Anastylosis of the northeastern tower of the ancient fortress of Aegosthena in Attica: construction analysis, collapse mechanism and interventions for the improvement of the foundation conditions

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ABSTRACT

The northeastern tower of the Hellenistic fortress of Aegosthena in Attica presents significant conservation and structural challenges due to its complex foundation conditions. In the present study, the structural analysis of the tower demonstrated that the original foundation system was insufficient to safely support the superstructure, contributing to the observed instabilities and collapse mechanisms. To address these issues, a detailed geotechnical investigation was carried out, considering both shallow and deep foundation alternatives. Shallow foundations were found to require extensive earthworks and remediation of the heterogeneous fill, which would result in substantial disturbance and loss of the original ancient filling and construction techniques. Conversely, deep foundations using micropiles were shown to provide uniform bearing conditions, effectively supporting the superstructure while minimizing intervention into the existing fill. The micropile system not only stabilizes the tower but also functions as a confinement enhancing the stability of the surrounding soil without compromising the archaeological integrity of the site. Comparative analyses of the two foundation strategies confirmed the superiority of the deep foundation approach in terms of structural performance, preservation of heritage materials, and minimal impact on the historical context. The findings provided a technical basis for the anastylosis of the northeastern tower and demonstrate how modern geotechnical and structural interventions can be integrated with the conservation of ancient masonry to ensure both safety and preservation..
Keywords: Hellenistic, tower, anastylosis, intervention, foundation, micropiles

INTRODUCTION

The Aegosthena fortress was built at the eastern side of the Corinthian Gulf. It is dated at around 340 B.C. and was part of the Athenian defensive system, which extended all over the Attica region. It consists in an acropolis built on top of a steep hill, to protect a vast fortified area extending from the foothill to the sea westwards. The northeastern tower is one of the four towers reinforcing the acropolis' eastern wall. It is a small, quadrangular dry-stack construction, with external dimensions approximately 7.5m at its lowest level and total remaining height around 11m (Figure 1). The tower had two accessible floors but – being built on the slope of the hill - it is composed of three distinct levels. The eastern and northern elevations are founded outside the fortified area on rock substratum (Figure 1 left). Below this foundation, the slope extends another 10 m above the ground. The western and southern elevations are built on top of the hill, inside the fortified area, approximately 3,5 m above the foundations of the previously mentioned elevations and 13.5-14 m above the ground. The ground level formed between the hill slope and these two (eastern, northern) elevations is approximately 5m high (corresponding to 10 rows of stone blocks east and north) and is filled with carefully built layers of stones, aggregate and earth. The western and southern elevations are founded on this filling material, with the exception of the S.W. corner (Figure 2).The entrance to the tower was located, as shown in Figure 1 (right) on the first level. The second level, only partially preserved, hosted the catapults used for the defence of the tower. Its existence is proven by the beam cases carved on the 18th course of stones, 9 m above the ground level (Figure 1 right). The protruding stones at the top of the S.E. corner (Figure 1 left) suggest that, close to the tower, the fortification wall was rising gradually until the 19th row, to protect the external staircase leading to the entrance of the second floor.

Figure 1. *The southern & eastern elevations of the tower in 1933 (courtesy of the German Archaeol. Inst., Athens). In dotted line: the missing fortification wall (left). The tower after collapse of the S.E. corner and eastern elevation (1981)-in white dotted line the entrance to the first level is shown (right)*

The tower was standing as shown in Figure 1 (left) until February 24th, 1981. This day, the eastern Corinthian Gulf earthquake (Papazachos & Papazachou (1997), Ambraseys (2009) has struck the building, resulting mainly in the collapse of the free-standing southeastern corner and a big part of the eastern elevation (Figure 1 right). The pathology of the remaining structure was extremely severe and further collapse in case of a future, even low intensity, earthquake was certain. In addition, the foundations of the western and southern elevations had gradually settled due to the deformability of the filling material, resulting in block subsidence up to 40 cm in the middle of the western elevation. Sliding of the blocks of the still standing southern elevation was evident by the opening of the joints (Figure 1 right). Moreover, the N.E. corner top blocks (on the upper right of Figure 1 right) were in an extremely unstable situation due to the severe opening of joints, the rotation of the remaining blocks outwards and the subsequent loosening of the corner connection.

The study for the monument's restoration was undertaken by the Directorate for Restoration of Ancient Monuments (DRAM) and comprised two phases: the first one consisted in (1.a) the documentation of the collapsed blocks and study of their original position; and (1.b) the constructional and structural analysis and assessment of the behaviour of the monument, including earthquake behaviour, in view of the selection and pre-dimensioning of the interventions (Toumbakari 2012).

The second phase was carried out following the approval of the aforementioned studies by the Central Archaeological Council and comprised the following components: (2.a) a detailed investigation of the tower's structural behaviour, including the dimensioning of the interventions approved during the first phase; (2.b) a geological and geotechnical study aimed at defining foundation conditions, deriving the mechanical parameters required for the design of the new foundation, and providing quantitative criteria for selecting either a shallow or a deep foundation system; (2.c) the structural design of the new foundation; (2.d) a study for the anastylosis of the collapsed blocks; and (2.e) worksite design and management (Toumbakari 2015).

To connect the structural diagnosis with the foundation choice, the paper first summarizes the constructional/structural assessment (Phase 1.b) and then presents the targeted geotechnical investigation and comparative evaluation of shallow versus micropile-based alternatives (Phase 2.b) – (Egglezos 2015). Accordingly, the paper contributes an evidence-based decision pathway for foundation intervention in a seismically exposed, dry-stacked masonry monument with mixed rock/fill support conditions. It combines (i) constructional analysis and structural modelling used to demonstrate the governing role of foundation deformability, with (ii) a targeted geotechnical campaign and code-consistent seismic actions, and (iii) staged 2D FE/DE soil–foundation–superstructure interaction analyses. The comparison quantifies the performance gap between shallow and micropile-based solutions in terms of permanent seismic displacements and settlements, while explicitly treating the heritage constraint of preserving the historic internal fill. The outcome is a technically justified foundation scheme aligned with minimum-intervention conservation doctrine and verified through implementation.

