength parameters. At the second stage, based on the above determination, appropriate remedial measures were selected and dimensioned, in order to stabilize the tunnel.

The finite difference code FLAC 2D (ITASCA Consulting Ltd., 1998) was used for the simulation of the excavation and support process of the tunnel. One of the advantages of this code is the bult-in programming language called FISH that enables the user to define new variables, functions and constitutive models. A multi-staged back-analysis was performed consisting of 36 simulation steps and including full excavation and primary support of the two bored tunnels, application of the stabilization measures and investigation of the seismic effect in the tunnel. The Mohr-Coulomb plasticity failure criterion was used for the simulation of the rockmass.

Based on the interpretation of the measurement data and the construction phase, the tunnel was divided in three main parts:
Part A: Section in the entrance portal area where the excavation process has already been completed, and large deformations had occurred followed by cracks of the primary support shell and the ground surface.
Part B: Section in the entrance portal area where the excavation process has already been completed in the right bore while in the left bore only the top heading has been excavated.
Part C: Section in the exit portal area where the excavation process has already been completed, and less deformations have been measured in comparison to the entrance area.

The original primary support design that applied to the excavated part of the tunnel consisted of a 25 cm thick layer of fibre reinforced shotcrete, steel sets HEB140 per m, grouted rockbolts and forepole umbrella (steel tubes of 114 mm outer diameter and thickness of 6,5 mm installed per 35 cm axial spacing fully grouted).

The aim of the back-analysis was the numerical reproduction of the measured displacements of the temporary support shell as a function of the construction stages. To succeed this, a specific FISH routine that gradually reduces the strength parameters of the rockmass elements that failed in plastic yield at the end of every simulation stage was developed. The geotechnical parameters used for the simulation process are presented in Table 1. Values in column [2] have been derived by back analysis. Potential seismic effect in the entrance portal area (column [3]) was taken into account by reducing the shear strength parameters of column [1] by 20% (Hoek and Marinos, 2002), while in the exit portal area this reduction was of the order of 10% due to the better quality of the rockmass in this area. Simulation steps are given in Table 2 for every case investigated.

(a) (b)

Figure 4 (a) Severe invert cracking near the Entrance Portal of the S3 tunnel invert (b) Surface tension crack upstream of the right bore.

Table 1 Geotechnical parameters used for simulation process.

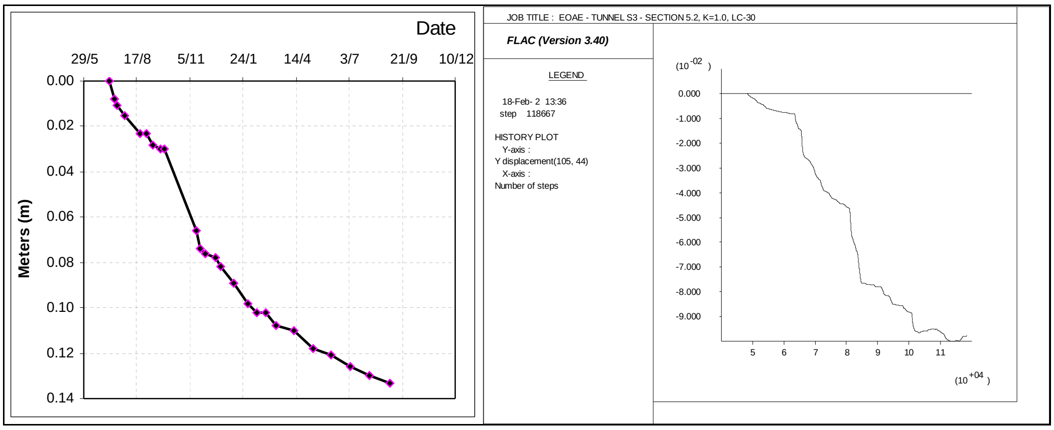
	[1]*	[2]*	[3]*	[4]*	[5]*	[6]*
Young modulus, (GPa)	0,3	0,3	0,3	0,3	2,0	15,0
Poisson ratio, (ν)	0,30	0,30	0,30	0,30	0,30	0,30
Unit weight (kN/m^3)	26,0	26,0	26,0	26,0	26,0	26,0
Friction angle (^{o})	27	30	24	27	38	44
Cohesion (kPa)	30,0	0	0	0	200,0	730,0
In situ stress ratio, K_o	1,0	1,0	1,0	1,0	1,0	1,0

*[1] Rockmass [2] Failure zone [3]Failure zone under seismic loading (NE portal) [4] Failure zone under seismic loading (SW portal) [5] Cement grouted rockmass [6] Cement grouted rockmass with forepole umbrella

Steps 1 to 30 for parts A and C and 1 to 23 and 26 to 32 for part B, refer to the back-analysis process. In these steps the multi-staged process of excavation and support of the two tunnels, the maturation of shotcrete strength and the gradual increment of the relaxation of the excavation as the advancing face moves away from the analysis section, has been simulated. A FISH algorithm was used to identify the elements of the model that failed in plastic yield and reduce their shear strength properties in order to simulate the formation of the sliding surface. A comparison between the calculated and measured temporary shell displacements is presented in Figure 5. In Figure 6 the calculated displacement vectors of the primary support shell are presented showing that both tunnels are moving downhill, which is in agreement with the field measurements. The geometry of the determined sliding surface is shown in Figure 7. The obtained values of the shear strength parameters are presented in column [2] of Table 1.

