https://doi.org/10.1051/e3sconf/202338212006

Large-Scale Plate Load Tests to Determine the Collapse Potential at Santorini 150 kV GIS Substation Site

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Abstract. Two large-scale plate load tests were carried out at the foundation level of the Santorini 150 kV GIS Substation, where the subsoil was identified as collapsible. The purpose of these load tests was to establish in-situ, the magnitude of the expected settlements due to the collapse of the subsoil structure after inundation, while applying a bearing pressure equal to the maximum expected according to the structural design. After completion of the tests, which are described in detail, the resulting settlements were found to be acceptable, so no soil improvement measures were required and it was concluded that with the design and execution of these innovative load tests, it was possible to measure under actual field conditions the magnitude of settlement due to collapse of the soil structure. This led to a more economical foundation solution, compared to the one that would have been adopted were only laboratory tests taken into account in the calculation of settlements.

1 Introduction

According to the executed geotechnical investigation at the Santorini 150 kV GIS Substation site ([1]) the subsoil consists of volcanic ash deposits in the form of lightly cemented silty sand of low unit weight and high porosity, which was identified as a collapsible soil (see figures 1 and 2). A soil that, although in a dry state exhibits satisfactory mechanical characteristics, it exhibits a sudden and large reduction of its volume when it gets wet, or even worse when it is saturated with water, due to loss of its apparent cohesion and collapse of its open structure ([2]).

Fig. 1. The subsoil as seen in an open cut excavation in the area of the project.

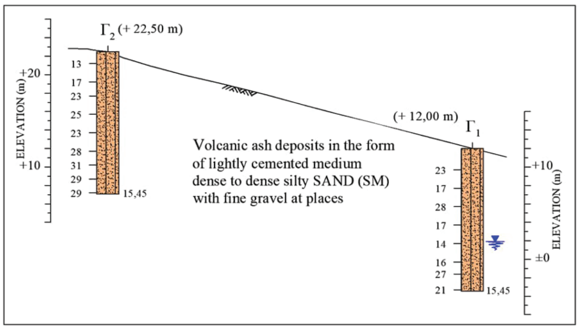

Fig. 2. Typical geotechnical section ([1]).

2 Soil conditions

According to the executed geotechnical investigation ([1]), the soil consists of lightly cemented medium dense to dense silty sand with fine gravel at places. The grain size distribution of the volcanic ash deposits is presented in figure 3, from which it is observed that there is little variability in terms of gradation. The unit weight of the volcanic ash deposits exhibited a significant variation, ranging between 14.0-21.0 kN/m^3 with an average value of 17.0 kN/m^3.

The ground water table was encountered at elevation +0.50 m in October 2020, which means that it slightly higher than the mean sea water level (±0.00 m).

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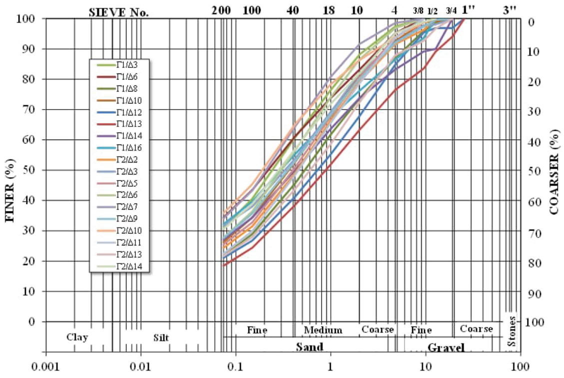

Fig. 3. Grain size distribution of the soil deposits.

3 Collapse potential estimation

Based on laboratory consolidation tests, where the procedure of ASTM D-5333 standard ([3]) was followed, the collapse potential of the soil was determined, expressed by the collapse index as follows:

$$I_c = s_c/h \quad (\%) \tag{1}$$

where:
s_c = settlement due to the collapse of the soil structure after wetting and
h = thickness of collapsible layer

Given that the collapse index depends on the magnitude of the applied pressure during saturation, multiple collapse potential tests were performed in the laboratory, in order to estimate the collapse potential for various applied vertical pressures. The results of these laboratory tests are summarized in the diagram below (figure 4). From this diagram it is observed that by increasing the applied ground pressure, the collapse index (I_c) increases as well.

