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Investigation, analysis and rehabilitation of the failures of St Dimitrios Bastion in the old city of Chania, Crete

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ABSTRACT

Saint Dimitrios Bastion is a landmark of Chania, Crete. It forms part of the Venetian fortifications of Chania, constructed between 1538 and the end of the 16th century. These fortifications followed the new Venetian defensive system, comprising curtain walls (cortine), bastions (baluardi) and elevated secondary defensive structures constructed above the bastions, known as cavalieri. From a technical perspective, the Saint Dimitrios Cavaliere is an artificial hill enclosed at its base by a circular masonry retaining wall. The wall rises approximately 7.5 m above the level of the bastion and retains sloping backfill extending upwards to a plateau located a further 6–7 m above the wall crest. The circular wall has a diameter of approximately 70 m, while the elevated plateau that accommodated the cannon positions has a diameter of approximately 40 m. The wall experienced repeated failures from as early as approximately 1600, necessitating the construction of a sloping berm at its toe to prevent collapse. This supporting berm remained in place until the early 20th century. Its subsequent excavation, undertaken to expose the masonry wall around the base of the Cavaliere, was followed by numerous failures of both the wall and the sloping backfill behind it. The most recent failures occurred at three locations in 2017 and 2019, following periods of intense rainfall. This paper presents the results of the geotechnical investigation undertaken to identify the soil strata beneath the wall foundations and within the retained backfill. It also describes the geotechnical analyses performed to investigate the failure mechanisms and evaluates the engineering solutions considered for the rehabilitation works, including the selected solution and its detailed design.

Keywords: Venetian fortifications, rainfall, slope failures, monuments in modern cities.

INTRODUCTION

Venice ruled the island of Crete from 1212 to 1669. During the period of Venetian rule, an extensive building programme was undertaken, focusing primarily on fortifications and public and administrative buildings. The Venetian fortifications of Chania, in western Crete, were constructed between 1538 and the end of the 16th century. They followed the innovative Venetian fortification system of the period, comprising curtain walls (cortine), bastions (baluardi) and elevated secondary defensive structures built above the bastions, known as cavalieri. The Saint Dimitrios Cavaliere formed part of these fortifications and was located at their south-western corner. It survives to the present day and constitutes a landmark of the old city of Chania. Locally, it is known as Trouli (Greek: «Τρουλί»).

From a technical perspective, the Saint Dimitrios Cavaliere, also known as the Lando Cavaliere, is an artificial hill enclosed at its base by a circular stone masonry wall. The wall rises approximately 7.5 m above the level of the bastion and retains sloping backfill extending upwards to a plateau located a further 6 m above the wall crest. The circular wall has a diameter of approximately 70 m, while the elevated plateau that accommodated the cannon positions has a diameter of approximately 40 m.

As documented in a surviving report written in 1602 by Benetto Moro, the Venetian Governor-General of Crete at the time (Spanakis, 1958), the wall had experienced repeated failures since approximately 1600, and possibly earlier. These failures necessitated the construction of a sloping berm at the toe of the wall to provide additional support and prevent its collapse. This berm probably remained in place until the 20th century (Figs. 1–3). Subsequent excavations undertaken to expose the masonry wall around the base of the Cavaliere were followed by numerous failures of both the wall and the sloping backfill behind it, the latest of which being the failures at three locations in 2017 and 2019, following severe rainfall incidents. It is also noted that during the siege of the City of Chania by the Ottomans in 1645, the fortifications were breached at the southwestern corner causing damages to the bastion and the cavaliere (including those caused by undermining), damages that were later rehabilitated during the Ottoman rule of Crete.

Figure 1. *Satellite image of the old city of Chania, Crete (Google Earth) with the location of Chania indicated by the red arrow in the map of Crete (bottom right). Close in on the area of the St Dimitrios Bastion indicated by the rectangle in the dashed yellow line.*

Figure 2. *Close in on the area of the St Dimitrios Bastion indicated by the rectangle in the dashed yellow line in Fig. 1. The bastion itself, the cavaliere and the locations of the 2017 & 2019 failures are indicated.*

