Revisiting Ancient Dam Engineering: Stability Assessment of the Alyzia Dam Using Modern Geotechnical Methods

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ABSTRACT

Ancient hydraulic structures such as dams represent remarkable achievements of early engineering, reflecting local geological knowledge and innovative construction techniques. Despite their age, many ancient dams remain structurally significant, necessitating robust methods for assessing their stability and informing conservation strategies. This paper presents a systematic geotechnical approach for evaluating the stability of ancient dams, combining historical information, geotechnical characterization, nonlinear finite element analysis and supplementary analytical checks. The Alyzia Dam in Western Greece, a Classical-period dry stone masonry dam, is analysed as a representative case study. Comprising stepped downstream courses of dry-laid limestone blocks, the dam stands approximately 11 m high with a crest length of about 25 m and was probably built in three construction stages. Historical and archaeological sources provide the basis for reconstructing its geometry and probable hydraulic function, either as a sediment-retention or energy-dissipation structure. Material properties are estimated through conservative engineering judgment and analogues from ancient masonry and modern rubble dams. Two-dimensional coupled seepage–stress analyses simulate staged construction, seepage patterns and deformation under both historical operating conditions and current design situations according to Eurocode 7 (CEN 2004a) and Eurocode 8 (CEN 2004b). A verification framework is adopted that mirrors contemporary design of rock-fill flow-attenuation dams, including checks for piping, hydraulic fracture and foundation bearing capacity. Results highlight both the inherent resilience and potential vulnerabilities of ancient dry-masonry dams, offering insight into their original design philosophy and providing quantitative guidance for present-day preservation and monitoring strategies.

Keywords: Ancient dams; Alyzia Dam; Heritage preservation; Finite element analysis; Eurocode 7; Eurocode 8.

INTRODUCTION

From the early second millennium BC onwards, dams and embankments were constructed in Greece to manage floods, reclaim land and regulate water resources. These structures range from small diversion weirs to large-scale hydraulic systems, such as the drainage works of Kopais (Kountouri, 2012) and the embankments of Mantineia and Stymphalos. Despite the absence of formal engineering theory, many ancient dams exhibit sophisticated use of topography, geology and construction staging. Extensive drainage and dam works in ancient Greece, including large-scale systems such as those at Kopais, have been investigated from hydraulic, geomorphological and landscape perspectives (Knauss, 2002; Koutsoyiannis et al., 2008; Kountouri et al., 2012).

The engineering assessment of ancient dams is increasingly important for three reasons. First, several structures remain partially functional or influence modern land use and flood behaviour. Second, their long-term survival provides rare full-scale evidence of the performance of dry masonry and rockfill systems under repeated loading and seismic activity. Third, as protected monuments, they require quantitative evaluation to inform conservation strategies, monitoring and risk management.

Previous research on ancient Greek dams has primarily focused on archaeological description and hydraulic interpretation. In contrast, geotechnical analyses addressing stability, failure mechanisms and structural response remain limited, particularly for dry stone masonry dams governed by block interaction and joint behaviour.

This paper applies a modern geotechnical assessment methodology to the Alyzia Dam, integrating the reconstruction of the model based on archaeological information, finite element modelling and code-based verification. The aim is to understand both the original engineering logic and the present-day stability of the monument.

ANCIENT DAMS IN GREECE AND THE ALYZIA DAM

Ancient dams in Greece

Ancient Greek hydraulic works include earth embankments, rubble-masonry gravity dams, diversion weirs and complex drainage systems (Knauss, 2005). Many were constructed within narrow valleys or gorges to minimize crest length and exploit competent rock abutments (Kerisel, 1985; Schnitter, 1994). Staged construction was common, reflecting sedimentation processes and evolving hydraulic needs (Moutafis, 2006; Tsatsanifos, 2015). Representative examples are found at Kopais, Olympia, Mantineia, Taka, Thespiai, Stymphalos and in eastern Crete (e.g. Choiromandres), many of which date to the Mycenaean or Classical periods and demonstrate careful foundation selection, integration with natural topography and hydraulic control provisions such as spillways and staged raising (Knauss, 2002; Knauss, 2005; Kountouri et al., 2012; Parry, 2005; De Feo et al., 2012).

These works sit within a broader eastern Mediterranean tradition of early dam building, with notable parallels at Jawa in Jordan and Sadd el-Kafara in Egypt (Kerisel, 1985; Schnitter, 1994). Some ancient dams remain in partial operation today, while others form the backdrop for modern dams or reservoirs, illustrating the ongoing interaction between hydraulic engineering and cultural heritage (Kerisel, 1997; Tsatsanifos, 2015).

Historical and Archaeological information on Alyzia Dam

The Alyzia Dam dates back to the mid 4th to 3rd century B.C.E, and is located near the ancient city of Alyzia in the Xirovouni region of western Acarnania, Greece, along the narrow “Glosses” gorge of the Varnakas torrent, a few kilometres inland from the modern coastal plain near Mytikas (Murray, 1984).

The structure is visible from the downstream side and comprises large limestone ashlar blocks arranged in approximately fifteen horizontal courses forming a stepped downstream face. Individual blocks are typically 0.6–0.8 m in height and length, and no mortar is present, indicating pure dry masonry.

The dam has a total height of approximately 11 m and a crest length of about 25 m. No mortar is present, indicating dry stone masonry construction. Archaeological and geomorphological evidence suggests at least three construction phases (Moutafis, 2006; Moustakas, 2012), consisting of an initial weir of approximately 4 m, followed by two successive raisings.

A natural or modified spillway exists at the left abutment, directing overflow onto competent limestone and limiting uncontrolled overtopping of the dam crest.

