

Failure, retrofitting and strengthening of the Evinos Dam drainage tunnel - a case study of cooperation between academics and professional engineers

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ABSTRACT: This article concerns a case study of the stabilisation works that took place in an area of extensive failures within the stabilisation and drainage tunnel of the existing landslide in the dam area of Evinos, in Western Greece. The tunnel presented serious structural problems due to failure of drainage measures on the surrounding slope area that increased the imposed lateral pressures on the walls of the tunnel. During the necessary retrofitting and strengthening, the drainage tunnel was re-developed in a systematic manner that took place through valuable cooperation between academics and professional engineers. The cooperation was initiated with an acknowledgment of the causes that led to the failures of the tunnel, and continued with the study of the project, and completed with the implementation of the project, and systematic application of available research and design tools for the optimisation of the final technical solution.

INTRODUCTION

As part of the stabilisation and restoration projects of the landslide in the area between Agios Dimitrios - Arachova (*road* landslide), a drainage tunnel with two branches of total length 713m was constructed [1]. The two branches were developed in the region below the landslide and joined into a common branch, which heads west to the exit of the tunnel at the adjacent stream. The cross section of the tunnel has a radius ranging from 1.85-1.90 m at the dome and a height of 1.85 m at the vertical sides. The width of the floor is 3.7-3.8 m. The areas of failures inside the tunnel were located at the south left abutment of the reservoir dam of Evinos, located about 140 m from the exit of the tunnel (Figure 1) [2]. A typical section of the tunnel with shotcrete lining before retrofitting is shown in Figure 2.

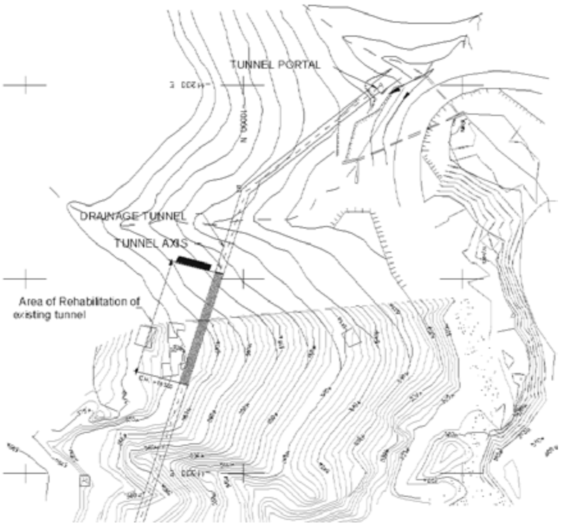


Figure 1: Plan view of the Ag. Dimitrios - Arachova drainage tunnel.

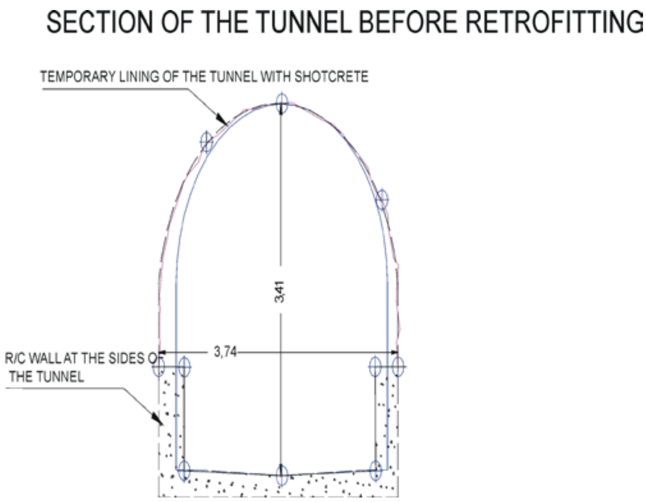


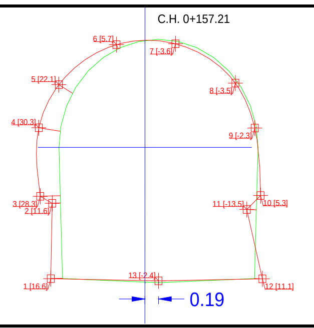
Figure 2: Typical section of the tunnel with shotcrete lining that suffered failures.

