# Laboratory and field investigation of the backfill behind the ancient walls of the Acropolis, Athens

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## ABSTRACT

Geotechnical investigation was performed in the backfill behind the ancient walls of the Acropolis in Athens. It consisted of two excavation pits that were excavated, DCP testing and local surface sampling. One of the pits was adjacent to the north facing wall and one adjacent to the south facing wall at the point separating two different periods of the construction of the walls. The pits were a combination of archaeological and geotechnical investigation. The paper presents the results of the geotechnical tests performed allowing classification of the backfill material and determination of their mechanical properties. This laboratory investigation was complemented by long-term field measurements of suction and volumetric water content from instruments installed in the walls of the excavation pit adjacent to the north facing wall.
*Keywords: Monitoring, soil suction, large-size direct shear device, isotropic compression.*

## INTRODUCTION

The Acropolis of Athens is one of the defining monuments of classical antiquity and stands out until today as the absolute landmark of the modern city of Athens, the capital of Greece. Archaeological and visitor interest focuses vastly on the monuments of Parthenon, Erechtheion and Propylae. Still it is the ancient walls surrounding the citadel that frame it as a landmark in the skyline of Athens until today (Fig. 1). These fortifications support the backfill forming the plateau defined by the perimeter of the walls and the limestone rock outcropping on the Acropolis hill. The depth of the backfill is offered by Kavvadias & Kawerau (1907) in their account of the excavation of the Acropolis between 1885 and 1890 and its findings. This paper presents the results of the geotechnical investigation and evaluation that was carried out in 2015 and part of the monitoring of pore-water pressure, soil suction, volumetric water content and horizontal total stress from the instruments installed in-situ in 2015 (Bardanis, 2015, 2016 & 2019).

This geotechnical investigation was based on two excavation pits, one adjacent to the North part of the wall, and one adjacent to the South part of the wall (Fig. 2) supported by local surface sampling at other locations. These excavation pits were a combination of archaeological and geotechnical investigation, as they were opened by archaeological investigation crews which as soon as they had finished their work at each depth, geotechnical logging and sampling would follow down to the full length of both excavation pits. The progress of excavation was supported by light Dynamic Cone Penetration Testing (as per ASTM D6951, Bardanis & Kokoviadis, 2026). Laboratory testing included soil mechanics classification and compaction tests, mechanical testing focusing on parameter testing to create an understanding of the expected behaviour of the fill under various conditions and testing in large-size devices given the coarse-grained nature of the material. This investigation was supported by field

***Figure 1.*** *Typical view of the north side of Acropolis as viewed by millions of Athenians and visitors of Athens. Erechtheion in the foreground, Parthenon behind it and Propylae on the right attract most of the interest, but it is the circuit walls (even from the north side as in the photograph -shorter than those on the south or western side) that frame the citadel and form the Athens skyline.*


***Figure 2.*** *Satellite image of the Acropolis of Athens (Google Earth) showing the location of the excavation pits and the major monuments.*

measurements of in-situ density and water content, light Dynamic Cone Penetration Testing, and the measurements from the field measurements instruments included in the excavation pit adjacent to the North part of the wall.

The backfill behind the walls is a material that has been practically ‘remoulded’ (in geotechnical terms) because of the great excavation between 1885 and 1890 (Kavvadias & Kawerau, 1907) that involved moving large volumes of original, historically placed backfill to excavate all the way to the bedrock on the hill. These excavations were combined with various important archaeological findings and as the excavation progressed with backfilling of the excavated back to its place on the hill. The same source reports the material of the backfill as man-made ground of fine-grained to sandy origin with large fraction of gravel, cobbles and fragments from building materials (porous stone of marly origin, marble, limestones and dolomites) which can be observed in the limited photographs of the time (Fig. 3). The same photographic material documents the ability of the excavation faces to stand for considerable height without failing (Fig. 3). Andronopoulos & Koukis (1976) report the backfill behind the walls as *recent* earthfill of fine-grained to sandy origin with gravel, *of low permeability*, with fragments of porous stone of marly and limestone origin towards its base. More recent observations (Egglezos, 2013) seem to verify previous reports, with the backfill material being considered as well compacted, clayey to sandy gravel (GC). All these observations are in agreement with the findings in the 2015 excavation pits (Fig. 4) and in terms of a geotechnical context seem to be summarised as follows:

- The presence of very coarse-grained material, apparently as residue of the building activities, is evident and should be expected to dictate a large value of the angle of shearing resistance.

- Fine-grained material is also evident, offering a more ‘closed’ fine-grained matrix between the coarsest material, decreasing the permeability of the whole mass and offering extra strength to it due to partial saturation at low values of water content.
- The ‘remoulding’ because of the great excavation of 1885-1890 most probably ‘disturbed’ (in a geotechnical context) whatever structure one could expect of a soil material brought in-situ at least 2.5 millenia before. Similarly, it probably ‘disturbed’ any reasonable layering of the backfill material that may have been in place before the great excavation due to the gradual increase of the backfill thickness during the major building programmes and destruction by invaders in antiquity. Given that the works of the great excavation were performed practically only manual labour, it should be expected that the backfill placed again after the great excavation to its place had most probably an initial low density and water content making it vulnerable to hydroconsolidation (‘collapse’) under the action of rainfall.