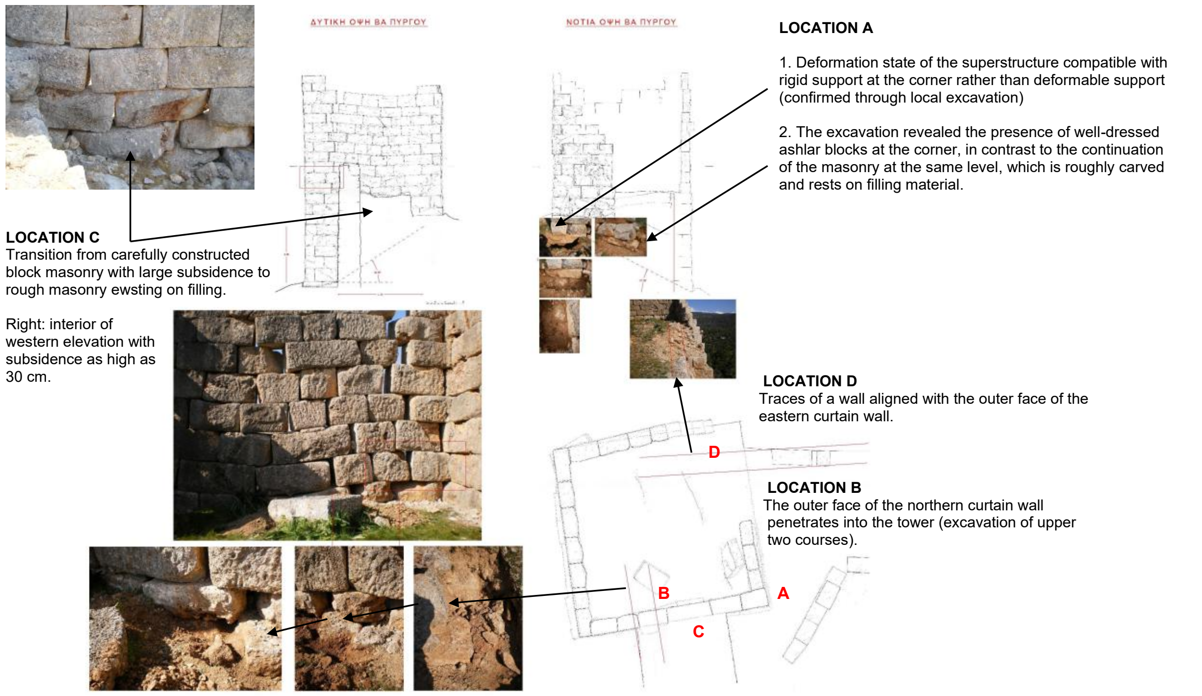

Figure 2. *Drawing reconstruction of the southeastern corner of the tower on the basis of archival photos*

DESCIPTION, ASSESSMENT & INTERPRETATION OF DAMAGES

Constructional analysis & collapse mechanism

The constructional analysis of the tower comprised a detailed survey of the structure’s dimensions and deformations, an assessment of the foundation condition—supplemented by a limited archaeological excavation—and systematic documentation of the individual blocks. On the basis of these data, the study focused on reconstructing the monument’s original configuration prior to 1981. Using archival photographs, the configuration of the missing eastern part of the southern elevation was gradually reconstructed; the results of this analysis are shown in Figure 2.

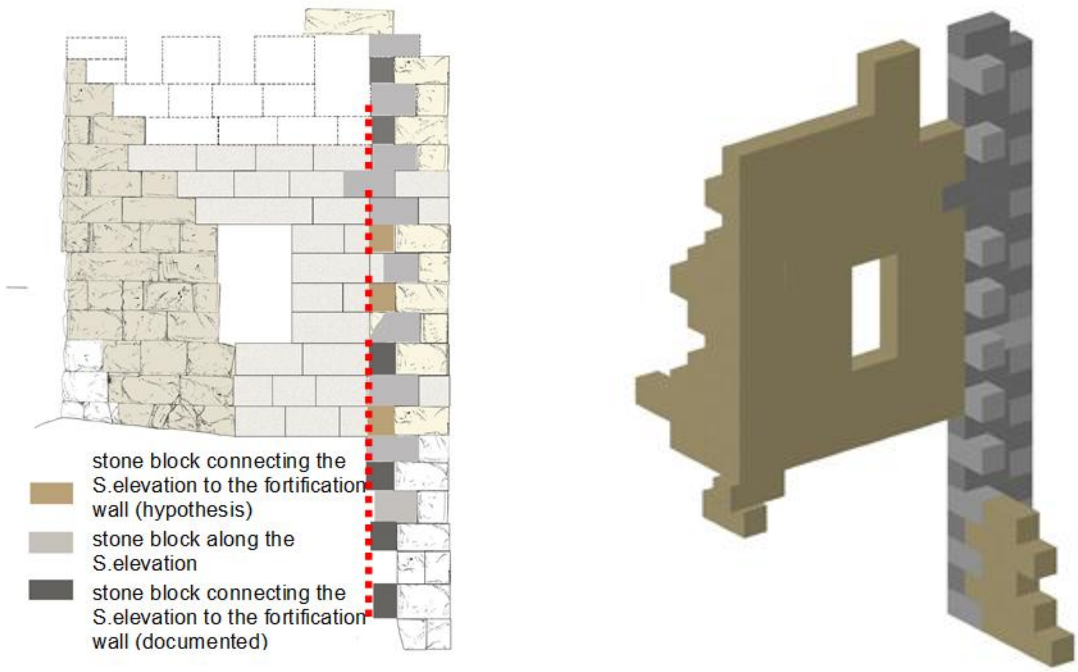

Figure 3. *Drawing reconstruction of the southeastern corner of the tower. A joint is formed between the blocks of the tower’s SE corner and the southern elevation (Toumbakari 2015)*