The cracks and detachments were more intense on the sides than on the dome of the tunnel. Furthermore, significant size overturning occurred in the toe concrete walls that led to convergence of the tunnel sides. Failure also occurred in the area of maximum convergence of the tunnel sides at the temporary steel supportive measures of the tunnel. This failure took the form of local buckling and shear failure of the steel members that are placed in the perimeter of the domed tunnel. It must be stressed that the permanent lining of the tunnel had not been constructed as it was originally planned. The understanding of the mechanism of failure required cooperation between the field and the academic engineer. Their cooperation aimed to study and recommend the appropriate measures of tunnel rehabilitation and strengthening. Therefore, a detailed inspection programme was addressed by the academic engineer that was performed by the field engineer on site. Area measurements of the hollow section of the tunnel were undertaken in order to approximate the level of convergence of successive sections of the tunnel in their existing state. Finally, the performance of the drainage bores was recorded at successive sections of the length of the tunnel. The field engineer elaborated an overview table that included the drainage calibration of drop flow and the tunnel inspection that was provided to the academic engineer for interpretation, discussion and further study for the retrofitting and strengthening measures.

INTERPRETATION OF THE FAILURE MECHANISM

The height of overburden strata at the area of failure is 45 m. In the area of failure different phases of mudrock and sandstone flysch, which are characterised by a different permeability are brought together [2]. For these reasons, a large accumulation of water is observed in this region. This flow probability is due either to the presence of fractured zone characterised by a considerable porosity in relation to the adjacent healthier rock mass or to the proximity of the relatively permeable sandstone flysch relative to practically impermeable or slightly permeable mudrock flysch, the contact of which constitutes *negative* border on the movement of groundwater. These overlying conditions of the surrounding strata were observed *in-situ* during the drilling of the drainage bores at the stage of implementation of the project. The pathology and the accumulation of failures inside the tunnel indicated that the applied temporary support measures that were not followed by the final support measures were not able to be received in the long term and secure the loads applied by the rock mass. The most significant convergence of the sides of the tunnel was observed at CH 150-168 (distance from the tunnel's exit) as emerged from the in situ measurements (Table 1). Convergence was obvious together with the cracks in the contact of vertical walls with shotcrete lining. Additionally, the buckling and shear failures of the existing temporary steel protective measures were observed behind the shotcrete lining. The above convergence was performed partly by the convergence of rock mass, as well as of the vertical wall in the sides of the tunnel, which was greater. The reason for the above mentioned phenomenon was the low resistance and the further deterioration of the quality of the surrounding rock mass, to which the presence of water contributed. The phenomenon was not connected to the adjacent landslide and was restricted between the CH 139 m - 178 m of the tunnel's length.

Table 1: Overview table of tunnel inspection and drainage calibration in CH 150-168.

Chain age	Drainage category	Section deformation profile before retrofitting	Special observations
150-168	CH150 ++ CH. 152 ++ (1 bore) CH. 154 ++ (2 bores) CH. 156 ++ (2 bores) CH. 158 ++ (2 bores) CH. 158 +++ (1 bore) CH. 160 ++ (1 bore) CH. 162 + (1 bore)		C.H.151-166: Buckling and shearing of metal temporary supportive measures C.H.159-163: Cracks in the shotcrete at the left side of tunnel C.H.158-162: Cracks in the shotcrete at the right side of tunnel C.H.162: Overturning of the left wall C.H.168: Exposure of metal net

\+ Single drops in the bore
++ Weak drop flow
+++ Intense drop flow

RETROFITTING AND STRENGTHENING WITH PERMANENT TUNNEL LINING

Initial Design Study

The area of extensive failures of the temporary supportive measures (shotcrete, low concrete wall, steel members) was found necessary to be lined using in-situ reinforced concrete (CH 139 m -168 m). The estimation of the magnitude of the loads that were applied in a permanent liming of the tunnel was made in principle using the empirical method of Bieniawski [3] and, then, by estimation through separate numerical analysis the loads that were active in the tunnel lining before the implementation of the retrofitting measures. This was done in the study of Dounias and Tzanis in an effort to simulate the existing stress field on the perimeter of the tunnel [4]. This separate analysis was undertaken using the final elements software, PLAXIS V8. This approach for the estimation of the overburden load that was exerted on the final lining of the tunnel was a methodological approach with two advantages. Firstly, it improved the estimation of the overburden loads that a professional engineer could estimate as a first approach and, secondly, it provided a useful tool to estimate the overburden loads with better precision utilising specialised finite element software. In this part of research, the knowledge and the academic engineers' guidance was required. In Table 2, the estimated vertical loads in the final lining of the tunnel are shown.

Table 2: Estimated vertical loads in the final lining of the tunnel (kPa).

