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Numerical Investigation of the In-situ Restoration and Strengthening Schemes of a Damaged Drainage Tunnel

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Abstract The numerical investigation of the structural schemes that are studied and proposed for the restoration and strengthening of the existing drainage tunnel are presented in this paper. In the area of St. Dimitrios village near the Dam area of Evinos in Western Greece a drainage tunnel was constructed for the stabilization of the active landslide that initiated soon after the construction of Evinos Dam. The stabilization works that are presented in this study took place in an area of extensive failures within the drainage tunnel that runs below the toe of the landslide. The drainage tunnel presented serious structural problems due to failure of drainage measures of the surrounding slope area that increased the imposed lateral pressures on the walls of the tunnel especially in areas in the interface between sandstone flysch and mudrock shale. There was an overturning phenomenon of R/C walls in the tunnel sides, failure of the shotcrete casing and buckling of some steel members that composed the temporary stabilization measures. After the initiation of works inside the tunnel and especially after the placement of the contractor in-site, significant problems have emerged concerning the structural integrity of the tunnel section. The main problem was the continuous convergence of the tunnel sides that obliged the supervisory authority to change the restoration and strengthening scheme during the course of the works and modified the R/C tunnel lining section and R/C material parameters.

Keywords Restoration, Drainage Tunnel, Numerical Investigation, Non-linear Analysis

1. Introduction

In 1993 during the construction of Evinos Dam, a landslide activated in the area between St. Dimitrios and Arachova village ("Road" landslide) in Western Greece. The area of the landslide lays in the left abutment of Evinos Dam which is part of the Athens water supply system. Part of the stabilization works that took place was the construction of a drainage tunnel below the landslide area. The drainage tunnel was consisted by two branches of total length 713m [1]. The two branches were constructed in the region below the landslide and were joined to a common branch which headed west to the exit of the tunnel at the adjacent stream. The cross section of the tunnel had radius ranging from 1.85 - 1.90m at the dome and a height of 1,85m at the vertical sides. The width of the floor was 3, 7 - 3,8m. The areas of failures inside the tunnel were located at the south left abutment of the reservoir dam of Evinos, about 140m from the exit of the tunnel (figure 1) [2] at the common branch of the drainage tunnel. This area laid on the side of the landslide. The initial lining of the drainage tunnel was done using steel temporary retaining measures that were accompanied by shotcrete for the dome and part of the sides of the vertical walls. A low R/C retaining wall was constructed at the toe of the vertical sides (figure 2). In figure 3, a longitudinal section of the St.Dimitrios –Arachova drainage tunnel is shown together with the zone where damages have been observed. The permanent lining of the tunnel using R/C structural members had not been constructed although it was originally planned.

Soon after the construction of the tunnel cracks and detachments presented in the tunnels shotcrete lining that were more intense on the sides than the dome of the tunnel. Furthermore, significant size overturning occurred in the R/C toe 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. These failures took the form of local buckling and shear failure of the temporary steel members that were placed in the perimeter of the domed tunnel. A detailed inspection program was addressed by the designer that was performed on site by the supervisor engineers. Area measurements of the hollow section of the tunnel were undertaken in an effort to approximate the level of convergence of successive sections of the tunnel in their existing state. Additionally, the calibration of drainage flow through the existing drainage bore system of the tunnel was recorded at successive sections of the tunnel's length (Table 1).

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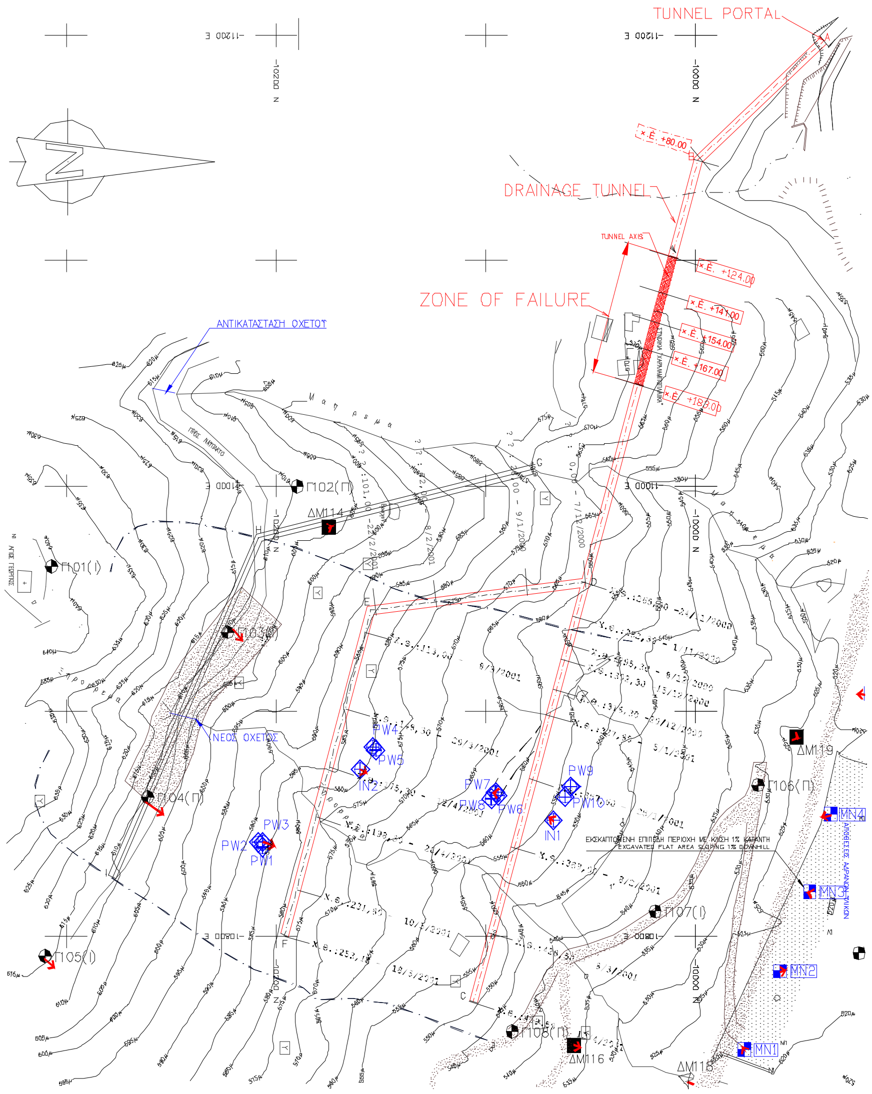