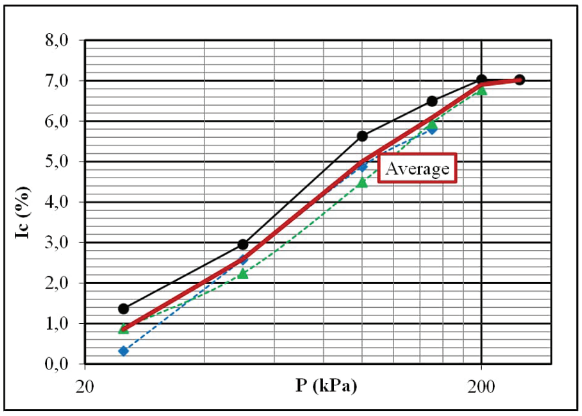

Fig. 4. Collapse index Ic vs. applied pressure P (from laboratory tests).

4 Problem description

Based on the average estimated collapse index and parametric analyses that were carried out, regarding the influence of soil unloading due to excavation and the depth of possible soil saturation under the foundations, the maximum acceptable allowable bearing pressure of the structures was determined (50 kPa), so that the expected total settlements after an accidental soil saturation would not exceed 100 mm. This settlement was considered as the maximum acceptable one, taking into account the type of the structures (mainly monolithic bases) and the stiffness of the building foundations (mat foundation combined with a basement).

However, this allowable bearing pressure could not be satisfied by the structural design that followed in a number of cases (GIS and SVC buildings as well as transformers, where the estimated mean applied pressure ranged between 65-75 kPa – [4]). In order to address this problem, soil improvement (such as soil replacement, dynamic compaction or rapid impact compaction) or deep foundation solutions (such as rigid inclusions or piled foundation) would have to be implemented which, inevitably, would seriously affect both the cost and the duration of the project.

5 The solution

In order to address the problem at hand, two innovative load tests were proposed and then performed at the foundation level of the GIS and SVC buildings, using a 2.00x2.00 m concrete slab ([5]). These tests were performed in two phases: ***Phase 1***: Gradual application of the maximum anticipated foundation pressure for each building (according to the structural design – [4]) at the subsoil's natural water content. ***Phase 2***: Saturation of the soil by inundation, while maintaining at the same time the applied maximum foundation pressure.

The purpose of these load tests was to measure the magnitude of settlements due to the collapse of the soil structure under actual field conditions (i.e., (a) without any disturbance of the soil structure as a result of sampling and laboratory preparation and (b) in terms of the soil tested volume). In case that the magnitude of the settlements, due to the collapse of the subsoil structure under saturated conditions and with an applied pressure equal to the maximum expected by the structural design, were acceptable, then no further soil improvement or deep foundation measures would have to be taken.

It should be noted here that the main objective of the load tests was to observe the behavior of this particular soil under saturated conditions, since it had been identified as collapsible, and not its behavior at its natural water content (for which there was no particular concern). Consequently, the 2nd phase of the tests (inundation phase) was the one of primary interest.

6 Description of the load tests

6.1 Preparation

A 4.00x4.00 m excavation was formed at the location of each load test, the bottom of which was at the foundation level of each structure (GIS and SVC buildings respectively). A non-woven separation geotextile, weighing 150 gr/m^2, was placed at the bottom of each excavation and then a layer of 20-40 mm gravel, 0.20 m thick, was placed on the geotextile. This layer facilitated the spreading of water under the loaded slab during the soil saturation phase. The geotextile was placed in order to separate the permeable gravel layer from the subsoil and to prevent sinking of the gravels into it, especially during the saturation phase. Finally, a square 2.00x2.00 m reinforced concrete slab was placed upon the gravel layer (see figure 5). This slab was designed in such a way that on one hand it would be fairly stiff, but on the other hand it would not be too heavy, so that the initially applied pressure, after its placement on the ground, would not exceed 5 kPa.

