Geotechnical investigation of a fissured highly expansive clay profile

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ABSTRACT

This study details some observations and challenges in interpreting site investigation data for highly expansive soils. Expansive soils exhibit large volume changes in response to changes in water content. This leads to a high degree of reworking and often a strongly fissured macrofabric in situ. A site underlain by a ~7.0 m thick active clay deposit, containing highly plastic and highly expansive smectites, was chosen for large-scale pile tests. Two identical areas were identified for the tests. The first was kept at natural water content conditions. The other was kept submerged under water through infiltration wells for six months, aiming to facilitate maximum swell of the active layers. As part of the site investigation, continuous surface wave (CSW) tests and standard penetration tests (SPT) were conducted at various points in time. These results have been compared for the natural water content and the submerged profiles. Small-strain moduli interpreted from the CSW tests showed no significant difference between the two profiles, contrary to the intuitive expectation that inundation of the highly expansive clay layers and dissipation of suctions in the order of several megapascals might significantly reduce the stiffness. Due to the small strains imposed during the test, the characteristics of the fissures and joint infill material are measured, rather than that of the intact masses. The large-strain SPT results showed a softer response for the inundated profile, as expected. Challenges in obtaining intact tube samples from the stiff and highly fissured unsaturated profile have also been discussed.

Keywords: expansive soils; continuous surface wave (CSW) testing; fissured clays.

1. Introduction

Expansive clays are problem soils that experience large volume changes with variations in water content. Seasonal heaving and shrinkage of these soils often cause damage to civil engineering infrastructure. As a result, expansive soils have been reported to be the most severe geohazard in Southern Africa (Diop et al. 2011), the UK (Jones and Jefferson 2012), China (Miao et al. 2012) and the US (Nelson and Miller 1992).

Although the understanding of expansive clays (aided by unsaturated soil mechanics principles) has developed significantly over time, thorough geotechnical investigations are required to properly identify and mitigate the consequences that may be caused by the presence of these soils. This paper aims to identify and discuss some aspects and challenges of site investigations in expansive clays, with reference to a case study on a research site in South Africa.

2. Site layout

A field-testing site near Vredefort in the Free State province of South Africa was selected for large-scale pile testing as part of the WindAfrica research project. The site is adjacent to a bentonite quarry, and was thus known to be underlain by an active expansive clay deposit. Two separate areas on the site were utilised for testing. One area was kept at unsaturated natural water content conditions for the duration of the project, and has been referred to as the 'dry site' in this study. Swelling was facilitated at the other area, which was kept submerged under water by provision of surface water and through infiltration wells to a depth of 6 m for a period of six months prior to testing. This area has been referred to as the 'wet site' in this study. The two areas were sufficiently far apart such that the inundation of the wet site did not influence the ground conditions at the dry site. Some details of the installations and testing have been provided in previous publications, including in-situ measurements of swell (Murison et al. 2022a), instrumentation of the piles (Murison et al. 2022b), behaviour of full-scale piles at the site due to seasonal heave (da Silva Burke et al. 2022) and cyclic lateral loading (Gaspar et al. 2024).

3. Profiling and classification

The initial profiling as part of the site investigation indicated that the profile is layered with various expansive clay soils to a depth of approximately 7 m. A simplified borehole log is presented in Fig. 1. As indicated, samples were taken from depths of 1.0, 3.0 and 5.0 m for classification testing to BS 1377 (1990), presented in Table 1. The particle size distributions are presented in Fig. 2. The mineralogical compositions in Table 2 were determined through X-ray diffraction (XRD) analyses from the same sample depths.

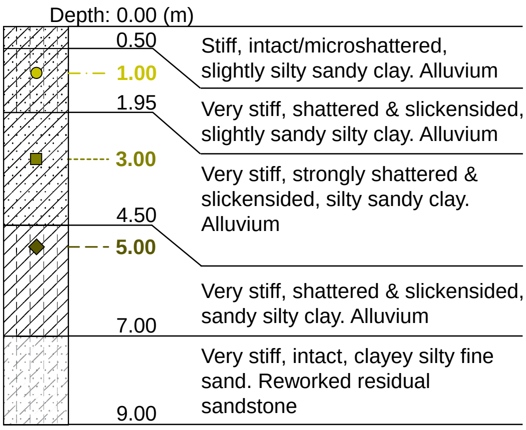


Figure 1. Soil profile for the test site, with sampling depths indicated.