Given these observations, the geotechnical investigation of 2015 focused on identifying the effect of the presence of a large fraction of coarse-grained material, the behaviour of the backfill under various water contents and densities and the effect of partial saturation and especially its change to full saturation due to inundation.

***Figure 3.*** *Views from the 19th century great excavation demonstrating the macroscopic nature of the earthfill (left) and the big height of the free standing vertical faces of the excavated earthfill (right).*


***Figure 4.*** *Views from the 2015 pit adjacent to the North part of the circuit wall when the depth was 0.90m in April 2015 (left) and during backfilling of the pit in October 2015 with the wall of the pit being 2.00m high (right). Both views of pit walls shown are photographed from north looking south.*

## CLASSIFICATION AND FIELD TESTS

Several samples were taken to measure the grain-size distribution, Atterberg limits and other soil index properties. The results are summarised in Table 1. The average grain size distribution curve with the extents of its scatter are shown in Fig. 5. It is emphasised that this average shown is very close to an approximately 1000 kg sample taken in order to perform one-dimensional and direct shear tests in a large-size device (Fig. 6a) -for comparison normal-size samples for this kind of coarse-grained material. In order to avoid damaging any fragments of archaeological importance that could still be in this sample, the *whole* of this sample was sieved after it was left to dry in atmospheric conditions (drying in a geotechnical oven was avoided in case there could be any fragment that could be affected by the 110 $^{o}$C temperature in these ovens). Fragments that could possibly be of archaeological interest were indeed found (Fig. 6b) carefully stored and returned to the Acropolis Restoration Service. Most of the samples were classified according to the Unified Classification System as silty gravel with sand (GM) with only one sample classified as silty sand with gravel (SM) and only one as GC-GM (clayey to silty gravel with sand because of Atterberg limits values placing it in the ML-CL region of Casagrande's plasticity chart). These results are in general agreement with previous observations, with the exception that no significant clay fraction was found or Atterberg limit values measured (out of all the samples tested, only two showed plasticity with $w_L$ of 22% and 20% and $I_p$ of 5% and 6% respectively).

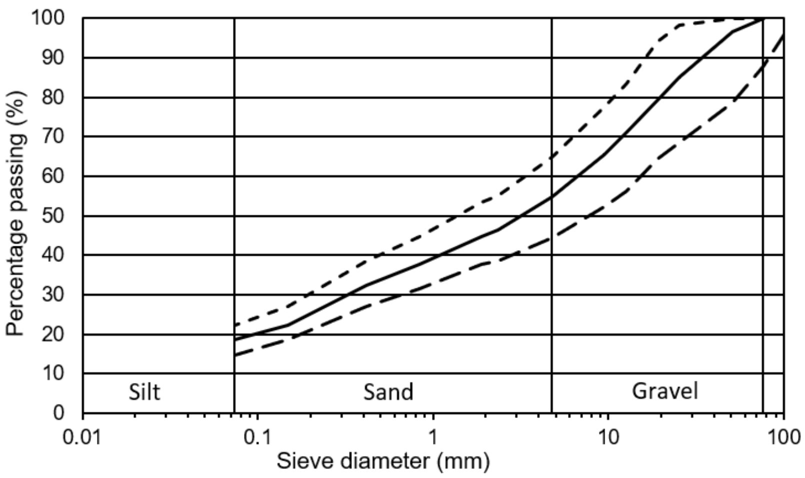


***Figure 5.*** *Range of grain size distribution for the backfill material (average in continuous line, upper limit in dotted line with small dash, lower limit in dotted line with big dash).*

***Figure 6.*** *Big bag of excavated material from the excavation pit adjacent to the North Wall (left) and findings during the sieving of all of the material in the big bag (right) consisting of marble, limestone, dolomite, marl fragments, ceramics (painted and non-painted) with traces of metallic objects and bones (in the small bowls at the top of the photograph).*

***Table 1.*** *Summary of soil mechanics classification and compaction test results from the 2015 Geotechnical Investigation campaign.*

| Property | Range of values and average |
|---|---|
| **Natural water content, $w_n$** | 4.5 to 12.0%, 8.2% on average |
| **Liquid limit, $w_L$** | Non plastic (for 2 samples only 20 and 22%) |
| **Plasticity index, $I_p$** | Non plastic (for 2 samples only 5 and 6%) |
| **In-situ unit weight (kN/m$^3$)** | 18.7 to 23.4, 20.5 on average |
| **Maximum dry unit weight from Proctor compaction tests with modified compaction energy on fraction passing #¾´´ (kN/m$^3$)** | 20.3 το 22.0, on average 20.8 (corresponding degrees of compaction of material in situ: 90 to 106%, 97% on average) |
| **Optimum water content from Proctor compaction tests with modified compaction energy on fraction passing #¾´´ (kN/m$^3$)** | 5.2 to 9.5%, 8.1% on average (with all specific samples where Proctor tests were performed found with a water content 1-0.5% dry of optimum) |
| **Specific gravity** | 2.69 to 2.71, 2.705 on average |
| **Gravel** | 35.1 to 55.5%, 45.3% on average |
| **Sand** | 29.5 to 42.5%, 36.2% on average |
| **Fines (Clay and Silt)** | 14.7 to 22.4%, 18.5% on average |
| **Classification** | All samples GM, only one SM and one GM-GC |

## MECHANICAL TESTING

Given the findings from the classification tests performed, it was decided to perform a series of tests on the fraction of the material passing #12.5mm and a series of tests on the fraction passing #75mm.