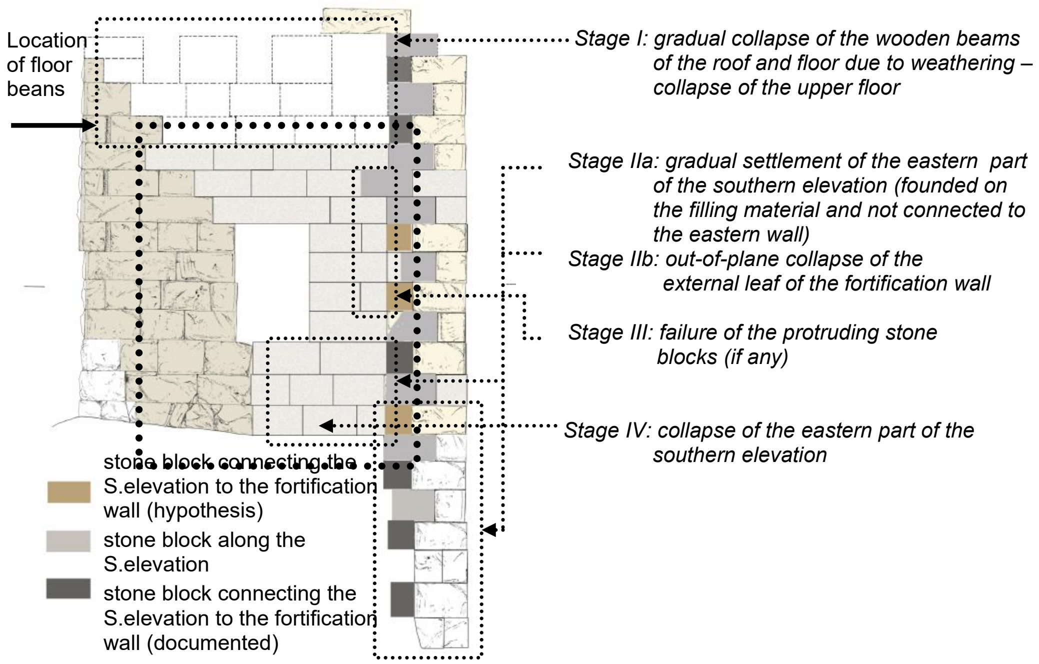

Figure 4. *The mechanism of failure of the N.E. tower of the Aegosthena fortress (Toumbakari 2015)*

It was demonstrated (Toumbakari 2015) that the blocks of the eastern elevation were not interlocked with those of the southern elevation. Instead, they were constructed to interlock with the fortress's eastern defensive wall, which runs approximately perpendicular to the southern elevation. This constructional feature is common to all towers along the eastern fortification wall of the acropolis and can be attributed to the ancient builders' strategy of reinforcing the external leaf of the curtain walls through this connection. While this approach is understandable in terms of construction speed and the reduction of damage from catapult attacks, it results in the formation of a joint between the southern and eastern elevations (Figure 3, left). Consequently, interlocking between the two elevations is weakened in favor of the connection to the curtain wall. When the uneven foundation conditions—both in terms of height and deformability—are also taken into account, the structural vulnerability of the N.E. tower becomes readily apparent. The theoretical reconstruction of the missing southern elevation and the understanding of the connection between the fortification wall and the eastern elevation of the tower permitted the study of its mechanism of collapse.As shown in Figure 3, four successive stages of deterioration can be proposed. Stage I consists of the gradual collapse of the wooden roof and floor beams due to weathering. Stage II comprises two distinct sub-stages, which may have developed in parallel and not necessarily in direct causal sequence. In Stage IIa, gradual settlement of the southern elevation—founded on fill material—may have occurred; the absence of a structural connection at the south-eastern corner facilitates localized settlement in this area. Stage IIb involves the out-of-plane collapse of the external leaf of the fortification wall, likely promoted by increased lateral pressure from the stone–earth infill and, decisively, by seismic actions. Stage III is characterized by pronounced settlement, which renders the eastern portion of the southern elevation—between the entrance and the vertical joint—completely free-standing. Stage IV corresponds to the failure of the south-eastern elevation, leading to the collapse of the remaining standing courses of the second floor. These mechanisms highlight that the damage cannot be attributed only to constructional choices; the presence of deformable foundation conditions played a critical and complementary role in the tower's failure.

STRUCTURAL ANALYSIS & ANASTYLOSIS APPROACH

Computational strategy for structural analysis & main results

The analysis of the structural pathology demonstrated a strong dependence on foundation conditions. To clarify this relationship and guide intervention design, the structural analysis was conducted in two phases, progressively refined through the integration of geotechnical considerations and anastylosis principles (Toumbakari 2015, 2016).