			Assuming Bieniawski formulae	Analysis by PLAXIS	Final Design
Strata	Overburden (m)	GSI	P_B	P_P	P_D
Clay shale	45	(-)	(kPa)	(kPa)	(kPa)
		15	75	200	200

A vertical load of $200kN/m^2$ was selected as a most probable and slightly conservative estimate of the load of the overburden strata. In the static calculations, potential active future loads of the rock mass were applied assuming that they were induced as vertical loads at the dome of the tunnel and as horizontal loads on the walls (sides) of the tunnel. Both loads were considered uniformly distributed throughout the perimeter of the above mentioned structural parts of the tunnel. The horizontal loads were considered as a percentage λ of vertical loads applied. This ratio was considered to vary in a relatively wide range, as there was insufficient data to select a single value. The static analyses were made assuming two loading coefficients $\lambda = 0.5$ and $\lambda = 1.00$ for the calculation of the worse combination of loading actions for the dimensioning of the permanent lining. The new tunnel lining was designed to fully drain the surrounding ground water level from the permanent lining and, therefore, it was expected that no hydrostatic pressures would be built. The finite elements software SAP 2000 was utilised to perform the static analyses to dimension the final lining of the tunnel [5]. The concrete tunnel lining was simulated by thick beam elements. The surrounding rock strata were simulated as equivalent joint elements (Figure 3). Again successive cooperation between professional engineers and academics was established to carry out parametric analyses assuming different support conditions at the base of the tunnel. The reinforcement of the permanent lining of the tunnels' sides was designed to be anchored to the tunnel's floor utilising vertical steel anchors. The fixity level between the sides of the tunnel and the floor of the tunnel could not be ensured. Thus, it was considered necessary to estimate the level of stress state built on the tunnel lining assuming that: a) the sides of the tunnel were fixed in the floor of the tunnel; and b) the sides of the tunnel were pinned in the floor of the tunnel.

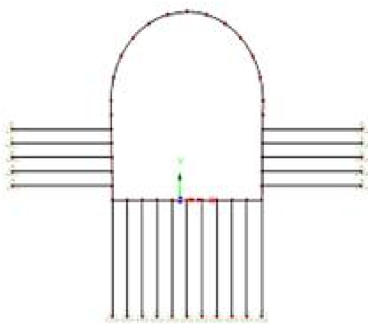


Figure 3: The tunnel lining simulation with linear elements using SAP 2000 [5].

The stress values of the internal forces used for the design of the tunnel lining are shown in Tables 3 and 4.

Table 3: Stress values in the tunnel's section under horizontal forces assuming fixed support conditions.

	Vertical sides			Dome		
	M_d(KNm/m)	N_d(KN/m)	Q_d(KN/m)	M_d(KNm/m)	N_d(KN/m)	Q_d(KN/m)
λ=0.5	214	-600	207	60	-285	111
λ=1.00	248	-608	329	14	-513	77

Table 4: Stress values in the tunnel's section under horizontal forces assuming pinned support conditions.

	Vertical sides			Dome		
	M_d(KNm/m)	N_d(KN/m)	Q_d(KN/m)	M_d(KNm/m)	N_d(KN/m)	Q_d(KN/m)
λ=0.5	30	-608	87	32	-580	80
λ=1.00	112	-639	212	70	-600	150

The two different numerical analyses showed a different distribution of internal stresses in the perimeter of the tunnel together with different deformation patterns. For that reason, it was decided to utilise the appropriate academic advice for the use of reinforcement that corresponded to the worse stress condition in the perimeter of the tunnel.

Proposed Retrofitting Scheme

The initial study of the tunnel retrofitting and strengthening scheme was completed in January 2012 [4]. However, it was not until February 2015 that the project was awarded to the contractor. During this period, the deformation of the tunnel in certain areas evolved due to insufficiency of the existing drainage bores that led to overburdening hydraulic pressures and movement of the surrounding rock in the perimeter of the tunnel. There were three questions that needed to be asked by the academic engineers: a) how the geometry of the proposed tunnel lining should be changed to take into account the deformation status of the lining of the tunnel incorporating the temporary support measures?; b) what type of formwork (mechanised or in-situ) should be utilised for the tunnel lining?; and c) whether there was a need to perform additional analyses to evaluate the newly proposed permanent lining that incorporated the changes in the geometrical and shape characteristics? The first question was answered by slightly decreasing the perimeter of the retrofitted lining, while keeping the appropriate distances that allowed the movement of vehicles and tools inside the tunnel. The answer to the second question was the use of *in-situ* formwork for the concrete lining that could match the changes in the profile and the diameter of the tunnel. The *oval* tunnel lining (Figure 4) as originally described in the relevant study should be changed to *polygonal* lining (Figure 5). The third question required the advice of academics, who needed to propose a new tunnel lining with the same stiffness and bearing capacity as that proposed in the initial study, but taking this time into account slightly decreased geometrical profile and polygonal shape of the lining. The new *polygonal* tunnel profile should have kept the general *oval* shape, the stiffness and the bearing capacity of the tunnel profile described in the initial study.

