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EVALUATING TECHNICAL AND ECONOMICAL ASPECTS OF CONVENTIONAL AND MTBM METHODS FOR DAM DIVERSION TUNNELS - THE CASE OF THE TRIANTAFYLLIA DAM

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Abstract

Hydraulic tunnels and particularly diversion tunnels are crucial for dam operations, since they ensure dry conditions during construction and served as water intakes or evacuators during operation. It is undeniable that the dam type and construction rate significantly affect diversion tunnel design which is highly related with the hydrological and hydraulic calculations. This paper aims to evaluate technical and economical aspects of either conventional or mechanized by microTBM excavation methods. The case study of this research is the Triantafyllia dam in Greece, which is built on gneissic rock masses. Key components included analysing hydrological processes and designing the hydraulic and geotechnical aspects of the diversion tunnel, considering the rock mass qualities. Tunnel diameter alternatives were determined based on the dam type, construction time, and failure mechanisms. Engineering geological sections were created from ground investigation data, while deformations and support measures requirements were assessed using software for critical areas. The suitability of several microTBM machines was also examined. Finally, costs for each scenario were examined.

Key words

Conventional and Micro-Tunneling methods, Dam Diversion Tunnels, Gneiss rock masses, GSI, TBC, Technical – economical comparison, TBM selection, Hydrology, Hydraulics.

1 Introduction

Hydraulic tunnels are among the most significant components in the suite of projects associated with a dam. These tunnels are categorized into intake tunnels, reservoir drainage tunnels, and diversion tunnels, whose functions overlap and complement each other upon the completion of construction works. According to Afshar et al. (1994) diversion systems significantly impact the construction timeline, and its cost constitutes a substantial portion of the overall project budget. Typically, the choice of diversion system capacity and the dimensions of its components involves both socioeconomical and technical considerations. This kind of tunnels, have a special interest as their proper design and efficient operation ensure the required dry conditions for the optimal construction of the dam. It is observed that the diameter of the diversion tunnel is directly dependent on the hydraulic design of the upstream area, a critical factor of which is the parameter of the return period. This, in turn, is heavily dependent on the type of dam as it is related to the risk of failure and the required construction time of dam. However, it is widely accepted that in the scientific community, there has been a lack of research comparing the technical and economical aspects of tunnel excavation methods and the consequent excavation time, while considering the type of dam and its effect on the hydraulic design of the upstream area.
The purpose of this paper is to further examine the dependency of these factors and additionally to conduct a technical and economical comparison of conventional and mechanical tunnel excavation

methods, using microTBM, for various diversion tunnel diameters. This framework is applied to the case of the Triantafyllia dam in Northern Greece, which is founded on a gneissic rock masses, slightly to moderately fractured.

2 Methods

2.1 Input data and utilized software

The primary data for this study were collected from the preliminary and final design reports of the Triantafyllia dam (Hydroexygiantiki L. S. Lazaridis & Co., 1999) which were processed and utilized for the geotechnical evaluation with the aim of proposing new alternative design scenarios for the diversion tunnel.
More specifically, the primary data included drawings such as the horizontal layouts and typical cross-sections of the dam and the diversion tunnel in order understand the current location and the dimensions. Moreover, through the evaluation of the ground investigation program, and particularly photographic material from the geological core boxes, useful data were obtained for designing the geotechnical longitudinal section of the diversion tunnel. Additionally, the results of laboratory tests, such as point load tests, uniaxial compression tests, tensile tests (Brazilian test) as well as the results of on-site tests, specifically permeability tests (Lugeon, Maag) were included to examine the appropriate type of microTBM, for the geotechnical assessment and also for the study of tunnel failure mechanism. Finally, the hydrological and hydraulic calculation reports of the project, along with its technical report, were particularly useful for the up-to-date hydrological and hydraulic design.
In our study, advanced geotechnical software played a pivotal role in the analysis and assessment of key parameters. Geotechnical software, specifically Unwedge by Rocscience Inc., was employed to evaluate the stability of wedges, while also RS2, another software by Rocscience Inc., was utilized for conducting geotechnical simulations. These simulations were conducted to investigate the resistance provided by the support system and to calculate the convergence and ultimate deformations following excavation. Regarding the subject of the hydrological study, the QGIS software was used in order to calculate the basic geometrical characteristics of the basin of Triantafillia and also the HEC-HMS software from the U.S. Army Corps of Engineers was used, while for the flood wave routing through the diversion tunnel, an appropriate programming environment in R was developed.