Fig. 5. Preparation for the load test.

Monitoring of settlements was carried out with 3 dial gauges of 0.01 mm precision, attached on two wooden beams 6.00 m long (see figures 6, 7 and 8). The choice of wooden beams was made with the purpose of minimizing temperature effects (low thermal expansion coefficient).

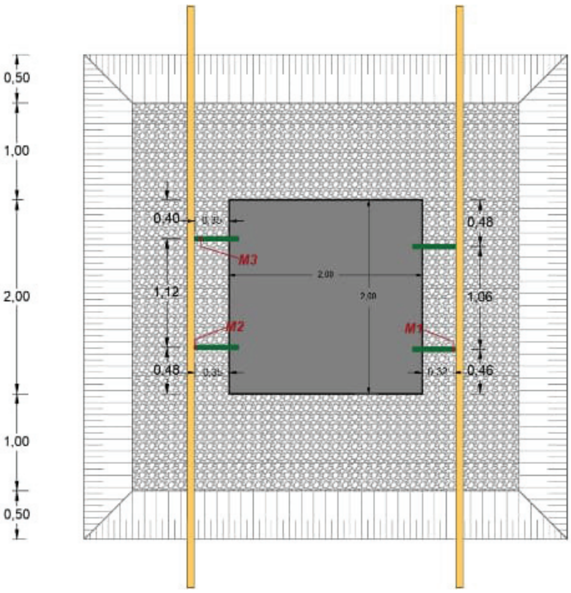

Fig. 6. Layout of the settlement monitoring arrangement.

Fig. 7. View of the settlement monitoring arrangement.

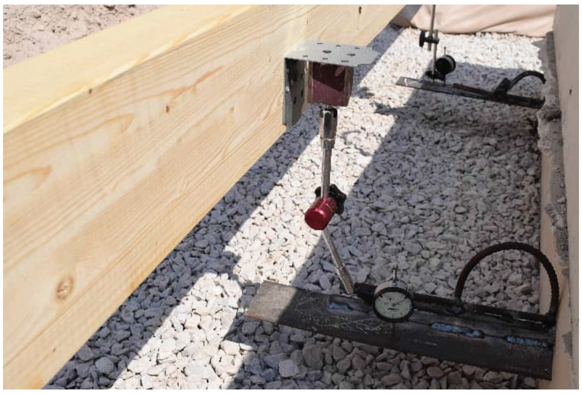

Fig. 8. Close up view of the settlement monitoring arrangement.

The test load was applied with 4 precast concrete blocks weighing 48 kN each και 4 steel counter weights of a construction crane weighing cumulatively 104.5 kN, in order to achieve the required test load (see figure 9).

During the 2nd phase of the test (saturation of subsoil), the gravel layer was inundated and was kept covered by water at all times with a continuous supply of water from water tank trucks (see figure 10).

Fig. 9. View of the elements used for the application of the test load.

Fig. 10. Inundation of the gravel layer under constant vertical pressure.

The required amount of water throughout each test for maintaining the gravel layer inundated depended on the rate of water infiltration into the subsoil and it ranged between 25-36 m^3.

6.2 Test Procedure

The maximum applied load was such that the applied pressure would be at least equal to the corresponding maximum design foundation pressure of each structure. More specifically, the maximum applied pressure was 79 kPa (as compared to the maximum estimated by the structural design 75 kPa) at the GIS building location (test Δ1) and 72 kPa (as compared to the maximum estimated by the structural design 65 kPa) at the SVC building location (test Δ2).

During the 1st phase of the test (loading phase under natural water content) the load was applied in 5 stages. The load at each loading stage was maintained for 15'. The maximum applied load (5th stage) was maintained for 120' before commencement of the 2nd phase of the test (inundation). Dial gauge readings were taken every 5' throughout the 1st phase of the test.