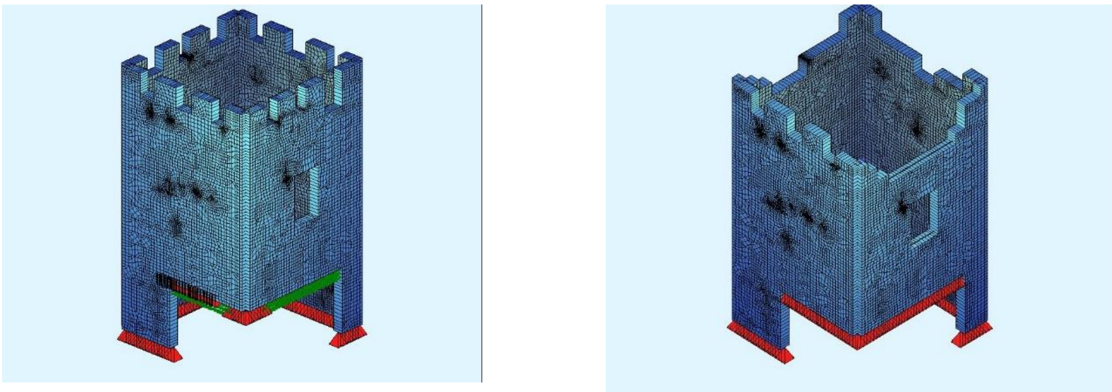

Figure 4. *The model used for spectral analysis before (left) & after (right) the interventions (view from SW. The rigid and deformable foundation conditions are depicted in red and green respectively.*

The first phase was intrinsically linked to anastylosis, balancing the reuse of authentic material with structural safety. Conservation of the N.E. tower in its post-1981 ruinous state was rejected due to the availability of collapsed blocks, while restoration to the pre-earthquake configuration was dismissed because of excessive seismic vulnerability. A balanced anastylosis solution was therefore adopted, involving partial reconstruction of the southern elevation, preservation of the characteristic joint of Hellenistic fortifications in Attica, and selective use of clamps and dowels to ensure structural continuity. Numerical modelling showed that inhomogeneous, deformable foundation conditions—combined with the absence of diaphragmatic action—adversely affected the superstructure's response. A second model, incorporating a rigid underfoundation and targeted connections, produced more uniform displacements and quasi-diaphragmatic behaviour, thereby validating both the intervention strategy and the need for a rigid underfoundation.

The rationale of the intervention on the foundations in relation to anastylosis principles

The N.E. Aegosthena restoration represented a key moment for understanding ancient construction practices. The filling material at the interior of the tower bears by definition crucial testimony to ancient technology. According to the principles of the Venice Charter (Venice Charter, 1964), particularly those concerning minimum intervention and respect for original material, the preservation of this stratified fill is essential. Foundation solutions such as rockfill or reinforced concrete beams on elastic soil would necessitate the removal of significant portions of the ancient deposits, resulting in irreversible loss of evidence. By contrast, the use of deep foundation (micropiles) minimizes disturbance to the archaeological layers and allows the original fabric to remain largely intact. This approach therefore complies with international conservation doctrine by ensuring structural stability while safeguarding the monument's historical and archaeological integrity. To substantiate the selection of a deep foundation system over a shallow alternative, a comprehensive geotechnical investigation was undertaken with the objective of generating quantitative data for each solution and, through their comparison, identifying the respective advantages and limitations (Egglezos 2016). This process enabled an objective and evidence-based evaluation of the proposed foundation schemes.

INVESTIGATION FOR THE IMPROVEMENT OF THE FOUNDATION CONDITIONS

Content of the geotechnical study

The geotechnical study was carried out in two phases. During the first phase, the following tasks were undertaken: (a) implementation of a minor geotechnical investigation program within the project area; (b) execution of geotechnical laboratory testing; (c) evaluation of the results of the geotechnical investigation; (d) determination of characteristic values of the mechanical and physical parameters of the geomaterials relevant to the study; (e) compilation of data concerning the general geology of the area; (f) performance of in situ geological measurements for the identification and mapping of rock discontinuities and for the classification of the rock mass according to the Geological Strength Index (GSI); (g) preparation of geostatic calculations for the bearing capacity of: (i) shallow foundations, and (ii) deep foundations with micropiles; (h) formulation of a proposal for the comparative evaluation of the two potential foundation schemes; and (i) preparation of a concise Technical Report presenting the above activities (a–h). The second phase consisted of the following activities: (a) additional laboratory testing (seven tests) on samples obtained from the soil fill of the Northeast (NE) Tower; (b) completion of the evaluation of the geotechnical investigation on the basis of the full set of investigations from both phases; (c) analyses of foundation–subsoil–superstructure interaction, undertaken to ensure the completeness of the study; and (d) calculations for the stabilization of unstable rock blocks on the rocky slope forming the foundation base of the NE Tower aimed at improving the stability of the foundation subsoil. In the following paragraphs, the main results of the analyses will be presented.

Geotechnical units

Two geotechnical units were identified (Egglezos, 2015). Unit I consists of soil fills (GC, GM, GP, SC, CL), for which the results of nine (9) classification tests performed on samples from excavation trenches were used. Based on statistical processing and the resulting mean values, the soil fill may be classified as clayey sandy gravel.

Table 1. *Parameters of the geotechnical units*

	Ia[a] (GC)	Id[b] (GP)	II
γ (kN/m^3)	21.0	21.0	26.0
c´ (kN/m^2)	3	3	280
φ´ (°)	36	40	45
c_u (kN/m^2)	-	-	2600
GSI	-	-	50
E_i (MPa)	30	50	12000
σ_{ci} (MPa)	-	-	30.0
E_{RM} (MPa)	-	-	3600
σ_{cRM} (kPa)	-	-	1800
$\sigma_{cRM\text{-}tx}$ (kPa)	-	-	5200
σ_{tRM} (kPa)	-	-	60
v[c]	0.3	0.3	0.25

[a] Used for calculations of foundation stability
[b] Used for calculations of the soil - foundation – superstructure interaction
[c] estimation.

The determination of the mechanical parameters of Unit I is based on macroscopic observations indicating a medium-density granular material. It is assumed to correspond to an idealized SPT value of N* = 20, and, on the basis of this assumption, empirical correlations were employed (Bowles, 1996) to derive the values of the mechanical parameters. Unit II consists of conglomeratic limestone. For the determination of the physical properties of Unit II, six (6) tests were conducted to determine rock porosity and density (wet and dry). For the determination of the mechanical properties of Unit II, five (5) point load tests, three (3) uniaxial compressive strength tests (one of which included measurement of the modulus of elasticity and Poisson's ratio), and one (1) indirect tensile strength test (Brazilian test) were performed. Table 1 presents the characteristic values of the parameters defining the aforementioned geotechnical units (Egglezos, 2015).