Figure 1. *Top (left) and front (right) view of the Alyzia Dam and gorge setting. Adapted from Tsatsanifos (2015).*

GEOLOGICAL AND GEOTECHNICAL SETTING

Site geology and geomorphology

The dam is founded within a narrow rock gorge cut into the foothills of the Acarnanian mountains. The foundation consists predominantly of bedded limestone of generally good quality, locally karstified and fractured, overlain by colluvial and alluvial deposits in the wider valley. Steep rock slopes at the gorge outlet provide lateral confinement and a natural abutment configuration, favouring a relatively short crest and a slender structure.
Upstream of the dam, naturally transported torrent sediments have accumulated, ranging from sands and gravels to cobbles and boulders. The original construction may also have included internal rubble or earth backfill between stone skins; however, the present analyses focus on the observed upstream alluvium and the masonry body.

Adopted material parameters

No direct geotechnical investigation has been carried out at Alyzia. Mechanical and hydraulic parameters were therefore derived from published values for limestones and ancient masonry, combined with engineering judgement and back-analysis recognising the dam's long-term performance (De Feo et al., 2012; Parry, 2005).
The dam body and foundation rock are represented by two materials:

- Intact Limestone: the intact limestone matrix of both foundation and blocks, using the Generalised Hoek–Brown criterion (1980a; 1980b);

- Open Joints and gaps: zones coinciding with the open gaps and discontinuities between blocks and along major joints, sharing the same elastic and strength parameters as Rock but with distinct hydraulic behaviour.

Two soil-type materials represent the natural colluvial/alluvial foundation and the anthropogenic transported fill upstream of the dam.
Since hydraulic properties cannot be directly assigned to the current model, a 5 cm envelope was modelled around each block, characterized by very high permeability so that it behaves mechanically as intact rock, while hydraulically functioning as a preferential flow path.

Table 1 summarises the key parameters as implemented in the model.

Table 1. *Adopted mechanical and hydraulic parameters (characteristic values) for the 2D coupled analyses*

Material	**Intact Limestone (Rock-mass & Blocks)**	**Open Joints / gaps between blocks**	**Natural colluvial / alluvial foundation**	**Transported upstream fill**
γ (kN/m^3)	26	26	21	21
k (m/s)	1×10^{-10}	1×10^{-3}	1×10^{-8}	1×10^{-6}
v	0.30	0.30	0.30	0.30
$\sigma_{c,intact}$ (MPa)	50	50	-	-
E_{intect} (MPa)	25000	25000	100	40
GSI	90	90	-	-
$GSI_{reduced}$	85	85	-	-
$\sigma_{reduced}$ (MPa)	28.6	28.6	-	-
$\sigma_{reduced,residual}$ (MPa)	21.7	21.7	-	-
$E_{reduced}$ (MPa)	23965	23965	-	-
$E_{reduced,residual}$ (MPa)	23164.8	23164.8	-	-
c (kPa)	-	-	5	5
φ (°)	-	-	35	35

*Partial factors for design situations follow Eurocode 7 and the relevant National Annex. Hoek–Brown parameters (1980a, 1980b); dilation angle 3° for rock 0° for joints

NUMERICAL MODELLING STRATEGY

Modelling philosophy and software

Analyses were carried out adopting a two-dimensional plane strain representation of a typical dam cross-section. While the structure is inherently discontinuous, a continuum finite element approach with explicit representation of jointed hydraulic zones was adopted as a pragmatic compromise.

The model incorporates:

- elastic–plastic behaviour (Hoek–Brown for rock, Mohr–Coulomb for soils),
- coupled groundwater–stress analysis,
- staged construction and loading,
- and strength-reduction stability assessment.

Tensile failure is allowed to reduce shear strength to residual; joint opening reduces stiffness by a factor of 0.01, permitting partial separation of blocks.

Joint and Block modelling

Each masonry unit is represented as a separate FE block. Adjacent blocks are connected by linear joint elements defined by Mohr–Coulomb shear strength and negligible tensile capacity. Reduced kn/ks allow relative displacement and limited opening. This enables block sliding and local separation to develop naturally under hydraulic and seismic actions.

Due to the inability of the interface elements to carry independent hydraulic properties, thin hydraulic bands are introduced along joint traces and around block perimeters. These share mechanical properties with limestone but have permeability several orders of magnitude higher, reproducing preferential seepage paths and attenuating uplift pressures in a manner consistent with drained dry-masonry behaviour.

Geometry and staged construction

The model (Figures 2 ÷ 5) represents a 2D cross-section roughly perpendicular to the dam axis. The downstream stepped face and near-vertical upstream face are discretised with individual stone blocks that follow the Hoek–Brown failure criterion (1980a; 1980b). In between, joints and gaps are modelled as a zone with the same mechanical properties as the stone blocks but with more permeability (Figure 6). The zone width (5 cm) was selected as a plausible value to represent the openings between the somewhat irregular stone blocks of the dam and as the minimum width required to ensure adequate meshing; however, it should not be regarded as a fixed or binding value.
The surrounding Rock-mass is extended laterally and at depth to minimise boundary effects. The dam's construction history is represented by three stages:

- Stage A – Initial weir ≈4 m high (Figure 2)
- Stage B – First raising ≈6 m total height (Figure 3)
- Stage C – Final raising ≈11 m total height (Figure 4)

At each stage, new masonry is activated; the upstream naturally deposited alluvium and local geometry are updated to reflect progressive sediment accumulation and channel adjustments. The model referring to the current condition of the dam is depicted in Figure 5.