Table 2 Simulation steps for every investigated part.

Description of task	Step No
Excavation and support of the full section for right and left bore of the tunnel. Back analysis.	1 to 31 for parts A and C (1 to 23 and 26 to 32 for part B)
Application of stabilization measures consisting of: Ø Cement grouting in a radius of 6 m around the tunnels and in the intermediate pillar Ø 30 m long concrete piles of 1 m diameter per 2 m, upstream of the right bore. Ø 30 m long pre-stressed anchors	31-35 for parts A and C (24 to 25 and 33-35 for part B)
Reduction of the shear strength by 20% (10% in part C) to take into account potential seismic loading	36



(a) (b)

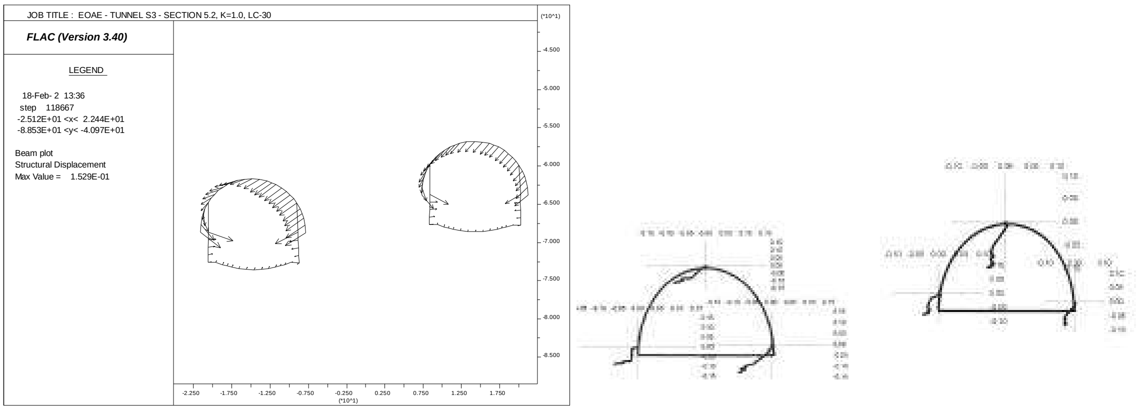
Figure 5 (a) Measured support shell displacements at the crown of right bore and (b) calculated by using back analysis (length units: m).

As it is obvious from the above figures the numerically calculated displacements are in sufficient agreement with the actual measurements. Calculated stresses in the support shell locally exceeded the strength of shotcrete. This is in agreement with the cracks observed at the invert and the sidewalls of the shotcrete shell.

Steps 31 to 35 for parts A and C, and 24 to 25 and 33 to 35 for part B, concern the simulation of the stabilization measures. Taking into account the depth of the above calculated failure zone, additional support measures were selected to stabilize the tunnel. These measures consist of:

- 30 m long concrete piles with 1 m diameter placed every 2 m, penetrating the failure surface and anchored in the more competent underlying bedrock.
- 30 m long pre-stressed permanent anchors applied from inside the tunnel at the area of the cracked invert and from the cap pile.
- Cement consolidation grouting applied from inside the tunnel. The role of the grouting is to strengthen the surrounding rockmass of the bores and the intermediate pillar so that the damaged shell can be removed and replaced safely.

A schematic representation of the selected stabilization measures is given in Figure 8. It is noted that final lining was not selected as a stabilization measure. According to general design practice concerning the tunnels of Egnatia Odos Highway Project, the tunnel must be stabilized before the installation of the final lining.



(a) (b)

Figure 6 Vectors of displacements, units: m (a) Calculated by back analysis (b) actual measurements.