Seismicity

According to the currently applicable Greek Seismic Code EAK 2000 (OASP/YPEHODE, 2000), the project area falls within Seismic Hazard Zone II, with a reference design ground acceleration of a_{gR} = 0.24g. The foundation subsoil in the project area is classified as Ground Type B (stiff clays of significant thickness and semi-rocky formations). Furthermore, for permanent new construction on a monument, such as the proposed new foundation system, an importance factor of Σ4 = 1.30 is specified. On the basis of the above, the design seismic action is defined as $a_g = a_{gR} \times \Sigma4 = 0.24 \times 1.30 = 0.312g$.
According to the provisions of Eurocode 8 (Eurocode 8, EN 1998-5), which was concurrently applicable, the soil forming the slope supported by the retaining structure is classified as Ground Type E (soil deposits of 5–20 m thickness with shear wave velocity in the range 180–360 m/s, underlain at great depth by a formation with shear wave velocity of approximately 800 m/s). For this ground type, a soil factor of S = 1.40 (Type 1 spectrum) and S = 1.60 (Type 2 spectrum) is specified. EC8 provides for a topographic amplification effect due to the slope morphology, taken as ST = 1.20 at the crest of the slope. Finally, for permanent new construction on a monument, such as the examined new foundation system, an importance factor of γ_I = 1.40 was adopted. Based on the above, the design seismic action (for a Type 1 spectrum) is determined as follows: $a_g = a_{gR} \times S \times ST \times \gamma_I = 0.24 \times 1.40 \times 1.20 \times 1.40 = 0.565g$.

Actions

Owing to the relatively small height of the slope, its stability may be assessed using a pseudostatic seismic analysis. In this context, both EAK 2003 and Eurocode 8 (unfavourable case) prescribe the following seismic actions: a horizontal action a_h = 0.50 ag and a vertical action a_v = ±0.25 ag. For the slope stability analyses, it was decided to adopt the prescriptions of Eurocode 8, as these more adequately represent the seismic response of the soil to the seismic excitation of the geological substratum and are, moreover, on the conservative side; accordingly, a_h = 0.283g and a_v = 0.141g have been used. The seismic actions applied on both shallow and deep foundations have been derived from the corresponding structural analysis (Toumbakari, 2012). In addition to carrying the loads of the superstructure, the foundation piles also function as a quasi-stabilizing diaphragm for the fill material, which is in an active geostatic state, resulting in the transfer of earth pressures to the foundation piles under both static and seismic conditions. For the required calculations of the micropile foundation system, a behaviour factor r = 1.0 was adopted (flexible wall founded on rock, EC8 Part 5, Table 7.1), together with the seismic acceleration used for the slope stability assessment.

GEOTECHNICAL ANALYSIS OF A SHALLOW FOUNDATION

Bearing capacity calculations against soil failure were carried out for a rectangular shallow foundation on the western and southern sides, following Hansen's method (Brinch Hansen, 1970), with an overall factor of safety of FS = 3.0 for static conditions and FS = 1.3 for seismic conditions. The following geometric characteristics (B × L = width × length) were adopted: for the western side, B × L = 0.70 m × 4.00 m, and for the southern side, B × L = 0.70 m × 4.50 m. In addition, the foundation depth D was examined parametrically for depths of 0.0 m, 0.50 m, and 1.0 m below ground surface under static conditions, while for seismic conditions the most unfavourable foundation depth of D = 0.50 m was adopted.

With regard to the underlying soil, it was assumed that the foundation response is governed solely by the soil fill layers. The seismic actions affecting the foundation were derived from the structural analysis study and have been increased by 50% with respect to the results of the seismic analyses in order to account for possible effects related to modelling assumptions. The calculations yielded (a) the bearing capacity of the foundation under static loading with a centrally applied vertical load, and (b) the verification of the foundation under appropriate combinations of seismic loading. The results of the bearing capacity calculations demonstrate that the examined foundation is adequate under both static and seismic conditions. They are summarized in Table 2.

Settlement calculations were carried out based on the Schleicher equation (Schleicher, 1926) for a foundation resting on a soil layer of limited thickness overlying a rock substratum of "infinite" thickness. For the settlement analysis, both (a) geostatic at-rest conditions (K_0) and active earth pressure conditions (K_a) were considered, taking into account the confinement of the fill by the walls of the N.E. Tower. In the case of active earth pressure conditions—which more realistically simulate the in-situ conditions—the calculated settlement is increased by approximately 10%. In total, four stratigraphic cases per side were examined, corresponding to fill thicknesses of 2 m, 3 m, 4 m, and 5 m over the rock substratum (eight calculations in total). A vertical, centrally applied pressure of 200 kPa was assumed on the foundations, corresponding to loading from the self-weight of the walls of the N.E. Tower (assuming a wall height of at least 5 m). Selected results, including the mean foundation settlement δ_v under the prevailing active geostatic conditions, are presented in Table 2.

GEOTECHNICAL ANALYSIS OF A DEEP FOUNDATION

Description of the deep foundation system

The calculation of the ultimate and allowable vertical load of an individual micropile for the deep foundation system, under both static and seismic conditions, was performed for two typical micropile diameters (Φ30 and Φ40), based on the following assumptions: micropile diameter D = 0.30 m and D = 0.40 m; effective pile length defined as the portion of the pile extending below the pile cap, with the following parametric cases examined: (i) a pile passing through the maximum thickness of fill (5 m) with an embedment of 3 m into the rock mass, resulting in a total pile length of L = 8 m; (ii) a pile passing through a fill thickness of 3.5 m with an embedment of 4.5 m into the rock mass, again with a total length of L = 8 m; and (iii) a pile passing through the minimum fill thickness (2 m) with an embedment of 6 m into the rock mass, with a total length of L = 8 m.