Table 1. Soil classifications (to BS 1377: 1990)

Sample depth:	1.0 m	3.0 m	5.0 m
In-situ water content (%)*	20.4	21.2	20.2
In-situ void ratio, e *	0.90	0.86	0.83
Specific gravity, G_s	2.662	2.692	2.669
Liquid limit, w_L (%)	66	109	72
Plasticity index, I_P (%)	47	82	50
Clay fraction by mass (%)	34	39	30
Classification	CH	CE	CV
Activity (after Skempton 1953)	1.38	2.10	1.66
Potential expansiveness class (after Van der Merwe 1975)	Very high	Very high	Very high

* At end of dry season

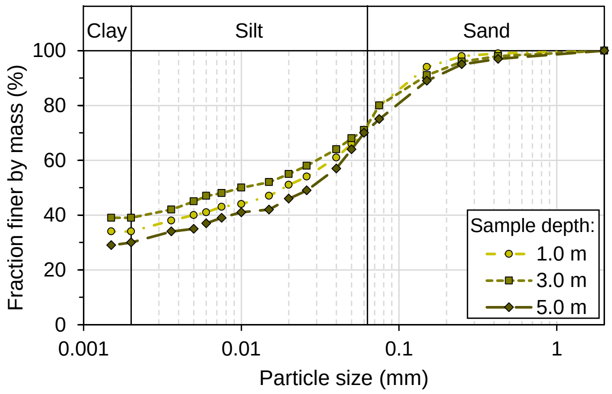


Figure 2. Particle size distributions, determined according to BS 1377 (1990).

Table 2. Mineralogy from XRD analyses

Sample depth:	1.0 m	3.0 m	5.0 m
Quartz (%)	37	36	41
Plagioclase feldspar (%)	7	6	7
Orthoclase feldspar (%)	6	5	5
Illite/illite-smectite/mica (%)	9	10	11
Smectite (%)	40	42	35
Kaolinite (%)	1	1	1

The dominant mineral constituent for all three sample depths was smectite, which is the mineral causing the expansive nature of the soil. The ground profile exhibited a strongly fissured macrofabric, which is typical of expansive clays due to high degrees of reworking through seasonal swelling and shrinkage. Despite similar visual appearance, grading, in-situ states and mineralogical compositions, a distinct difference between the clay layers was identified in their plasticity characteristics. The clay from a 3 m depth exhibited a significantly higher liquid limit and plasticity index. This greater plasticity, and thus activity, implies that the 3 m layer would exhibit greater inherent expansivity than the other two layers. Preliminary results from element tests have confirmed that this layer is significantly more expansive, which is evident through a greater swell pressure and greater swell potential under any given stress. These results highlight the value in obtaining multiple samples for classification testing. If the 3.0 m layer were to be improperly classified or not considered, there may be serious underprediction of in-situ heave upon changes in water content, and of the resulting pressures upon structures founded in the soil. Extensive and thorough classification testing may also prove to reduce the volume of element testing required for a project, or allow for greater value to be drawn from the laboratory tests, as testing programmes may be designed to focus on specific depths of interest.

4. In-situ testing and interpretation

In-situ testing was conducted at various stages throughout the project. The timeline of key events has been summarised in Fig. 3. Upon site establishment and borehole logging, the first sets of standard penetration tests (SPTs) were conducted at what would become the 'dry' and 'wet' site locations. No water table was observed at either location. A rain gauge was installed at the site to monitor daily rainfall. Total rainfall over each complete wet and dry season within the monitoring period is reported in Fig. 3. Approximately two months after establishment, the flooding of the wet site commenced through the infiltration wells. Water was continuously provided for a period of six months, keeping the water table at the ground surface and allowing the intact masses within the fissured profile to swell. Approximately two months after the flooding period had been completed, the first sets of continuous surface wave (CSW) tests were conducted. The water table at the wet site was still observed to be very near the ground surface at this stage. Hereafter, approximately 8-9 months passed before the next set of tests, during which the wet site profile had partially dried out and the water table had naturally subsided. The second sets of SPT and CSW tests were conducted within two weeks of each other. At this stage, the water table was observed at a depth of approximately 4 m during profiling of the wet site, whilst the water table at the dry site remained deep. The results and interpretation of the in-situ tests, with reference to the timeline and corresponding water table positions, are discussed in the subsequent sections.

