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NUMERICAL MODELING OF TUNNEL EXCAVATION AND SUPPORT USING THE DECONFINEMENT METHOD FOR STATIC AND SEISMIC CONDITIONS

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NUMERICAL MODELING OF TUNNEL EXCAVATION AND SUPPORT USING THE DECONFINEMENT METHOD FOR STATIC AND SEISMIC CONDITIONS

ABSTRACT

In this paper, an analysis of the phase construction of a tunnel with support in static and seismic conditions is made. The PLAXIS 2D software package was used for the problem's numerical modelling. A parametric analysis of the excavation using the deconfinement method (1-ß) was made on an actual tunnel with support in the excavation phase and primary lining with sprayed concrete and anchors. From the conducted analysis, it can be concluded that through the so-called method 1-β, i.e., the percentage enabled realisation of the deformations in the excavation, can successfully model the time needed to set up the support and obtain relevant parameters for dimensioning the elements of the lining for different load cases in static and seismic conditions and stages of performance.

Keywords: *tunnel, tunnel support, numerical modelling, deconfinement method.*

1. INTRODUCTION

This paper deals with tunnel excavation numerical analysis in static and seismic conditions using PLAXIS 2D software. The deconfinement method (1-ß) [1]-[3] was used for the parametric analysis of the excavation, which can simulate the tunnel's behaviour under different excavation conditions and duration until the support is placed.

The analysis is based on a case example of constructing an access tunnel to a hydrotechnical tunnel, which suffered certain damages during the operation phase, so its rehabilitation is needed. It concerns the Vlajnicki tunnel as part of the Mavrovo hydro system in Western Macedonia. Access to the rehabilitation parts is possible only through the construction of transverse entrances—in the case of tunnels, from where the necessary equipment will be brought in.

The Vlajnica tunnel is part of the main water supply from the Mavrovo dam reservoir to the Vrutok HPP, which has a total length of 3,163.00 m. It is intended to operate with flow Q=32 m^3/s and flow velocities of V=3.98 m/s, internal apparent diameter (clear opening) of 3,20 m, longitudinal slope of 3 ‰ and internal water pressure up to a maximum of 8 bar. The facility is of exceptional importance for HPP Mavrovo, especially for the operation of HPP Vrutok and HPP Raven, as well as for transporting water from the Shar and Jelov supply to the reservoir (reverse direction). During the construction of this tunnel, six access tunnels were excavated and built to increase the number of workplaces and reduce transport lengths.

2. METHODOLOGY

In this paper, an innovative numerical approach was used to analyse tunnel excavation, as it effectively simulates the percentage of deformation realised during the excavation phase and assesses the deformations and internal forces in the lining elements.

To improve the stability and maintain the self-support of the rock mass near the limits of the underground excavation, a primary support consisting of reinforced shotcrete and anchors was applied.

The analysis was performed for both static and seismic conditions and different values of the $1 - \beta$ parameter.

3. GEOLOGICAL, HYDROGEOLOGICAL AND SEISMIC CHARACTERISTICS OF THE LOCATION

The Vlajnicki tunnel passes through terrain with a different structure. According to its global geological-tectonic structure, it belongs to a geotectonic unit of the first rank in western Macedonia, more specifically, to the so-called Sharsko-Pelisterska zone.

The geological structure of the terrain mainly includes rocks of the Paleozoic age, represented by quartzites, schistose quartzites with ularitoschists, black clay schists, silicified black clay schists, chlorite schists, marbleised limestones, chloride schists, serpentinites less often and other rocks. It is significant for this environment to be significantly tectonically damaged with the presence of fault zones, faults, and increased cracking. This entire Paleozoic complex has undergone intense multi-grade tectonic shaping so that on the terrain today, they are represented as folds, cracks, and fault structures with the most different patterns and sizes.

Generally speaking, the black clay shale and chlorite shale are the most common for the whole tunnel, and the occurrences of more complex quartzites can be considered relatively impermeable to water, while the carbonate complex (marbles, limestones, and their mixtures) and very cracked hard environments have increased water permeability and often appear to have underground water in them.

During the excavation of the tunnel, no underground water appeared in the marbled limestones and quartzites, which indicates increased cracking and cavernousness in the limestones and quartzites, so that at this height, they had the function of a hydro-collector-conductor. Such sections are completely dry and, as an environment, are extremely water permeable. In the rest of the rocks, especially in the chloride shale, there was also water in the form of wetting or seepage. This heterogeneous engineering-geological composition of the environment was also reflected in the diversity of the strength-deformable characteristics of the rocks after the completion of the tunnel construction. Consolidation injection of the tunnel along its entire length was carried out on profiles with a mutual distance of 4 m and boreholes with a length of 0,5 m.