Table 2. *Parameters of the geotechnical units*

Foundation location	Dimensions [m]	Foundation depth [m]	Loading		Settlement [mm]			soil index [MN/m³]
			Static conditions[1]	Seismic conditions[2]	fill thickness	at center	at mid-length	
South	4,5 X 0,7	0	138	1,28[3] –	2m	7,29	5,95	31,5
		0.5	314	4,31	3m	8,17	6,67	28,1
		1	497		4m	8,62	7,04	26,6
					5m	8,76	7,16	26,2
West	4,0 X 0.7	0	137	1,87 –	2m	7,27	5,93	31,6
		0.5	315	5,19	3m	8,12	6,63	28,2
		1	500		4m	8,55	6,98	26,8
					5m	8,69	7,10	26,4

[1] σ_{all} [kPa] for FS=3
[2] FS check (FS >3; (DIN1054, 1976)) for 11 loading combinations
[3] acceptable at the limit

For the calculation of the ultimate and allowable vertical load under seismic conditions, no reduction in bearing capacity was considered, since the soil layers in contact with the micropiles are not classified as seismically susceptible. The calculation of the ultimate unit shaft resistance (skin fricrion) τ_{mf} and the ultimate tip resistance (end bearing pressure) σ_s was carried out in accordance with DIN 4014 (DIN 4014, 1990), while all calculations were based on the geotechnical model described in the preceding sections. Factors of safety equal to FS = 2.0 for static conditions and FS = 1.75 for seismic conditions were adopted, the latter reflecting the alternating compressive–tensile behaviour of the micropiles under seismic loading.

Pile bearing capacity and soil modulus (vertical subgrade reaction modulus)

First, the ultimate and allowable vertical and horizontal loads of individual foundation piles were calculated. Subsequently, the bearing capacity of pile groups was assessed. In the examined arrangement of micropiles, the center-to-center spacing between adjacent piles is greater than or equal to 3D (where D is the pile diameter). Therefore, no reduction in vertical load-bearing capacity due to group effects is expected. The arrangement of piles in a line, whether transverse or parallel to the direction of soil-induced forces from the slope, affects (according to relevant finite element analyses and scale experiments; cf. Poulos & Davis, 1980) the ultimate lateral capacity of each pile in the line ($H_{u,g}$). The reduction in performance is determined using a reduction factor, η_g, in relation to the single-pile performance, where η_g is a function of the normalized center-to-center spacing s/D of consecutive piles. The vertical soil modulus k_v, used for elastic static foundation calculations, was calculated directly from the formula $k_v=q/s$ (kN/m³),:where q = contact pressure on the pile (kPa) corresponding to the allowable vertical load and s = corresponding settlement of the pile. For selecting the horizontal soil modulus in Unit I – Soil Backfill for geostatic calculations of deep pile foundations, the granular nature of the backfill and a medium relative density (65%) are considered. Based on these criteria, the horizontal soil modulus is taken according to the Reese-Matlock correlations (Matlock & Reese, 1960) for unsaturated granular materials: $k_h=6zD$ (MN/m³), where: z(m) = depth from the surface (m) and D (m) = pile diameter. For Unit II – Rock Foundation, the horizontal modulus can be estimated using: $k_h=E/D$, where E is the Young's modulus of the rock mass (E = 3600 MPa).

Evaluation of the rock slope stability

The stability of the rock slope in the area of the northeast tower was evaluated through analyses for different loading scenarios. The geometry considered corresponds to the most unfavorable cross-section in the west–east direction, as provided by the Directorate for Restoration of Ancient Monuments. The section includes the northern wall and is illustrated in Figure 5 (left). The analysis focuses on the eastern slope. Three analysis cases were examined: a) static conditions, in which a conservatively increased vertical load was applied to also account for temporary construction equipment loads; b) seismic loading with horizontal seismic acceleration a_h = 0.283 g and vertical acceleration a_v = 0.141 g, as already discussed; c) intense rainfall conditions. The effect of the superstructure was also considered, introducing an additional horizontal stress on the soil due to the tower during an earthquake, calculated as q = 0.283 × 325 = 92 kPa. Stability checks indicate high safety factors for all examined cases.

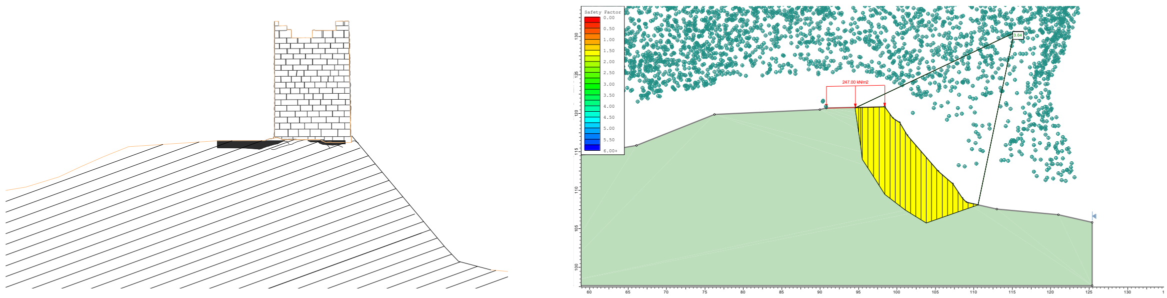

Figure 5. *Section for slope check (left).Eastern slope, stability check without earthquake: FS = 3.29 (right)*