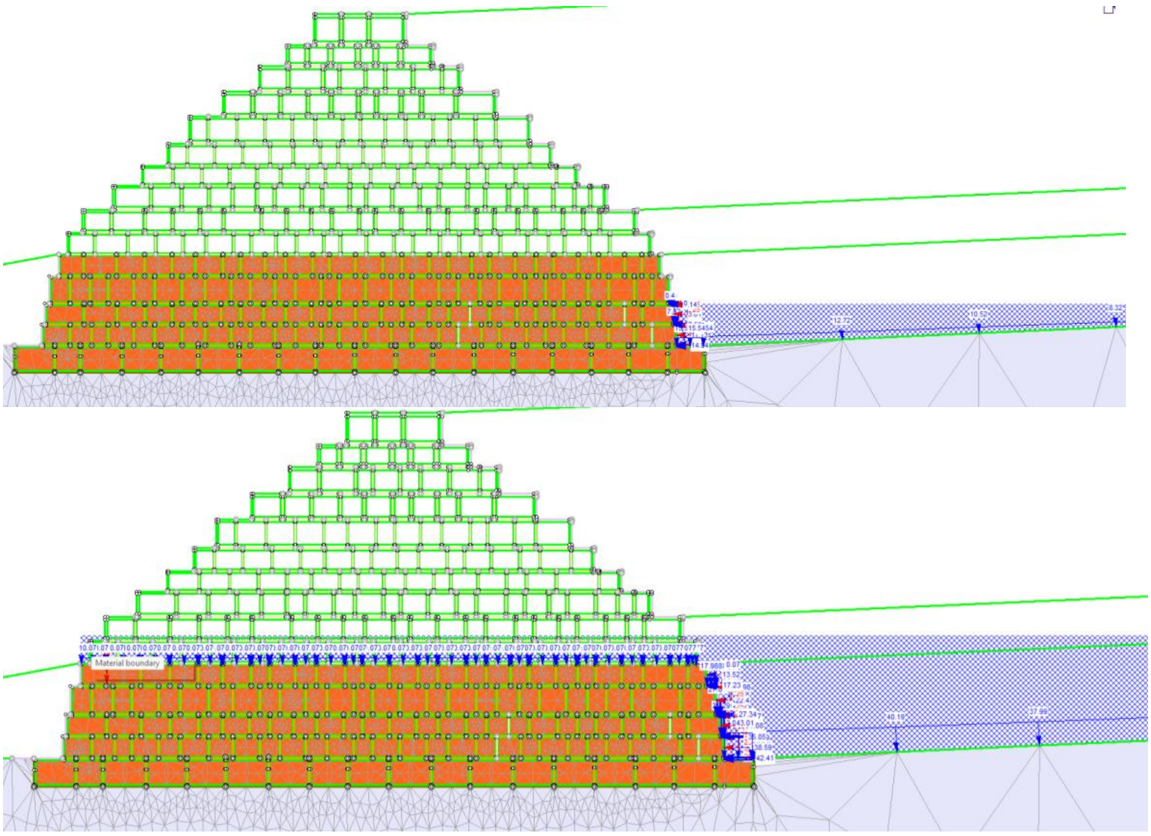

Figure 2. *Phase A - Initial structure of the dam. Upper: Normal flow, Bottom Overflow.*