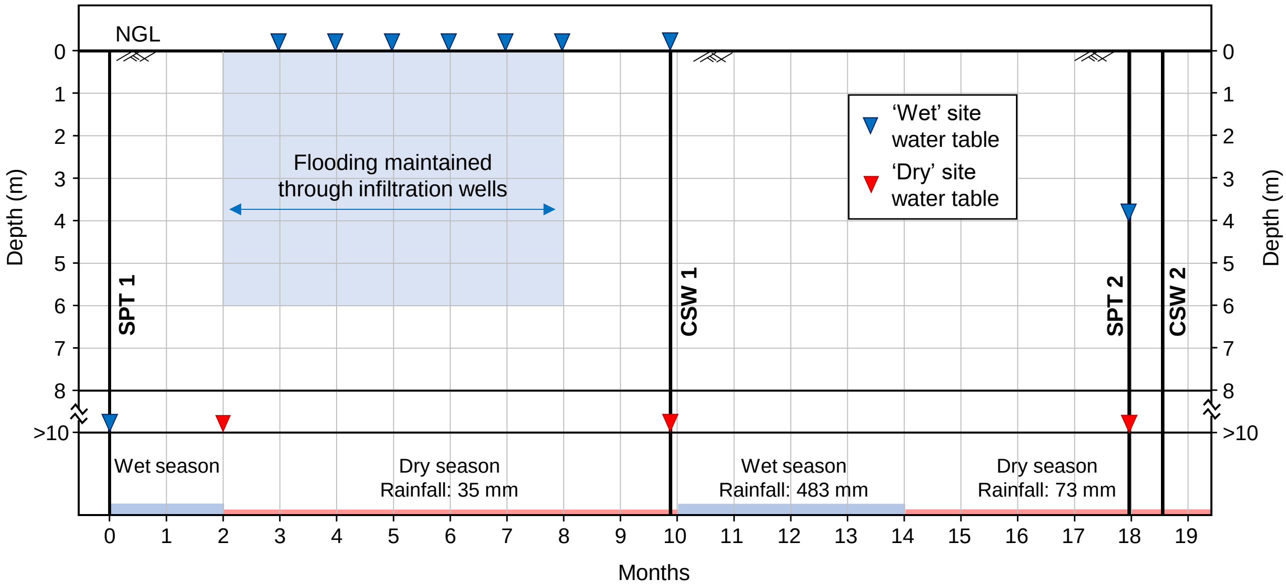


Figure 3. Timeline of in-situ testing, flooding of the wet site and seasonal rainfall.

4.1. CSW results

Continuous surface wave tests were conducted in accordance with guidelines by Heymann (2007). Sets of harmonic single frequency tests, as well as a 'sweep' through a range of frequencies, were conducted using a low frequency (7-22 Hz) and intermediate frequency (20-90 Hz) shaker. Five geophones were used to determine the resulting phase velocities and wavelengths of Rayleigh waves through a fast Fourier transform (FFT) algorithm. The dispersion data for each of the CSW tests are presented in Fig. 4. Each profile was observed to be normally dispersive, with no higher-mode vibrations observed.

The inversion analysis was conducted using a neighbourhood search algorithm proposed by Wathelet (2008) and Wathelet at al. (2004). The best-fit modelled dispersion curves from these inversion analyses have been included in Fig. 4.

The small-strain shear moduli (G_0) with depth for each of the profiles in Figs. 5 and 6 were determined using Eq. 1, where ρ_b is the bulk density of the material and V_s is the shear wave velocity determined from the inversion analysis.

$$G_0 = \rho_b \cdot V_s^2 \tag{1}$$

All profiles trialled in the inversion search algorithm which contained a theoretical dispersion curve within 10% of the best-fit have been reported in Fig. 5. This aims to give an indication of the confidence level of the final small-strain modulus profiles. At both instances in time and water table positions, the G_0 profiles at the dry site and wet site were similar enough in magnitude that the engineering decision made based upon the results would likely not be different. For scientific purposes, the wet site profile was marginally stiffer in both cases. This observation is seemingly counterintuitive, as one would expect the stiffness of the wet site profile to reduce during wetting due to the dissipation of suctions in the order of several megapascals.