Figure 1. Plan view of the St. Dimitrios –Arachova drainage tunnel

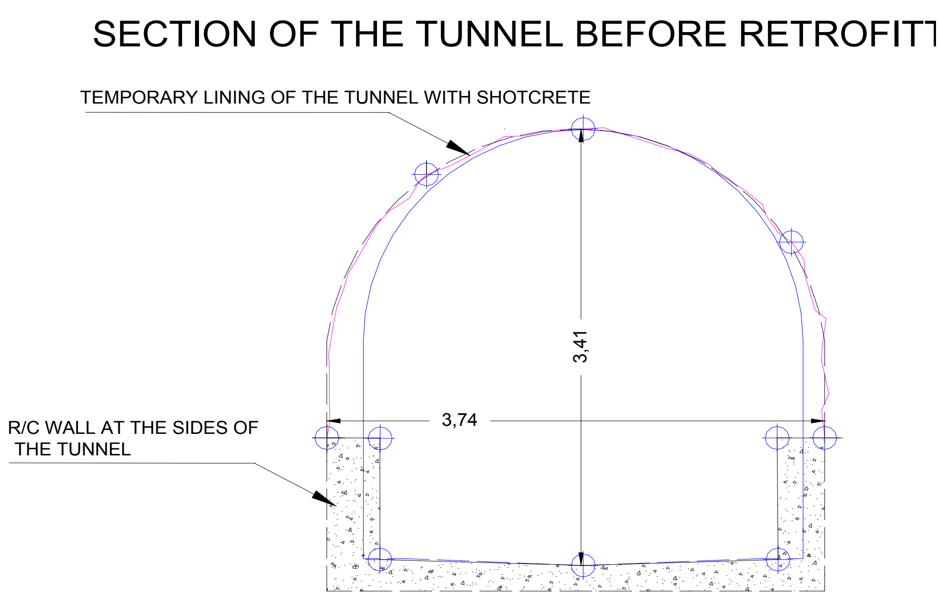


Figure 2. Typical section of the tunnel with shotcrete lining that suffered failures

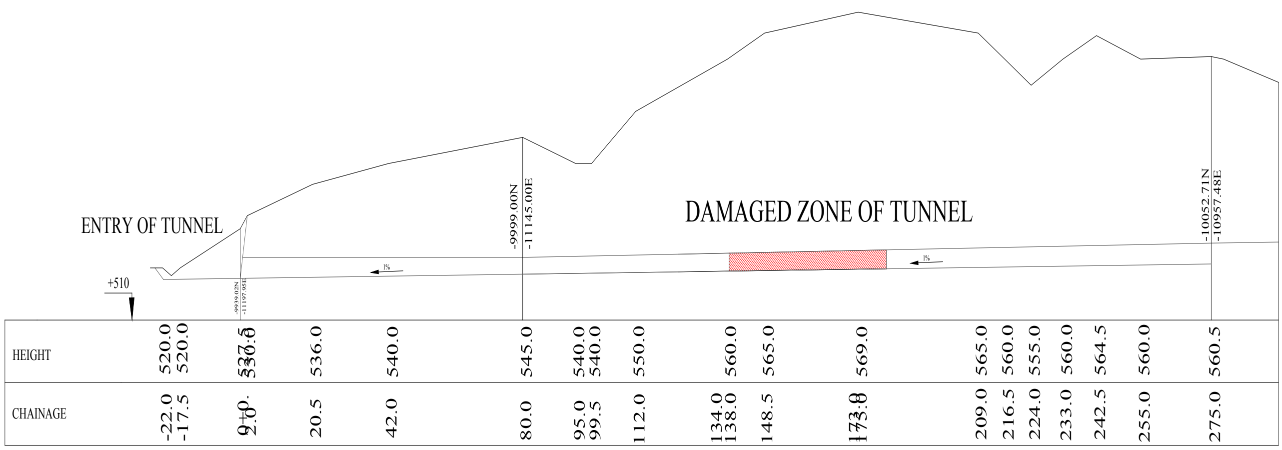


Figure 3. Longitudinal section of the St.Dimitrios –Arachova drainage tunnel

The initial study of the tunnel restoration and strengthening scheme [3] was completed in January 2012. This study concluded with the proposal of R/C tunnel lining in the perimeter of the structure. However, it was not before February 2015 that the project was awarded to the contractor. During this period the convergence of the tunnel sides in certain areas continued to evolve due to the insufficiency of the existing drainage bores that led to overburden hydraulic pressures and surrounding rock movement in the perimeter of the tunnel. Therefore questions raised that needed to be asked by the designers of the project and the supervisor engineers. The geometry of the proposed tunnel lining should be changed to take into account the new deformation status of the existing tunnel's profile incorporating the temporary support measures that were different from that originally designed. The space unavailability and the evolution of the structural deformation inside the tunnel rendered the use of pre-cast mechanized formwork impossible. Finally, there was an urgent need to perform additional analyses to evaluate the newly proposed permanent lining that incorporated the changes in the geometrical and shape characteristics. All these questions are answered through an analytical procedure that followed two phases of investigation which are presented in the next sections.