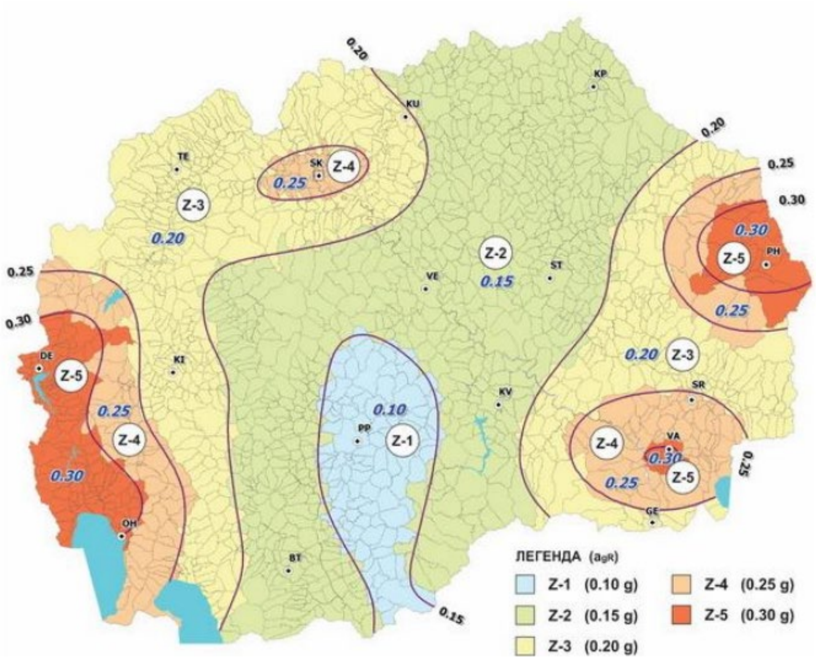


Figure 1. Seismic zoning map of Macedonia, ground-type A, Vs ≥ 800 m/s [4]. Adopted for National Annex to MKS EN 1998-1:2012 Eurocode 8

In terms of seismic characteristics, according to the seismicity map for a return period of 475 years (Figure 1), the location belongs to zone 3, with a seismicity coefficient of 0,2g.

4. GEOTECHNICAL PARAMETERS AND NUMERICAL ANALYSIS

In order to perform a geostatic calculation of the access tunnel, a numerical analysis was performed that determined the stress-deformation state of the soil masses from which the internal forces in the elements needed for the dimensioning of the lining will be obtained. The problem is modelled using the PLAXIS program package, based on FEM, which is specialised for application to continuous soil environments for various geotechnical problems, giving a simple representation of the loads and the stress-strain state depending on the strength and deformable characteristics of the soil materials [5]. It is also possible to model structural elements: reinforced concrete, geosynthetics, etc. There are many models to define soil materials, and triangular surface isoparametric finite elements with 9-node and 15-node points are also available. For this calculation, a two-dimensional analysis of the planar state of deformations was performed, where the Mohr-Coulomb model approximates the soil as a continuous medium. At the same time, a linear elastic one was applied for the shotcrete and tunnel lining.

For the analysis's needs, the geometry of the models represented by the characteristic (representative) cross-section of the tunnel together with all its elements is defined first, while the soil materials from the surrounding environment are defined through their physical-mechanical strength and deformable characteristics.

Data from previous geotechnical research and tests were used to define appropriate environmental parameters. From there, the strength-deformable parameters shown in Table 1 were adopted (on the side of reliability due to the usual indeterminacy in constructing such structures).

Table 1. Adopted ground material parameters

Parameter	Symbol	Unit	Native ground material
Material model			Mohr - Coulomb
Conditions			Drained
Modulus of elasticity	Eref	[kPa]	50 000
Water-saturated volume weight	γsat	[kN/m3]	22
Natural state volume weight	γunsat	[kN/m3]	22
Poisson's ratio	ν	[/]	0.35
Cohesion	c	[kPa]	10
Angle of internal friction	φ	[°]	20
Dilation angle	ψ	[°]	0

Table 2. Adopted shotcrete parameters

Parameter	Symbol	Unit	Shotcrete
Material model			Linear Elastic
Axial stiffness	EA	[kN/m]	3150000
Bending stiffness	EI	[kN/m]	2625
Thickness	d	[cm]	10

Table 3. Adopted seismic parameters

Symbol	Seismic parameters
Kx	0.20 g
Ky	0.10 g

The analysis is done for a tunnel with primary support that includes anchors and shotcrete. The anchors are modelled with the embedded beam row option. Steel anchors with a diameter of Ø32 and a length of 2 m are adopted, and they are placed at a mutual distance of 2 m in the longitudinal direction. The shotcrete is modelled as a plate element; the adopted thickness is 10 cm (Table 2).

The tunnel construction in PLAXIS is analysed in several stages: excavation and placing the structural elements of the tunnel.

In PLAXIS 2D, it is possible to enter a Deconfinement value during staged construction (as 1-β) in the model explorer. This enables part of the stresses (β) in the soil polygon inside the tunnel to be retained as a support pressure.

The Deconfinement (1-β) method simulates the three-dimensional soil arching behaviour around an unsupported tunnel face using a 2D model, like PLAXIS 2D.

There are two ways to set this up in PLAXIS 2D:

- using the method of partial staged construction (setting the phase's general ΣMstage value to 1 – β);
- or using the Deconfinement option for a deactivated soil cluster.