Following completion of the 1st phase of the test, the gravel layer beneath the loaded slab was inundated while maintaining the maximum applied load. During this phase dial gauge readings were taken continuously until the rate of settlement dropped below <0.2 mm/hr and continued to drop, but no less than 24 hours.

7 Presentation and interpretation of the results

The results of the two load tests are presented graphically in figures 11 (1st phase) and 12 (2nd phase).

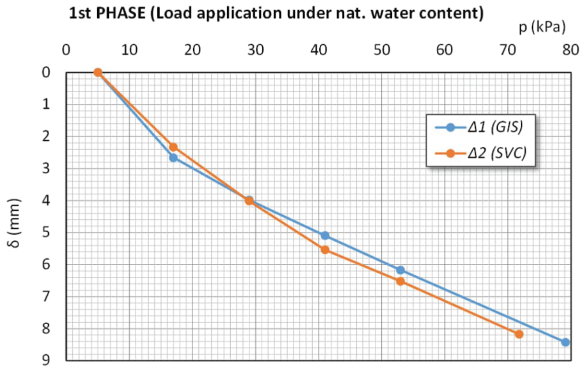

Fig. 11. Settlement vs. applied pressure under natural water content of the subsoil (1st phase of the tests).

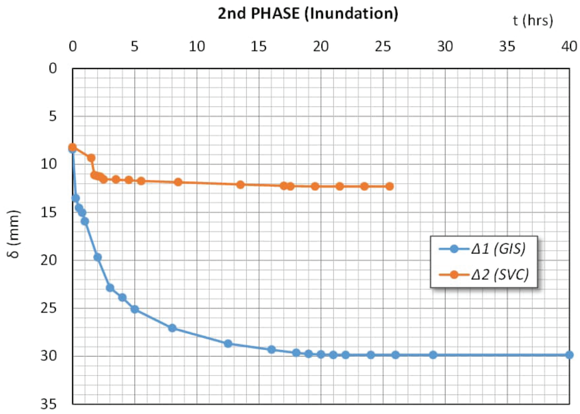

Fig. 12. Time evolution of settlement during inundation under constant load (2nd phase of the tests).

More specifically, in figure 11 the settlement of the slab as a function of the applied load under natural water content of the subsoil is presented. It is mentioned that the majority of settlements during each loading stage in this phase developed fairly fast. As it can be observed from figure 11, the response of the subsoil was fairly linear, implying that it was far from failure, and there was no distinct difference in behaviour between the two test locations. The estimated subgrade reaction modulus at the maximum applied pressure under natural water content conditions is in the order of Ks = 10 MN/m^3.

In figure 12, the settlement of the slab as a function of time after inundation, under maintained load (the maximum applied in phase 1), is presented. From this figure the following are observed:

- The phenomenon of additional settlements due to collapse of the soil structure after wetting was confirmed, as expected based on the laboratory test results.
- Most of these additional settlements developed fairly quickly after commencement of the inundation phase (about 55%-75% in 2 hours and 90%-95% in 10 hours).
- The observed increase of settlements due to soil wetting was distinctly different at the two test locations, ranging between 50% for test Δ2 (SVC) to 250% (!) for test Δ1 (GIS).
- The subgrade reaction modulus at the maximum applied pressure decreased from the original value Ks ≈ 10 MN/m^3 to Ks ≈ 2.5-5.0 MN/m^3.

 https://doi.org/10.1051/e3sconf/202338212006

The observed difference in behavior between the two test locations during the inundation phase, can be attributed to the following factors:

- The SPT blow count values (N_{SPT}) beneath test level were generally higher (up to twice) in the SVC building (borehole Γ2 – test Δ2) as compared to those in the GIS building (borehole Γ1 – test Δ1) (see figure 13), denoting a denser soil structure at the SVC building location. This assumption is also supported by the observed lower water infiltration in the SVC area during inundation (25 m^3 in the SVC area as compared to 36 m^3 in the GIS area), which is attributed to a lower void ratio and thus lower permeability.