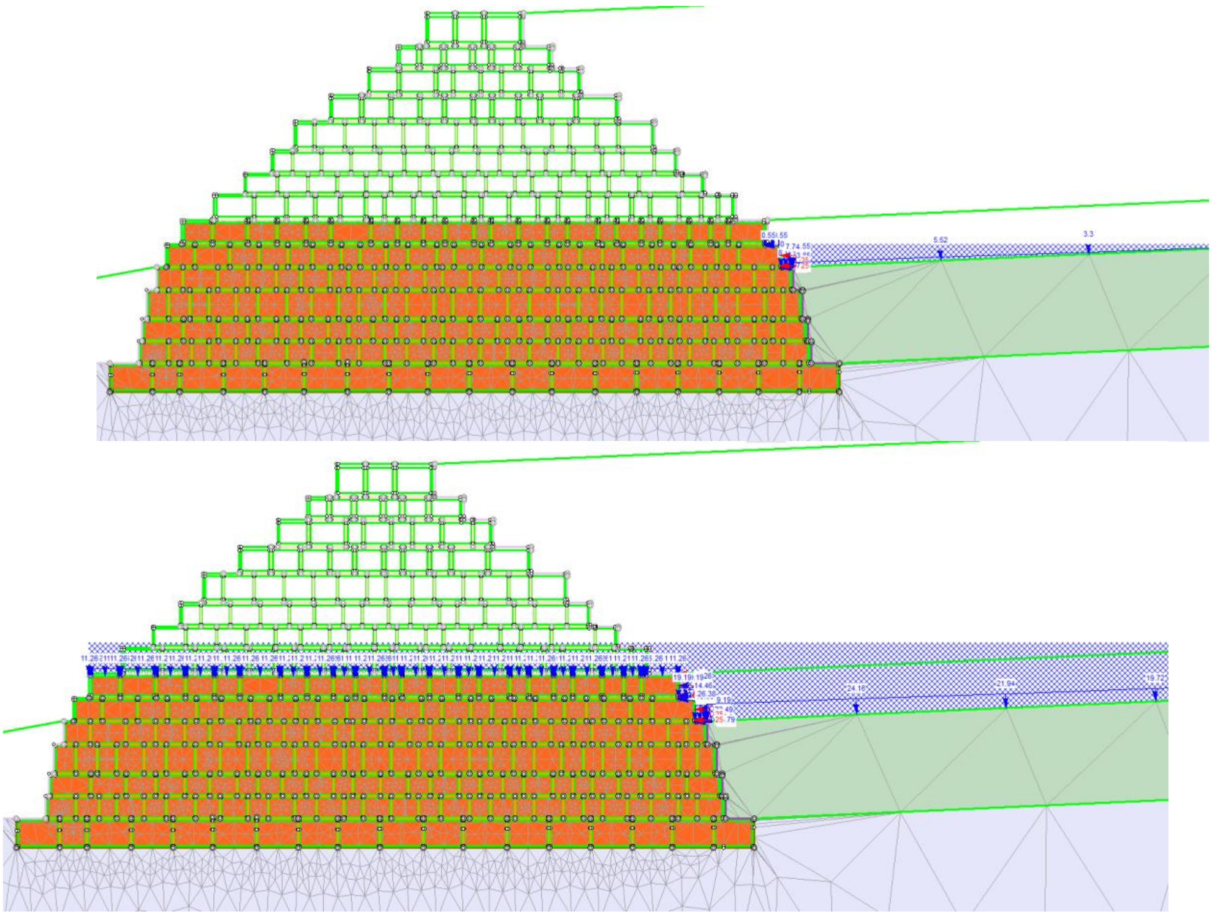

Figure 3. *Phase B – First raising of the dam. Upper: Normal flow, Bottom Overflow.*

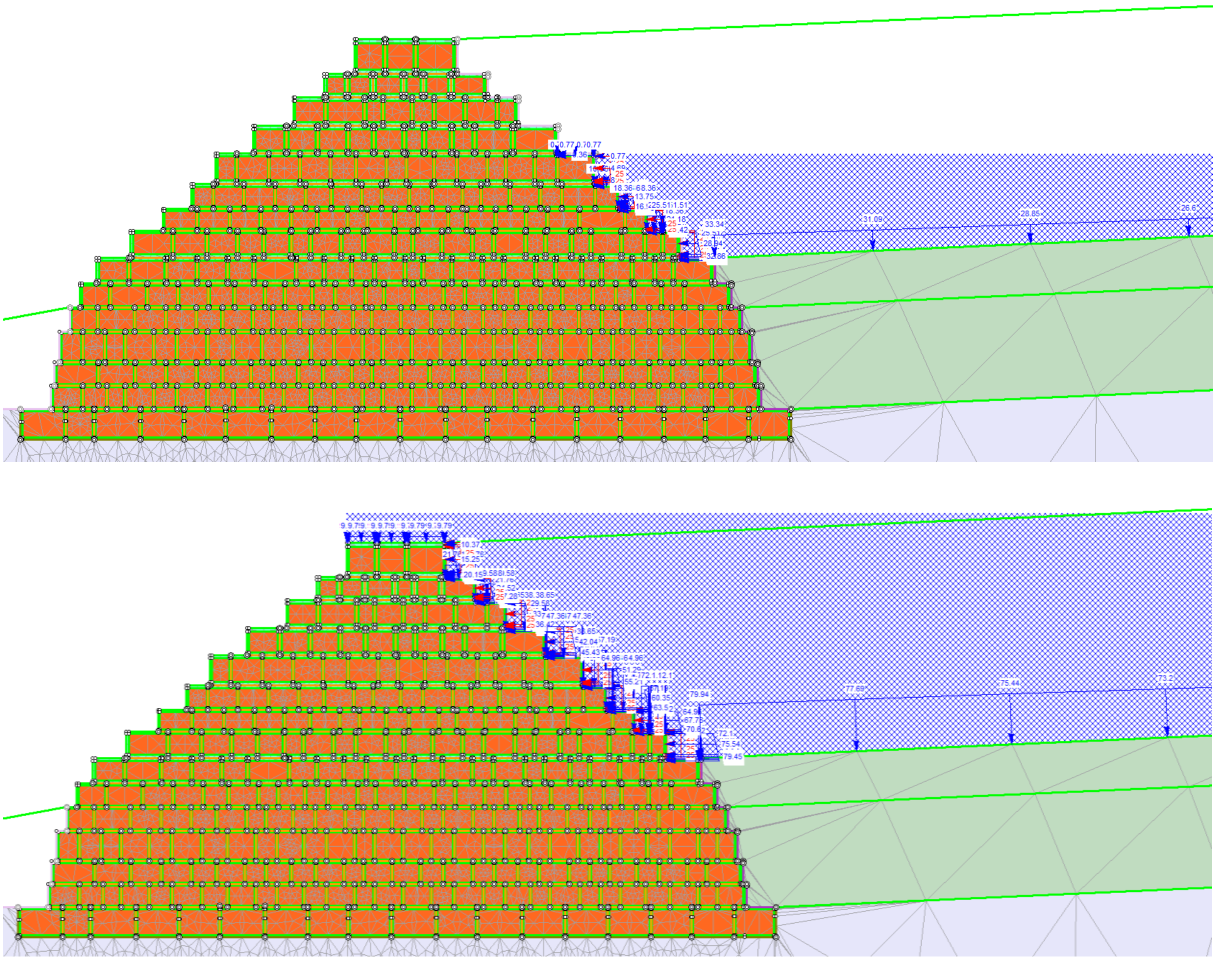

Figure 4. *Phase C – Final raising of the dam. Upper: Normal flow, Bottom Overflow.*