A deconfinement value can be entered during staged construction as 1-β in the model explorer for any selected and deactivated soil cluster. This method intends to give similar results to those obtained using the partial staged construction (ΣMstage) method while bringing additional flexibility since different deconfinements can be applied to different tunnels or tunnel sections in the same phase.

Various methods are described in the literature for analysing tunnels constructed according to the New Austrian Tunnelling Method. One is the so-called Convergence confinement method or β-method. The idea is that the initial stresses pk acting around the location where the tunnel is to be constructed are divided into a part $(1-\beta)p_k$ that is applied to the unsupported tunnel and a part βp_k that is applied to the supported tunnel (Figure 2). The β-value is an 'experience value,' which, among other things, depends on the ratio of the unsupported tunnel length and the equivalent tunnel diameter.

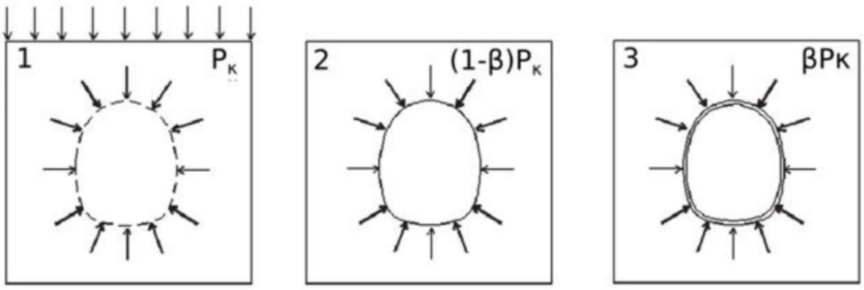


Figure 2. Schematic representation of the β-method for the analysis of NATM tunnels

This coefficient is given in percentages, and it can be explained in the simplest way as the percentage allowability of deformations of an unsupported excavation, while the tunnel support will accept the rest of the percentages of deformation.

The model helps determine soil stresses acting on the tunnel lining, which determines static quantities in the sections: bending moments and axial and transverse forces. The total displacements, i.e., deformations, were also calculated, which defines the complete picture of the construction's response.

The geometry of the numerical model is given in Figure 3, where its discretisation with finite elements is also shown. The standard fixities option in Plaxis is used to set the model limits. The light opening of the tunnel is 3.2 m.

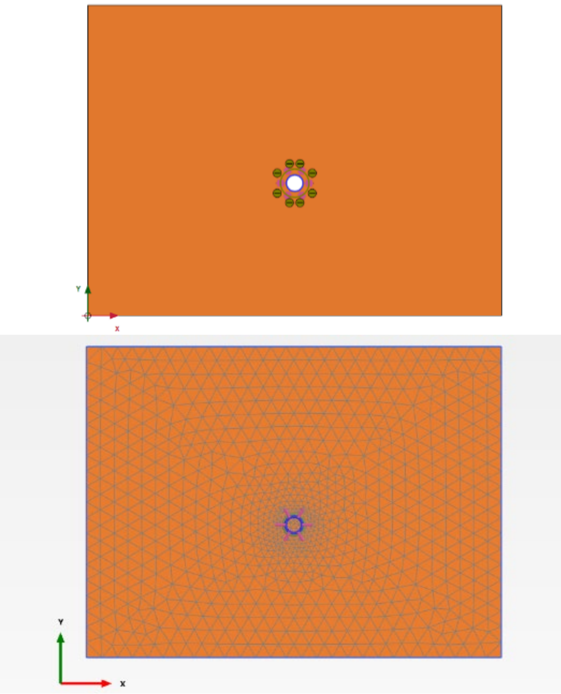
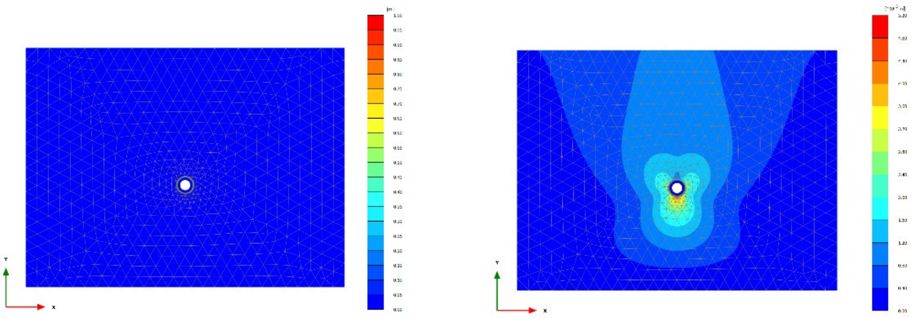


Figure 3. A view of the tunnel and the generated finite element mesh in PLAXIS

5. RESULTS

The following Figure 4 presents the deformations of the medium for two different coefficients 1 – ß.



A) 1 – ß=0%

B) 1 – ß=100%

Figure 4. Display of the tunnel deformations for different values of the coefficient 1–ß

From the diagram of deformations, it can be seen that for case 1 – ß=0 %, no deformations occurred after the excavation of the tunnel opening, while for case 1 – ß=100 %, all the deformations were realised before the support was installed.

The results are presented for all analysed values of 1-ß through tables and different relationships. In contrast, graphical outputs from the software are shown only from the phase where 1-ß= 100 %, aiming to best compare the results in static and seismic conditions (equivalent static).