2.2 Methodology

In order to investigate the engineering geological background, we evaluated the boreholes that were executed as part of the geological study and we compared this information with the Geological Map of Greece by the Institute of Geological and Mining Research. The Geological Strength Index (GSI) system to classify the rocky formations of gneiss and granitic gneiss (V. Marinos, 2007) was used. Based on the combination of these data, we separated three engineering-geological units, TS I, TS II and TS III, as elaborated bellow. Moreover, considering the height of the overburden and the uniaxial compressive strength σ_{ci}, we determined the tunnel behavior modes, based on the Tunnel Behavior Chart (TBC) (Marinos, V. 2012) for each geological section.
The hydrological study was conducted for a return period of 100 years, in accordance with the Greek specifications for the development of flood protection projects and international and Greek common practice, as well as the engineering judgment depending on the required construction time of the tunnel. Based on the method named Soil Conservation Service (SCS) and utilizing software HEC-HMS, we calculated the final design flood hydrograph for a range of return periods. For the same range, we examined the effect of the diameter of diversion tunnel at the appropriate function of diversion tunnel. The return period is related with the type of the dam and the required construction time.
Following, we examined three different scenarios based both on the method of excavation and on the type of the dam. More specifically, the scenarios I, II, and III were examined, corresponding to an earthen dam with a 3-meter diameter tunnel excavated by conventional method, an earthen dam with a 3-meter diameter tunnel excavated using a micro-Rock TBM, and an RCC dam with a 2-meter diameter diversion tunnel excavated similarly using a micro-Rock TBM, respectively. Also, firstly we examined

the construction of an earthfill dam and the conventional excavation, secondly an earthfill dam and the mechanized excavation method using Micro-TBM and finally a RCC dam and the mechanized excavation method using Micro-TBM. For these cases we determined the required support systems and the suitability of the appropriate Micro-TBM types. We also calculated the cost for each scenario.

3 Case study – Triantafyllia Dam

The Triantafyllia dam is constructed on the homonymous river, approximately 2 km upstream from its outlet to the plain of Florina and about 1.5 km northwest of the settlement of Ano Ydrousa (Figure 1). The purpose of the project is water storage for the irrigation and water supply needs of the plain of Florina, as well as ecological provision, amounting to a volume equivalent to 1 hm³ annually during the irrigation months.

Figure 1. Project location map

The complex of the dam of Triantafyllia consists of individual projects. More specifically, the dam is constructed by stones, its central core is impermeable and it is distinguished into specific main zones. The height of the dam is 75 meters from the foundation of the core, with a crest length of 510 meters and a crest width of 16 meters. Dam reservoir has a dual role in order to provide water for irrigation and water supply. The maximum operating level of the reservoir is 838 m, corresponding to a surface area of 0.486 km² and a volume of 10.08 hm³, while the usable volume is 9.56.Furthermore there is a spillway, which width is 31.5 m and its capacity is up to 375 m³/s). Also, there is a diversion tunnel with a horseshoe cross-section, which diameter is 3 m and its length is up to 705 m.

3.1 Geological background

In the context of the geological study, there were drilled 19 sampling boreholes (Figure 2) from which we mainly evaluated 4, G-12, G-1, G-6 and G-10 at the area of diversion tunnel, two at the right abutment of the dam and one near the ending point of the diversion tunnel.

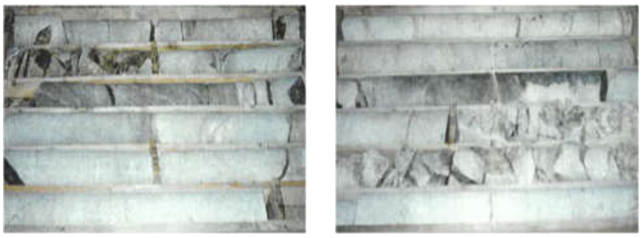
Figure 2. General layout of the diversion tunnel of the Triantafyllia dam (in red). In dots the sampling boreholes.

According to the geological report of the Preliminary Study and the reports of the consultant, Professor P. Marinos, the wider area is composed of metamorphic, Paleozoic-aged rocks of the Pelagonian Zone.

The gneiss-schist bedrock is extensively covered by weathered lateral colluvium and older fluvial deposits. Specifically, the rocky bedrock at the dam site consists of gneisses-granite gneisses with intercalations of amphibolitic and biotite schists, while aplite veins and lens-shaped concentrations are also present. On the right abutment, the percentage and thickness of amphibolitic-biotite schists increase, while the rocky bedrock is overlain by a thin weathering mantle. Gneiss is characterized by high strength, heterogeneity from its texture, irregular weathering, and shear zones that create fragmented and weathered areas up to 10 meters thick, composed of residual weathered gneiss and clay materials.
The area's tectonic structure features folding and faulting. Specifically, a longitudinal fault with a disturbance zone up to few meters wide is present at the lower right abutment, addressed with typical remediation and reinforced with a cement grouting diaphragm. The gneissic rock mass generally has moderate fracturing, resulting in high strength, low deformability, and low permeability. RQD values vary from moderate to high but are lower in intensely fractured zones.