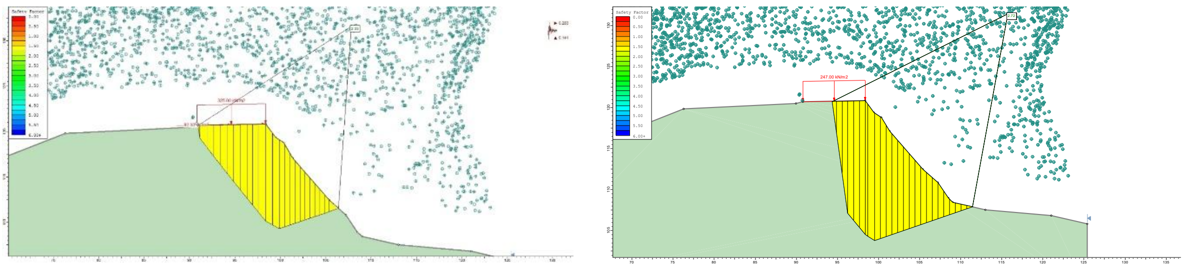

Figure 6. *Eastern slope, stability check with earthquake: FS = 2.33 (left). Eastern slope, stability check without earthquake, considering water pressures on discontinuities: FS = 2.51 (right)*

Results for static loading, seismic loading, and intense rainfall are presented in Figures 5 (right) and 6 respectively. Overall, the analyses show that the slope provides an adequate safety factor, at least twice the minimum required for all scenarios. Consequently, slope stabilization measures are not required, except in the case of standing boulders on the slope, which are not discussed here.

Analysis of the NE Tower–deep foundation–slope system interaction

This section summarizes the geostatic analyses carried out to assess the response of the system comprising the NE Tower, its foundation, and the underlying rocky slope under applied loading conditions. The analyses included two-dimensional finite- and discrete-element simulations in the West–East direction along the southern wall of the NE Tower. The objective of the analyses was the qualitative and quantitative comparative evaluation of the two proposed foundation systems, namely shallow foundation and deep foundation using micropiles connected by a pile cap.

All geomaterials and structural materials were modelled as elastoplastic and were assumed to obey the Mohr–Coulomb failure criterion, with their properties given in Table 1. The analyses were performed in successive stages in an effort to represent as realistically as possible the mechanical history of the monument, including the development of geostatic stresses in the rocky slope, construction of the fill and the external wall, completion of the superstructure, demolition of the superstructure, construction of the foundation, reinstallation of the superstructure, and finally the application of seismic loading based on the forces derived from the structural analysis of the superstructure. The applied loads considered in the analyses comprise self-weight (geometry and unit weight of geomaterials), seismic ground acceleration according to EC8 (a_h = 0.283), and seismic actions transmitted from the superstructure, namely bending moment M = 870 kNm, shear force V = 1350 kN, and horizontal force H = 320 kN. These actions were based on the most unfavourable internal force results obtained from seismic load combination analyses and were increased by 50% for safety reasons. The numerical analyses were carried out using the PHASE2.8 software developed by RocScience Inc. (Rocscience, 2011). The results of the two-dimensional analyses include, among others, the permanent horizontal displacement of both the shallow foundation and the pile cap due to seismic loading, the corresponding settlements, and the estimated internal forces developed in the piles of the deep foundation system during the seismic event. The computed results indicate that the horizontal displacement at the level of the pile cap of the deep foundation is approximately 4.3 cm, whereas the corresponding displacement at the level of the shallow foundation reaches about 19 cm; the settlement at the pile cap level of the deep foundation ranges between 0.55 and 6.74 mm, while the settlement at the level of the shallow foundation ranges between 0.12 and 10.7 cm, indicating significant rotation of the shallow foundation. The permanent seismic displacements for both foundation alternatives are presented schematically in Figures 7-10.

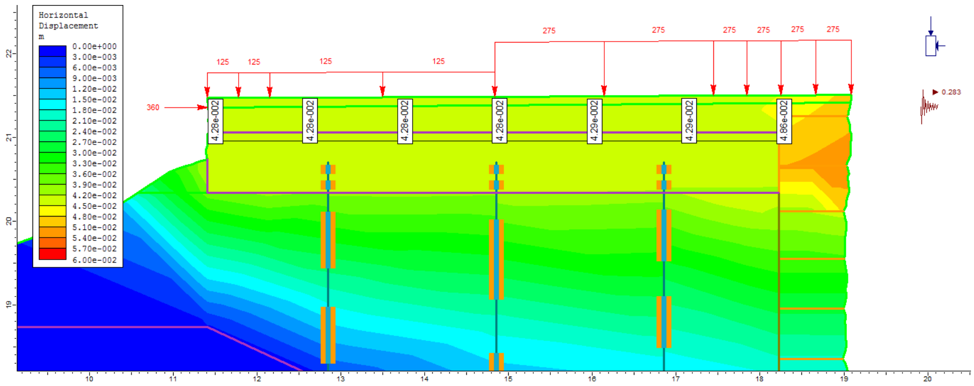

Figure 7. *Horizontal displacement at the foundation level due to earthquake (piles)*