Table 4. Maximum displacements of the local environment

1-ß	[%]	0	20	40	60	80	100
d seismic	[m]	0.2685	0.2686	0.2684	0.2681	0.268	0.2677
d static	[m]	0.000434	0.002014	0.004608	0.007152	0.00984	0.01272

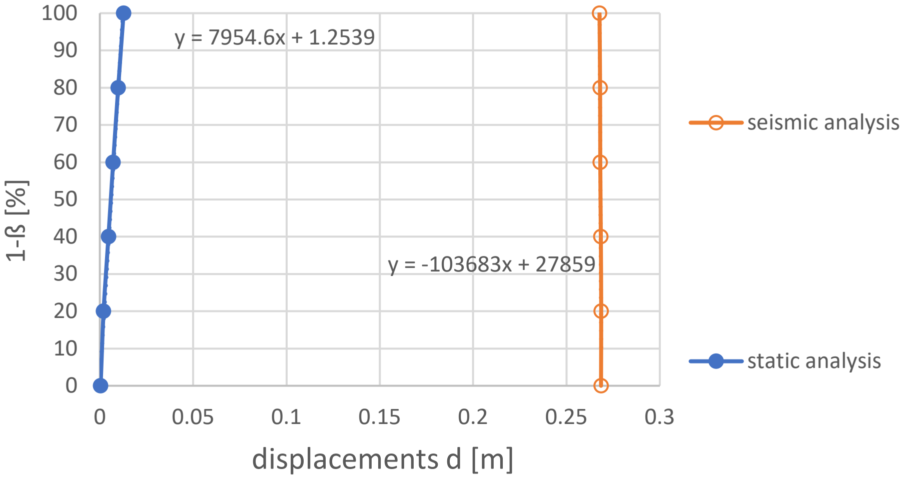
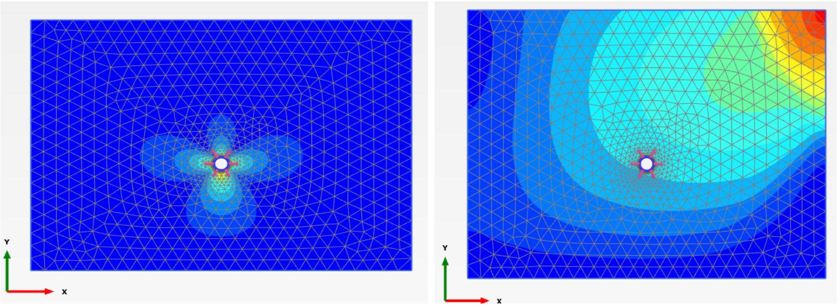


Figure 5. Dependence (1-ß) - d of the surrounding ground in both static and seismic conditions



A) Static analysis *B) Seismic analysis*

Figure 6. Total displacement phase 1-ß= 100%

Based on the results presented in Table 4 and Figures 5 and 6, the deformations of the surrounding ground increase insignificantly with the increase of the coefficient 1-ß in static conditions. In contrast, in seismic conditions, they are significantly larger and increase linearly with the increase of the coefficient 1-ß.

Table 5. Maximum deformation of the anchor

1-ß	[%]	0	20	40	60	80	100
d static	[m]	0.000434	0.001808	0.003562	0.005535	0.007534	0.009692
d seismic	[m]	0.09067	0.09104	0.0914	0.09181	0.09215	0.09256

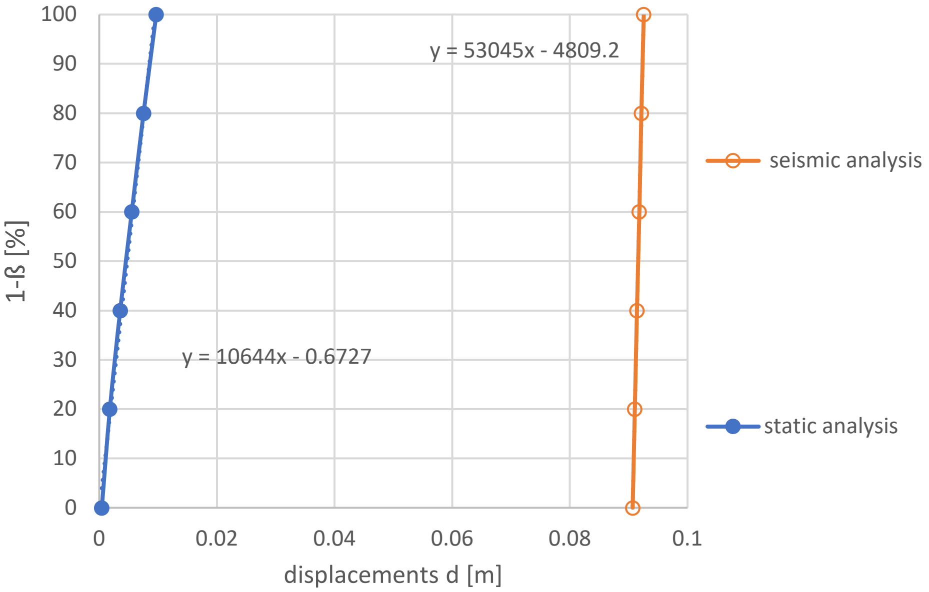
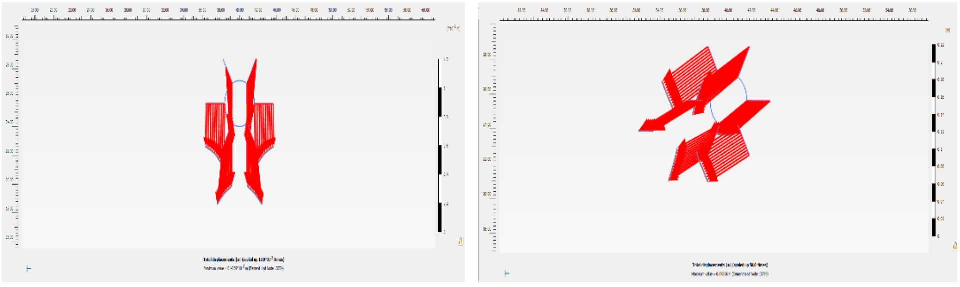