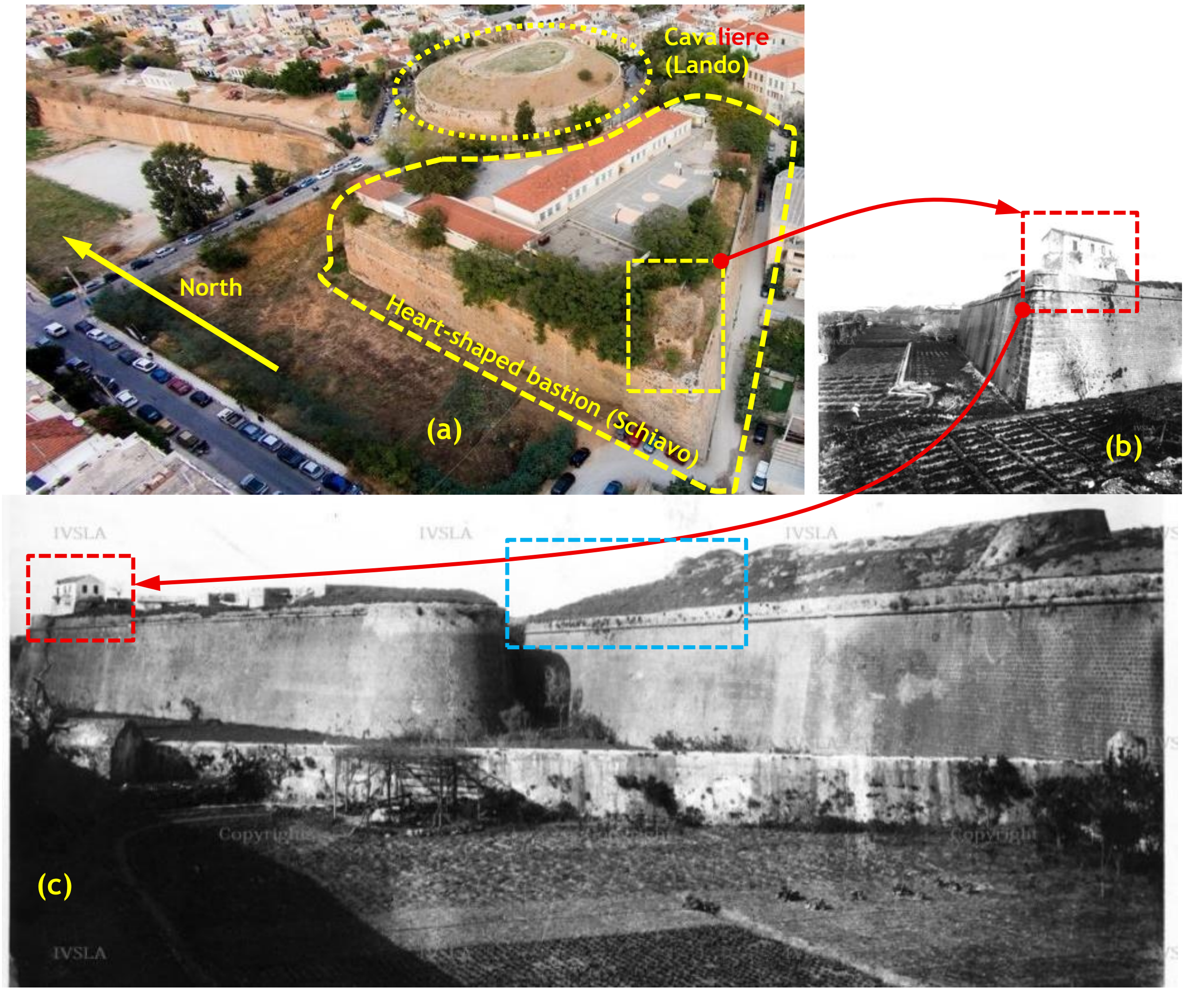


Figure 3. *a) Aerial view of the southwest part of the Venetian fortifications in the old city of Chania (before the failures of the cavaliere walls) with yellow rectangle indicating the remains of a building at the tip of the heart-shaped bastion, b) historic photo of the early 1910's showing in the red rectangle the same building indicated in Fig. 3a (taken from south-west of the bastion tip), & c) historic photo of the early 1910's showing in the red rectangle the same building indicated in Fig. 3a (taken from south-east of the bastion) and the toe berm of the cavaliere wall in the light blue rectangle.*

BRIEF HISTORY OF THE ST DIMITRIOS BASTION AND THE FAILURES OF ITS WALLS

Investigation in the archives of the Ephorate of Chania and the literature after the 2017 and 2019 failures revealed the aforementioned toe berm constructed by Venetians to support the wall and failures that had taken place after it but before the 2017 and 2019 ones. A slope failure occurred in the 1980's at the exact location of the eastern failure with reconstruction of the cavaliere wall after it (exact date not known). Also, at the exact location of the northern failure, another failure of practically the same size occurred on 10/8/1992 with reconstruction of the cavaliere wall after it. These previous failures indicate a wall and a slope behind it practically at limit equilibrium, easily failing with a small trigger, the usual culprit being excess pore water pressures due to rainfall.

