Proceedings of the 20th International Conference on Soil Mechanics and Geotechnical Engineering– Rahman and Jaksa (Eds)
© 2022 Australian Geomechanics Society, Sydney, Australia, ISBN 978-0-9946261-4-1

Comparison of swell behaviour of highly expansive clay through field monitoring and centrifuge modelling

Comparaison des propriétés de gonflement de l'argile très expansive grâce à la surveillance sur le terrain et à la modélisation par centrifugation

Ruan A. Murison, S.W. Jacobsz & Talia S. da Silva Burke
Department of Civil Engineering, University of Pretoria, South Africa. Corresponding author: ruan.murison@tuks.co.za

Tiago A.V. Gaspar & Ashraf S. Osman
Department of Engineering, Durham University, United Kingdom

ABSTRACT: Expansive clays are problem soils that undergo significant volume changes with seasonal wetting and drying. These volume changes cause damage to civil engineering infrastructure and pose design challenges for deep foundations. A site underlain by highly expansive clay near Vredefort, South Africa was monitored over a period of 12 months. The soil movement of the active layer was recorded in situ to gain an indication of heaving and shrinkage. Centrifuge modelling was used to simulate the in situ behaviour of this expansive clay profile in the laboratory. The laboratory and field results were compared to two methods of heave prediction used in engineering practice. The centrifuge model exhibited more uniform expansive behaviour throughout the profile, with greater swell than the field site profile. The empirical heave methods provided satisfactory predictions, depending on the assumption of expansive potential.

RÉSUMÉ: Les sols argileux expansibles sont des sols problématiques qui subissent d'importantes variations de volume en rapport avec l'alternance de cycles saisonniers d'humidification et d'assèchement dans les climats semi-arides à arides. Ces changements de volume sont susceptibles d'endommager les infrastructures et poser des difficultés de conception pour les fondations profondes. Un site reposant sur de l'argile très expansible près de Vredefort, en Afrique du Sud, a été surveillé sur une période de 12 mois. Le mouvement du sol de la couche active a été enregistré in situ pour obtenir une indication du soulèvement et du retrait. La modélisation par centrifugation a été utilisée pour simuler le comportement in situ de ce profil d'argile expansif en laboratoire. Les résultats au laboratoire et sur le terrain ont été comparés à deux méthodes de prédiction du soulèvement utilisées dans la pratique de l'ingénierie. Le modèle de centrifugeuse a présenté un comportement d'expansion plus uniforme sur tout le profil, avec une houle plus importante que le profil du site sur le terrain. Les méthodes empiriques de soulèvement ont fourni des prédictions satisfaisantes, en fonction de l'hypothèse d'un potentiel d'expansion.

KEYWORDS: expansive clays, field monitoring, centrifuge modelling, heave prediction

1 INTRODUCTION

This study was conducted as part of the WindAfrica project, which aims to address and overcome the challenges of the design of wind turbine piled foundations within expansive clay profiles. An overview of the project is given by Gaspar et al. (2022).

Expansive (or swelling) soils occur most abundantly across Australia, North America, and Southern and Central Africa (Nelson et al. 2015). However, reports of the occurrence and associated implications of these soils at several locations across every continent show that expansive soils are a worldwide problem (Gaspar 2020). Experience has shown that expansive clays are the problem soil that causes the greatest damage to infrastructure in North America (Fredlund et al. 2012; Jones & Holtz 1973), Britain (Jones & Jefferson 2012), Southern Africa (Diop et al. 2011) and China (Miao et al. 2012). To fulfil serviceability design requirements and limit structural damage, engineers require a reliable indication of the magnitude of seasonal heaving and shrinkage that these clays can exhibit.

This study aims to gain a better understanding of the swell behaviour of expansive clay present at the selected field testing site near Vredefort, South Africa, through both in situ field monitoring and laboratory work in the geotechnical centrifuge.

2 EXPANSIVE CLAYS

Expansive clays are problem soils that experience significant volume change with changes in moisture content. The swelling and shrinking behaviour of these soils is predominantly governed by the complex dioctahedral aluminium silicate clay minerals in the smectite group, such as montmorillonite (Das 2006). These are 2:1 lattice clay minerals (Williams et al. 1985) where the clay platelets, which have negative surface charges, are loosely bonded by polar water molecules between the plates. This allows for significant expansion and shrinkage with the introduction and removal of moisture, as would seasonally occur in annual wet (rainy) and dry seasons. Figure 1 illustrates the concept.

