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Behaviour of a building foundation on unsaturated expansive soil

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Abstract. The paper presents a case study of the behaviour of a building foundation on expansive unsaturated clay. The load-bearing masonry building started exhibiting severe cracking in its superstructure, immediately after its completion around 1950. Despite the interventions, the problems continued to exist sixty years later. In the context of identifying the causes of these problems, the paper presents the results of the laboratory tests conducted on the expansive clay in order to estimate its swelling pressure and also understand its behaviour. It is shown that the seasonal variation of the water content of the foundation soil, combined with the intrusion of the root system of the nearby trees at the level of the foundation, subjected the soil to wetting-drying cycles, resulting in its corresponding swelling-shrinking and consequently the settlement of the building. Finally, the proposed countermeasures for the solution of problems are presented, which aimed mainly in minimizing the variation of the moisture of the soil around and at the foundation of the building.

1 Introduction

Engineering problems associated with unsaturated expansive soils have been reported worldwide due to the damage of structures founded on them. Unsaturated expansive soils exhibit significant swell potential and in addition shrinkage potential. As a result, they swell and thus increase in volume when they become wet, and shrink when they dry. According to [1] the behaviour of unsaturated expansive clay is closely associated with the interaction between a capillary-controlled macrostructure which involves the ensemble of particles and/or aggregates together with interaggregate pores and a microstructure where physicochemical and other phenomena occurring at particle level take place. The swell-shrink potential of expansive soils is determined by their clay mineralogy which controls the amount of trapped water molecules by the clay particles at the microstructure level and the initial water content which controls the suction and therefore the state of effective stress at the macrostructure level [2]. Clay minerals of the smectite group indicate a high natural expansiveness of the soil.

In saturated expansive soils, their shrink-swell behaviour is controlled by the clay mineralogy and swelling strains are observed when the soil is unloaded. In unsaturated expansive soils, changes in water content (or suction) induce significant volume changes and therefore increase the probability of damage of any structure resting on them. Water content changes may be induced by seasonal variations (rainfall, evapotranspiration of vegetation), or caused by changes of local site, such as leakage from water supply pipes or drains, surface drainage, landscaping and planting [3]. Damage to foundations in expansive soils often results from tree growth. Such foundation movements are cyclic on a seasonal basis and are due to swelling during the rainy season and shrinkage during the dry season, when vegetation imposes a permanent moisture deficit.

In practice, the laboratory testing methods to estimate soil volume changes are based on either one-dimensional oedometer tests, or on soil suction/water content tests [4]. Another group of methods to infer the swell potential is based on the physio-chemical properties, such as molecular adsorption, or cation exchange capacity [5].

The paper presents a case study of the behaviour of a building foundation on expansive unsaturated soil. The building houses a high school in Greece and consists of two sections: the west (I), one-story with basement, and the east (II), one-story, Fig. 1. Inside the yard and along the north fence wall there was a series of cypress trees at a distance of 3m-8m from the north side of the building. Immediately after its completion in 1951, Section I exhibited problems with its load bearing masonry walls. In particular around 1956, this section exhibited settlement, indicated by cracking in the walls and the roof slab along the axis D. The problems continued and in 1970, a wooden tiled roof was added for the protection of the roof slab from incoming rainwater. Two interventions followed in Section I, the first was cement injection of the ground floor masonry walls in 1971 and the second was foundation reinforcement in 1988. The latter included the construction of five reinforced concrete columns (three along the axis C and two along the axis 3) in contact with the masonry walls and the underpinning and widening of the masonry strip foundation from 0.65m-0.70m to 1.50m along the axes C, 3 & 4. The estimated foundation pressure was of the order

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of 200kPa to 250kPa before the underpinning and was reduced to half of the above values after the underpinning.

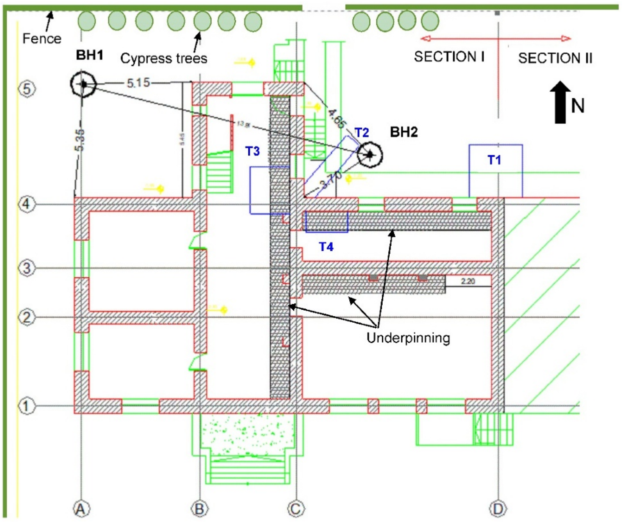

Fig. 1. Foundation of Section I and points of geotechnical investigation.