### *Tests on the fraction passing 12.5mm*

Quick unconfined compression tests and consolidated undrained triaxial compression tests were performed on 100mm diam by 200mm height samples compacted at various dry unit weights and water contents.

Quick unconfined compression tests performed on samples from various depths in the Northern excavation pit compacted at a combination of dry unit weight and water content as that measured in the field density tests yielded unconfined compression strength, $q_u$, values of 200 to 220 kPa with an average of 210 kPa. Attempting to identify the effect of increasing dry unit weight (as caused by gradual collapse of the material under the effect of precipitation), tests were performed on samples compacted at 8% water content with a varying dry unit weight from 15.6 to 18.6 kN/m$^3$, yielding $q_u$ values from 10 kPa for the lowest dry unit weight up to 280 kPa for the largest. Similarly, in an attempt to identify the effect of increasing water content (as caused by gradual collapse of the material under the effect of precipitation), tests were performed on samples compacted at a dry unit weight of 17.5 kN/m$^3$ with a water content from 6.5 to 11.5%, yielding $q_u$ values from 95 kPa for the lowest water content down to 30 kPa for the largest. These test results can be considered by no means representative but indicate that the material with the density and water content it has in-situ seems to have medium unconfined compression strength, which however should generally be considered very much lower at the beginning of the placement of the material after the great excavation of 1885 to 1890 (with the material gaining strength after that due to density increase because of collapse) and may decrease from these values if water content increases considerably.

Consolidated undrained triaxial compression tests were also performed. Two different unit weights were decided for the tests performed; 17.5 kN/m$^3$, corresponding to what was expected to be more representative of the initial dry unit weight of the backfill after placed again in its original position after the great excavation, and 18.5 kN/m$^3$, as a more characteristic value (still conservative) of the dry unit weight of the backfill nowadays. Isotropic consolidation with unloading-reloading cycles for both dry unit weight values tested yielded a value of isotropic compression modulus in the range of 20 MPa along the unloading-reloading path (Fig. 7a) with a $\lambda$ of 0.047 for 17.5 kN/m$^3$ of initial dry unit weight and a $\lambda$ of 0.065 for 18.5 kN/m$^3$ of initial dry unit weight, with $\kappa$ measured along the unloading-reloading cycle for both dry unit weights 0.01 (Fig. 7b). The shearing stage of the consolidated undrained triaxial compression tests (characteristic stress paths for two $\sigma_3'$ values for both initial dry unit weight values of the tests shown in Fig. 8) yielded typical behaviour corresponding to normally consolidated materials for

17.5 kN/m³ initial dry unit weight and overconsolidated materials for 18.5 kN/m³ initial dry unit weight. The Critical State Line seems to have been identified with an *M* values of approximately 1.5.

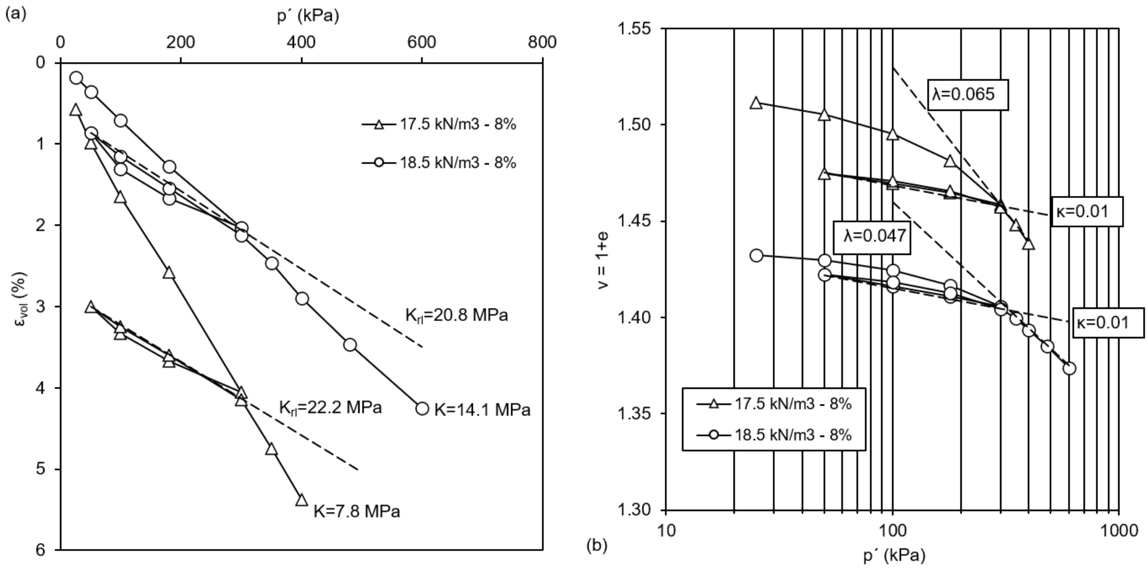


***Figure 7.*** *Results of isotropic compression tests on 100mm diam and 200mm samples on the fraction of the material passing #12.5mm in terms of a) $\varepsilon_{vol}$ – p´, and b) v – p´.*