3.2 Engineering geological conditions

In order to evaluate the quality of the formations and to proceed with the longitudinal geotechnical section, and to determine the failure mechanisms, the assessment of RQD, GSI, weathering degree, and σci was conducted.
The degree of fragmentation of the gneiss varies throughout the depth examined. The condition of the rock mass on the right abutment is characterized as compact to fractured, with RQD values ranging from 25% to 98%, and locally fragmented (RQD<25%). The fragmentation zones either have significant thickness (G1), or finally, have small thickness but are frequently interchanged with zones of lesser fragmentation (G6). The fragmentation is less apparent at the toe of the right abutment, although the presence of fragmented zones cannot be entirely ruled out.
Based on the results from the borehole analysis, the rock mass appears slightly to moderately fragmented with an average RQD of 60%. Representative samples are shown in Figure 3 .



a b

Figure 3. Samples from the drill cores showing that the rock mass is moderately fractured and slightly weathered

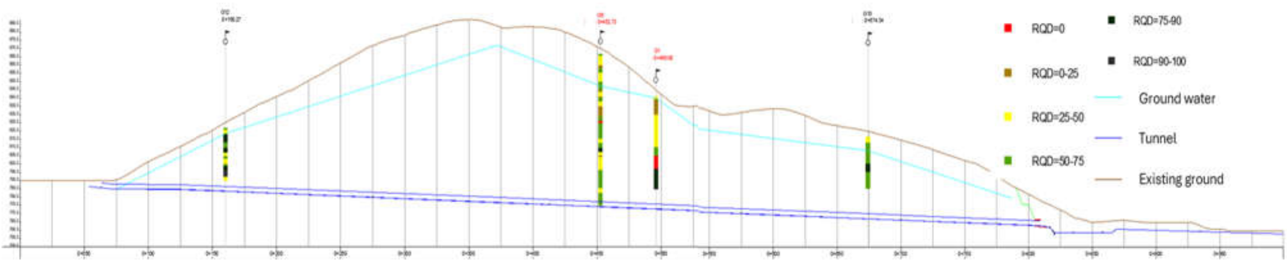


Figure 4. Longitudinal Section of RQD Distribution

Based on the geological map of the Institute of Geological and Mineral Exploration, the site specific geological conditions as well as the data from the boreholes, the geological section of the examined tunnel was evaluated. During the excavation of the diversion tunnel, good-quality rock masses of gneissic and granitic-gneissic nature are expected to be encountered, while at 0+495, a fault is expected to be encountered, as evident from the geological map and the reduced mechanical characteristics of the rock mass.

For the classification of the rocky formations of gneiss and granitic-gneiss along the tunnel, the Geological Strength Index (GSI) method was used since both gneissic and granitic-gneissic rocks have a clear rocky structure. The GSI for gneissic rock masses was used based on diagram of Figure 5**5**, considering the structure and degree of weathering of the rock mass (V. Marinos, 2007). Specifically, it was considered that a higher degree of weathering occurred in the portal area (category III), while in the rest of the tunnel length, weathering was considered to be of category II. Regarding the structure of the rock mass, it was generally considered blocky-disturbed-seamy and slightly fragmented-undisturbed-stratified in areas outside the portals, gneiss and granitic-gneiss outcrops, and the fault zone.

Based on the above, three engineering geological units were considered for the design of this tunnel, depending on the structure and the mechanical properties of the respective rock mass.

In the portal area, a higher degree of weathering (category III) and disturbed/stratified/faulted rock mass were considered, hence the category TSI. In the TSII, i.e., in sections where healthy gneiss and granitic-gneiss are encountered, weathering is significantly lower (category II), while the structure is considered intact to slightly blocky. Finally, in the TSIII where gneissic and granitic-gneissic rocks are present on the surface and in the fault zone area, weathering is again moderate (category II), lower than that of TSII, while in terms of structure, it was characterized as blocky-disturbed-seamy rock mass. The projections of each engineering geological unit appear at Figure 5

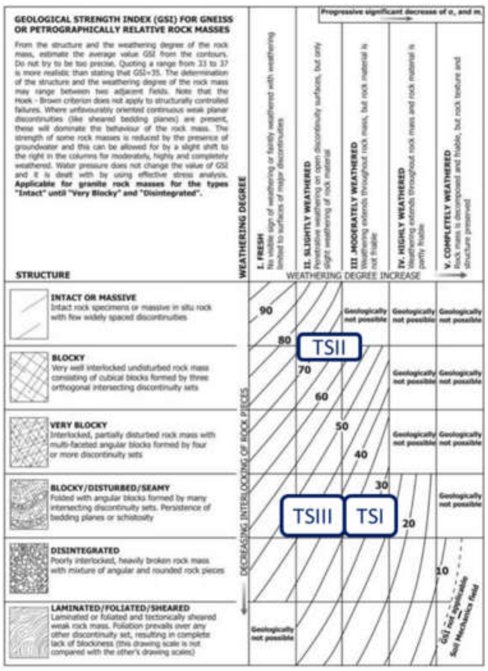


Figure 5. Geological Strength Index (GSI) for Weathered Gneiss Rock Masses (V. Marinos, 2007)

Table 1. Geological properties

Variable	T.S. II	T.S. III
Special Weight (γ) (MN/m³)	0.026	0.026
Rock mass disturbance (D)	0.5	0.5
Elasticity measure of rockmass (E_m)(MPa)	2187.2	192.2
Strenth of intact rock (σ_{ci}) (MPa)	110	30
Strenth of rockmass (σ_{cm}) (MPa)	34.2	2.9
GSI	70	40
m_i	28	28
Poisson ratio (ν)	0.25	0.25
Coefficient of lateral earth pressure (K_o)	0.8	0.8
Friction angle (φ)	40	35
Dilatancy (δ)	10	8.75

To complete the geotechnical assessment process, the magnitude of uniaxial compressive strength σ_{ci} per geological section was determined based on laboratory tests (point load test) conducted during the final study. From the data of the final study approximate values of uniaxial compressive strength σ_{ci} were considered equal to 40, 110, and 60 MPa for TSI, TSII, and TSIII, respectively.