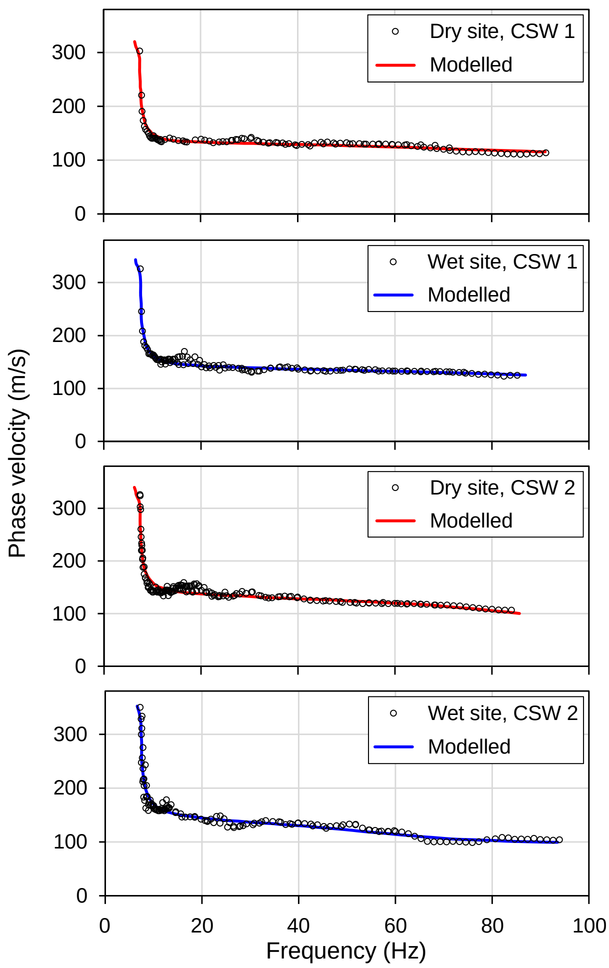
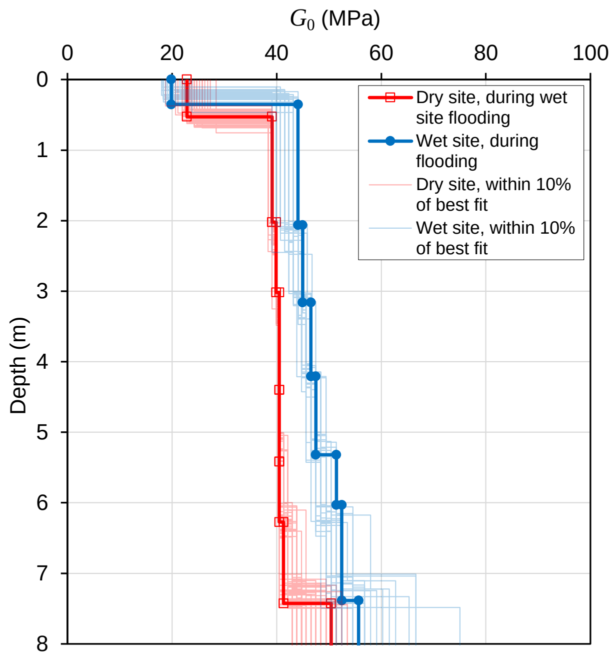
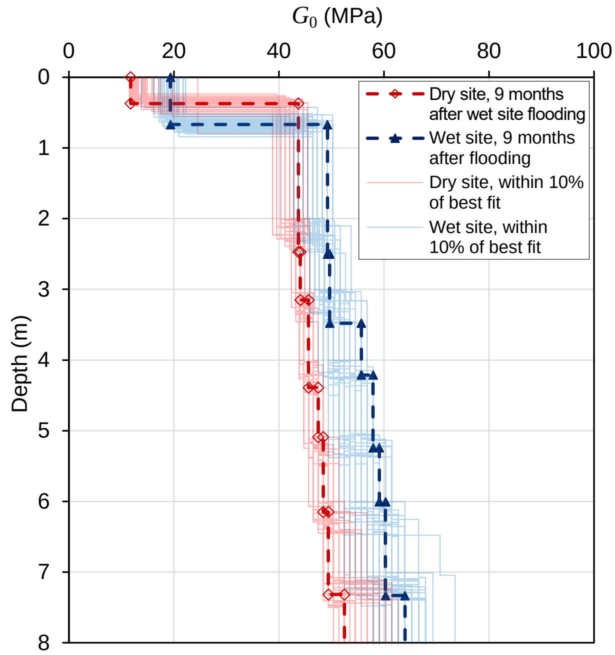


Figure 4. Measured and modelled dispersion data.



a) CSW 1, water table at surface for wet site



b) CSW 2, water table at ~4 m for wet site

Figure 5. Small-strain shear modulus profiles for the wet and dry sites from continuous surface wave tests.