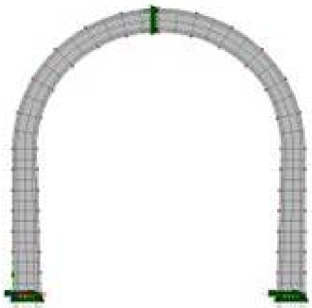


Figure 4: Simulation of *oval* tunnel lining using Lusas 14.7 [6].

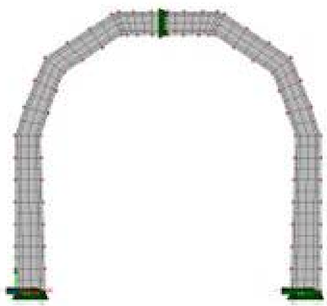


Figure 5: Simulation of *polygonal* tunnel lining using Lusas 14.7 [6].

Simulation of the R/C Tunnel Lining

The pushover non-linear analysis method [7] was utilised to investigate the performance of the originally proposed tunnel *oval* lining (Figure 4) and compared with the performance of the newly proposed *polygonal* (Figure 5) tunnel lining under permanent vertical loads and gradually increasing lateral loads that approximately represent the induced earth pressures. The purpose of pushover analysis was to evaluate the expected performance of the two structural systems in terms of initial stiffness, strength and deformation demands. The two analyses were carried out by applying incremental displacements up to 3.5‰ total drift value at the dome of the tunnel for both types of linings. The finite element simulation employed for the approximation of the newly proposed *polygonal* R/C tunnel lining is shown in Figure 5. The Lusas 14.7 finite element software has been utilised to perform the non-linear push over numerical analyses of the reinforced concrete tunnel linings [6]. In this numerical model the sides and the dome of the tunnel are simulated utilising plane stress elements (Figures 4 and 5).

It was assumed that a single non-linear material law including an isotropic multi crack concrete failure criterion governed the behaviour of the concrete tunnel lining (Figure 6) [8][9]. The longitudinal and shear reinforcement were simulated separately utilising bar elements that could only deform axially. It was assumed that the elements simulating the reinforcement were behaving elasto-plastically. The linear and non-linear mechanical properties of the concrete and reinforcement that were utilised in this numerical simulation are listed in Tables 5 and 6. A plot of the total base shear versus top displacement was obtained for the two types of linings (Figure 7). The comparison between these two analyses showed a slight insufficiency in terms of bearing capacity of the proposed *polygonal* type of lining in respect with the *oval* type of lining of the initial study. It was, therefore, proposed by the academic engineers to upgrade the concrete class of the *in-situ* concrete used in the *polygonal* type of lining by one class. The new pushover analysis that was performed for the upgraded *polygonal* lining showed satisfactory agreement with the *oval* tunnel lining of the initial study and not undesirable failure pattern (Figure 7).

Table 5: Strength of concrete used for the lining of the tunnel.

Stage of implementation	Concrete class	Young modulus of concrete (N/mm^2)	Compressive strength of concrete (N/mm^2)	Tensile strength of concrete (N/mm^2)	Strain at peak uniaxial compression ε_{cr} (%o)	Strain at ultimate uniaxial compression ε_{cu} (%o)	Fracture energy per unit area N/mm
Design *oval*	C20/25	30500	25	3.30	0.001	0.003	0.005
As built *polygonal*	C25/30	32000	30	3,80	0,001	0,003	0,005

Table 6: Tensile strength of the reinforcement used for the lining of the tunnel.

A/α	Young modulus (N/mm^2)	Yield tensile stress f_{sy} (N/mm^2)	Strain at yield ε_{sy} (‰)	Strain at ultimate stress ε_{su} (‰)
Φ18	$209X10^6$	500	0.0024	0.1
Φ12	$209X10^6$	500	0.0024	0.1
Φ12 (stirrups)	$209X10^6$	500	0.0024	0.1

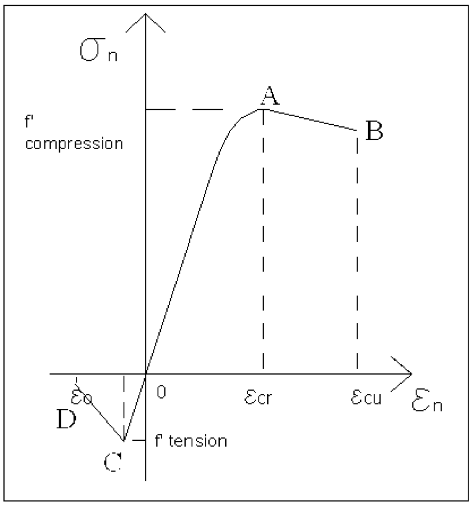


Figure 6: Stress-strain curve for the simulation of plane stress elements representing *concrete*.