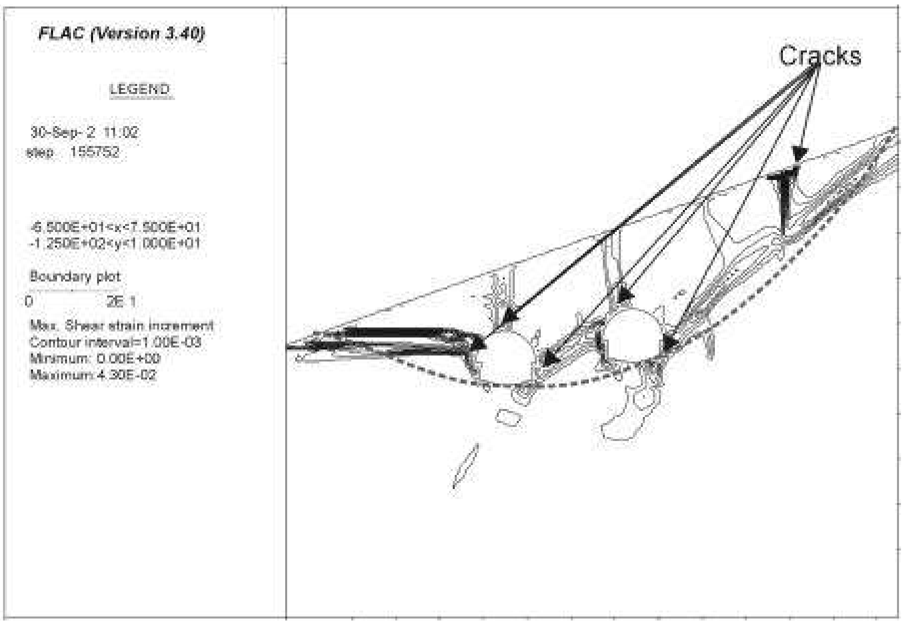


Figure 7 Contours of calculated shear strain increment after the implementation of back analysis.

The last step of simulation comprised the potential seismic effect on the tunnel stability. The analyses showed that the application of additional measures resulted in the stabilization of the tunnel in all the parts under consideration. Maximum additional displacements of the support shell, after the application of the additional measures, are not exceeding 5 mm. In case of seismic loading the maximum displacement does not exceed 67 mm. The application of cement consolidation grouting results in the rockmass model, by eliminating the yielding elements. The additional calculated shear strain is limited. Stresses are not exceeding the bearing capacity of the stabilization measures.

A critical point for the design was the appropriate construction sequence of the stabilization works as well as the repair and rehabilitation of the already cracked primary support shell. Based on the numerical analysis results, the most appropriate general construction sequence was determined as follows:

1. Construction of 30 m long concrete pile wall, anchored every 2 m in the sound bedrock. Simultaneously cement grouting and backfilling is performed in the bench of the left tunnel.

2. Application of cement grouting at the bench of the right tunnel. Construction of 30 m length pre-stressed anchors from the pile cap at 1 m centers, and the bench of the right bore. Backfill of the bench of the right tunnel.
3. Application of cement grouting at the top heading of both tunnels. Step by step demolition and repair of the damaged support shell at the top heading.
4. Backfill removal of the bench of both tunnels and step by step demolition and repair of the primary support shell at the bench.

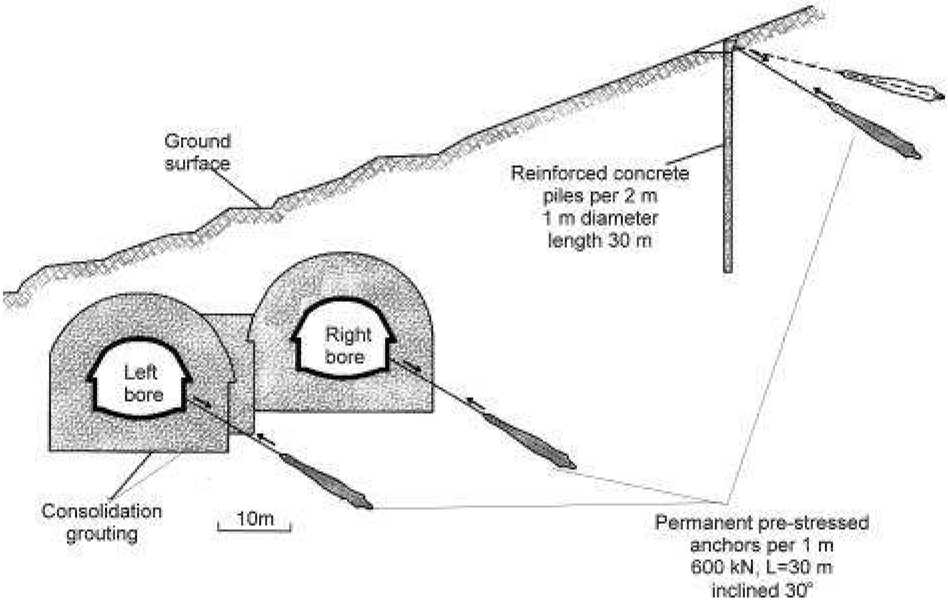


Figure 8 Stabilization measures.

5. CONCLUSIONS

Extensive multi-staged numerical back-analysis by using the two-dimensional code FLAC, proved to be successful in reproducing numerically the tunnel movements, and determining the geometry of the failure zone in the vicinity of the tunnel. Back-analysis was based on a FISH algorithm that automatically progressively reduces the strength parameters of the elements that failed in plastic yield, at the end of every calculation stage, until the calculated displacements coincide with the measured ones. The failure zone that moves the tunnel downhill was determined as the area of the model that is in plastic yield and the shear strain is relatively increased. Selection and design of the stabilization measures was based upon the back-analysis results. According to the numerical results the combination of long concrete pile walls, pre-stressed anchors from inside the tunnel and cement grouting around the two tunnels, is sufficient to stabilize the tunnel and the surrounding slope. The static adequacy of the selected stabilization measures was checked under seismic loading. A critical point and requirement for the effectiveness of the above measures is the proper implementation of the sequence of construction according to the design. Construction of the stabilization measures is in progress at this time (July 2003).

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