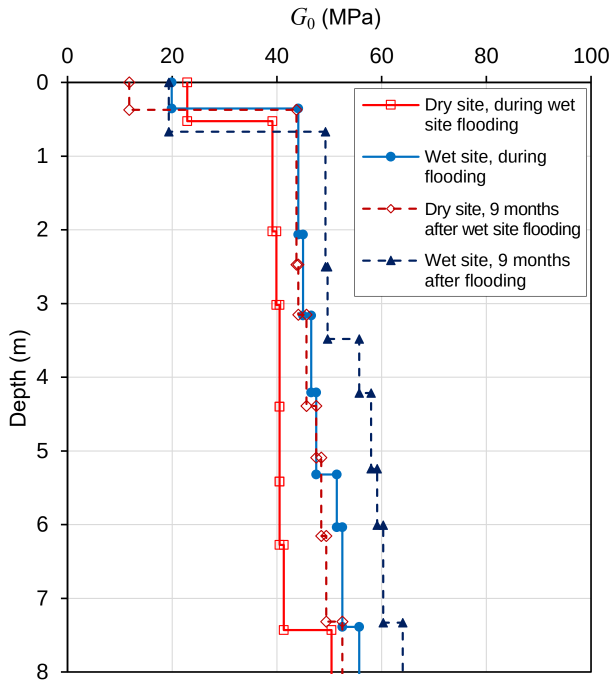


Figure 6. Combined CSW test results.

However, it is important to consider the nature of the test, the macrofabric of the clay profile and the strain range over which the measurement of shear moduli occurs. Matthews et al. (2000) found that the in-situ stiffness of jointed and fissured hard soil and weak rocks, when measured using seismic techniques, is significantly lower than stiffness measured in triaxial tests. The very small strains imposed during CSW testing do not sufficiently deform the soil to close the fissures, and the measured shear wave velocity is thus dominated by the joint infill material rather than the intact material. Wetting may have influenced the behaviour of intact material, but if the fissure characteristics were not significantly changed, the measured stiffness would likely be similar. The slight differences in stiffness can be explained through the sketches in Fig. 7. Figure 7a shows a fissured clay profile, with joint widths exaggerated. Although the voids are largely air-filled in this case and the water table is deep, seasonal rainfall will still cause some swelling in the intact masses and change the joint characteristics. This would explain the minor differences between the two dry site CSW results. Figure 7b shows a hypothetical case where the first profile has been instantaneously submerged under water, and swelling is yet to occur. Theoretically, shear wave velocities and small-strain moduli within the profiles shown in Figs. 7a and 7b would be identical. Figure 7c shows the inundated profile after some swelling of the intact masses has occurred. The wet site CSW 1 result is suggested to reflect such a case. Finally, Fig. 7d shows a case where the water table has subsided, but swelling had continued to occur below the water table. The uppermost layers in the profile may have encountered some shrinkage, but on the most part the fissures are narrower than in the previous cases. This case is suggested to reflect the CSW 2 result for the wet site. It is suggested that the changes in the fissure characteristics between the cases in Figs. 7c and d, and the potential increase in continuity between the intact masses, are responsible for the marginal increase in shear wave velocity (and thus G_0) between the wet site results from CSW 1 to CSW 2.

Note that in any of the given cases, some fissures may have swelled completely closed, and intact mass behaviour may have started to drive the measured shear wave velocity.