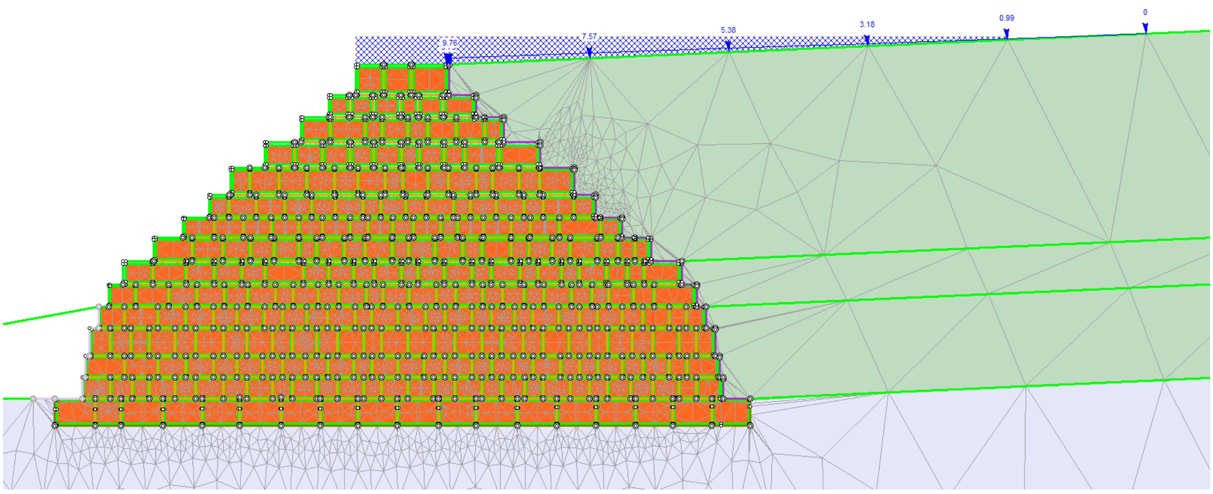

Figure 5. *Current condition of the dam. .*

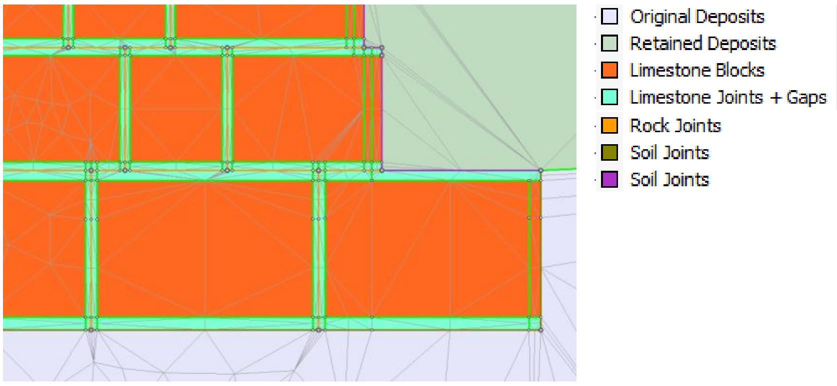

Figure 6. *Detail of dam modelling with legend for the materials used in the modelling of Alyzia Dam: stone blocks (Limestone Blocks), contact interfaces (Rock Joints, Soil Joints) and thin permeable zone (Limestone Joints+Gaps).*

Groundwater and seepage modelling

Groundwater flow is solved using finite-element groundwater formulation, coupled with the stress analysis.

Boundary conditions impose a design upstream water level (varying by scenario) along the retained sediments and adjacent rock, and a downstream water level at the exposed channel. The model computes pore-pressure distributions, phreatic surfaces and seepage velocities for each load case. Figure 7 presents the discharge velocity and the flow surface as estimated by the coupled analyses of the current condition under hydraulic loads.

The contrast between low-permeability stone blocks and high-permeability joints/gaps produces a realistic pattern of seepage concentrated along jointed paths and within the upstream alluvium, while uplift pressures at the base are reduced.

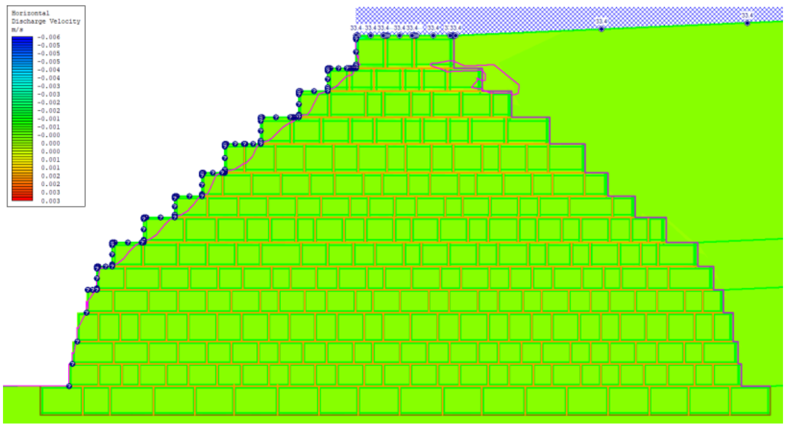

Figure 7. *Horizontal discharge velocity (reduced volumetric flow rate). The purple line indicates the surface of the flow.*

Staged Construction Simulation

The analytical workflow originally comprised a sequence of historical construction and loading stages, followed by code-based ultimate limit state verifications. For the purposes of the present work, and to maintain clarity without loss of technical content, the analyses are organised into a compact matrix combining structural configuration and hydraulic condition. Three representative construction configurations, together with the current condition of the dam, are examined under three characteristic hydraulic states. In addition, two code-consistent verifications are carried out for the current configuration according to Eurocode 7 and Eurocode 8. In total, eleven representative analyses are reported, which fully capture the governing deformation and stability mechanisms while avoiding unnecessary repetition of intermediate stages.

The detailed correspondence between the historical construction stages and the condensed analysis matrix is omitted for brevity, as it does not affect the comparative assessment of stability and deformation trends discussed herein.

Table 2 summarises the verification framework and load conditions adopted in the present work, grouping the analysed configurations and hydraulic states into characteristic situations and Eurocode-based verification cases.