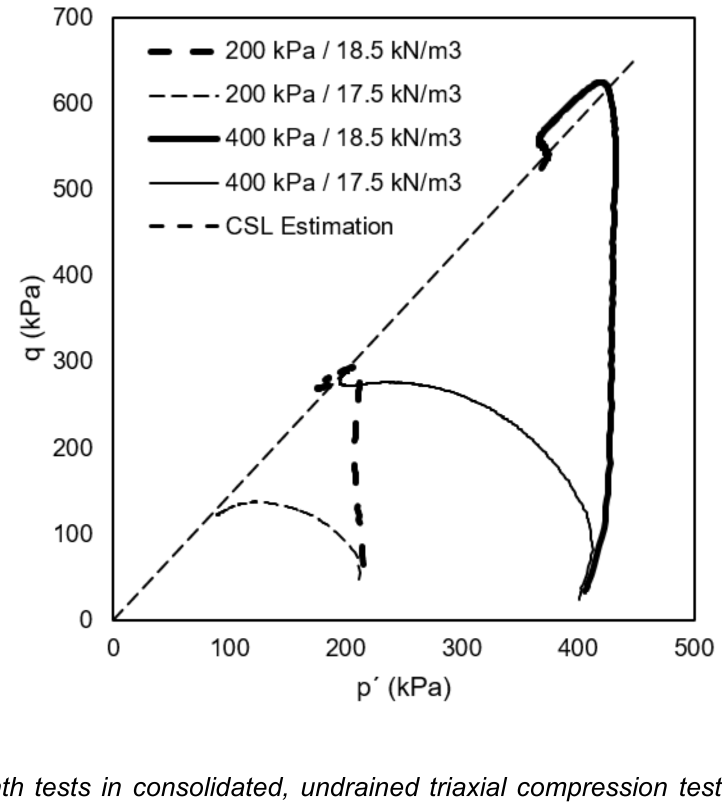


***Figure 8.*** *Stress path tests in consolidated, undrained triaxial compression tests on samples with initial dry unit weight of 17.5 and 18.5 kN/m³.*

## *Tests on the fraction passing 75mm*

Tests on the fraction of the material passing #12.5mm were significant in order to obtain an understanding of the general mechanical behaviour of the material. Still in order to try and understand its behaviour in the presence of its coarser fractions, a large series of tests was performed on the fraction passing #75mm. To test this material the EDAFOS S.A. large-size direct shear device was used (Bardanis et al., 2013, Bardanis, 2014, Fig. 9). It is a displacement-controlled device capable of applying as low shear strain rates as 0.1%/min with a 45cm diameter circular shaped cell, generally compliant to ASTM D3080. Vertical load is applied through an oil jack, electronically controlled to counter for pressure changes due to contraction or dilation during the shearing stage and compression during the loading stage. The maximum load applied electronically yields 1500 kPa vertical stress and 3000 kPa when the load is applied by means of manual control of the oil jack. The horizontal load is measured by a load cell measuring a force corresponding to a shear stress of 2000 kPa. This combination of electronically controlled vertical load application and horizontal load measurement allows for measuring values of the angle of shearing resistance considerably larger than 45$^o$ even up to the vertical stress of 1500 kPa.