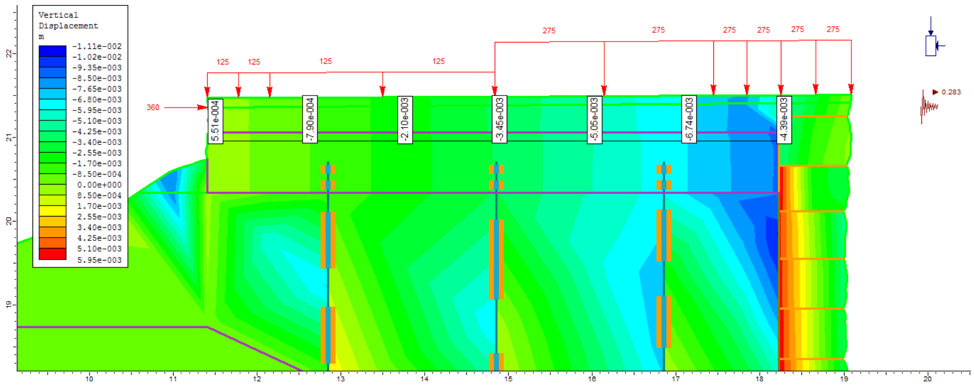

Figure 8. *Settlement at the foundation level due to earthquake (piles)*

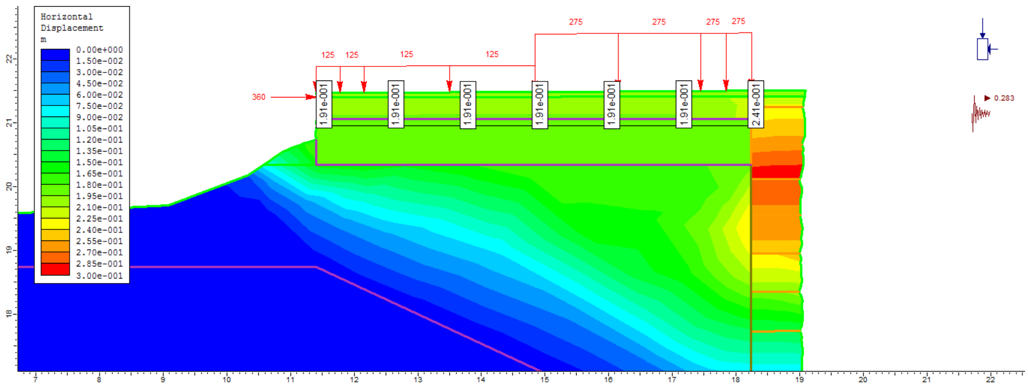

Figure 9. *Horizontal displacement at the surface foundation level due to earthquake*

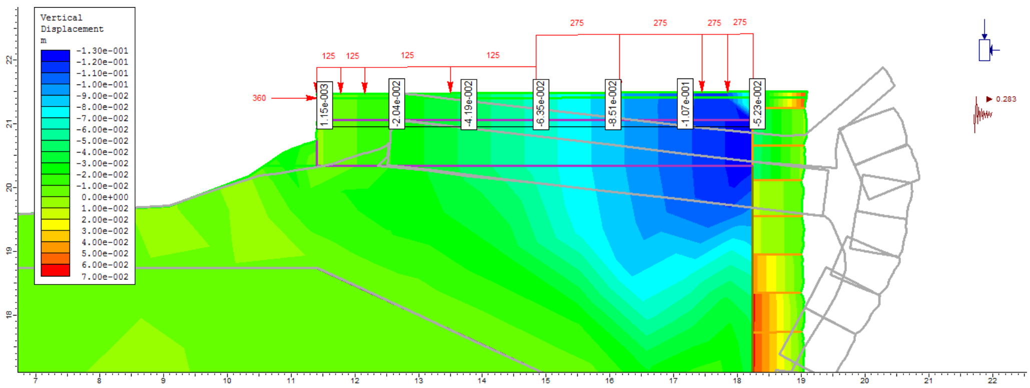

Figure 10. *Settlement at the surface foundation level due to earthquake*

CONCLUSIONS

The geotechnical analysis provided important quantitative evidence in support of the deep foundation concept promoted in the framework of the structural restoration / anastylosis study. The main conclusions are summarized as follows:

1. The shallow foundation, provided that appropriate footing width and embedment depth are adopted, may in principle safely carry the loads of the superstructure if additional measures are taken.
2. The shallow foundation is founded on heterogeneous fill of variable thickness, which may lead to potential rotation and differential settlement of the foundation; addressing this issue would require the placement of a remedial layer of sufficient thickness beneath the foundation to ensure uniform bearing conditions, resulting in extensive earthworks and the destruction of the ancient fill, and consequently of evidence of historical construction techniques.
3. Moreover, on the southern side the shallow foundation would temporarily rest on inclined ground due to the existence of voids created by the absence of the filling of the adjacent curtainwall, which may induce rotation and settlement of the foundation towards the south.
4. The fills in the area of the NE Tower are in an active earth pressure state due to their confinement by the northern and eastern walls of the tower, and any potential outward movement of these walls would result in corresponding displacement of both the fill and the foundation, since a shallow foundation cannot contribute to improving the stability of the confined fill.
5. The deep foundation system employing micropiles can safely carry the loads of the superstructure, without the occurrence of rotation or differential settlement of the foundation.
6. The deep foundation provides uniform bearing conditions, as the piles act as end-bearing piles founded on rock.
7. No additional earthworks are required beyond those necessary for the construction of the pile cap, since improvement of the underlying soil is not needed, thereby limiting disturbance and destruction of the ancient fill.
8. In addition to their primary foundation function, the group of piles also acts as a stabilizing diaphragm contributing to the stability of the fill; for this reason, earth pressures acting on the piles was taken into account in the static design of this foundation system.

The study was approved by the Central Archaeological Council and subsequently implemented (Figure 11). The anastylosis of the tower (Figure 12) was completed in 2016. From the outset of the project, particular emphasis was placed on the significance of the internal fill, which constitutes valuable evidence of ancient construction technology. Its preservation, to the greatest extent possible, was adopted as a key criterion guiding the design of the proposed interventions, with the geotechnical study proving to be a critical tool in achieving this objective. The internal fill yielded important archaeological and technical insights, shedding light on construction methods, material selection, and building practices employed during the original erection of the tower. Its conservation therefore contributes not only to the structural integrity of the monument, but also to the documentation and broader understanding of ancient engineering knowledge.

Figure 11. *The deep foundation under construction (view from the east) (left). The pile cap of the eastern elevation before the anastylosis and, in the background the western one (right).*

Figure 12. *The restored south and western elevations from the interior of the acropolis (left).The restored eastern and northern elevations (right)*

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