Table 2. *Loading Conditions*

Structural Phase	Normal Static Condition	Normal Hydraulic Condition	Extreme Hydraulic Condition
Initial Construction	Stage 3	Stage 4	Stage 5
First raising	Stage 7	Stage 8	Stage 9
Final raising	Stage 11	Stage 12	Stage 13
Current Condition	Stage 16	-	-
Current Condition – EC7 ULS (DA2)	Stage 17	Stage 18	-
Current Condition – EC-8 pseudostatic	Stage 19	-	-

Stages 1 and 2 represent the initial geostatic conditions prior the construction of the dam. Stages 6, 10 and 14 represent the state were in each construction phase the upstream reservoir has been silted up Stage 15 represents the stage in which some historical repairs have been made on the downstream side of the dam.

RESULTS OF COUPLED ANALYSES

Historical stages: structural performance and deformation

For the initial 4 m-high weir, the computed displacements are negligible and plastic deformation is essentially absent. The stress field is dominated by self-weight and geometric confinement within the gorge, while seepage and uplift effects remain minor.

The first raising to approximately 6 m results in increased compressive stresses within the lower masonry blocks and the foundation, without inducing extensive yielding. Seepage localises near the upstream heel and along preferential block joint and gap zones, but remains limited overall. The phreatic surface exhibits a gently sloping profile, intersecting the downstream face near mid-height.

The final raising to 11 m introduces significantly higher hydraulic heads and increased potential for bending effects within the masonry. Numerical analyses reveal pronounced compressive arching along the stepped downstream face, with stresses preferentially transferred toward the abutments. Localised yielding develops at the downstream toe and within jointed zones near the upstream heel; however, no continuous failure mechanism or global instability surface is formed (Figure 8).

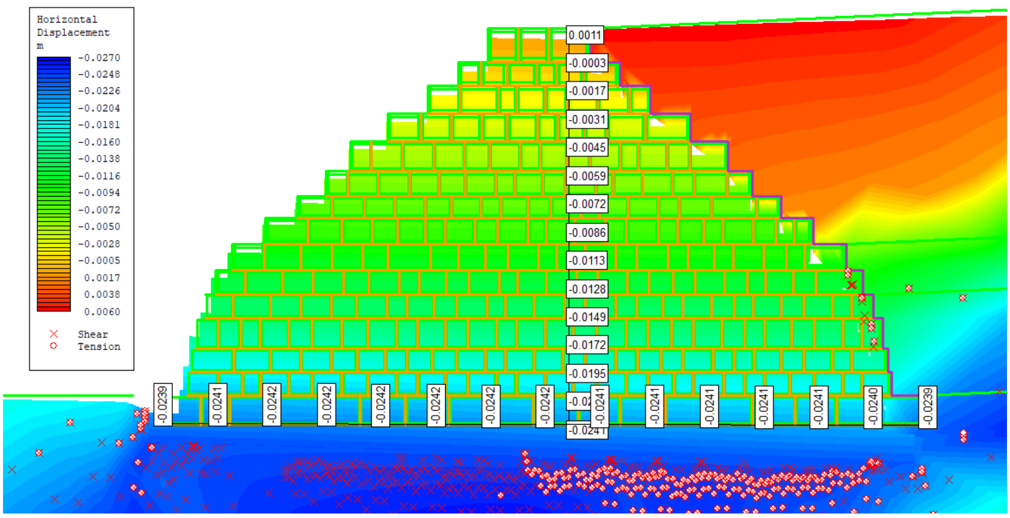

Figure 8. *Horizontal displacement contours and phreatic surface for the final historic configuration at full upstream level. The yielded elements are depicted in red marks.*

EC-7 design combinations – current condition

Although the Alyzia Dam a historic monument to which modern design frameworks are not formally applicable, for deeper understanding of the current condition, supplementary analyses were performed following Eurocode 7 (CEN, 2004a) design philosophy. Two loading situations were examined: Normal Geostatic Conditions ($NGC_{cur,EC\text{-}7}$) and Normal Hydrostatic Conditions ($NHC_{cur,EC\text{-}7}$).

Verification was carried out at the Ultimate Limit State (ULS) following Design Approach 2 (DA2). Partial factors were applied to actions, material parameters, and resistances in accordance with EC7 provisions for retaining structures (A1 + M1 + R2 combination). For overturning verification, the prescribed EQU partial factors were additionally adopted.

EC-8 seismic response

In Stage 19, the Eurocode 8 seismic design scenario (EQcur,EC-8) is examined using Spectrum Type I. Hydrodynamic pressures and pseudo-static inertial forces associated with flood conditions are not considered, since their simultaneous application would lead to an excessively conservative and physically unrealistic loading scenario.