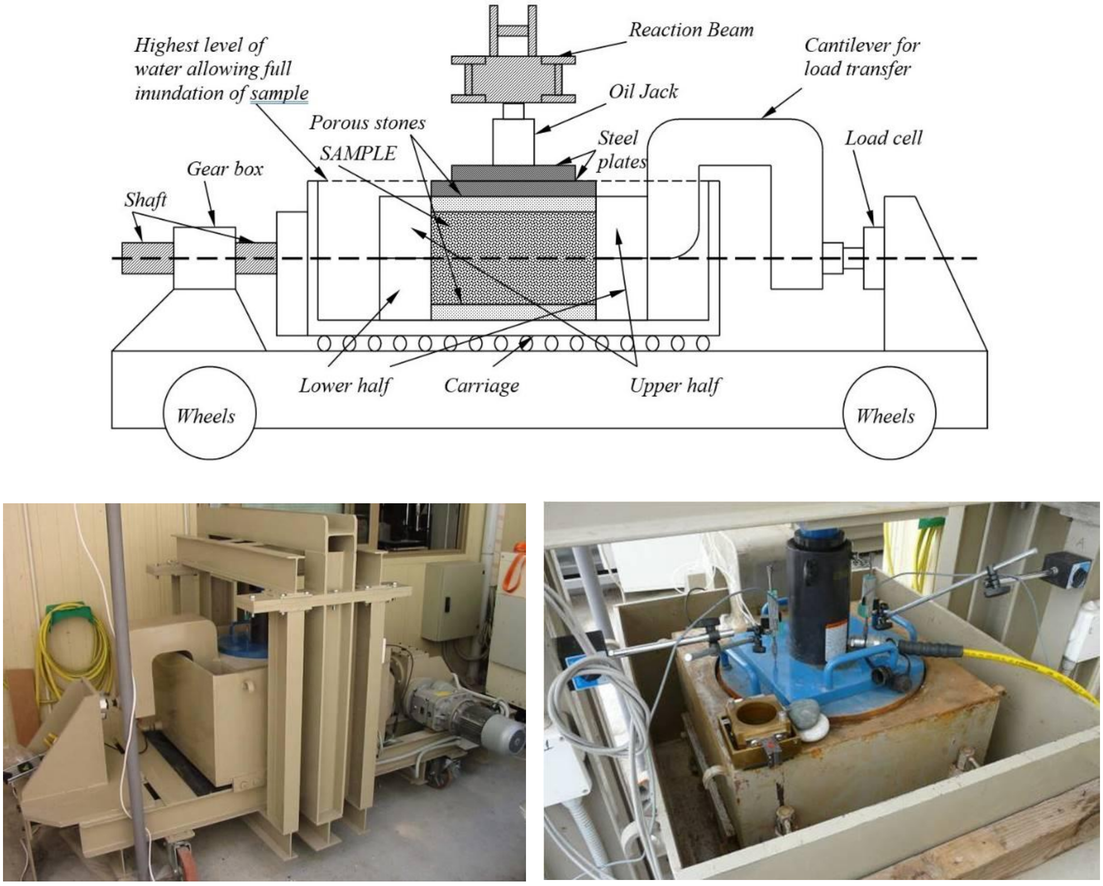


***Figure 9.*** *A schematic of the EDAFOS S.A. large-size direct shear device (top), and actual views of the large direct shear device: overall view (left), view of direct shear cell (right) -for comparison two cobbles of the largest size that may be included in the sample in this device and the cell of a conventional small-size direct shear device.*

The large size of this device allows samples containing the fraction of material even to that passing #75mm to be tested and in combination with the circular shape of the cell, it allows for measuring the minimum and maximum unit weight. Minimum dry unit weight was found 13.1 kN/m$^3$ and maximum dry unit weight 20.5-21.0 kN/m$^3$. Following this, consolidated, direct shear tests were performed on samples with the minimum dry unit weight, the maximum dry unit weight and an intermediate value of 19 kN/m$^3$. Initial water content in all case was selected to be 8% -approx. the average of the water content measured in situ. Fig. 10 summarises the results from the peak values from these tests and the failure envelope of the shear strength after large strain. The latter corresponds to c´=0 and φ´=46$^o$, with values of shear strength parameters for tests with high initial dry unit weights corresponding to φ´=42-44$^o$ and cohesion intercept values of tens of kPa. On top of these tests, using the circular section of the device cell, one-dimensional compression tests were performed on samples with the aforementioned dry unit weight values and total earth pressure cells installed inside them to measure the coefficient of earth pressure at rest, Ko, along normal compression branches of the tests and unloading-reloading cycles at various stress levels to relate Ko with the overconsolidation ratio of the material. One-dimensional compression curves are shown in Fig. 11.