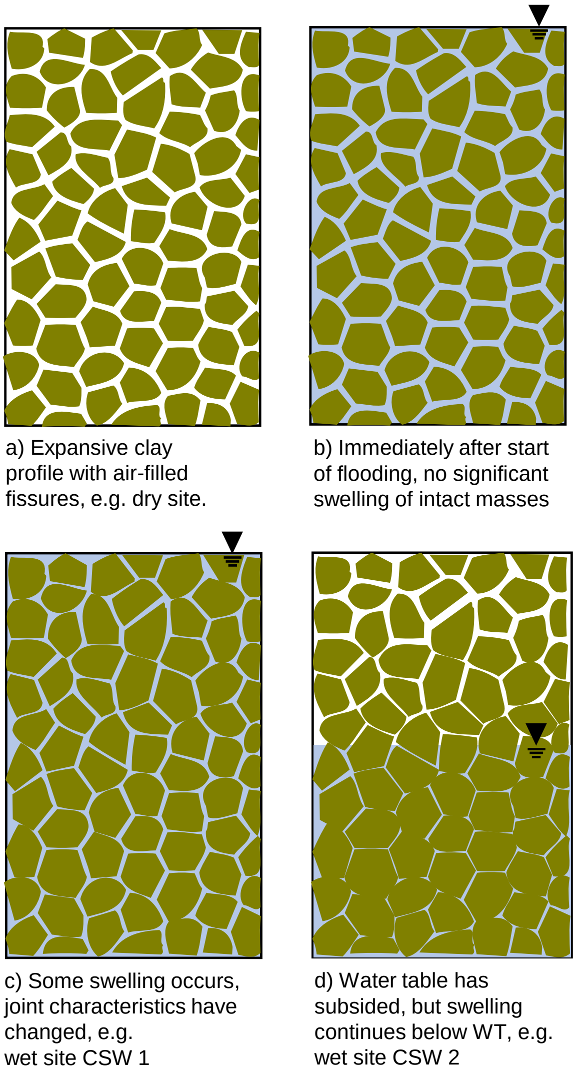


Figure 7. Simplified fissured clay profile throughout flooding process.

4.2. SPT results

The standard penetration tests were conducted in rotary core-drilled 54 mm diameter boreholes using an automatic trip hammer, with an energy ratio of 75%. The blow counts obtained at various depths were converted to equivalent N_{60} values presented in Fig. 8 (i.e. corresponding to the standard energy ratio of 60%).

The SPT results during site establishment showed similar profiles for the wet and dry site, apart from the point at approximately 4.5 m. This can be attributed to some natural variability in the ground profile at the test site or some localised feature at the dry site borehole. The second set of tests showed a significantly stiffer/stronger response at the dry site, and a significantly softer/weaker response at the wet site.

The changes in strength can be attributed to changes in suction occurring within the intact clay masses. As an example, gravimetric water contents for samples taken from a depth of 3 m at the site have varied between approximately 27% after the wet season and 21% after the dry season. Soil-water retention curves for material from this depth showed that this variation would cause an increase in total suction from approximately 1.5 MPa to 6 MPa (Murison et al. 2023), which would induce a significant increase in shear strength and thus blow count. The first SPT tests were conducted in the middle of the wet season, and the second at the end of the dry season. Such variations in water content would thus be expected. The converse is also true: samples taken from the wet site during the second set of SPT tests were approximately fully saturated, and suctions would thus be significantly lower. This would explain the reduction in blow count in the wet site at greater depths.

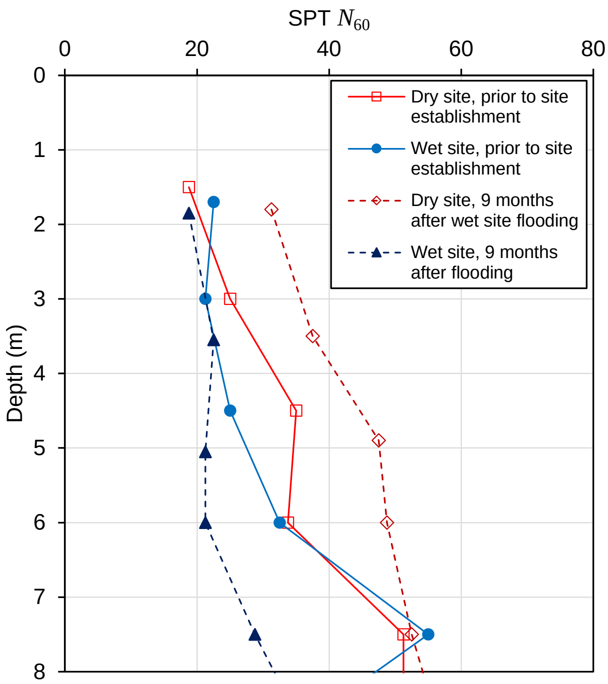


Figure 8. Combined SPT test results.

4.3. Discussion

The two sets of in-situ tests which show seemingly opposing trends illustrate the challenges in interpreting site investigation data for complex ground conditions, and the danger of considering any single test in isolation. Consider the CSW tests for the wet site and dry site at either of the instances at which the tests were conducted. If no large-strain test was conducted, or if laboratory testing or determination of ground water conditions were not considered, near-identical designs would likely be implemented to address serviceability issues for structures founded at either site. Figure 9 shows a double oedometer test (i.e. one sample loaded under natural water content, and one allowed to swell under the in-situ total stress before loading) on material from 3 m, after data by Murison et al. (2024). The test illustrates the differences in stiffness, under K_0 conditions, of an intact soil mass at natural water content (such as the dry site) and a sample that has been soaked and subjected to swelling (such as the wet site).

Prior to yielding of the soaked specimen, the stiffness values for the two samples were nearly identical (slopes of 0.036 and 0.039 for the 50-100 kPa increment). This suggests that even for the intact masses within the fissured profile, elastic moduli are similar, which aligns with the observations at very small strains for the in-situ profiles from the CSW tests. Interestingly, this also aligns with the assumption in the Barcelona Basic Model (Alonso et al. 1990) that the slope of loading paths in elastic states (i.e. κ) is independent of suction.