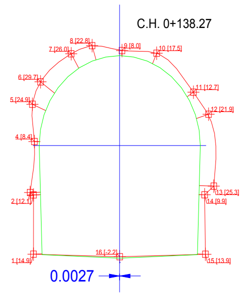
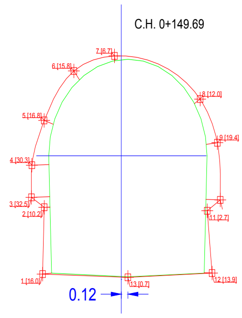
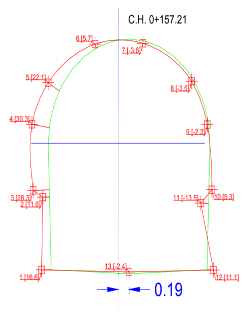
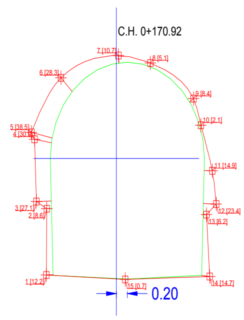
In 1976 Bieniawski [4] published the geomechanical classification of rock mass (Geomechanics Classification of the Rock Mass Rating - RMR system). Over the years, this system has been successfully refined as more data have been examined and Bieniawski [5] has made significant changes in the classifications assigned to different parameters. The RMR classification of the surrounding rock strata of the drainage tunnel under consideration was poor and did not exceed the 15 level. However, under severe in-situ conditions that caused the deformation of the tunnel section, there was a need to propose a suitable R/C lining section that adopted a "polygonal" tunnel shape with four radii of curvature for the dome, the sides and the base of the tunnel instead of the "oval" shape proposed initially.

S.C. Möller, P.A. Vermeer [6] reviewed empirical and analytical approaches for the approximation of settlements and forces on tunnel linings, placing emphasis on the sequences of excavation and installation of support on tunnel linings. Such installation procedures range from open face to close face tunnelling and they tend to have a significant effect on deformations and lining forces. In their research they have performed two-dimensional FE-analyses based on characteristic case studies. Additionally, P. Kounis [7] in his postgraduate thesis proposed a methodology for the implementation of static analysis on different road tunnel sections. In each tunnel section elastic and non-linear analyses were performed. A non-linear numerical analysis is an accurate analysis method that takes into account the non-linear behaviour of the materials (steel and concrete), stress levels that are built in the tunnel sections as a result of second class phenomena and the interaction between the R/C tunnel lining and the surrounding strata.

Dawei Mao et.al [8] carried out numerical simulation on the failure of a Hanekleiv road tunnel. The effects of weakness zones containing swelling clay on tunnel stability were examined. The researchers had focused on several selected mechanical states, particularly the one representing the long term loading on rock support. Both the detected shotcrete cracks during tunnel excavation and the tunnel collapse have been verified. The construction of heavy rock support system that was achieved either by reinforced shotcrete ribs or by cast-in-place concrete lining is proposed by the researchers as remedy mean.

In the proposed investigation a numerical analysis was performed in two phases to evaluate different R/C tunnel lining sections and propose one that could be easily implemented on-site without losing its structural reliance. In the first phase a non-linear numerical analysis is performed to evaluate the stress levels that are built in the perimeter of the tunnel assuming different tunnel lining shapes. In the second phase a non-linear push over analysis is performed to evaluate the stiffness, bearing capacity and the failure modes of the proposed tunnel lining section that was selected from the first phase.

Table 1. Overview table of tunnel inspection and drainage calibration

CHAINAGE	DRAINAGE CALIBRATION	SECTION DEFORMATION PROFILE BEFORE RESTORATION	SPECIAL OBSERVATIONS
131 - 142	-		Ch 139 – 142: Failure of shotcrete in the dome of tunnel, in the area where supplementary shotcrete has been sprayed in 2006
142 - 150	CH. 142-145 ++ CH. 145 – 146 +++ ~ C.H. 148 +++ ~ C.H. 150 ++		C.H.142 – 145 : Water drops in the drainage bores C.H.142 – 143 : Exposure of metal net C.H. 148 – 150: The left wall presents a significant overturning C.H. 148– 150: Mudrock geological strata
150 -168	CH150-152 ++ CH. 154 ++ (2 bores) CH. 156-158 ++ (4 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.156-158: Mudrock geological strata C.H.158-162: Cracks in the shotcrete at the right side of tunnel C.H.162: Overturning of the left wall
168 - 178	-		C.H.168: Overturning of the left wall, Exposure of metal net at the dome

\+ Single drops in the bore
++ Weak drop flow
+++ Intense drop flow
Where there is no drainage calibration, no water drop was observed in the drainage bores

2. Interpretation of Failure Mechanism

The height of overburden strata at the area of failure is 45m. In the area of failure different phases of mudrock and sandstone flysch, which are characterized by a different permeability [2] are brought together. For these reasons large accumulation of water is observed in this region. This flow probability is either due to the presence of fractured zone characterized by a considerable porosity in relation to the adjacent healthier rock mass, or due to the proximity of the relatively permeable sandstone flysch relative to practically impermeable or slightly permeable mudrock shale, the contact of which constitutes "negative" border on the movement of groundwater. The pathology and the accumulation of failures inside the tunnel indicated that the applied temporary support measures that were followed by the final support measures were not able to receive in long term and safely the loads applied by the rock mass. The convergence of the sides of the tunnel has exceeded 20cm, as emerged from the in situ measurements. 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 in the dome of the tunnel as well as in the vertical sides of the tunnel which was greater. The reason of the above mentioned phenomenon was due to the low resistance and the further deterioration of the quality of the surrounding rock mass. The adjacent landslide was not connected directly to the phenomenon of damages. However, the underground water flow that has common origin with the mechanism of landslide had significant contribution to the damages observed between CH 139m-178 m of the tunnel's length. After all, possible closure of the drainage tunnel near its exit could cause interruption of the function of the tunnel and built up of hydraulic pressures in the previous sections of the drainage tunnel.

3. Restoration and Strengthening with R/C Tunnel Lining

In this section the methodological approach is presented in an effort to propose the final R/C tunnel lining. The procedure is composed by two phases. Initially, a non-linear analysis is adopted to evaluate different structural restoration and strengthening R/C lining schemes. Secondly, a fully non-linear "push-over" analysis is taking place to examine the stiffness, bearing capacity and failure modes of the finally proposed R/C tunnel lining.