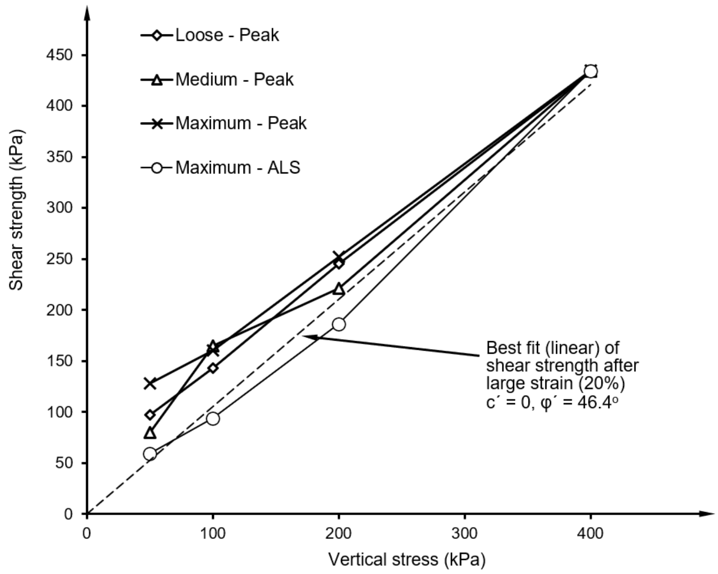


***Figure 10.*** *Direct shear test results on all samples with all dry unit weights.*

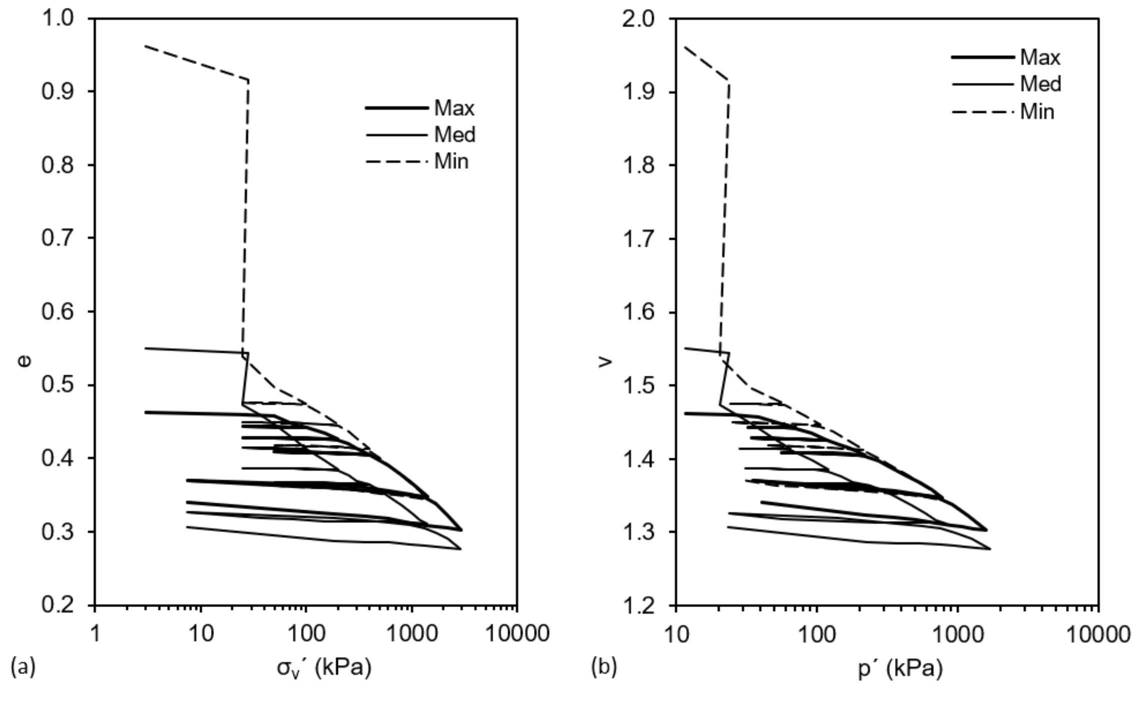


***Figure 11.*** *One-dimensional compression tests performed on samples with all dry unit weights in terms on void ratio e vs vertical stress (a) and specific volume v vs mean effective stress (b) -mean effective stress calculated from applied vertical stress and horizontal stress measured on total stress cell installed in the samples. Decreases of e or v under 25 kPa of vertical stress are due to collapse due to inundation under this stress that was introduced during the tests.*

# MONITORING

## *Instruments installed*

In the excavated pit adjacent to the North wall, two couples of suction (DECAGON MPS6) and volumetric water content sensors (DECAGON GS3) were installed at 0.80 and 2.00m depth, along with a vibrating-wire piezometer at 2.00m depth and a total earth pressure cell installed on the wall face at 2.60m (Fig. 12) and connected to automatic loggers for continuous measurements recording (Bardanis & Kokoviadis, 2023). Automatic logging started in February 2016 and continues to this day on the suction sensors. It is the longest-running suction measurement station for geotechnical engineering purposes in Greece and has measurements over more than 10 years despite gaps showcasing practically all possible problems associated with long-running monitoring schemes with continuous measurements:

- Electrical failures. Modern field sensors and loggers have come a long way and are generally fairly robust instruments, especially regarding their electrical parts. Still, electrical failures are and will be a possibility and this is what happened in one of the logger channels. Abundance of channels is something that should generally be incorporated in the design of monitoring schemes (although it increases the cost) as in this case that it allowed simply changing channel for the particular sensor on the channel that failed electrically.
- Contractual problems. A smooth contractual procedure ensuring continuous involvement of specialised personnel guarantees maintenance of stations (cleaning, changes of batteries etc). Gaps in contracts often leave stations without basic maintenance that may result in loss of data. On the other hand, very long-running monitoring schemes may outlast contracts initially scheduled for a limited time on the basis of anticipated data gathering duration or budget restrictions.
- Natural phenomena. The Acropolis station “survived” the April the 17th 2019 lightning strike north-east of the Erechtheion. Although not something common, especially in Athens, a lightning did actually strike and led to the electrical failure of the volumetric water content sensors, the vibrating-wire piezometer and the total earth pressure cell installed in the ground, despite the presence of lightning rods. Suction measuring sensors and the logger itself survived electrical loading of the ground during the strike and continue to take measurements to this day.