An additional essential characteristic necessary for the geotechnical assessment of the rock mass is the intact rock properties of the geomaterials along the tunnel. Constant m_i, which is related to the frictional properties of the geological formation, and its value is associated with its fabric and the interlocking of its particles. Its determination was based on the method of Hoek and Marinos (2000) which considers results from laboratory tests. More specifically, for the TSI (tunnel portal area), was considered to be 23 (28-5) as the mechanical properties of the gneiss there are more weakened, for the TSII (healthy gneiss and healthy granodiorite), m_i was assumed to be 28, and finally, for the TSIII (fault zone area), a value

of m_i equal to 23 was adopted again due to the diminished mechanical properties.
The deformation modulus Ei of the intact rock $w$as evaluated 12, 5, and 2.5 GPa for TSI, TSII, and TSIII. Finally, the measure of elasticity, the specific weight of the rock mass, was considered to be 0.026 MN/m³, based on the results of the geotechnical study.

3.3 Tunnel behavior and failure modes

Taking into account the structure, the maximum height of the overburden, and the uniaxial compressive strength of the intact rock, potential failure mechanisms per TS were determined based on the TBC chart failure mechanism diagram shown in Figure 6
In general, structurally controlled failures can be only expected along the tunnel. Specifically, the failure type Ch – Wg involves the collapse of a chimney type in combination with wedge-like slides or falls of blocks due to gravity. The rock mass is heavily jointed, maintaining its structure in most cases, while usually exhibiting an open cross-section without good interlocking. This structure, combined with small lateral stresses, may progressively lead to collapses and progressively to over-excavations of the chimney type. The failure type Wg – St is encountered in stable sections with local gravitational failures, which may be characterized by wedge-like slides or falls of blocks due to gravity. Finally, the mechanism Ch – Sh includes chimney-type failure combined with small to moderate deformations and the occurrence of shear failures in small areas around the tunnel. The correlation between each engineering geological unit and the failure mode is shown in Figure 6.

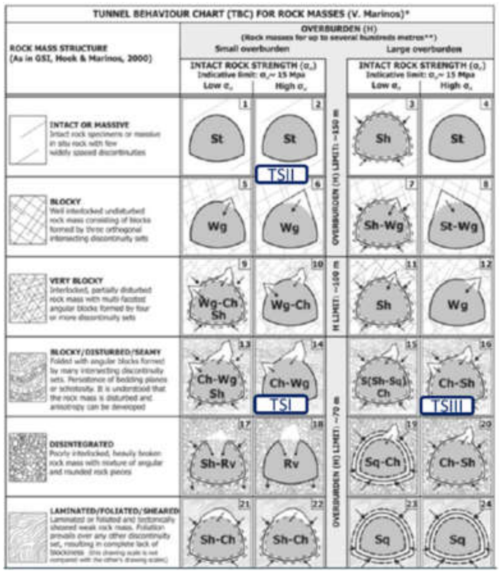


Figure 6: Failure modes (Marinos, V., 2012) – Wg: Wedge failure ,Ch: Chimney failure ,Sh: Shearing failures in shallow zone around the tunnel perimeter ,St: Stable

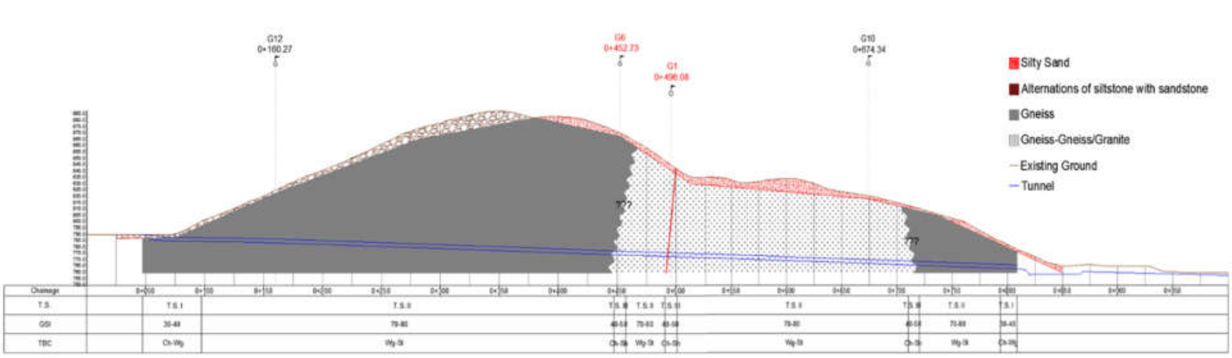


Figure 7: Engineering geological longitudinal section added with tunnel behavior appraisal.