Obviously until that time, as indicated by the type of the intervention, the settlements of the building were attributed to the exceedance of the bearing capacity of its foundation. However, despite the underpinning, the problems continued until 2008. Then, the pathology of the building included:

- differential settlement between Sections I and II, indicated by the separation joint along axis D and its increasing width from foundation to the roof level, and the absence of cracking in Section II. Levelling of the ground floor showed higher settlements in the areas, defined by the axes C, D, 3 and 4, and the axes B, C, 4 and 5. The settlement reached 57mm along axis 4 in the first area and 62mm along axis C in the second area.
- extensive cracking of the walls of Section II and
- extensive cracking of the reinforced concrete and masonry walls along the north-west fence, as well as the step corridor walls leading to the basement.

In 2008, it was decided to conduct a ground investigation for the identification of the causes of the observed structural damage.

2 Geotechnical investigation

The geotechnical investigation comprised of two sampling boreholes (BH1 and BH2) down to a depth of 7.60m from ground surface and four trial pits (T1 and T2 outside the building and T3 and T4 inside it) down to the foundation depth of either the masonry walls, or the undepinning. The ground water table was not encountered. At BH1, the soil profile consists of fill, marl and volcanic tuff below it, whereas at BH2 only fill and volcanic tuff below it were found. Fig. 2 shows the cross section of the subsoil along the NS direction and close to BH2 and T2. The volcanic tuff consisted of two layers with distinct physical and mechanical properties. The upper volcanic tuff had variable degree of weathering and consisted of either a weathered light green clay with sand and silt [CL-MH], or a very weathered green high plasticity green clay [CH], with vains of calcium carbonate, or both types of soils. The lower volcanic tuff layer was a green sandy silt to silty sand [ML] with vains of calcium carbonate and was much stronger. The green high plasticity clay with calcium carbonate pockets of the upper volcanic tuff was also encountered at and below the basement floor in BH2, below the foundation level of underpinning in trial pit T2 and close to it in trial pit T3. In BH1 this soil was found at a greater depth, 2.30m below the basement floor.

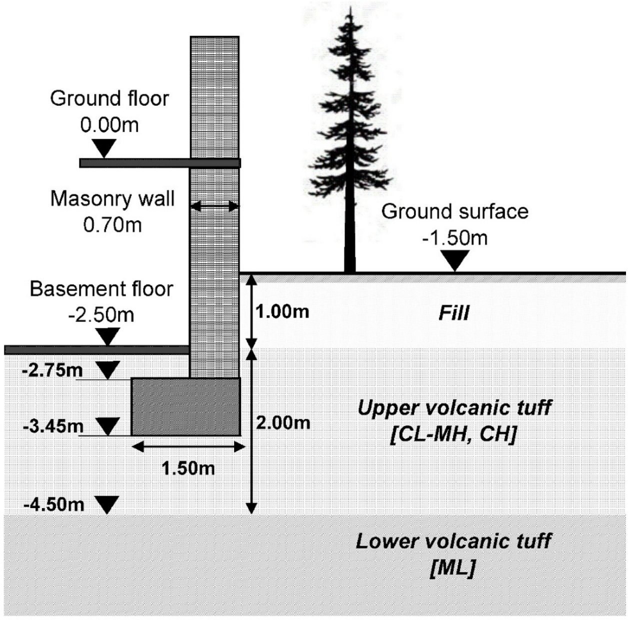

Fig 2. Cross section of the soil profile along the NS direction and close to BH2 and T2.

The physical properties of the samples retrieved from the upper volcanic tuff, summarized in Table 1, indicated that these were unsaturated soils of high to very high expansion potential, Fig. 3. Mineralogy analysis of a sample (T2-S1), obtained from a depth close to the foundation level of the underpinning in trial pit T2, showed that 52% of clay minerals were of the smectite type, characterized by high swelling/shrinking capacity.