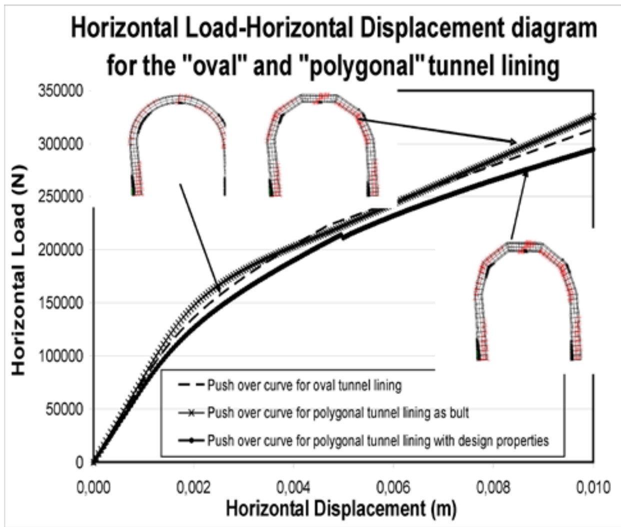


Figure 7: Horizontal load-displacement diagram for the *oval* and *polygonal* tunnel lining with failure pattern.

STEP BY STEP COOPERATION BETWEEN ACADEMICS AND PROFESSIONAL ENGINEERS

The steps that defined the cooperation between the academics and professional engineers in the described project are depicted in Figure 8. The main characteristic is that the project required academic proficiency and advice at two different stages; one at the stage where design of the retrofitting measures was scheduled, and one during the construction process where a review of the design solution was required. More specifically, in the above project, academic advice was needed initially to evaluate the conditions that led to the failure of the tunnel and propose measures to restrain the evolution of the movement of the tunnel's profile. During the construction stage there was a need by the academic engineers to propose a new tunnel (polygonal) profile that met the design requirements of the originally proposed *oval* tunnel profile.

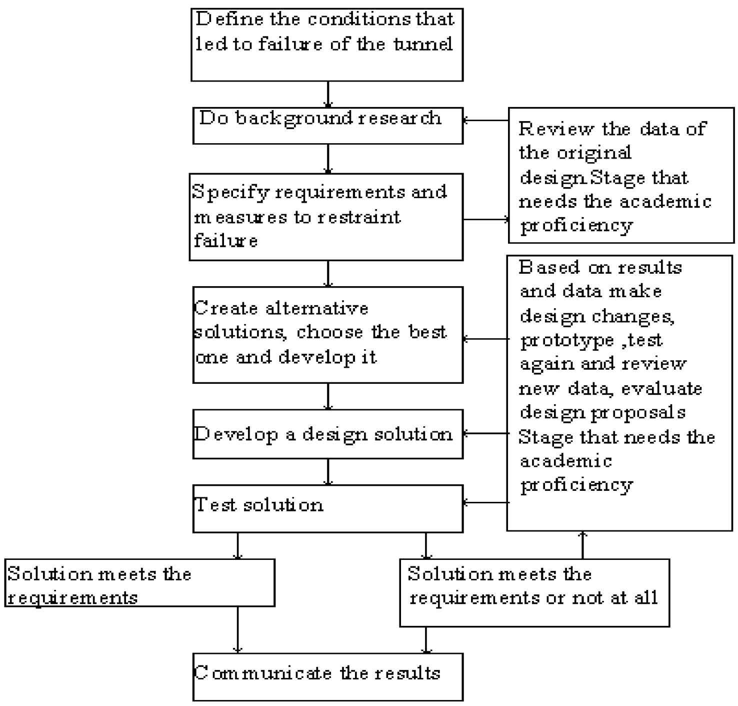


Figure 8: Steps that defined the cooperation between the academics and professional engineers.

CONCLUSIONS

Strong cooperation between professional engineers and academic engineers is essential in significant projects as the one described. Moreover, the proficiency and advice of academic engineers is demanded in cases where the professional engineer has to face *in-situ* engineering problems that were not foreseen at the original design stage. A typical case study of retrofitting and strengthening an existing tunnel that can be a valuable methodological tool for relative engineering problems is described in the previous paragraphs.

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