Figure 7. Dependence (1-ß) – d of the anchor in static and seismic conditions



A) Static analysis

B) Seismic analysis

Figure 8. Total displacement of an anchor phase 1-ß= 100%

Based on the results presented in Table 5 and Figures 7 and 8, the deformations of the analysed anchor increase approximately linearly with the increase of the 1-ß coefficient in both static and seismic conditions. In terms of absolute value, they are more significant in seismic conditions and change direction.

Table 6. Maximum Shotcrete deformation

1-ß	[%]	0	20	40	60	80	100
d static	[m]	0.000439	0.002092	0.004608	0.007152	0.00984	0.01272
d seismic	[m]	0.09845	0.08942	0.08971	0.09021	0.09109	0.0924

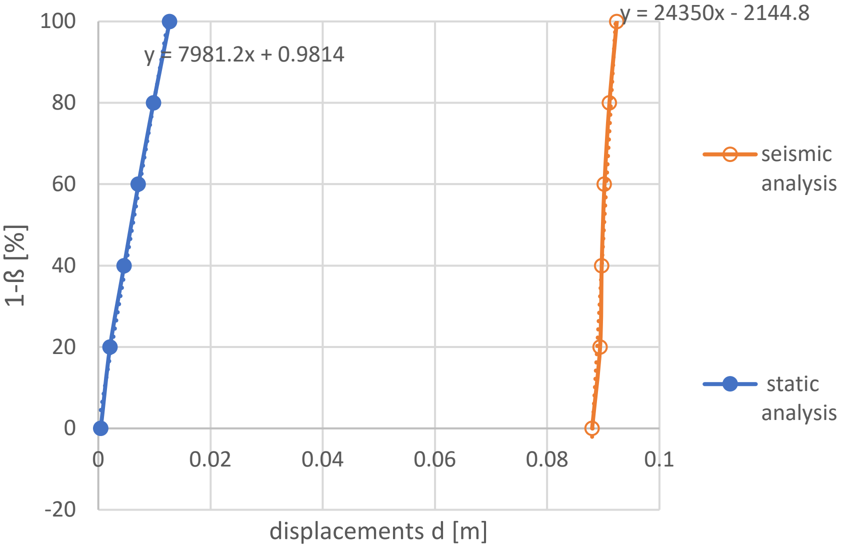
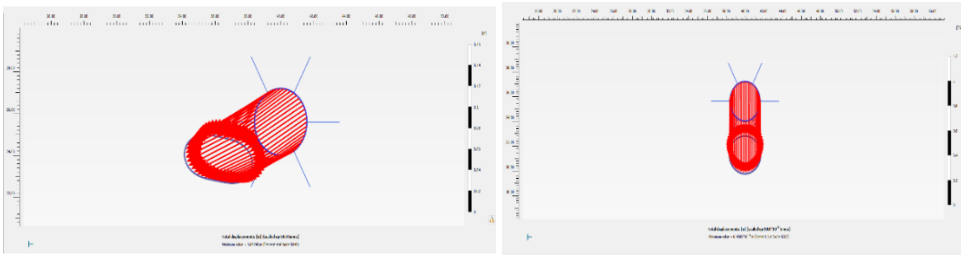


Figure 9. Dependence (1-ß) – d of shotcrete in static and seismic conditions



A) Static analysis B) Seismic analysis

Figure 10. Total displacement of shotcrete phase 1-ß= 100%

Based on the results presented in Table 6 and Figures 9 and 10, the shotcrete deformations increase approximately linearly with the increase of the coefficient 1-ß in both static and seismic conditions. In terms of absolute value, they are more significant in seismic conditions and also change direction.