Moreover, inside trial pit T1 roots of the nearby cypress trees were found at the top of the upper volcanic tuff layer and the on the external surface of the masonry wall, probably intruding below it at its foundation level, Fig. 4. Inside trial pit T2, roots were found both inside the fill and the top 1m of the upper volcanic tuff layer and close to the foundation level of the underpinning, Fig. 5. In trial pit T3, an extended horizontal root system was found below the basement floor, Fig. 6.

Bearing capacity calculations showed that the layer of the upper volcanic tuff (c'=39.9kPa, φ'=24^0) had sufficient strength to undertake the applied foundation loads. The high expansion potential of the soil in the upper volcanic tuff combined with the existence of extended horizontal

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tree root system at the foundation of Section I indicated that the soil might have been subjected to wetting-drying cycles, due to changes of its moisture, resulting in its corresponding swelling-shrinking. Thus, it was decided to explore the swelling/shrinking behavior of the soil by performing oedometer tests.

Table 1. Physical properties of the samples taken from the upper volcanic tuff.

Boring/ Trial pit	Depth (m)*	w %	WL %	PI %	CF %<2μm	USCS
BH1	3.30-3.60	18.7	53.1	26.6	29.0	CH
BH2	3.90-4.35	15.3	47.5	23.3	29.0	CL
	1.40-2.00	19.0	51.8	22.4	21.5	MH
T2-S1†	2.50-2.80	21.9-24.5	70.5	37.0	29.5	CH
T3-S1	1.00	19.8	57.1	30.3	26.0	CH

*Depth below ground surface
† S_r=74.4-86.9%

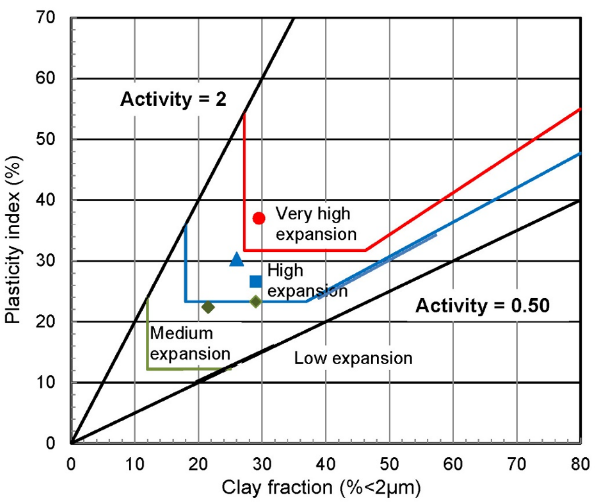

Fig. 3. Expansion capacity of the samples taken from the upper volcanic tuff [6].

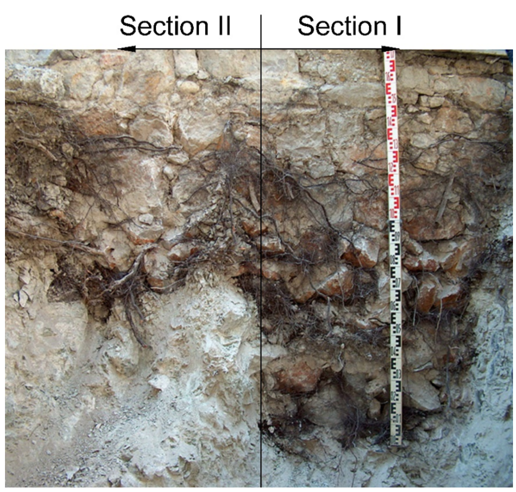

Fig. 4. Soil profile and roots system in trial pit T1.

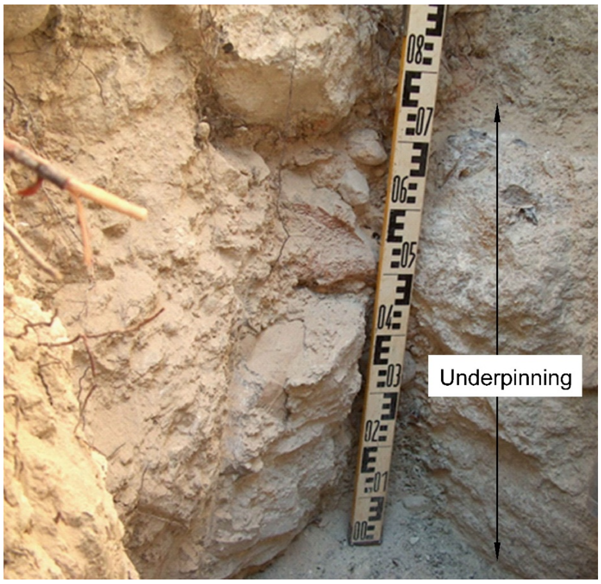

Fig. 5. Soil profile in trial pit T2.