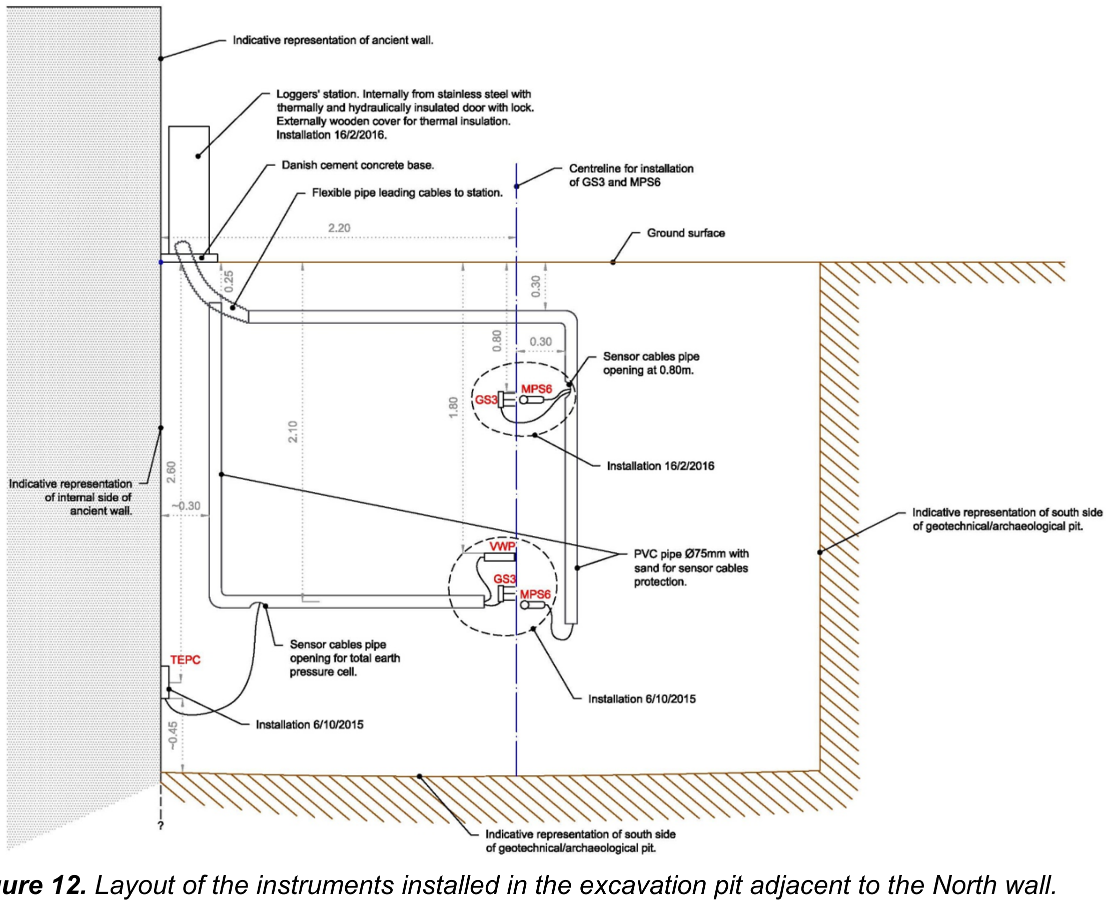


***Figure 12.*** *Layout of the instruments installed in the excavation pit adjacent to the North wall.*

### *Monitoring results*

These problems taken into account, the Acropolis station with its 10 year long-running measurements of suction offers perhaps the most important information from the suction measurement stations installed so far in Greece (Bardanis & Kokoviadis, 2023). First it is affected by the microclimate of Athens, which similarly affects practically half of the economic and engineering activities in Greece due to the size and population concentration of its capital, Athens. Second, it records suction evolution in a relatively free draining material, that is a material that normally would not be expected to be able to develop and maintain high values of suction. Still, as data show in Fig. 13a, suction can indeed be considerably high in this material and on top of that it can be maintained for a very long time. Suction remained very high at both depths during the wet seasons of 2016-2017 and 2017-2018 (400-450mm of cumulative rainfall with some very high intensity incidents) and it took the very rainy wet season of 2018-2019 (580mm of cumulative rainfall) for suction values to decrease to 30-40 kPa. Following that, for the wet seasons of 2020-2021 and 2022-2023 that full data for each period are available, again suction was not lost especially at the depth of 2m where practically negligible changes seem to have taken place. Corrected