3.1. Numerical Investigation of Different R/C Tunnel Linings

The parameters of the deformability and strength of the surrounding rock strata was determined utilizing data of the initial geotechnical study. These data and in-situ estimates of the designer of the project in cooperation with the supervisory authority were also utilized for the dimensioning of the shotcrete lining of the tunnel (Dounias et al [3]). The classification of GSI (Hoek et al) [9] and the failure criterion of Hoek-Brown [10] was utilized. In table 2, the assumed values of strength under compression σ_c, coefficient mi and Strength Geological index, GSI can be depicted. In the same table two sets of values are presented; the variation of strength and deformability parameters that are assembled for the surrounding rock strata and the adopted values that are used for the consequent numerical analyses. In the same table the equivalent cohesion c and friction angle φ values of the Mohr-Coulomb criterion are presented, as obtained by the Hoek - Brown failure criterion assuming 45m of overburden rock strata.

The lower strength values are adopted in the numerical analyses. This conservative approach resulted from the in-situ observations and measurements that acknowledged an extended zone of plasticity that caused significant overburden loads on the temporary supporting measures [2].

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 139m-178m). 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 [4], and then through a separate numerical analysis. The scope of this analysis is to determine the loads that are active on the tunnel lining before the implementation of the restoring measures. This was done in an effort to simulate the existing stress field on the perimeter of the tunnel. This separate analysis was undertaken using the final elements software, PLAXIS V8 [11].

The vertical load that is exerted on the dome of the tunnel corresponds to the loose rock strata with unit weight of 25.0kN/m^3. The depth of the surrounding loose rock strata is assumed to be 8 m.

Table 2. Material Strength Values Adopted for the Numerical Analysis

	σc (MPa)	GSI	mi	c (KPa)	φ (o)	v	E (MPa)
Variation Range	5 - 10	15 - 25	6 – 8	50-100	22-33	0,3	298-750
Adopted values	5	15	6	50	22	0,3	298

Table 3. Estimated Vertical Loads in the Final Lining of the Tunnel (kPa)

			Assuming Bieniawski formulae	Analysis by PLAXIS	Final Design
Strata	Overburden (m)	GSI (-)	PB (kPa)	PP (kPa)	PD (kPa)
Mud rock	45	15	75	200	200

The tunnel lining in this section of the tunnel under consideration ranges in depth that spans from 31-45m. Earthquake loads are not considered to act on the tunnel lining under study, in that depth.

A vertical load of 200 kN /m^2 was selected as a most probable and slightly conservative estimate of the load of the overburden strata (Table 3). In the static calculations, potential active future loads of the rock mass were applied assuming that they were induced as vertical load at the dome of the tunnel and as horizontal load 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 non-linear 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 tunnel lining. The proposed 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 are built on the proposed tunnel lining. The finite elements software Lusas 14.7 [12] has been utilized to perform a non-linear analysis to dimension the final lining of the tunnel. The R/C tunnel lining was simulated by thick beam elements that varied in width in the perimeter of the tunnel lining (0,30m-0,25m). A sensitivity analysis was carried out assuming different support conditions at the base of the tunnel section. The reinforcement of the permanent lining of the tunnels' sides was designed to be anchored to the tunnel's floor utilizing vertical steel anchors. However, 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 stresses built on the tunnel lining assuming that: a) the sides of the tunnel were fixed in the floor of the tunnel (figure 4), and b) the sides of the tunnel were pinned in the floor of the tunnel (figure 5).

The interaction between the surrounding rock strata and the R/C tunnel lining was simulated with equivalent non-linear joint elements (Winkler type figures 4, 5). These elements were assumed to behave non-linearly in the compressive range of their behaviour. The adopted tensile strength of these joint elements was assumed to be equal to 0.

The axial stiffness of the joint elements is given by Kavvadas [13]:

$$K_{axial} = \frac{E}{(1+v)R} \tag{1}$$

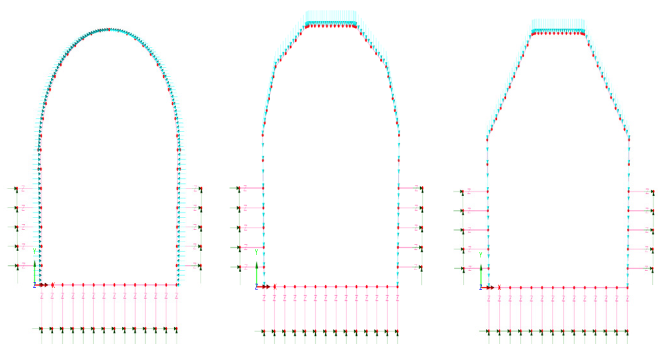
The transverse stiffness of the joint elements is given by Kavvadas [13]:

$$K_{transverse} = 0.3 * K_{axial} \tag{2}$$

Where : E= Young Modulus of Rock Strata
v= Poisson's Ratio of Rock Strata
R= Curvature Radius of tunnel lining

The curvature radius of tunnel lining was assumed to be equal with half the length of the largest tunnel opening. The compressive strength of non-linear joint elements was defined by multiplying the measured compressive stress σ_c of surrounding rock with the effective area of the tunnel wall where each joint was active.

Three different R/C tunnel linings were evaluated in terms of their structural performance and the stress values that are developed under the above mentioned combination of vertical and horizontal loading. The first tunnel lining was assumed to have the "oval" shape as originally proposed by the designer. The second tunnel lining was assumed to have a "polygonal" shape to compensate the design requirements in respect to the site limitations. The third tunnel lining that was studied was assumed to be of "trapezoidal" shape that would have beneficial results in terms of construction time and effort but it deviated from the designer's proposed solution. In figure 4, the different R/C tunnel linings that were tested numerically are shown assuming fixed support conditions. The internal stresses as they resulted from the numerical analysis for the three different shapes of tunnel linings are shown in table 4.



a) Oval R/C tunnel lining b) Polygonal R/C tunnel lining c) Trapezoidal R/C tunnel lining

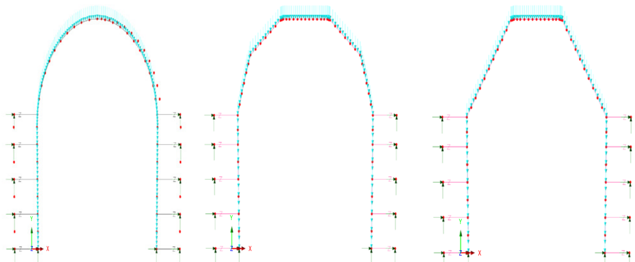
Figure 4. Numerical simulations of R/C lining shapes that were tested in terms of structural performance assuming fixed support conditions