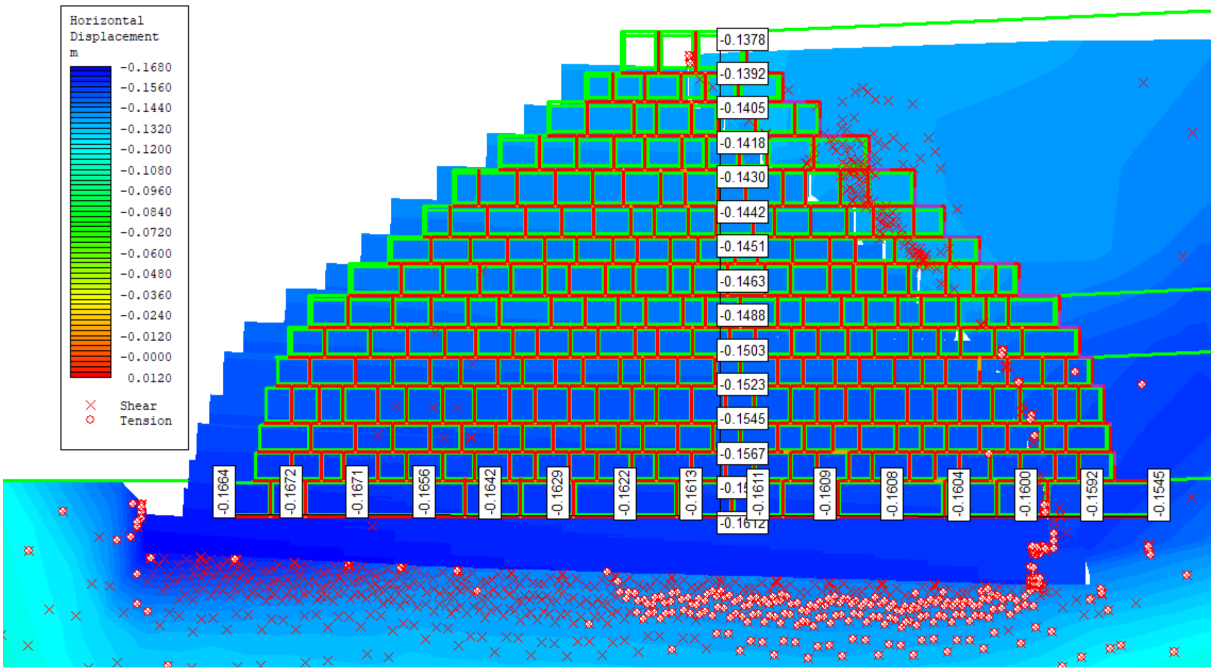

Figure 9. *Horizontal displacement during the EC-8 seismic action. The yielded elements are depicted in red marks.*

The horizontal component of the design seismic acceleration is a_{hd}=0.338g, whereas the vertical component is a_{vd}=0.169g. The estimated maximum historical peak ground acceleration (PGA), based on historical seismicity data (Papazachos, 2003) and ground-motion attenuation relationships applicable to the Greek region (Kolliopoulos, 1998; Rinaldis, 1998; Skarlatoudis, 2003), ranges from 0.098g to 0.165g. These values are significantly lower than the design value prescribed by the relevant code provisions (EC8) for the area in which the dam is located (a_d = 0.36g).
It is further noted that the seismic acceleration is assumed to be uniform over the dam height, in accordance with the provisions of EAK 2000 (Greek Seismic Code, 2000) for the pseudostatic analysis of embankments with a height of less than 15 m.
The numerical results indicate that the seismic response is characterised by increased tensile stresses and a high likelihood of crack initiation at the upstream face near the crest, elevated shear stresses along the masonry-rock interface and within critical rock hydraulic bands, and localised plastic yielding at the downstream toe and in the upper part of the upstream alluvial deposits. As shown in Figure 9, the displacement pattern is dominated by downstream movement, while the yielded elements are concentrated in critical zones and are marked in red. The horizontal displacement at the base reaches approximately 1.5% of the free height of the dam, whereas the displacement of the top wedge is

approximately 1.3% of the free height. The fact that the base displacement exceeds that of the top wedge suggests that sliding governs the failure mechanism, rather than dominant overturning or rocking.
The computed stress, plastic strain, and displacement patterns indicate that strong earthquakes may induce permanent displacements, joint opening, and limited rocking of the dam. However, the development of a global collapse mechanism would require an unfavourable combination of parameters and very strong ground shaking.

SUPPLEMENTARY ANALYTICAL CHECKS

To complement the finite element results and maintain full compatibility with contemporary practice, three additional families of checks were carried out. The seepage and stress fields are post-processed to obtain hydraulic heads, pore pressures and contact stresses at specific control points, and these are then used in classical analytical expressions. Partial factors are applied in DA-2 format, so that the numerical and analytical verifications speak the same "Eurocode language".

Hydraulic gradient, piping risk and hydraulic fracture

Piping is assessed by comparing the computed hydraulic gradients between characteristic points with the critical gradient

(1) $$i_{cr} = \gamma'/\gamma_w$$

where γ' is the submerged unit weight of the soil or block material and γ_w the unit weight of water.

For the foundation soil ($\gamma \approx 21$ kN/m³) the critical gradient is about 1.14. Characteristic gradients are evaluated between points at the upstream and downstream toes at the surface and at depth (P1–P10).

The aforementioned hydraulic checks are performed for the most unfavourable states (stages) of the dam, throughout its whole history (Stages 5, 9 13 and 18). The safety factors for those hydraulic checks are given below:

- Hydraulic gradient and piping risk: $FS = i_c/i > 1$,

and

- Hydraulic Fracture: $FS = \gamma_{G,dst}(i \cdot \gamma_w \cdot H) < \gamma_{G,stb}(\gamma' H) > 1$

In each examined state, the hydraulic gradient between four sets of points is estimated, along with the safety factors against piping and hydraulic fracture. The results are shown in the following Table 3.

Table 3. *Results of hydraulic checks against piping and hydraulic fracture.*

Location Check	**Factor of Safety against Piping**	**Factor of Safety against Hydraulic Fracture**
Stage 5: Extreme Hydrostatic Conditions of the initial construction (EHC_{ini})		
Horizontal Check at foundation level	6.96	
Horizontal Check at basin level	7.50	6.96
Vertical Check Between upstream basin and foundation levels	3.95	
Stage 9: Extreme Hydrostatic Conditions of the secondary construction (EHC_{sec})		
Horizontal Check at foundation level	11.30	
Horizontal Check at basin level	13.29	2.62
Vertical Check Between upstream basin and foundation levels	3.36	
Stage 13: Extreme Hydrostatic Conditions of the final construction (EHC_{fin})		
Horizontal Check at foundation level	3.68	
Horizontal Check at basin level	4.32	1.38

Table 3 (continue). *Results of hydraulic checks against piping and hydraulic fracture.*

Location Check	Factor of Safety against Piping	Factor of Safety against Hydraulic Fracture
Vertical Check Between upstream basin and foundation levels	2.46	
Stage 18: Normal Hydrostatic Conditions of the current condition ($NHC_{cur,EC\text{-}7}$)		
Horizontal Check at foundation level	32.52	
Horizontal Check at basin level	18.37	2.67
Vertical Check Between upstream basin and foundation levels	2.92	

In most cases, the average gradients over the foundation thickness remain well below i_c, yielding safety factors typically higher than 2.0.