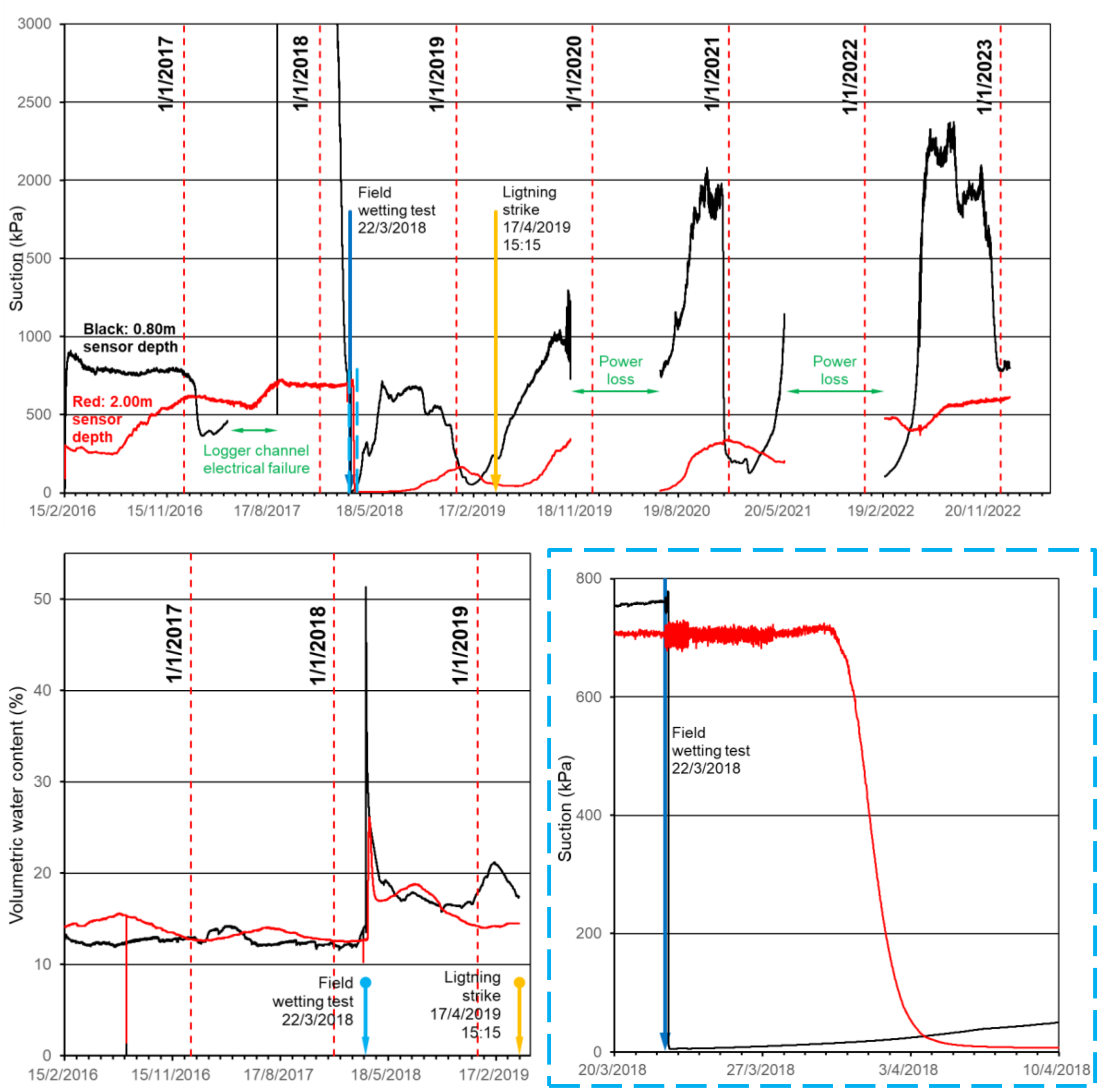


***Figure 13.*** *a) Suction evolution with time. Black solid lines correspond to 0.80m of depth and red solid lines to 2.00m of depth. Dashed light blue rectangle indicates detail in Fig. 13c. b) Volumetric water content evolution with time until the lightning strike of April the 17th 2019 when the sensors failed. c) Detail of suction evolution with time little before until 19 days after the wetting test of March the 22nd 2018.*

values of suction for temperature (Walthert & Schleppi, 2018) reached and exceeded 2000 kPa at 0.80m and 600 kPa at 2.0m. Volumetric water content followed a similar trend matching suction evolution (Fig. 13b). These values of suction are in general agreement with other measurements of suction in Greece (Bardanis & Kokoviadis, 2023, Belokas, et al., 2025) and Cyprus (Bardanis & Loukidis, 2020).

In this particular station, a wetting test was performed on the 22nd of March 2018 (Fig. 14). It allowed the study of the response of the sensors at the two depths (Fig. 13c) indicating a response within 3.5 hours for the depth of 0.80m (after providing a total of 738lts of water from the surface) but a response after 9 days for the depth of 2.0m (which itself lasted 3 days until suction dropped to zero). The total supply of water reached 2177 lts and was completed in the morning of the 23rd of March 2018. Recovery of "normal" field suction values took approximately 3 months for the depth of 0.80m and 9 months for the depth of 2.0m. The drying stage was used to estimate field evolution of permeability of the material with suction. The water front diameter at the ground surface was 75cm (Fig. 14).

***Figure 14.*** *The wetting test of March the 22nd 2018 at the Acropolis station.*

## CONCLUSIONS - PARAMETERS FOR THE BACKFILL

A geotechnical investigation in the backfill of the circuit walls of the Acropolis of Athens was performed in 2015. It included installation of field instruments, some of which keep taking measurements until today. Soil mechanics classification testing indicated that the backfill material is a silty gravel with sand (GM) with only one sample classified as silty sand with gravel (SM) and only one as GC-GM (clayey to silty gravel). These results are in general agreement with previous observations, with the exception that no significant clay fraction was found or Atterberg limit values measured (out of all the samples tested, only two showed plasticity with wL of 22% and 20% and Ip of 5% and 6% respectively).

Regarding mechanical properties, many of the tests performed were presented in this paper. Instead of conclusions the geotechnical parameters proposed after the evaluation of all tests is presented in Table 2.

***Table 2.*** *Geotechnical parameters proposed for the backfill material behind the circuit walls.*

| Property | Value | Observations |
|---|---|---|
| **$\gamma_d$ (kN/m$^3$)** | 18.5-19.5 | |
| **c´ (kPa)** | 5-20 | Peak strength values |
| **φ´ (°)** | 42-44 | |
| **c´ (kPa)** | 0 | Strength values after large strain |
| **φ´ (°)** | 42-43 | |
| ***κ*** | 0.001-0.002 | |
| ***λ*** | 0.04-0.05 | |
| **$C_r$** | 0.0025-0.005 | |
| **$C_c$** | 0.09-0.13 | |
| **Ko** | 0.31-0.32 | On normal compression branches |
| **$E_s$ (MPa)** | 15-25 | |
| **$E_s$ (MPa)** | 7-8 times the value during loading | On unloading/reloading branches |

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- “Maintenance, Measurements Recording, Training of Personnel and Presentation of Recorded Data in Printed and Electronic Format for the Geotechnical Instruments Station at the North Wall of the Acropolis of Athens”

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