Table 7. The axial force of the anchor

1-ß	[%]	0	20	40	60	80	100
N static [kN/m]	max	-0.00018	-0.1368	-0.2733	-0.4062	-0.5496	-0.6956
	min	-0.00207	-1.157	-2.312	-3.4664	-4.28	-4.805
N seismic [kN/m]	max	-0.1271	-0.06311	0.09757	0.7655	1.502	2.032
	min	-3.798	-3.868	-4.486	-4.926	-5.244	-5.523

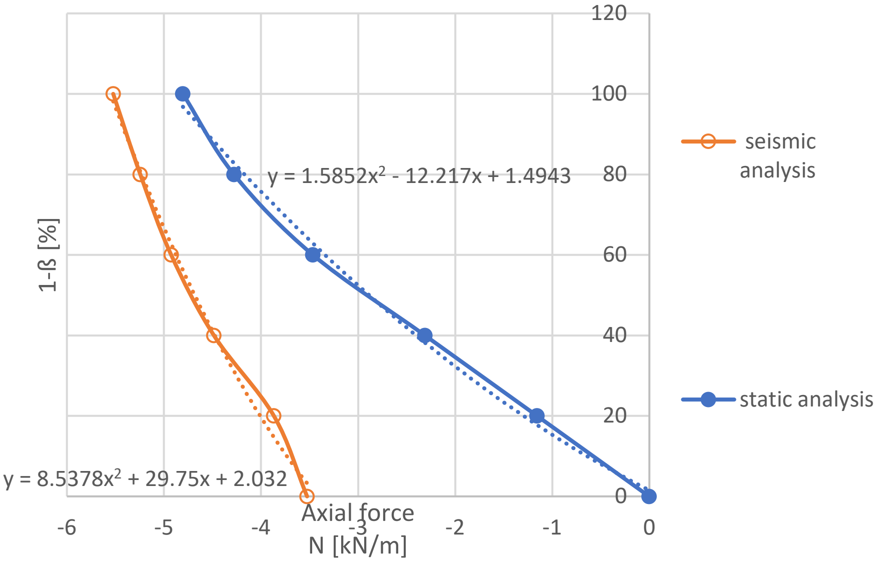
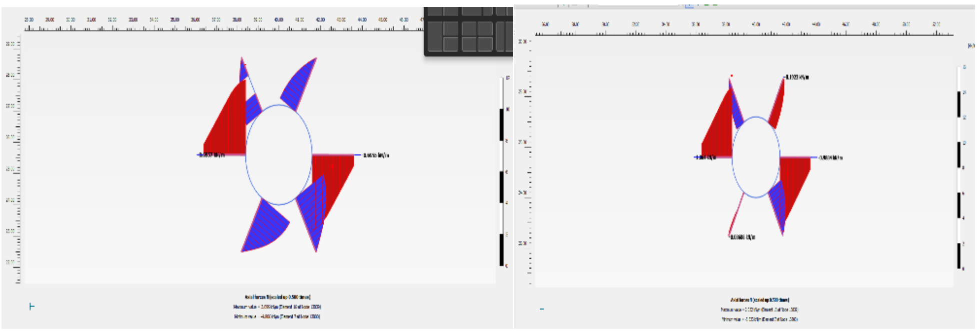


Figure 11. Dependence (1-ß) – N in static and seismic conditions on the anchor



A) Static analysis B) Seismic analysis

Figure 12. The axial force of an anchor phase 1-ß= 100%

Based on the results presented in Table 7 and Figures 11 and 12, it can be concluded that there is an increase in axial force as a function of the increase in the value of 1-ß, which is more significant in static conditions than in seismic conditions.

Table 8. Transverse force in shotcrete

1-ß	[%]	0	20	40	60	80	100
Q static [kN/m]	max	0.05454	7.732	15.43	23.33	34.23	47.69
	min	-0.05448	-7.734	-15.53	-23.38	-34.20	-47.58
Q seismic [kN/m]	max	18.4	18.99	23.35	28.87	34.93	43.20
	min	-16.52	-18.61	-24.03	-30.26	-37.52	-50.42