The lowest safety factor against piping appears locally, close to the downstream toe and within short distances, where peak gradients approach i_c in the most unfavourable conditions, suggesting that localised erosion is conceivable but global piping is unlikely.

For all three construction (historic) stages in normal static and hydrostatic conditions as well as extreme hydrostatic conditions, computed displacements are negligible and plastic zones are not developed.

Supplementary Limit-Equilibrium Hydraulic and Stability Checks

In addition to the finite element analyses performed in the present study, an independent verification of the hydraulic and overall stability behaviour of the Alyzia Dam was carried out using limit equilibrium methods and dedicated software (MacRA Design, Maccaferri), which is specifically developed for grade-control and flow-attenuation structures.

The hydraulic parameters were selected so that the structure behaves as a dry stone boulder dam and not as a typical gabion structure, by assuming limited but non-zero seepage through the body and no full internal drainage. This approach is more representative of an interlocked boulder structure with partially filled joints.

According to the MacRA (Maccaferri, 2024) hydraulic results, the upstream flow does not submerge the crest (main weir not submerged);downstream, the supercritical flow develops with Froude number Fr ≈ 3.07, followed by a hydraulic jump within the stilling basin; and the predicted tailwater level and the geometry of the counterweir are sufficient for proper control of the hydraulic jump, satisfying the minimum required tailwater depth. In addition, MacRA (Maccaferri, 2024) computes upstream and downstream hydrostatic pressures, and uplift pressure due to seepage along the foundation, which are subsequently used in the stability checks.

The limit equilibrium stability checks indicate that the Alyzia Dam exhibits adequate safety against sliding ($FS_{sl} \approx 2.10$), overturning ($FS_{ov} \approx 20.6$), seepage / piping - critical gradient method ($FS_{seep} \approx 1.45$) and uplift ($FS_{uplift} \approx 1.65$).

These results demonstrate that even under unfavourable hydraulic conditions, the structure presents substantial safety margins against the main failure mechanisms.

It should be noted that although the geometry of the Alyzia Dam resembles that of a gabion dam, its hydraulic behaviour is fundamentally different. Gabions are characterised by very high and controlled permeability, which results in rapid dissipation of pore pressures and practically negligible upstream hydrostatic thrust. In contrast, in dry stone masonry dams the permeability depends on the degree of block interlocking and joint filling, which may lead to partial development of hydrostatic pressures and thus higher hydraulic loading.

To account for this difference, a conservative permeability value was adopted in the MacRA model (porosity n = 0.25 instead of the typical n ≈ 0.40 for gabions), in order to better represent the behaviour of an interlocked boulder structure.

For this reason, the MacRA Design results (Maccaferri, 2024) are used in the present work as an independent reference check, whereas the final assessment of the hydraulic and geotechnical behaviour is based on the 2D groundwater stress analyses performed for Alyzia Dam.

DISCUSSION AND CONCLUSIONS

The analyses confirm that the Alyzia Dam constitutes a mechanically rational and structurally efficient system whose long-term survival can be interpreted through fundamental geotechnical behaviour rather than exceptional material strength. The initial 4 m-high weir already exhibits a well-proportioned geometry, negligible displacements and absence of plastic zones, indicating that the original conception was structurally sound and appropriately adapted to the confined limestone gorge. The selection of a narrow rock abutment setting, combined with adequate base width and a stepped downstream face, demonstrates an intuitive yet effective “design” logic that ensured stability from the earliest construction phase and provided a reliable foundation for subsequent raisings.

As the dam was elevated in stages, compressive stresses increased but remained favourably redistributed through arching toward the abutments, while no continuous failure surface developed under historic or current hydraulic. Coupled seepage–stress analyses indicate that hydraulic gradients are generally well below critical values, and supplementary analytical and limit-equilibrium checks confirm satisfactory safety margins against sliding, overturning, piping and uplift. The structural response is governed primarily by masonry–joint–foundation interaction, with the gorge confinement playing a decisive stabilising role.

Seismic loading represents the governing action. Under Eurocode 8 pseudo-static conditions, the response is characterised by sliding-dominated behaviour, localised yielding at the downstream toe and potential joint opening near the crest. Permanent displacements may occur during strong earthquakes, yet a global collapse mechanism does not readily form, indicating deformation-controlled resilience rather than brittle instability.

The study further demonstrates that a coherent framework combining staged historical reconstruction, coupled finite element modelling and Eurocode-consistent verification (EC7 DA2 and EC8) can be meaningfully applied to ancient dry-masonry dams despite limited in-situ data. Expressing the results within a contemporary design language allows objective assessment while respecting the monument’s character.

Overall, the Alyzia Dam appears stable under typical hydraulic conditions, with seismic effects and downstream toe behaviour representing the critical aspects for long-term preservation. The good structural conception of the initial construction, progressively enhanced through staged raising, explains the monument’s durability. Targeted monitoring of joint behaviour and limited local toe protection, if required, would further enhance robustness without altering the structural identity of the dam.

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