4 Impact of hydraulics on tunnel diameter

Due to the advancements in the science of hydrology since the period when the final study of the Triantafyllia dam was conducted (1999), it was deemed necessary to reevaluate the input flood hydrographs for various return periods according to the construction time of the diversion tunnel, based on new and improved methods.

First of all, the design hydrograph was calculated based on the updated rainfall intensity-duration-frequency curves of the Flood Risk Management Plans of the Ministry of Environment and Energy for the Western Macedonia Hydrological District (EL09), to which the Triantafyllia dam belongs. Following the use of the HEC-HMS software by the US Army Corps of Engineers, the watershed hydrographs were determined for return periods of 10, 20, 50, and 100 years using the PRF 484-unit hydrograph, and the characteristics previously mentioned. The design flood hydrographs for return periods of 10, 20, 50, and 100 years, the effect of the diameter of the diversion tunnel on the routing of the flood volume through it was examined. Specifically, key characteristics were collected, such as the peak outflow ($Q_{max\ out}$), the maximum water height upstream of the tunnel entrance (h_{max}), and the height of the forebay, in an R programming environment. In Figures 8 and 9**Error! Reference source not found.**, the correlation between the return period (T) and the peak outflow ($Q_{max\ out}$) and the maximum water height upstream of the tunnel (h_{max}) is presented for various tunnel diameters (1.5, 2.0, and 3.0 meters).

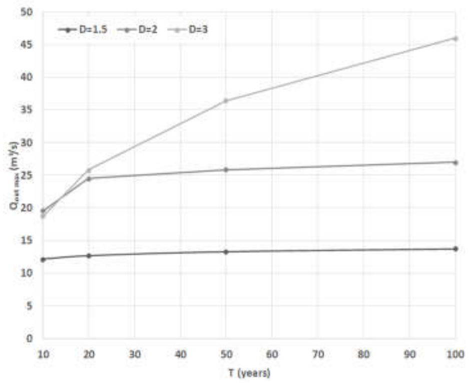


Figure 8. $Q_{max\ outflow}$ as size of return period

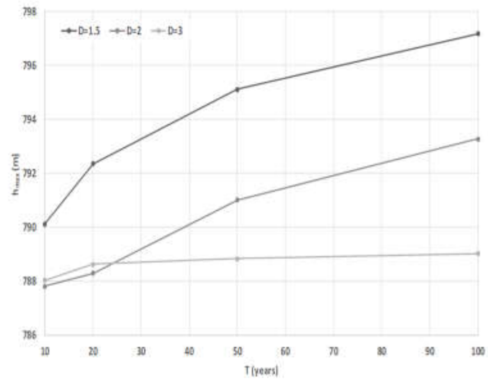


Figure 9. $h_{max\ outflow}$ as size of return period

A crucial factor in selecting the diameter of the diversion tunnel is the required construction time of the dam. More specifically, considering the daily construction rate of the dam depending on its type, a design flood hydrograph is selected with a suitable return period so that the risk of exceedance falls within certain acceptable limits. This risk is calculated according to Equation 1, where N is the expected completion time of the project and T is the return period.

$$r = 1 - \left(1 - \frac{1}{T}\right)^N \tag{1}$$

In the case of an earthen dam, the construction rate typically amounts to around 10 cm/day, while for an RCC dam, this is significantly higher, approximately 30 cm/day. Therefore, for a dam with a height of 75 m, the required time is expected to be 4 years for the earthen type and 2 years for the RCC type, considering an additional year in both cases for unforeseen issues. Based on the above information, it was calculated that considering a design flood return period of 100 years, for an earthen dam, a risk level of 3.9% is within acceptable limits. Similarly, to achieve the same risk level for an RCC dam, a return period of 50 years was chosen, corresponding to a risk level of 4.0%.

The calculations for the maximum peak discharge and the height of the cofferdam were carried out for the various return periods and different diameters of the diversion tunnel in order to understand the correlation between the individual parameters. Considering Figures 8 and 9 it was deemed reasonable to select a diversion tunnel diameter of 3.0 m and a return period of 100 years for an earthen dam. Additionally, for an RCC dam, a diversion tunnel diameter of 2.0 m was selected instead of 1.5 m since, as demonstrated in the right figure, in this case, the forebay height is 4 m lower, thus preferable from an economical point of view. It is noted that for the RCC dam, the design flood return period was selected as 50 years. All these data are required for the hydraulic calculations and geotechnical design of the diversion tunnel.

5 Comparing conventional and mechanized excavation methods

5.1 Scenario I

Given that the rock mass exhibits distinct joint patterns, it was deemed appropriate to assess their stability using the Unwedge software by RocScience Inc. As evident from the tunnel's layout, this does not maintain the same direction along its entire length. Therefore, the stability of wedges was examined in two sections, A and B, at chainages 0+000 to 0+375 and 0+375 to 0+780, respectively. The orientation in each section was measured at 130° and 55°, respectively, while the gradient along the length is common and equal to 2°. The friction angle was taken as 32°, and the cohesion as 0.06 MPa, according to the data from the final study of the trial tunnel. The details of the discontinuities studied are summarized in Table 2. Additionally, the length of the discontinuities was considered equal to one excavation step, which is 3 m in this case.