Table 4. Internal Stresses in the Tunnel's Lining under Combination of Vertical and Horizontal Forces Assuming Fixed Support Conditions

	Vertical walls								
	Md(KNm/m)			Nd(KN/m)			Qd(KN/m)		
	a	b	c	a	b	c	a	b	c
λ=0.5	66/69/61			600/637/580			142/146/126		
λ=1.00	134/129/130			600/636/582			245/241/236		
	Dome								
	Md(KNm/m)			Nd(KN/m)			Qd(KN/m)		
	a	b	c	a	b	c	a	b	c
λ=0.5	24/24/37			430/434/390			75/117/148		
λ=1.00	26/21/62			452/432/390			51/125/239		

(a)=oval
(b)=polygon
(c)= trapezoidal

The same structural configurations are tested assuming pinned support conditions. In figure 5, the different R/C lining shapes that were tested numerically are shown assuming pinned support conditions. The internal stresses as they resulted from the numerical analysis for the three different shapes of tunnel linings are shown in table 5.



a) Oval tunnel lining b) Polygonal tunnel lining c) Trapezoidal tunnel lining

Figure 5. Numerical simulations of R/C lining shapes that were tested in terms of structural performance assuming pinned support conditions

Table 5. Internal Stresses in the Tunnel's Lining under Combination of Vertical and Horizontal Forces Assuming Pinned Support Conditions

	Vertical walls								
	Md(KNm/m)			Nd(KN/m)			Qd(KN/m)		
	a	b	c	a	b	c	a	b	c
λ=0.5	27/26/40			597/612/609			79/87/53		
λ=1.00	96/101/83			600/614/610			180/187/172		
	Dome								
	Md(KNm/m)			Nd(KN/m)			Qd(KN/m)		
	a	b	c	a	b	c	a	b	c
λ=0.5	21/26/25			430/423/396			60/115/157		
λ=1.00	60/63/93			435/439/427			103/167/283		

(a)=oval
(b)=polygon
(c)= trapezoidal

3.2. Proposed R/C Tunnel Lining

The three numerical analyses for the three different cases of tunnel linings' showed different distributions of internal stresses in the perimeter of the tunnel together with different deformation patterns. The “oval” tunnel lining showed better redistribution of stresses with smaller shear forces exerted on the vertical walls and the dome of the structure. The “oval” tunnel lining also experienced higher axial forces at the vertical walls and the dome of the structure. However, the need for pre-cast, mechanized formwork that would be difficult to be assembled especially in the current case of restoration where the deformation of the tunnel’s profile was continuous and not uniform rendered this solution and the subsequent “oval” tunnel lining difficult to be applied. Additional obstacle to this solution was the short opening of the tunnel profile and the existence of a turn in plan just 20m before the zone of restoration and strengthening (figure 1). The “trapezoidal” tunnel lining produced higher shear stresses in the dome of the structure and lower axial stresses in the vertical walls and the dome of the structure. This solution resulted in shear stresses on the dome of the structure that were significantly higher than the shear stresses of the “oval” lining solution. The bending moments that were developed in the dome of the lining utilizing this solution was also high in respect with the “oval” solution. This structural behavior was undesirable especially for a tunnel with low RMR value (Kounis [7]). The “polygonal” lining was the compensation between the desirable structural behavior that was attained by the “oval” solution that did not produce high shear stresses and moments in the perimeter of the tunnel lining and the “trapezoidal” lining. The advantage of the “polygonal” lining was the ability to be constructed in-situ utilizing in-situ cast formwork that could be adjusted to embody the changes in the profile of the tunnel due to its lateral deformations.

However, the “polygonal” tunnel lining should be constructed with the same stiffness and bearing capacity as that proposed in the initial study and attained by the “oval” tunnel lining. Therefore, a fully non-linear "push over" numerical analysis should be performed for the newly proposed "polygonal" tunnel lining complementary to the fully non-linear "push over" numerical analysis performed for “oval” tunnel lining to examine their differences in respect to their structural performances.

3.3. Fully Nonlinear “push over” Analysis of the Proposed R/C Tunnel Lining

The pushover non-linear analysis method [14] was utilized to investigate the non-linear performance of the “polygonal” tunnel lining. As was mentioned above the originally proposed “oval” tunnel profile was unattainable due to the continuously induced lateral pressures of the surrounding rock mass that changed the existing in-situ tunnel profile. The objective of the non-linear analysis was to predict the behaviour of the originally proposed tunnel “oval” lining (figure 6) and compare it with the performance of the newly proposed “polygonal” (figure 7) tunnel lining. Initially, two different numerical simulations of the tunnel lining profiles were built, one assuming “oval” tunnel lining and one with “polygonal” tunnel lining. The numerical behaviour of these two numerical simulations were evaluated by the combination of loadings that included a vertical load and a gradually increasing lateral load that approximately represented the induced earth pressures on the sides of the tunnel. The purpose of pushover analysis was to evaluate the expected performance of the two structural schemes in terms of initial stiffness, strength, deformation demands and failure patterns. 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 simulations employed for the approximation of the non-linear structural behaviour of the originally proposed “oval” tunnel lining and the newly proposed “polygonal” R/C tunnel lining are shown in figure 6 and figure 7 respectively. The Lusas 14.7 [12] finite element commercial software has been utilized to perform the non-linear push over numerical analyses of the reinforced concrete tunnel lining. In this numerical model the sides and the dome of the tunnel were simulated utilizing plane stress elements (Figure 6, 7).