Fig. 6. Soil profile and roots system below the basement floor in trial pit T3.

3 Laboratory tests and discussion

Three oedometer tests were conducted on undisturbed block sample from trial pit T2-S1. In Test 1, the specimen (D/H:6.35cm/1.92cm) was placed in the apparatus at its natural water content of 23% and loaded in steps up to 202kPa, without water in the oedometer external cell, Fig. 7. Then, the specimen was inundated for 144hr (6 days) until swelling was complete. The measured swelling corresponded to a vertical deformation of 1%. Afterwards, the typical loading (up to a maximum vertical stress of 4904kN/m^2) /unloading sequence followed, with each loading/unloading step lasting 24hr.

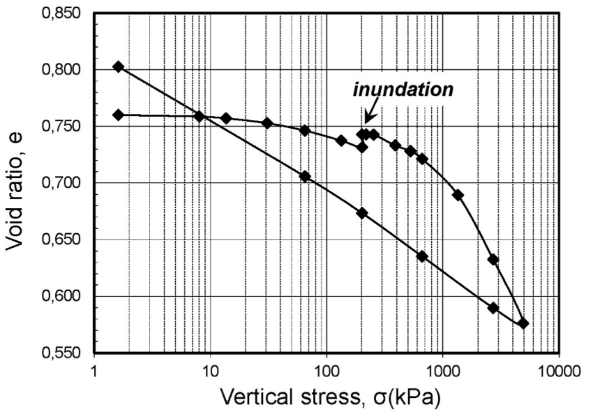

Fig. 7. Test 1 on sample T2-S1 (e_0=0.760, w_0=23%, S_r=81.7%).

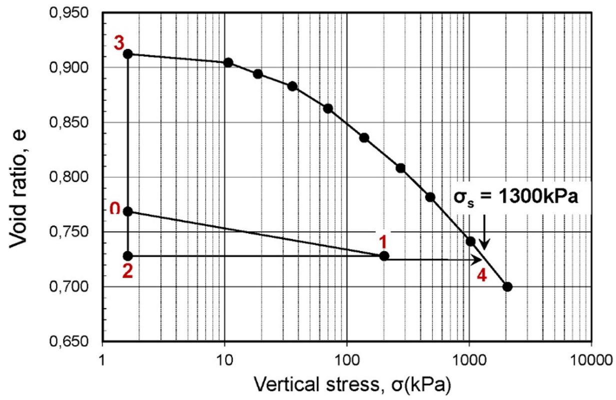

Fig. 8. Test 2 on sample T2-S1 (e_0=0.769, w_0=24.7%, S_r=86.9%).

Test 2 was a free swelling test [7]. The specimen (D/H:6.35cm/1.92cm) was placed in the apparatus at its natural water content of 24.7% (Point 0) and loaded in steps up to 202kPa, without water in the oedometer cell, Fig. 8 (Point 1). Then it was unloaded (Point 2) and immediately inundated for 144hr (6 days) until swelling was complete (Point 3). The measured free swelling corresponded to a vertical deformation of 10.71%. Afterwards, loading followed in steps with each loading step lasting 24hr, until the specimen reached its initial height (Point 4). The swelling pressure was of the order of 1300kPa significantly greater than the foundation pressure before and after the underpinning.

In Test 3, the specimen (D/H:5cm/2cm) was placed in the apparatus at its natural water content of 20.3% (Point 0) and loaded in steps up to 200kPa (Point 1), without water in the oedometer external cell, Fig. 9.

To simulate the cyclic alteration of wetting/drying periods, the specimen was indurated for 96hr (4 days) until full swelling. The measured swelling corresponded to a vertical deformation of 0.76%. Then the oedometer external cell was emptied from water (Point 2). This caused reduction of the specimen water content. For a period of 140hr (5 days and 20hr), after the water removal, no change of specimen height was observed (Point 3). However, then a compression of the specimen was observed for a period of 384hr (16 days). Afterwards, 19 swelling/wetting cycles, with a period ranging between 33 and 94 days approximately each, followed. During these cycles, the value of void ratio cycled between 0.656 and 0.517, and the vertical compression strain between 0.43% and 8.81%, Fig. 10.