THE FAILURES OF FEBRUARY 2017 AND FEBRUARY 2019

The failure at the south-eastern side was the first of the recent failures to occur during the night of 11/2/2017. It led to a major collapse of the toe wall of the cavaliere, closure of the street beyond the toe (Dounopapa st.) and destruction of several cars parked at the road side (Fig. 4a). Three days later, during the night of the 14/2/2017 the failure at the south-western side occurred, leading again to a major collapse of the toe wall of the cavaliere (Fig. 4b), this time without any other destruction as Dounopapa st. had been closed for traffic and pedestrians after the failure on 11/2/2017. Rainfall measured in the centre of Chania for the 10th and 11th of February amounted to 90mm and cumulative rainfall from 1/12/2016 until 11/2/2017 amounted to 390mm. Both these failures were cleared of fallen debris and steel buttresses were designed and constructed by the Ephorate of Chania to ensure the protection of the rest of the cavaliere wall outside the boundaries of these two failures (Fig. 4a & 4b). Following these failures, on 28/2/2019 the failure at the north-western side occurred leading again to a major collapse of the toe wall of the cavaliere and closure of the street beyond the toe (Meletiou Piga st.). Rainfall measured in the centre of Chania for the 24th to 26th of February amounted to 160mm and cumulative rainfall from 1/12/2018 until 27/2/2019 amounted to 670mm (the 2018-2019 winter was one of the rainiest winters in Greece, and especially western Crete, over the past decades with catastrophic consequences in western Crete). Again, steel buttresses were designed and constructed as immediate measures to ensure the protection of the rest of the cavaliere wall outside the boundaries of this new failure. These were complemented by sub-horizontal drainage holes through the intact wall to relieve any excess pore water pressures that could develop as the culprit for the failures seemed to be increased pore water pressures following heavy rainfall. Following all these, extensive geotechnical investigation was carried out both for the materials used in the cavaliere and the ground below it and alternative solutions for reconstruction of the areas of the failure and strengthening of the rest of the cavaliere walls were investigated until the final solution was decided and designed.

Figure 4. *View from the level of Dounopapa st in November 2017 a) of the eastern failure and b) of the western failure after removal of soil and wall debris and construction of the steel buttresses, and c) view of the northern failure in March 2019 before the removal of the debris. Small excavations on the slopes in Fig. 4a and 4b mark the locations of samples taken using an excavator from the various layers of the cavaliere materials.*

GEOTECHNICAL INVESTIGATION

The geotechnical investigations undertaken following the 2017 and 2019 failures comprised four boreholes and nine lightweight dynamic cone penetrometer tests conducted in accordance with ASTM D6951. The Cavaliere has an inclined access ramp extending from its toe on the south-eastern side to its summit on the north-western side. The ramp was narrow and highly uneven, while its entrance at the toe was almost completely obstructed by the base of the eastern steel buttress constructed after the 2017 failure. Consequently, heavy drilling equipment could not reach the top of the Cavaliere. The boreholes were therefore drilled around the perimeter of the Cavaliere at toe level. Boreholes GE-1 to GE-4, ranging from 10 to 20 m in depth, investigated the ground conditions beneath its foundations and within the body of the bastion. The materials forming the Cavaliere itself were investigated using lightweight dynamic cone penetrometer tests, supplemented by samples collected from the exposed faces of the eastern and western failures after removal of the debris. The lightweight dynamic cone penetrometer tests were performed using a manually operated Kessler K100 device equipped with additional rods, enabling penetration to depths of up to 9 m. Personnel carried the equipment to the summit of the Cavaliere on foot. The versatility of this testing method makes it particularly suitable for geotechnical investigations at archaeological sites, where restricted space and difficult access frequently preclude the use of conventional drilling equipment (Bardanis & Kokoviadis, 2026). At the Saint Dimitrios Cavaliere, the penetrometer reached a maximum depth of 8.85 m, which is unusually deep for this method and is seldom achievable because of the penetration resistance of the soil layers encountered. The locations of all boreholes and lightweight DCP tests are shown in Fig. 5.

Dynamic cone penetrometer tests provide information primarily on the resistance of the soil to cone penetration. This resistance may be correlated with geotechnical parameters such as the California Bearing Ratio (CBR) or unconfined compressive strength. However, the test results alone cannot identify the nature or composition of the soil. This limitation may be addressed by conducting selected tests close to boreholes, as in the case of DCP test P-1 in Fig. 5, or by combining the penetration results with classification tests performed on samples from soil layers known to have been penetrated. The latter approach commonly includes sampling the surface layer at each test location, allowing the near-surface penetration response to be related to the properties of the corresponding soil. By combining the DCP results with the classification test results on samples excavated from the various material layers exposed on the faces of the eastern and western failures (Figs. 4a and 4b), geotechnical sections were developed, including the representative section shown in Fig. 6. The alignment of this section in plan is shown in Fig. 5. This interpretation was possible because observations of all exposed failure surfaces indicated that the material layers forming the Cavaliere were either approximately horizontal or only slightly inclined. Their inclinations were readily determined from elevations obtained through surveying.