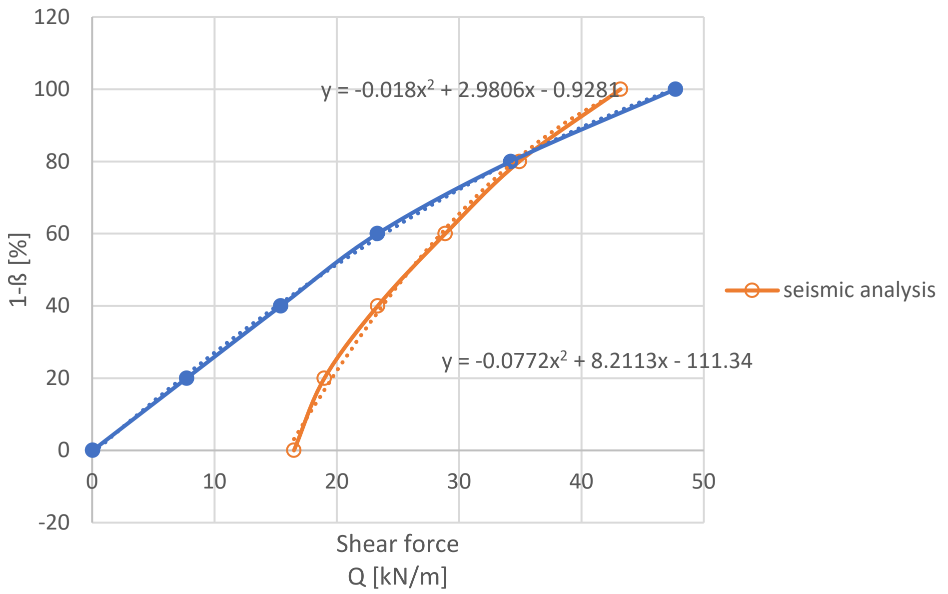
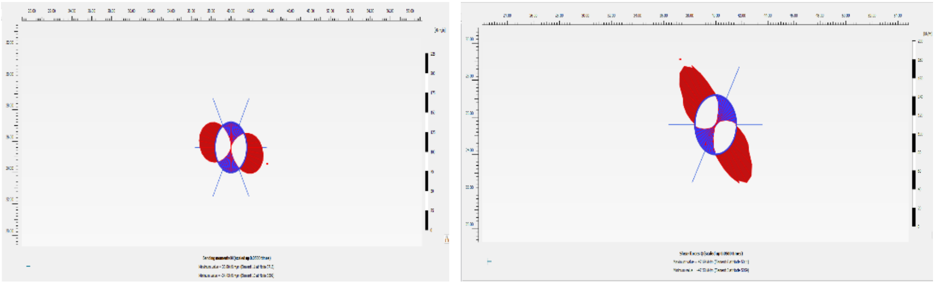


Figure 13. Dependence (1-ß) – Q in static (in blue) and seismic (in orange) conditions on the shotcrete



A) Static analysis B) Seismic analysis

Figure 14. Shear force of shotcrete phase 1-ß= 100%

Based on the results presented in Table 8 and Figures 13 and 14, it can be concluded that there is an increase in transverse force as a function of the increase in the value of 1-ß, which is more significant in static conditions than in seismic conditions. For 1-ß=80 %, approximately the same results are obtained in static and seismic conditions.

Table 9. Axial force in shotcrete

1-ß	[%]	0	20	40	60	80	100
N static [kN/m]	max	0.7788	-143.2	-287.3	-432.2	-578.9	-726.9
	min	-1.163	-218.5	-436.6	-654.1	-868.9	-1081.0
N seismic [kN/m]	max	-118.9	-287.2	-443.7	-592.6	-740.4	-887.0
	min	-233.7	-429.9	-637.0	-850.2	-1064.0	-1276.0

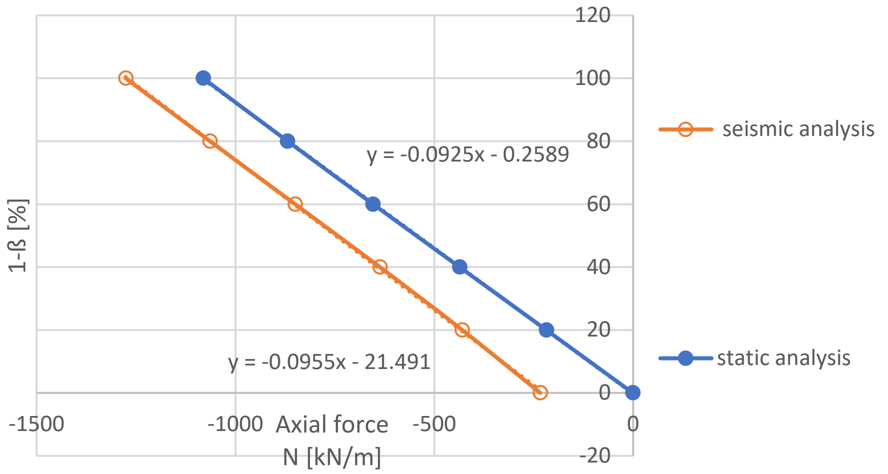


Figure 15. Dependence (1-ß) – N in static and seismic conditions on the shotcrete

A) Static analysis B) Seismic analysis

Figure 16. The axial force of shotcrete phase 1-ß= 100%

Based on the results presented in Table 9 and Figures 15 and 16, it can be concluded that there is an increase in axial force as a function of increasing the value of 1-ß, which is the same in static and seismic conditions. In absolute value, greater axial forces are obtained in seismic conditions.

Table 10. Bending moments in shotcrete

1-ß	[%]	0	20	40	60	80	100
M static [kN m/m]	max	0.02463	6.159	12.30	18.45	24.86	31.77
	min	-0.02541	-6.019	-12.02	-17.96	-23.83	-29.79
M seismic [kN m/m]	max	13.31	14.83	17.94	22.77	28.04	33.80
	min	-13.30	-15.37	-19.16	-23.72	-29.16	-34.43