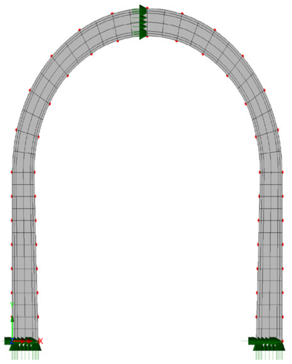


Figure 6. Numerical Simulation of "oval" tunnel lining

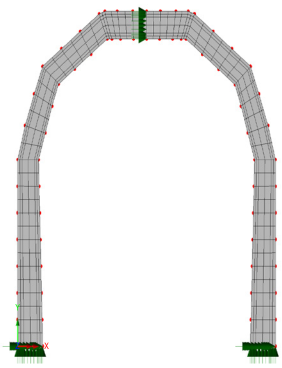


Figure 7. Numerical Simulation of "polygonal" tunnel lining

It was assumed that a single non-linear material law including an isotropic Multi crack concrete failure criterion [15, 16] governed the behaviour of the concrete tunnel lining (figure 8). The multi-crack concrete model is a plastic-damage-contact model in which damage planes are formed according to a principal stress criterion and then they are developed as embedded rough contact planes. The basic softening curve used in the model may be controlled via a fixed softening curve or a fracture-energy controlled softening curve that depends on the element size. The latter localised fracture model is applied to the un-reinforced cases of structures as the one that was studied in the current research effort, where the longitudinal and shear reinforcement were simulated separately.

The longitudinal and shear reinforcement were simulated separately utilizing bar elements that could only deform axially. It was assumed that the elements simulating the reinforcement were behaving elasto-plastically (figure 9).

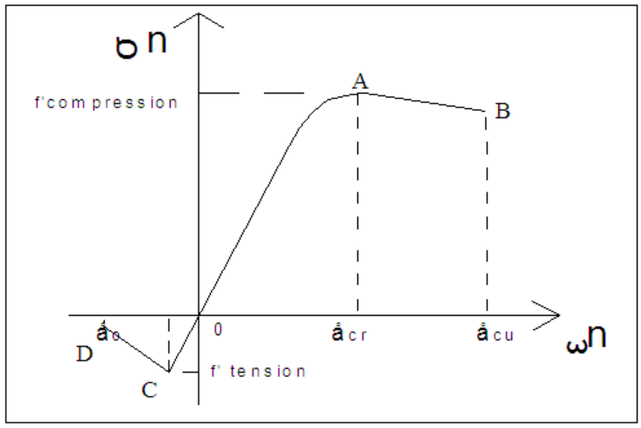


Figure 8. Stress-Strain curve for the simulation of plane stress elements representing "concrete"

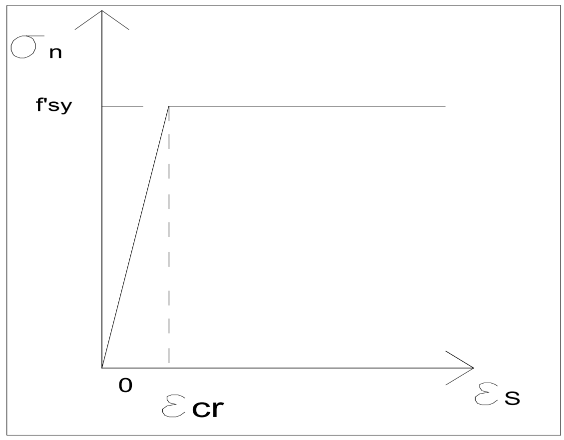


Figure 9. Stress-Strain curve for the simulation of bar elements representing "Steel reinforcement"

The linear and non-linear mechanical properties of the concrete and reinforcement that were utilized in this numerical simulation are listed in tables 6 and 7.

Table 6. 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)
Design "oval" tunnel lining	C20/25	30500	25	3.30
Design "polygonal" tunnel lining	C20/25	30500	25	3.30
As-built "polygonal" tunnel lining	C25/30	32000	30	3.80

Stage of implementation	Concrete class	Strain at peak uniaxial compression ε_{cr} (‰)	Strain at ultimate uniaxial compression ε_{cu} (‰)	Fracture energy per unit area N/mm
Design "oval" tunnel lining	C20/25	0.0022	0.01	0.1
Design "polygonal" tunnel lining	C20/25	0.0022	0.01	0.1
As-built "polygonal" tunnel lining	C25/30	0,0022	0,01	0.1

Table 7. 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	209X106	500	0.0024	0.1
Φ12	209X106	500	0.0024	0.1
Φ12 (stirrups)	209X106	500	0.0024	0.1

A numerical experiment was carried out to determine the failure surface of the concrete section adopting the material properties of the initial design and the multi-crack concrete model [15, 16]. A small concrete section (0.30 m x 0.30 m) was tested bi-axially. In figure 10, the numerical simulation of the concrete section that was tested under bi-axial load combinations is shown. In figure 11, the failure envelope that resulted from the bi-axial loading under different horizontal to vertical load ratios of the numerical simulation of the small concrete section is shown.

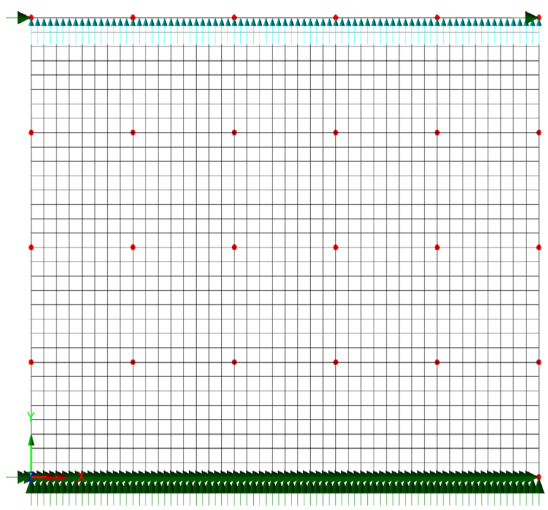


Figure 10. Numerical simulation of concrete section tested bi-axially

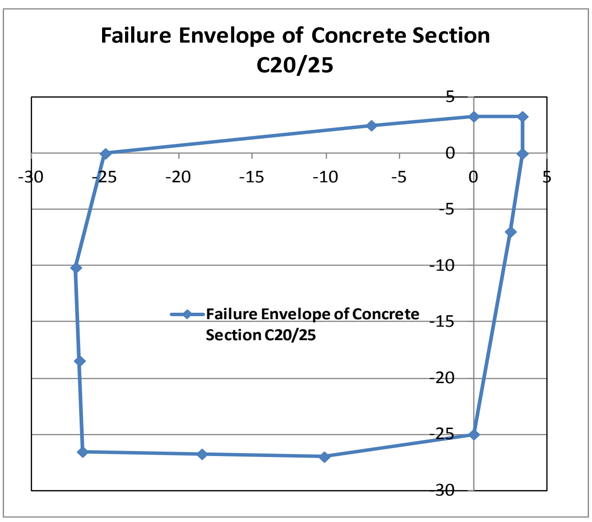


Figure 11. Failure envelope of concrete section tested bi-axially

Fully non-linear "push over" analyses were performed and the plot of the total horizontal load versus top horizontal deformation curves was obtained for the two types of linings (figure 12).