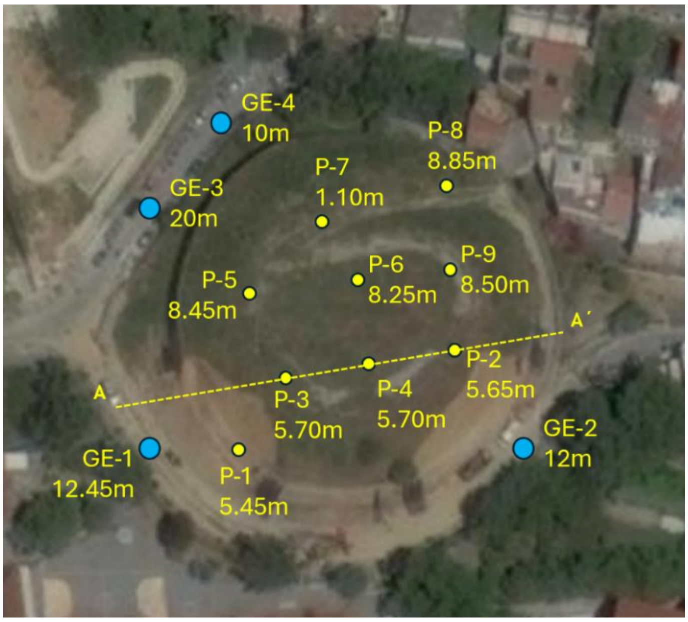


Figure 5. *Boreholes (blue circles, denoted GE-n) and lightweight dynamic penetrometer tests (smaller yellow circles, denoted P-n) performed as part of the 2017 and 2019 geotechnical investigation campaigns.*