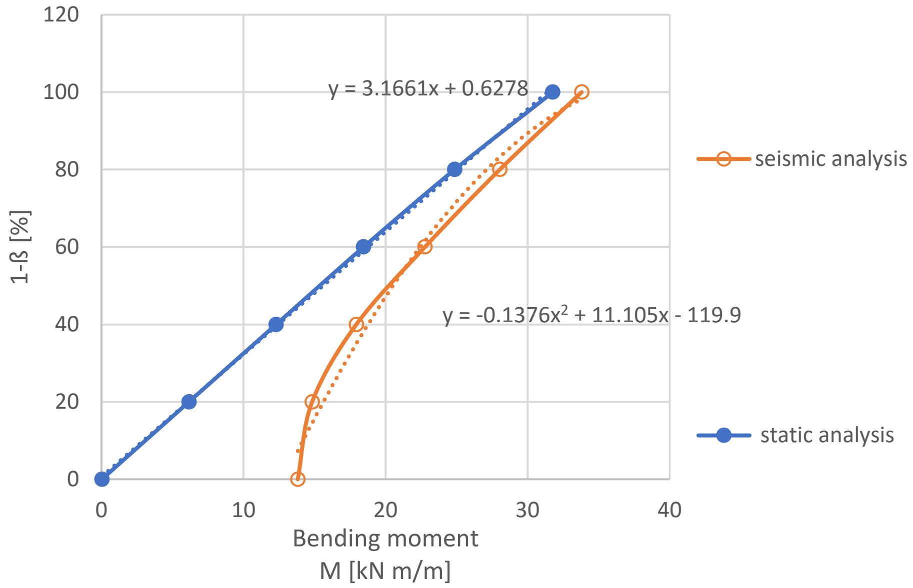
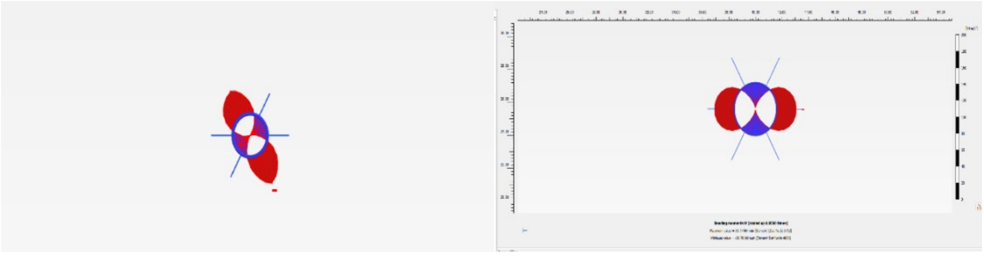


Figure 17. Dependence (1-ß) – M in static and seismic conditions on the shotcrete



A) Static analysis B) Seismic analysis

Figure 18. Bending moment of shotcrete phase 1-ß= 100%

Based on the results presented in Table 10 and Figures 17 and 18, it can be concluded that there is an increase in bending moments as a function of increasing the value of 1-ß, which is different in static conditions compared to the increase in seismic conditions. However, when 1-ß=100 %, approximate values are reached.

6. CONCLUSION

This paper analyses methods and procedures for tunnel construction through a practical example of constructing an access tunnel to a hydro-technical tunnel from the Mavrovo hydro system. During the exploitation phase, the tunnel suffered certain damages, so its rehabilitation was needed.

A case with support in the excavation phase and primary lining with shotcrete and anchors was analysed. The analysis of the tunnel is made in a software package based on the application of the finite element method and offers the possibility to simulate the phase performance of the tunnel and the simulation of the time required to place the support through the so-called method 1-β, that is, the percentage enabled realisation of the deformations in excavation.

A parametric analysis was made for different values of the coefficient 1-β, during which the changes in the deformations of the surrounding environment, the deformations of the support elements, and the internal forces of the structural elements were monitored.

The results show that as the 1-β coefficient increases from 0 to 100%, the deformations and internal forces in the shotcrete and anchors grow almost proportionally. The absolute values of the deformations and internal forces of the surrounding environment and the primary support are greater for seismic conditions than for static conditions.

Finally, from the conducted analysis, it can be concluded that through the so-called method 1-β, i.e., the percentage enabled realisation of the deformations in the excavation, can successfully model the time needed to set up the support and obtain relevant parameters for dimensioning the elements of the lining for different load cases in static and seismic conditions and stages of performance.

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НУМЕРИЧКО МОДЕЛИРАЊЕ ИСКОПА И ПОДГРАДЕ ТУНЕЛА ПРИМЈЕНОМ МЕТОДЕ РАСТЕРЕЋЕЊА У УСЛОВИМА СТАТИЧКОГ И СЕИЗМИЧКОГ ОПТЕРЕЋЕЊА

Сажетак: У овом раду анализирана је фазна изградња тунела са подградом у статичким и сеизмичким условима. За нумеричко моделирање проблема кориштен је софтверски пакет PLAXIS 2Д. Параметарска анализа ископавања користећи метод растерећења (1-β) спроведена је на стварном тунелу са подградом у фази ископавања и примарном облогом од млазног бетона и анкера. На основу спроведене анализе може се закључити да се тзв. метода 1-β, односно проценат омогућене реализације деформација током ископавања, може успјешно користити за моделирање времена потребног за постављање подграде, као и за добијање релевантних параметара за димензионисање елемената облоге при различитим случајевима оптерећења у статичким и сеизмичким условима, као и у различитим фазама извођења радова.

Кључне ријечи: тунел, подграда тунела, нумеричко моделирање, метода растерећења.