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 to upgrade the concrete class of the in-situ concrete used in the "polygonal" type of lining by one class (C25/30). A new non-linear "push over" analysis was performed assuming upgraded concrete class for the elements representing concrete and therefore upgraded numerical properties for concrete as can be depicted in table 6. The new push over analysis that was performed for the upgraded concrete class C25/30 of the "polygonal" lining showed satisfactory agreement with the "oval" tunnel lining of the initial study (figure 12).

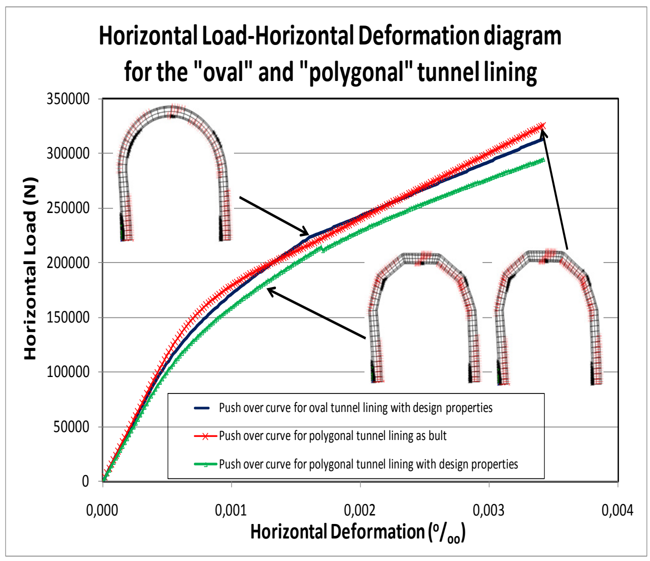


Figure 12. Horizontal Load- Horizontal deformation diagram for the "oval" and "polygonal" tunnel adopting two cases for "polygonal" lining with designed and as-built material properties

In figures 13, 14, 15 the failure modes that were recorded numerically for the three tunnel linings are also shown. In these figures, zones of crushing and cracking of the concrete tunnel lining are compared. The red signs represent areas of tensile failure of concrete (cracking), where black signs represent areas of compressive failure of concrete (crushing).

In terms of failure modes these seem to be in agreement regarding the types of failures (crushing or cracking) they are describing in the dome and the vertical walls of the tunnel lining but there is small quantitative difference that concerns the overestimation of the distribution of failures in the "polygonal" type of tunnel lining in respect with the "oval" type of tunnel lining. This overestimation is more pronounced in the case of "polygon" tunnel lining adopting design material properties C20/25 than "polygon" tunnel lining adopting as-built material properties (C25/30). Finally, the typical section of the originally proposed "oval" tunnel lining together with the as-built "polygonal" tunnel lining that incorporates the reinforcement details can be depicted in figures 16 and 17 respectively.

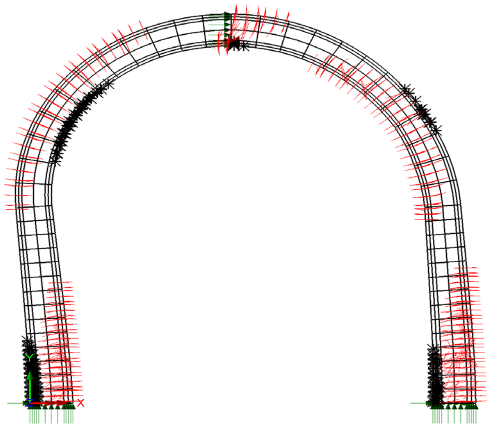


Figure 13. Cracking and crushing failure of designed "oval" tunnel lining at deformation level 3.5‰

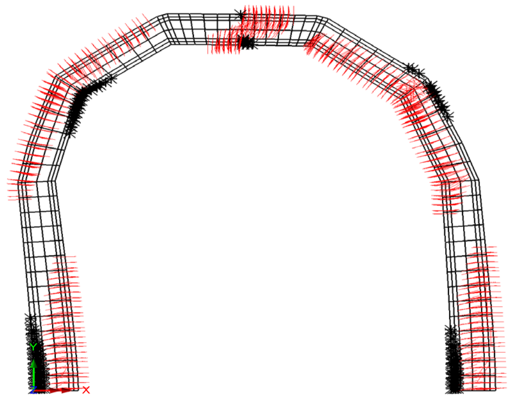


Figure 14. Cracking and crushing failure of designed "polygonal" tunnel lining at deformation level 3.5‰

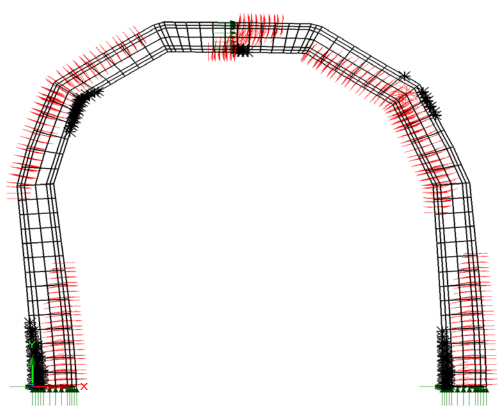


Figure 15. Cracking and crushing failure of as built "polygonal" tunnel lining at deformation level 3.5‰