Table 2. Elements of Discontinuities

Discontinuities	Orientation
J1	55/025
J2	75/195
J3	80/300
J4	75/115

From the analysis, the discontinuities J1J2J3 and J2J3J4 on the roof in both sections A and B were considered unstable so we proposed the placement of Swellex bolts, with a tensile strength of 0.12 MN, and a length of 2 m, installed in a 2 x 3 pattern (2 m perimeter spacing every 3 m step), as it is showed at Figures 10 to 12.

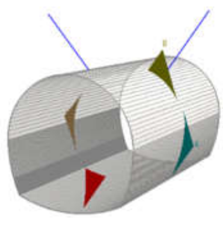


Figure 10. A – J1J2J3

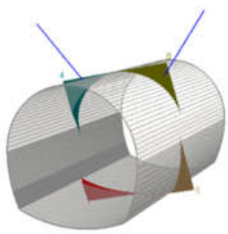


Figure 11. A –J2J3J4

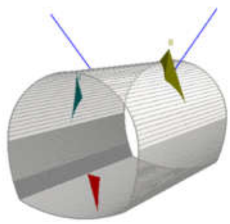


Figure 12. B –J2J3J4

We can expect mainly wedge problems but we also check places where deformations can develop, especially in fault zones and strong disintegration, where GSI, sci and mi are the smallest and the overburden are the highest. Specifically, the cases of maximum overhangs on healthy schist (0+350 – Analysis 1), maximum overhangs on healthy granite-schist (0+475– Analysis 2), and the fault zone in a granite-schist environment (0+495– Analysis 3) were examined. For these areas, longitudinal displacement profiles) and longitudinal development profiles of the failure degree were formed, based on the Chern et al. (1998) method.

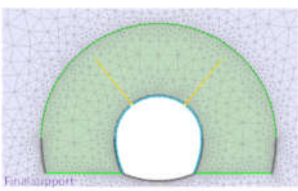


Figure 13. 1 – Final support

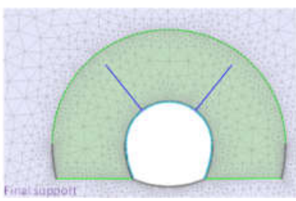


Figure 14. 2 – Final support

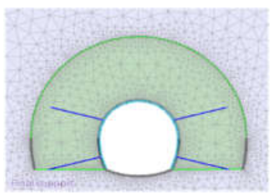


Figure 15. 3 – Final support

In the simulations, certain basic assumptions were adopted. Specifically, regarding the soil, the rock mass was simulated as elasto-plastic, assuming isotropic linear elasticity. For simulating the behavior of the geological structure, the Generalized Hoek & Brown failure criterion (2002) was utilized. The parameters used are summarized in Table 1. According to the operation mode of the RS2 software, the disturbance degree D due to excavation is applied across the entire extent of the geological structure. As evident, this does not make sense since the impact of excavation and the subsequent disturbance of the rock mass are concentrated within a disturbance zone, approximately 2.5 m thick around the excavation area (Georgiou et al., 2023), as depicted in Figures 13 to 15.

The support of the underground opening is achieved through the combination of two support sections, A and B. Specifically, section A includes a shotcrete lining with a thickness equal to the minimum recommended by Guidelines for road project studies standards, along with type Swellex expandable rock bolts. Additionally, section B comprises a shotcrete lining and steel lattice girders, along with full-length grouted rock bolts. The properties of the support measures for both proposed sections A and B are presented in Table 3.

The total displacements developed during excavation were calculated to be 3 mm at the crown and 1 mm at the sidewalls, in case of analysis 1 (0+350), 2 mm at the crown and 1 mm at the sidewalls in case of analysis 2 (0+475) and 1.4 at the crown and 0.7 mm at the sidewalls.

Table 3. Recommended support measures

	Analysis 1 & 2 - T.S. II	Analysis 3 – T.S. III
Opening phases	1	1
Excavation step	3	1.5
Shotcrete - C20/25 (cm)	5	15
Bolts	Swellex, 320 kN, L=2 m, 1.0 x 3.0 m	Fully bonded, 160 kN, L=2 m, 1.0 x 1.5 m
Steel sets	-	Lattice Girder, 3 bar, #50, 18.26 mm / Excavation step

Based on the descriptive pricing of public works according to Law 4412/2016 (Government Gazette 147 A / 8-8-2016), the total cost for excavating the tunnel, considering the loading and unloading of rocky materials, excavation, support measures, and labor, was evaluated to be around 8.1 million euros.

5.2 Scenario II

In this scenario, the excavation of the tunnel was examined using a micro-TBM. In order to ensure hydraulic consistency between scenarios I and II and to make the results comparable, the equivalent hydraulic radius of the tunnel was calculated assuming a circular cross-section with a diameter of 3.1 m.