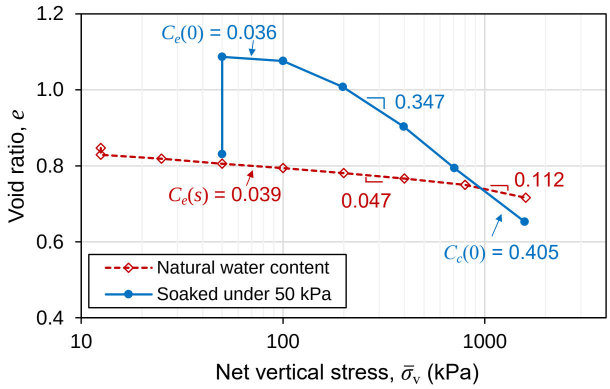
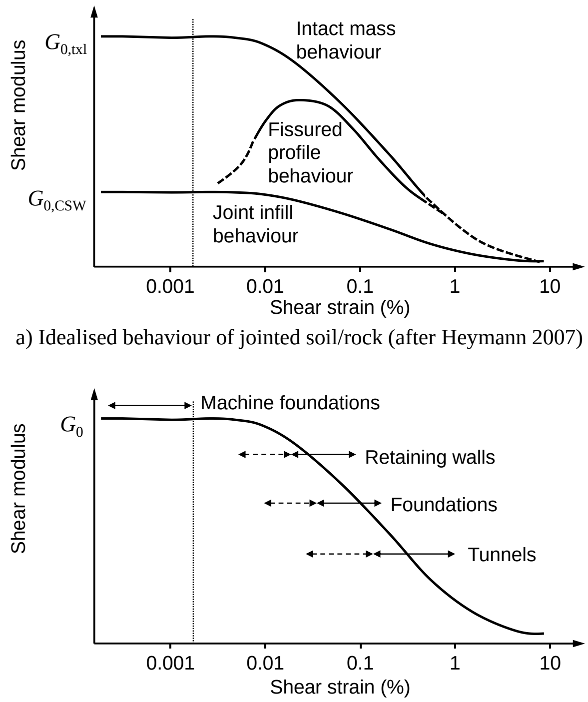


Figure 9. Double oedometer test on material from 3.0 m (after data by Murison et al. 2024).



a) Idealised behaviour of jointed soil/rock (after Heymann 2007)

b) Typical strain ranges for geotechnical structures (after Mair 1993)

Figure 10. Modulus reduction considerations for fissured soil.

However, once the swelled clay had yielded, significant variations in stiffness were observed. For the 200-400 kPa increment, the slope of the loading path for the soaked sample was 0.347, which is nearly an order of magnitude greater than the 0.047 of the sample loaded at natural water content. In any context where strains are sufficiently large that the stress-strain behaviour is dominated by the intact masses within the fissured profile, responses in the wet site and dry site could be vastly different depending on the stress level of the problem. Figure 10a shows a simplified set of modulus reduction curves (Heymann 2007), illustrating the stiffness behaviour of jointed profiles for the case where the joint infill stiffness is less than the stiffness of the intact material. This framework ties in with the observation by Matthews et al. (2000) discussed earlier. Figure 10b shows the typical strain ranges of some geotechnical structures (Mair 1993). It should be noted that these curves represent "rules of thumb" rather than actual quantitative limits. The purpose of these figures is to illustrate that for most applications (other than machine foundations), the behaviour of a structure founded in a fissured profile would be influenced by the stiffness of the intact masses. This is detail that would be completely lost when considering seismic tests in isolation.

Similar shortcomings for considering SPT results in isolation can be discussed. If, for example, SPT tests were only conducted at the end of the dry season (e.g. SPT 2), with no thorough characterisation of the profile to identify the presence of expansive clays and understand its sensitivity to moisture conditions, strength might be significantly overpredicted. Dry site resistances in the wet season were 30-50% less than in the dry season. If insufficient in-situ and laboratory tests were to be conducted, this might go unnoticed.

It is of course well-known that no in-situ test should be conducted in isolation, and the value of thorough site characterisation is well understood by the geotechnical community. The observations presented in this study aim to reinforce this sentiment.