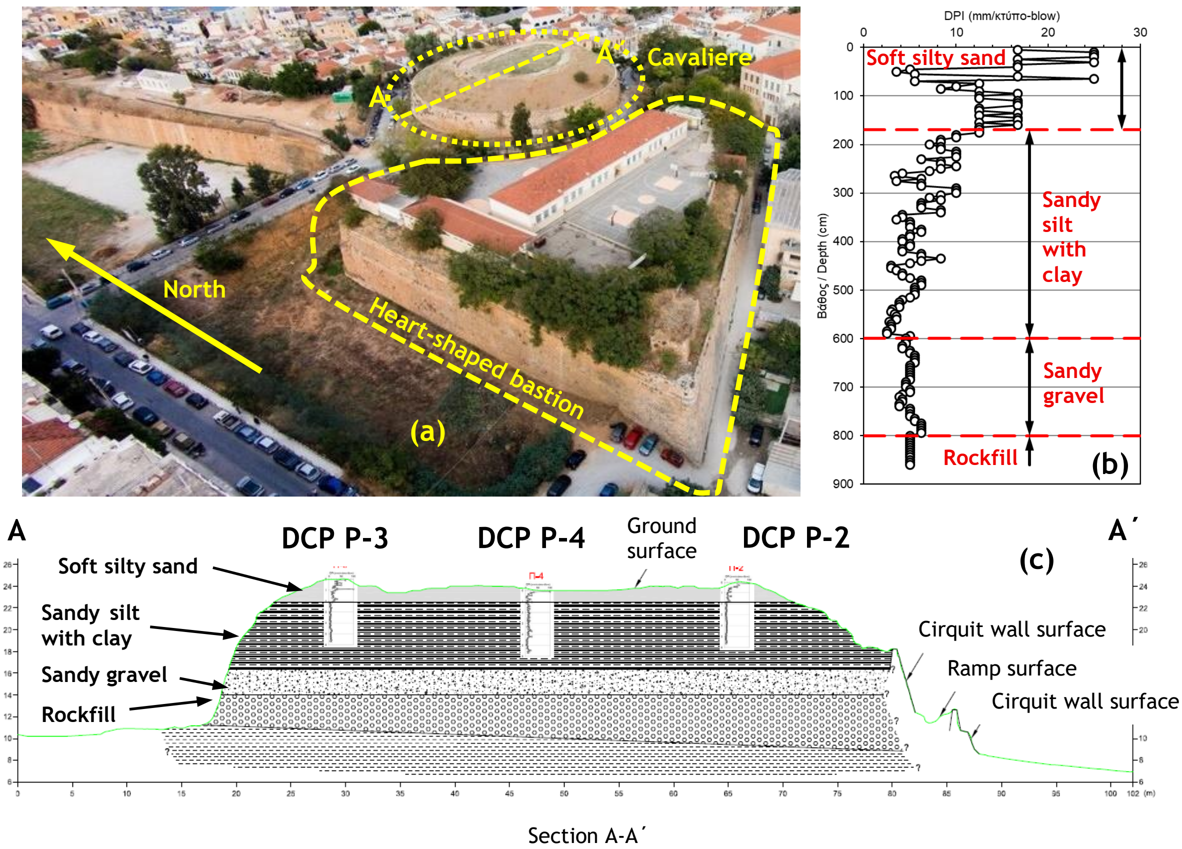


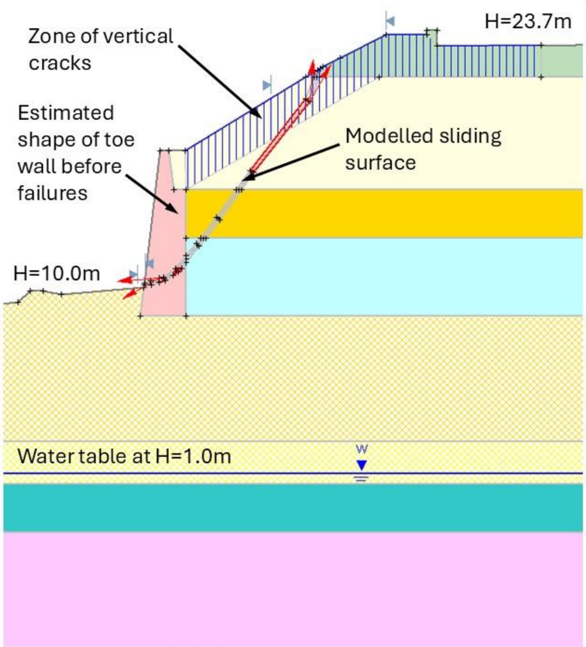
Figure 6. *a) Aerial view of the southwest part of the Venetian fortifications in the old city of Chania in Crete, Greece (before the failures), b) DPI with depth of DCP test P-9 performed at the crest of the St Dimitrios Cavaliere, & c) cross-section A-A´ across the St Dimitrios Cavaliere (indicated in Fig. 5a).*

Based on these observations and the results of the geotechnical investigation, comprising boreholes, DCP tests and surface samples, the geotechnical model shown in Fig. 7 was adopted for the back analyses and the design of the reconstruction measures. Visual observations indicated that the sandy silt with clay forming the main upper part of the Cavaliere contained a network of vertical cracks extending to a depth of approximately 2 m. These cracks are also shown in Fig. 7, together with the estimated geometry of the toe wall before failure and the modelled sliding surface.

The principal observation regarding the toe wall itself was that its width was considerably smaller than would be expected in relation to both its own height and the height of the sloping ground retained behind it. Figure 8a shows the western section of the wall that remained after removal of the debris from the western failure. The wall is approximately 1.5 m wide at its base and approximately 0.8 m wide at the crest of its main body. Above this level, it continues as a much thinner, parapet-like extension, which also appears to retain sloping ground. A wall of these dimensions would clearly have had only a limited capacity to withstand the earth pressures acting upon it. Furthermore, although the stones used to construct the wall appeared to be reasonably competent, consisting of marly limestone and limestone with unconfined compressive strengths ranging from 8 to 47 MPa, the lime-based mortar exhibited poor mechanical properties. In some locations, the wall appeared to have been constructed using soil-based mortar composed of silty clay (Fig. 8b). Based on these observations, the toe wall was represented in the back analyses of the failures as a Mohr–Coulomb material with relatively low strength parameters.

BACK ANALYSES OF THE FAILURES

Back analyses were performed by limit equilibrium analysis methods for slope stability using SLIDE software (by ROCSCIENCE). A series of analyses was performed using the modified Janbu and Bishop methods of analysis. The presence of water pressures was modelled by using the pore pressure coefficient, r_u, which was set 0 for the sandy gravel and the rockfill at the base of the cavaliere and various values between 0 and 0.5 for the upper layers of the sandy silt with clay and the soft silty sand.



Layer	USCS	γ (kN/m³)	c´ (kPa)	φ´(°)	E (MPa)
Soft silty sand	SC, SM	18-19	0-5	24-28	3-5
Sandy silt with clay	CL	18-20	10-20	20-24	6-8
Sandy gravel	GP, GM, GC	18-20	10-15	30-35	10-15
Rockfill	GP, GW	17-19	0-10	40-45	40-50
Silty sands to sandy silts	SM-SC, CL	18-20	10-15	30-34	15-20
Clay	CL	17-19	20-25	20-22	10-15
Silty Sand	SM, SC	18-20	10-15	30-33	15-20
Sandy silt	SM, ML	18-20	10-15	28-30	15-20

Figure 7. *Geotechnical model as introduced in the slope stability analyses with the features modelled and the geotechnical parameters used (section shown is a typical section at the western failure).*