It's worth mentioning that unlike scenario I, where excavation is carried out using the drill and blast method, resulting in rock mass disturbance (D=0.5), in scenario II, excavation is done using micro-TBM, hence the rock mass is undisturbed (D=0). The support of the underground opening is achieved with the direct support section type C, consisting of a shotcrete shell, the properties of which are summarized in Table 4.

The selection of the micro-TBM machine for the excavation of the tunnel of interest is a fundamental aspect. A fundamental element in determining immediate support is the selection of the micro-TBM machine chosen for the excavation of the tunnel in question. For this purpose, an investigation into the applicability of each micro-TBM was conducted, as presented in the longitudinal section in figure 16. From this investigation and based on the characteristics of each tunneling machine, the micro-Rock TBM was considered the most suitable, as it shows good performance over the majority of the tunnel length. The area of particular interest is the fault zone at Station 0+495, where there is a possibility of encountering water. In this area, it is deemed necessary to perform horizontal drilling (prop drilling) and carefully monitor the water flow. For this purpose, it is recommended to locally install piezometers and subsequently pump the water,or construct reservoirs in the case of low pressure. These horizontal drillings can be carried out either adjacent to the excavation after constructing an appropriate auxiliary tunnel or along the axis of the excavation using the micro-TBM itself.

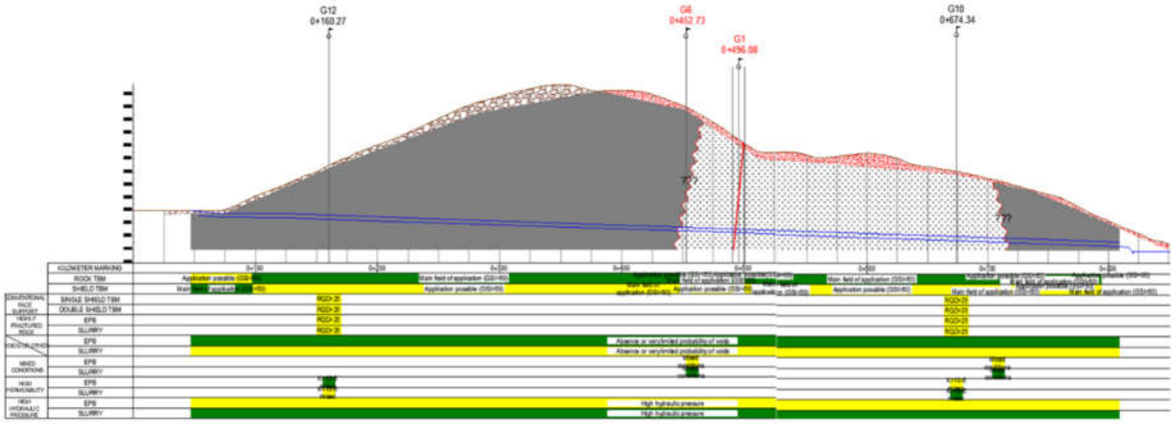


Figure 16. 1 – Micro-TBM feasibility

The total displacements developed during excavation were calculated to be 1.3 mm at the crown and 0.6 mm at the sidewalls, in case of analysis 1 (0+350), 1 mm at the crown and 0.5 mm at the sidewalls in case of analysis 2 (0+475) and 6 at the crown and 3 mm at the sidewalls.

Table 4. Recommended support measures

	Analysis 4-9
Opening phases	1
Excavation step	1.50
Shotcrete – C20/25(cm)	30

Based on the descriptive pricing of public works according to Law 4412/2016 (Government Gazette 147 A / 8-8-2016), the total cost for excavating the tunnel, considering the loading and unloading of rocky materials, excavation, support measures, and labor, was determined to be 1.9 million euros.

5.3 Scenario III

In this scenario, the alternative case of constructing an RCC (Roller Compacted Concrete) dam was examined. These types of dams are characterized by their high construction speed as they do not require the use of formwork or reinforcement. This significantly affects the diameter of the diversion tunnel, as we expect a lower flood discharge over a shorter period of time, leading to a smaller diameter. Among their advantages are the impermeable nature of the dam body, the minimization of cooling and vibration requirements compared to conventional concrete, and the reduction in cement usage due to the possibility of using industrial by-products. A typical construction rate for RCC dams is approximately 30 cm per day. To achieve a comparable risk level to that of constructing an earthfill dam, the design flood event chosen was a 50-year recurrence interval, and it was deemed reasonable to examine the construction of a tunnel with a diameter of 2 m instead of 1.5 m, due to the significantly smaller height of the headworks. Therefore, in this scenario, the construction of a 2 m diameter diversion tunnel along with the simultaneous construction of an RCC dam is being considered. Moreover, the excavation of the tunnel was examined using a micro-TBM.
The total displacements developed during excavation were calculated to be 0.9 mm at the crown and 0.5 mm at the sidewalls, in case of analysis 1 (0+350), 0.7 mm at the crown and 0.4 mm at the sidewalls in case of analysis 2 (0+475) and 4.8 at the crown and 2.4 mm at the sidewalls at analysis 3 (0+495). Based on the descriptive pricing of public works according to Law 4412/2016 (Government Gazette 147 A / 8-8-2016), the total cost for excavating the tunnel using micro-Rock TBM, considering the loading and unloading of rocky materials, excavation, support measures, and labor, was determined to be 1.0 million euros.