5. Intact sampling challenges

In addition to the challenges encountered during in-situ test interpretation, some challenges were encountered in obtaining intact or "undisturbed" samples of the expansive clay from the dry site. Shelby tube sampling was carried out at the time of the second round of SPT tests, such that intact samples for element testing could be obtained. Samples from the softer, nearly fully saturated wet site could be successfully extruded and trimmed into oedometer rings for testing, as illustrated in Fig. 11a. It was not possible to extrude any intact oedometer sample from the tube samples obtained from the stiffer, unsaturated dry site. These severely distorted samples are shown in Fig 11b.

The difficulties in obtaining intact element test samples from Shelby tubes can be attributed to two features. The first is the close joint spacing and presence of fissures within the clay being sampled, prior to insertion of the tube. The second is distortion induced by the strains during both sampling and extrusion. Clayton et al. (1995) presented sketches of shear distortions similar in appearance to those in Fig.11b, for tube samples of very stiff overconsolidated London clay. Figure 12 shows axial strains induced by the advancing edge of a simple tube sampler as a function of the diameter to wall thickness ratio (B/t), simulated using the strain path method by Baligh (1985). Even for a high B/t ratio of 40, as used in this study, nearly 1%

of axial strain is invoked upon the sampled soil in both compression and extension during insertion. While a soft and ductile sample (Fig. 11a) can sustain such strains, this magnitude is likely to cause significant shattering of a stiff and brittle sample (Fig. 11b).

a) Intact samples successfully extracted from the wet site

b) Severe distortions in tube samples from the dry site

Figure 11. Tube sampling difficulties for stiff, unsaturated clay.

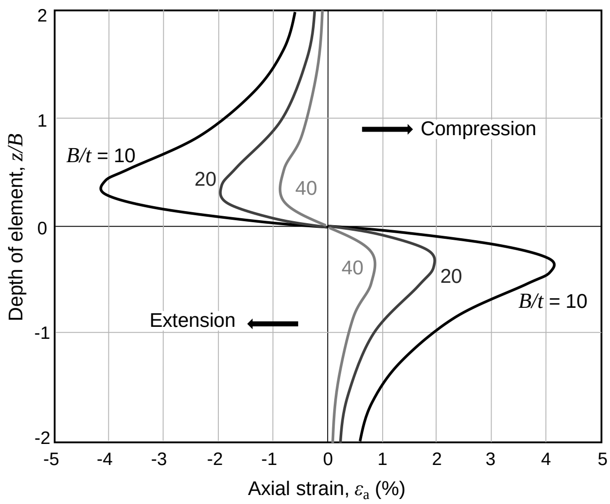


Figure 12. Simulated strain history along the centreline of a simple tube sampler (after Baligh, 1985).

It is thus suggested to obtain large block samples for careful trimming in the laboratory for element testing of stiff unsaturated expansive clays with the intact in-situ fabric. Such an exercise was conducted successfully for a similarly strongly fissured highly expansive clay profile by Gaspar (2020) and Gaspar et al. (2022). Large intact block samples have since been successfully obtained from the 'dry' site investigated in this study.

6. Conclusions

Continuous surface wave tests and standard penetration tests were conducted on a highly expansive clay profile under natural water content conditions, and after submerging the profile under water to induce swell. The two types of in-situ test seemingly exhibited opposing trends regarding changes in stiffness and strength. Marginal increases in small-strain shear modulus were recorded for the swelled profile through the CSW testing. This was attributed to the fissured nature of the profile and the small strains imposed by seismic tests. Shear wave velocities were likely governed by the joint infill characteristics rather than by the softened intact masses of swelled clay. However, element test results indicated that for low stresses (less than 100 kPa, prior to yielding), the stiffnesses of a flooded and swelled specimen and a specimen at natural water content were similar. At higher stresses, yielding of the swelled specimen caused the stiffness to reduce to nearly an order of magnitude less than the unsaturated sample. The SPT test results exhibited the more intuitive trend of a decrease in resistance to penetration in the profile that was submerged under water. This was attributed to the dissipation of suctions in the order of several megapascals within the intact clay masses.

Finally, Shelby tube samples obtained from the softer submerged profile could successfully be extracted for element testing, but tube samples at natural water content were severely shattered and could not be used. It was suggested that large block samples should be obtained for such stiff, fissured profiles, if element tests on intact or "undisturbed" samples are required.

Acknowledgements

The authors gratefully acknowledge the UK Engineering and Physical Sciences Research Council (EPSRC) for funding under the Global Challenges Fund programme for a project entitled 'Developing performance-based design for foundations of wind turbines in Africa (WindAfrica)', Grant Ref: EP/P029434/1.

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