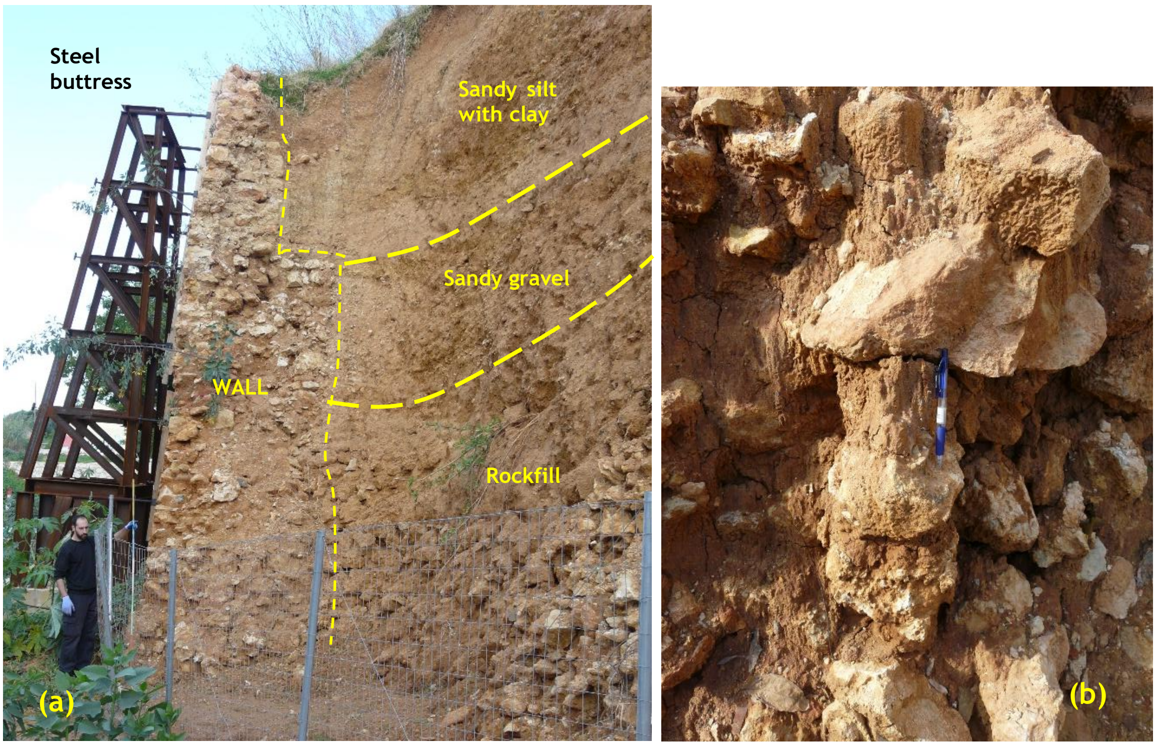


Figure 8. *a) Western section of the failed toe wall at the western failure after removal of the debris (human for scale). Boundary of the wall noted along with boundaries between soil layers in the cavaliere. b) Close-up photograph of the western section of the failed toe wall at the eastern failure after removal of the debris sowing extensive use of thick soil-based mortar with the soil being of silty to clayey nature (pen for scale).*

The shear strength values introduced were the minimum ones of the range shown in Fig. 7. Parameter analyses with r_u, indicated that even without water slope stability is already marginal close to 1.1 to 1.15, and reaches slightly below 1.0 as r_u becomes 0.45-0.5, a very large value indeed but rational for the magnitude of rainfall recorded before the failures and the presence of the cracks. Given the observations and limited laboratory data for the materials of the wall, it was modelled as a Mohr-Coulomb material with c=50 kPa and $\varphi=30^o$. Parametric analyses with values in that range indicated minimum effect on overall stability in comparison to the properties of the sand silt with clay layer or the value of r_u.

ALTERNATIVE OPTIONS FOR STABILISATION AND RECONSTRUCTION AND THE SELECTED SOLUTION

Given the results of the back analyses several alternatives for reconstruction of the failed areas and stabilisation of the remaining walls were examined.

The first alternative involved soil nailing. In the failed areas, where reconstruction would be required, the upper silty to clayey materials were considered to have poor geotechnical properties. Consequently, soil nails of relatively large diameter and length would have been required. Moreover, the number and size of the soil nails would have made this method intrusive and irreversible. In the areas where the existing wall remained intact, self-drilling anchors were considered, primarily as a means of increasing the apparent cohesion of the lower, coarser-grained materials.

The second alternative involved constructing a pile wall within the failed areas. Its exposed face would have been clad with stone masonry to blend with the surviving sections of the existing toe wall. The pile wall would have been constructed from a temporary berm located 3–4 m above the toe of the wall, thereby allowing the piles to develop maximum resistance at the base of the Cavaliere. Above the pile heads, the structure would have continued upwards for a further 3–4 m to the level of the existing toe-wall crest, forming a very stiff pile-cap structure. From an engineering perspective, this alternative offered the highest achievable level of stability. However, it was also by far the most intrusive and irreversible method of stabilisation and was ultimately considered disproportionate to the requirements and heritage value of the monument.