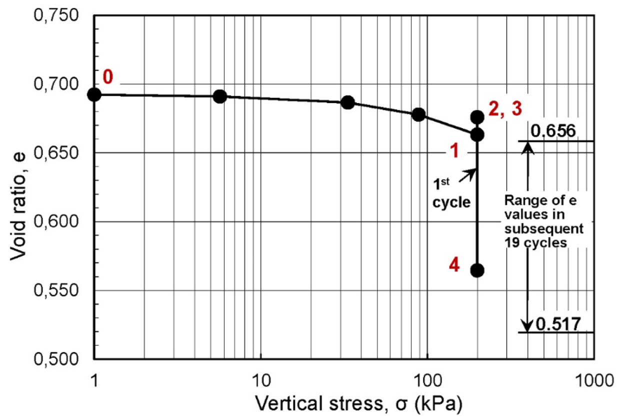

Fig. 9. Test 3 on sample T2-S1 (e_0=0.692, w_0=20.3%, S_r=79.3%).

The above test results together with the pathology of the section I indicate that the following process might have taken place. The moisture of the soil around and at the foundation of section I increased by downward infiltrating seasonal rain, as the basement was acting as a drain. The cypress trees absorbed moisture through their roots systems which extended below the basement floor and the foundation of this section. In dry seasons, the moisture of the soil decreased due to evaporation and the presence of the tree roots, which absorb moisture from the surrounding soil. This process subjected the soil in wetting-drying cycles, resulting in its corresponding swelling-shrinking. The estimated swelling pressure is significantly higher than even the initial foundation pressure of 200kPa-250kPa. For this reason, underpinning of the foundation deteriorated further its behaviour, and the problems continued to exist.

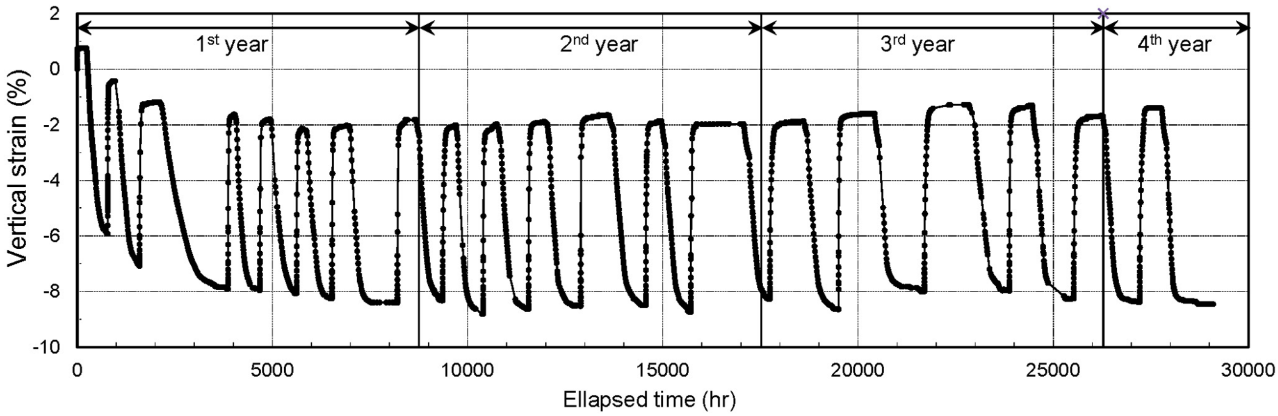

Fig. 10. Evolution of vertical strain of specimen in Test 3 during wetting/drying cycles.

Considering the maximum vertical strain of 8.81% in Fig. 10 and a thickness below the underpinning foundation level of 0.60m-1.00m for the upper volcanic tuff, a settlement of 52.8mm-88mm is estimated. These values which are close to the observed settlements, mentioned earlier, may be regarded as an upper bound to the observed, as the calculation is based on the laboratory results on the high plasticity green clay with the highest expansion potential.

The proposed countermeasures for the solution of problems aimed mainly in minimizing the variation of moisture of the soil around and at the foundation level. These included the construction of a drain outside and around the perimeter of section I for the collection of the surficial water, the placement of an impervious membrane beneath the drain and up to the external walls of this section to prevent migration of surface moisture into foundation and the removal of cypress trees located at a distance less than one to one and a half times their height.

4 Conclusions

This case study demonstrates that the unsaturated expansive soils pose a significant hazard to the foundations of structures founded on them, as well as the fact that incomplete knowledge of the subsoil conditions can lead to the proposal of ineffective countermeasures, which instead of improving the behaviour of the foundation soil, can worsen it.

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