5 Results and Discussion

This study presents a technical and economical comparison of conventional (drilling and blasting) versus mechanical (microtunneling) excavation methods for diversion tunnels, with a focus on how dam type affects tunnel diameter through hydrological and hydraulic considerations. Using the Triantafyllia dam in Florina as a case study, the "as built" design employed the drill and blast method for the diversion tunnel on the right abutment, and an earthen dam was selected. The new approach evaluated literature and project data to develop revised geotechnical conclusions and propose an updated geotechnical design for the tunnel.

Regarding the geotechnical conclusions initially drawn, the surrounding rock mass was categorized into three engineering geological units TSI, TSII, and TSIII, around the portals, in the section of the tunnel where gneiss and granite-gneiss were characterized by good mechanical properties, and in the area around the gneiss and granite-gneiss interface points as well as in the fault area, respectively. As for the main geotechnical characteristics of the individual geotechnical units, we distinguish that overall, it is about a good-quality rock mass except for the fault area and the gneiss and granite-gneiss interface area, where the rock mass is more weakened and has poorer mechanical properties.

Regarding on the behavior of the rock mass in the tunnels and the possible failure modes depending on the maximum height of the overburden, the structure, and the uniaxial compressive strength of the intact rock, the potential failure modes of the rock mass were predicted. More specifically, the dominant failure modes are Ch-Wg, Ch-Sh and St.

Regarding the hydrological aspect, considering the influence of the construction rate of each type of dam, suitable return periods were selected for hydrological and hydraulic calculations. Specifically, with the reasonable assumption of a construction rate of 10 cm/day in the case of an earthen dam and 30 cm/day in the case of an RCC dam, and accounting for delays due to unforeseen problems during construction amounting to 1 year, the completion time was determined to be 4 and 2 years, respectively. With the goal of equalizing the risk considered for design with a return period T and a completion time N, it was deemed appropriate to select return periods of 100 and 50 years for the earthen and RCC dams, respectively.

Subsequently, hydrological calculations were carried out starting from the rainfall curves of nearby stations, from which the design rainfall for each return period was determined and transformed into runoff using the HEC-HMS software. Then, to investigate the influence of the return period on the diameter of the diversion tunnel, an R programming environment model was developed to route the inflow hydrograph, yielding output parameters such as the maximum outflow discharge, the required height of the cofferdam, and the maximum water level at the tunnel entrance. From this investigation for various return periods, the selection of a diameter of 2 and 3 meters and return periods of 50 and 100 years was deemed reasonable for the RCC and earthen dam types, respectively.

Subsequently, the geotechnical design was implemented considering the peculiarities of the surrounding rock mass, while the analyses, both in the case of the drill and blast method and the microtunneling method, were carried out using the RS2 software. All the support measures of the typical sections A, B, C, and D were proved to be sufficient as demonstrated using the RS2 software.

The main criteria for selecting the optimal scenario are the feasibility of support measures, time, and cost. The relevant results for each scenario are presented in Table **5**. It is evident that, based on the assumptions, the excavation of the 2m diameter diversion tunnel using microtunneling with micro-Rock TBM and the construction of an RCC dam are clearly more cost-effective. It is noted that in the technical and economic comparison, only the costs related to the excavation of the diversion tunnel were considered, not the construction of the respective types of dams (earthfill and RCC) and their peculiarities, as this issue does not correspond to the objectives of this study.

Table 5. Results – Scenarios I, II, III

	Scenario I	Scenario II	Scenario III
Diameter	3.0	3.1	2.0
Excavation method	Drill & Blast	Micro-TBM	Micro-TBM
Cost (millions)	8	1.9	1.8
Required time	251 days	21 days	21 days

Obviously, the choice to construct an RCC (Reinforced Cement Concrete) dam as opposed to another type of dam, such as an earthen dam or another type of concrete dam, is also determined by geotechnical conditions and the suitability of construction materials in the surrounding area. Therefore, there is the cost of the diversion tunnel as well as the cost of the dam itself, which is affected by its type and construction materials. The calculation of the cost of the dam itself is not within the scope of this paper. Moreover, although the conventional method appears more time-consuming and costly, it is often chosen because the equipment is already available from previous construction projects, and it does not require more specialized personnel for the excavation and support sequence.

The present study marks the initial steps in developing a methodology for comparing the technical and economical aspects of conventional and Micro-TBM methods for dam diversion tunnels. To enhance its comprehensiveness, it is suggested to parameterize the characteristics of the rock mass, dam embankment, and reservoir area for a more accurate selection of dam type. This would move beyond purely economical and time-based criteria. Additionally, there's a need to focus on peripheral stations to better understand their influence on reducing surface runoff from rainfall curves, thus improving the accuracy of data representation. Further investigation and improvement of the stochastic framework regarding flood event categorization and correlation to flood risk levels is recommended. This could involve generating multiple synthetic hydrographs for each return period instead of selecting individual flood events.

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