The third and final alternative involved constructing a stronger masonry gravity wall of larger dimensions than the existing wall. The proposed wall would have a width of 3.0 m at its base and 1.5 m at its crest, with a stepped profile along its rear face. Despite their disadvantages, the first two alternatives would not have required excavation at the toe of the failed slopes following removal of the debris and displaced material. By contrast, the gravity-wall alternative required excavation to a depth of approximately 3–4 m at the toe. Such excavation would temporarily reduce the overall stability of the Cavaliere. Furthermore, it would have to be undertaken within the rockfill and sandy gravel, the two materials exhibiting the lowest cohesion. Following removal of the failed material, the temporary stability of these materials was considered to depend partly on the additional apparent strength provided by partial saturation. To address this risk, the construction of the larger masonry gravity wall was combined with improvement of the rockfill and sandy gravel. A temporary berm was designed at a level 3–4 m above the toe of the Cavaliere, with side ramps providing access for construction machinery. From the top of this berm, cement grout would be injected through a dense pattern of vertical and subvertical boreholes into the sandy gravel and rockfill. Sub horizontal drainage boreholes would also be installed within the upper, finer-grained layers of the Cavaliere. The cement grouting was intended to increase the cohesion and strength of the sandy gravel and rockfill. Following completion of the grouting works, the temporary berm would be removed progressively along the perimeter of the failed zones. This operation would be undertaken in stages, together with excavation at the toe of the existing slope and construction of the new stone masonry wall. Once all sections of the wall had been constructed up to (H = 14) m, construction would proceed layer by layer along the full length of each failed zone until the wall reached the crest level of the existing structure. After completion of the wall, the space behind it would be backfilled with high-quality, free-draining granular material. This material, together with the specified relief holes through the new masonry wall, would prevent the development of excess pore-water pressures. This alternative was ultimately selected as the proposed solution for reconstructing the toe wall and rehabilitating the failed areas of the Cavaliere. The design also included specific details for connecting the new masonry wall to the existing structure. Special provisions were developed for drilling and grouting close to the boundaries of the failure zones, thereby maximising the extent of ground improvement and drainage while limiting disturbance to the surviving historic fabric.

For the sections of the toe wall that had not failed, the selected stabilisation solution combined drainage with ground improvement. Visual observations indicated that, although relief holes existed in the wall, many appeared to be clogged. A series of 15 m long sub horizontal drainage holes was therefore specified at 2 m intervals along the perimeter of the toe wall. In addition, four rows of 6 m long self-drilling anchors were specified at depths of 1.5, 3.0, 4.5 and 6.0 m below the crest of the existing wall. Special construction details provided for the careful removal of individual stones, installation of each anchor and formation of its head through the resulting opening, followed by reinstatement of the removed stone. Self-drilling anchors were selected because they allow progressive grouting along their length. The section shown in Fig. 10 illustrates the proposed solution, including the anticipated cement-grout bulbs intended to improve the properties of the sandy gravel and rockfill.

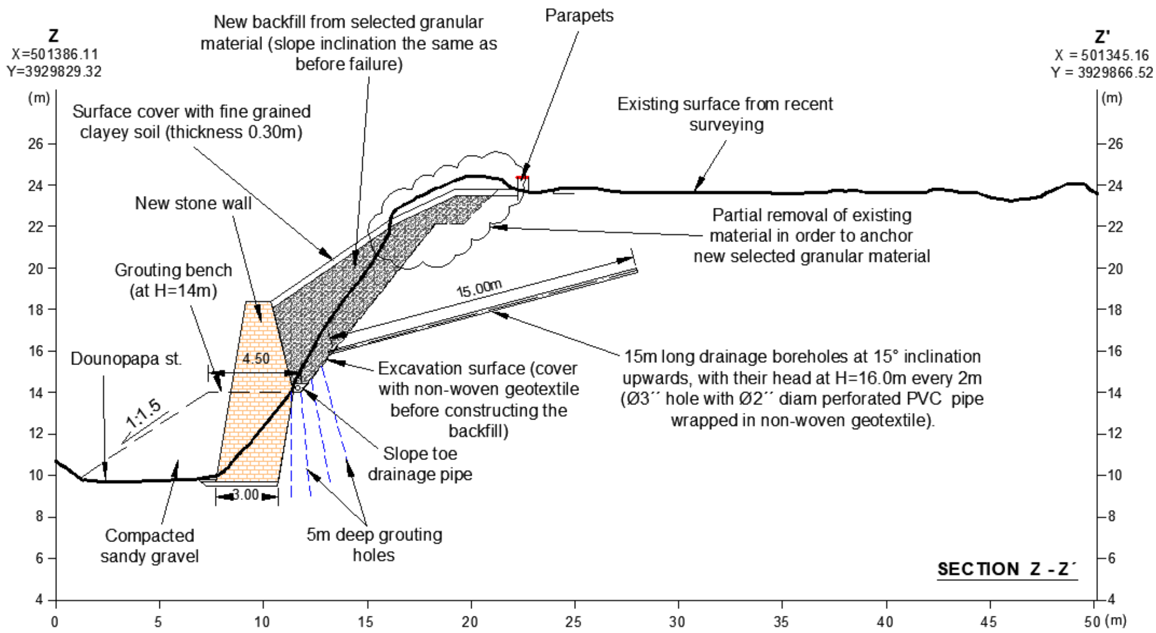


Figure 9. *Typical section of the selected solution for reconstruction across the western failure sowing all the major structures, properties' improvement and drainage measures.*