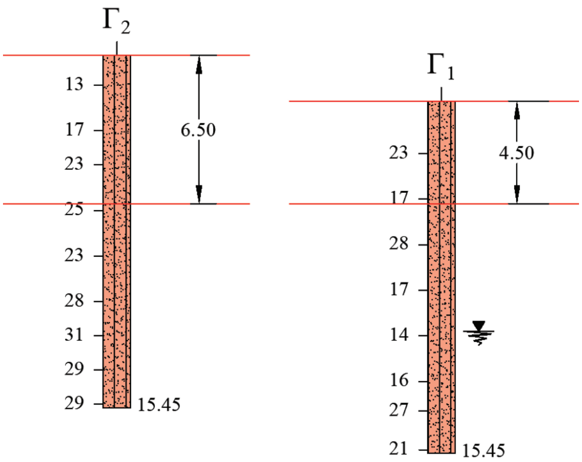

Fig. 13. N_{SPT} values vs. depth beneath load test level at the two test locations.

- The excavation depth, and by extension the magnitude of the unloading, was greater at the SVC building location (6.50 m) than at the GIS building location (4.50 m).
- The maximum test load in the SVC building location (test Δ2) was approximately 10% less (72 kPa) than in the GIS building location (test Δ1-79 kPa).

In order to evaluate these results in terms of the expected additional settlements of the buildings due to subsoil wetting, an estimation of the expected additional settlements due to collapse of the soil structure in the case of the load tests was then made based on the results of the laboratory tests. The procedure followed for this estimation was the same as the one followed in the original geotechnical design for the estimation of the settlements of the buildings due to collapse of the foundation soil. Following this procedure the following estimations were made, which are then compared to the actual load test measurements:

Test Δ1 (GIS)

- Maximum applied pressure: 79.125 kPa
- Influence depth for settlement calculation: 2.40 m
- Estimated additional settlement due to soil wetting, based on the laboratory test results: 80.9 mm
- Actually measured additional settlement due to soil wetting during the test: 21.43 mm

Test Δ2 (SVC)

- Maximum applied pressure: 71.75 kPa
- Influence depth for settlement calculation: 1.95 m
- Estimated additional settlement due to soil wetting, based on the laboratory test results: 62.8 mm
- Actually measured additional settlement due to soil wetting during the test: 4.13 mm

From the above it is observed that the actually measured additional settlements due soil wetting under field conditions were approximately 73%-93% smaller than the ones anticipated based solely on the laboratory test results. Based on this observation, settlement calculations of the two Substation buildings (GIS and SVC) were repeated taking into account the results of the load tests. The new thus estimated settlements due to soil wetting, under the maximum estimated by the structural design foundation pressures (75 kPa for GIS and 65 kPa for SVC), turned out to be acceptable and consequently these foundation pressures were eventually accepted.

It should be noted here that in the aforementioned comparison between measured and estimated settlements based on the laboratory test results, the inherent assumption has been made that the water front has reached at the end of the test the influence depths used in our calculations. In order to verify that, it would have been useful to have employed proper unsat instrumentation to measure the moisture and/or suction pressure of the soil within the aforementioned influence depths. Nonetheless, a gross check was made by taking into account the volume of water that dissipated into the gravel and the subsoil and by assuming that the water front spread out at an average angle of 45° and that the porosity of the soil was n = 0.50 (based on the laboratory tests). According to this gross check the water front reached a depth of:

- 2.15 m (< 2.40 m) in test Δ1 (GIS) and
- 1.75 m (< 1.95 m) in test Δ2 (SVC),

i.e. it was slightly shallower than the corresponding calculation influence depths, which means that the observed differences might have been slightly smaller.

8 Conclusions

With the design and execution of these innovative large-scale load tests, it was possible to measure, under actual field conditions, the settlements due to soil wetting and consequent collapse of its structure, leading to a more economical foundation design compared to the one that would have been adopted were only laboratory tests taken into account in the calculation of settlements.

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