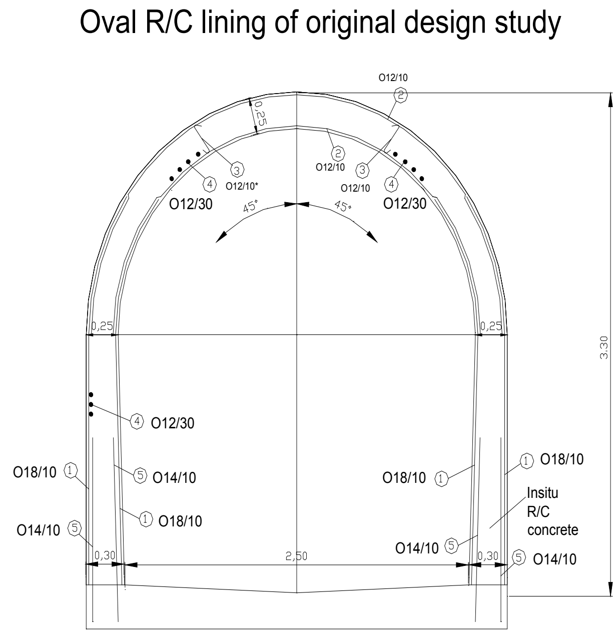


Figure 16. Originally proposed "oval" tunnel lining with reinforcement

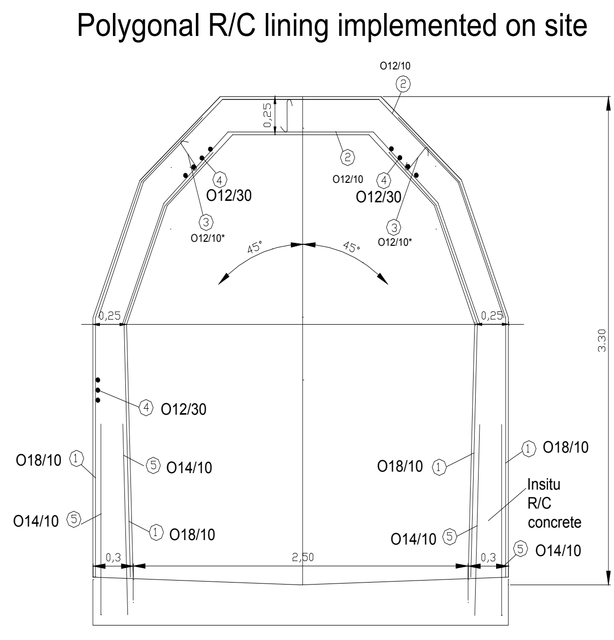


Figure 17. Final as-built "polygonal" tunnel lining with reinforcement

4. Conclusions

In this paper the restoration and strengthening of the existing drainage tunnel that spans below the landslide that occurred between St. Dimitrios and Arachova village ("Road" landslide) in Western Greece was presented.

The drainage tunnel presented serious structural problems due to failure of drainage measures of the surrounding slope area that increased the imposed lateral pressures on the walls of the tunnel especially in the interface areas of relatively permeable sandstone flysch in contact to practically impermeable or slightly permeable mudrock shale.

The restoration and strengthening scheme was concentrated in a zone of extensive damages that spanned for 39m. This involved the numerical investigation of different tunnel linings in an effort to propose a final R/C tunnel lining that could serve two purposes. Initially, to propose a tunnel lining and a construction method that could be applied inside the deformed and not uniform tunnel profile and secondly to restore and upgrade the structural performance of the existing shotcrete lining by applying an R/C tunnel lining.

The structural behaviour in terms of the internal stresses was evaluated for three different R/C tunnel linings. Namely an "oval", a "polygonal" and a "trapezoidal" tunnel lining were examined. It was found that although the "oval" tunnel lining presented upgraded structural performance as was expected, it was very difficult to be applied in-situ utilizing pre-cast and mechanised formwork due to the non-uniformity of the existing tunnel's profile, the short opening of the tunnel's profile, and the existence of a turn just 20m before the zone of proposed restoration and strengthening works.

The "polygonal" tunnel lining was the structural scheme that was selected. This structural scheme although it showed increasing internal stresses in terms of shear stresses in the dome of the structure in respect to the "oval" tunnel lining predicts stresses that can easily be accommodated by the structural design. Moreover, this solution had the advantage of the use of in-situ formwork that could be adapted to the deformed section of the tunnel profile especially in the zones of tunnel convergence and failure.

The stiffness, bearing capacity and failure modes of the proposed "polygonal" tunnel lining was compared with the same characteristics of the initially proposed "oval" tunnel lining. A fully non-linear push over analysis took place incorporating non-linear material properties for concrete and steel members.

By comparing the response of a "polygonal" tunnel lining, as predicted by the non-linear push over analysis, with the initially proposed "oval" tunnel lining, it was demonstrated that "oval" tunnel lining showed upgraded structural performance in terms of the bearing capacity.

It was shown that the resulted push over curve of the fully non-linear "polygonal" tunnel lining was in better agreement to the push over curve of the fully non-linear "oval" tunnel lining when upgraded concrete class C25/30 was utilized for the design of the "polygonal" tunnel lining.

The failure modes (crushing or cracking) as predicted by the fully non-linear analysis of the "polygonal" tunnel lining were in agreement with the failure modes as predicted by the fully non-linear analysis of the "oval" tunnel lining. However, a small quantitative difference was depicted that concerned the overestimation of the distribution failures in the "polygonal" type of R/C tunnel lining in respect with the "oval" type of R/C tunnel lining.

Following the proposed approach, it was regarded that the "polygonal" type of R/C tunnel lining could be adopted for the restoration and strengthening of the existing shotcrete lining of the drainage tunnel.

The proposed pushover analysis can be used as a useful design tool in order to assess the structural performance of different tunnel linings. The proposed approach can be utilized in the design of tunnel linings or for assessing the bearing capacity of existing tunnel linings. It can be also utilized for the evaluation of potential damage for either newly designed or existing structures as part of a potential damage screening process.

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