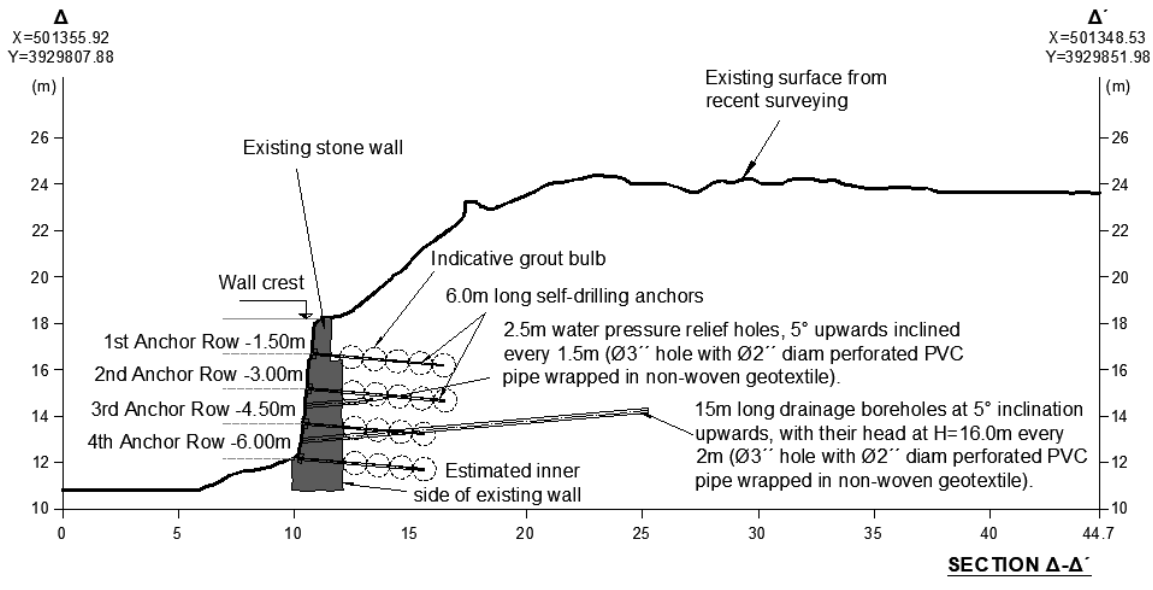


Figure 10. *Typical section of the selected solution or reconstruction showing properties' improvement and drainage measures.*

Finally, the designed solutions for the reconstruction of the failed areas and the stabilization of the rest of the toe wall are complemented by a monitoring system. It includes 10 bi-axial clinometers evenly distributed along the perimeter of the toe wall and installed at its crest and a pore water pressure monitoring station installed from the crest of the cavaliere including vibrating-wire piezometers for positive pore water pressures, and porous block and FDR sensors for soil suction and volumetric water content measurement at various depths.

CONCLUSIONS

The toe wall and retained slope of the Saint Dimitrios Cavaliere, in the old city of Chania, failed at three locations in 2017 and 2019 as a result of excess pore-water pressures generated by heavy rainfall. The subsequent geotechnical investigation comprised boreholes drilled at the toe of the Cavaliere, lightweight dynamic cone penetrometer tests conducted from its summit and sampling with excavators from the exposed failure surfaces after removal of the collapsed wall debris and landslide material. The approximately horizontal stratification of the materials used to construct the Cavaliere allowed the borehole data, DCP results and samples from the failure surfaces to be combined. This information provided a geotechnical model of the stratigraphy within the body of the Cavaliere and the underlying bastion. Using this model and the material properties derived from laboratory and field testing, the failures were back-analysed using limit-equilibrium slope-stability methods. The analyses indicated that, given the properties of the materials, the Cavaliere had already been close to limiting equilibrium following the removal of a historically documented toe berm. The Venetians had constructed this berm in the late 16th century, shortly after construction of the wall, to provide additional support. The development of excess pore-water pressures within the upper, finer-grained layers of the Cavaliere disturbed this marginal equilibrium and triggered the failures. Immediate measures were designed to secure the surviving sections of the toe wall. These included steel buttresses, sub horizontal drainage boreholes and removal of unstable debris. Several alternative solutions were subsequently examined for the permanent reconstruction of the failed zones. The selected solution comprised a large masonry gravity wall constructed with cement-based mortar at the base of the failed slopes. The wall was combined with drainage boreholes in the upper, lower-permeability layers and cement grouting in the lower layers to improve their shear-strength properties. In the areas where the toe wall remained standing, a combination of self-drilling anchors and drainage boreholes was designed to improve the stability of the Cavaliere. The self-drilling anchors would provide both reinforcement and a means of injecting cement grout into the surrounding materials. A monitoring system was also designed to complement the reconstruction and stabilisation measures. It includes monitoring of wall-crest movements using clinometers, together with measurements of pore-water pressure and water content within the central part of the Cavaliere.

ACKNOWLEDGEMENTS

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