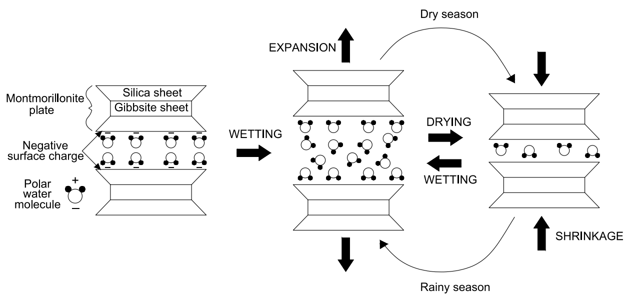

Figure 1. Structure, swelling mechanism and seasonally cyclic nature of montmorillonite in expansive clays.

In arid and semi-arid climates these soils exist in an unsaturated state, and exhibit swelling during the wet season, and shrinkage during the dry season. These vertical movements cause distress to roads and pavement structures, as well as differential settlement and cracking of buildings. The presence of expansive clay within a soil profile is evident with these signs of damage, or when there are fissures in the ground surface or subsurface profile, amongst other indicators (Williams et al. 1985). In addition, the analysis of piled foundations in swelling soil is complicated due to the variation in soil characteristics with changing moisture regimes. Lateral pressure exerted onto the piles, shaft friction, soil stress regimes and stiffness all vary with swelling and shrinkage. Understanding of unsaturated soil mechanics and quantifying the swell magnitude are required to overcome these challenges.

3 FIELD MONITORING SITE

The chosen site for the large-scale pile loading tests and long-term in situ soil monitoring was adjacent to an existing clay quarry near Vredefort, South Africa. The selection of the site was motivated by the knowledge of a thick deposit of highly plastic expansive clay (approximately 6 m) present at the natural ground surface, and the vast open area with no infrastructure in the surroundings that could be affected by the installation and testing (da Silva Burke et al. 2021).

3.1 *Site investigation*

Two rotary core boreholes were drilled at the site. Borehole BH1 was drilled to 16.5 m and BH2 to a depth of 15.5 m, both terminating in bedrock. The upper layers of the borehole logs, presenting a simplified soil profile of the active zone, are given in Figure 2. Potentially expansive material is indicated by the slickensided and shattered alluvial clay deposits. These were underlain by residual sandy material. The expansive layer is shaded in Figure 2, and is approximately 4 to 7 m thick.

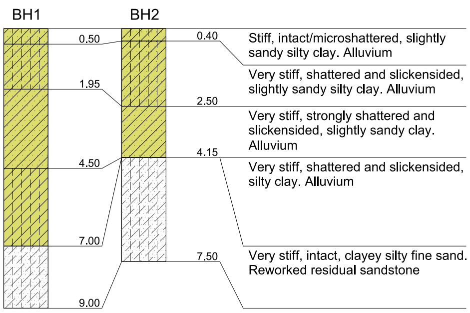

Figure 2. Simplified upper soil profile with the potentially expansive layer shaded.

Samples taken from a depth of 3 m during boring were used to conduct preliminary tests for soil classification. Van der Merwe (1964) presented an activity chart to determine the expansive potential of soils based on plasticity index (PI) of the whole sample and the percentage clay fraction ($\leq$ 0.002 mm). Table 1 gives the soil characteristics and expansive potential determined for each of the samples. The very high expansive potential confirmed that the clay on site was appropriate for the purposes of the study.

Table 1. Soil characteristics and expansive potential of samples from a sample depth of 3 m.

	Lab A	Lab B
Moisture content, w (%)	24.9	–
Liquid limit, LL (%)	76	82
Plasticity index, PI (%)	36	40
Clay fraction (%)	35	46
Expansive potential (Van der Merwe, 1964)	Very high	Very high

3.2 *Monitoring layout*

Two separate test areas for the large-scale instrumented piles and soil monitoring were installed. These were separated into a 'wet' and 'dry' site which were located approximately 50 m apart. A berm was constructed around the 'wet site', and water was introduced from above and radially through infiltration wells. The wet site was kept fully submerged under water for the first six months of the monitoring period, prior to pile testing. This allowed the clay to saturate and swell. The 'dry site' was kept at natural moisture conditions, and thus remained unsaturated throughout the monitoring period. Borehole BH1 in Figure 2 shows the profile immediately adjacent to the wet site, while BH2 corresponds with the dry site.

Heave and shrinkage of the profile were quantified through the examination of levelling data, with surveys conducted periodically on a number of points. The surveyed points included beacons on the ground surface (marked 'bmd' for the dry site and 'bmw' for the wet site) and tell-tale rods, which were installed to depths of 2 m, 4 m and 6 m to provide an indication of subsurface movement and swell with depth within the profile. The layouts of these surveying points at the dry and wet sites are given in Figure 3. A weather station and rain gauge, located between the two sites, were installed to record rainfall and temperature readings throughout the monitoring period.

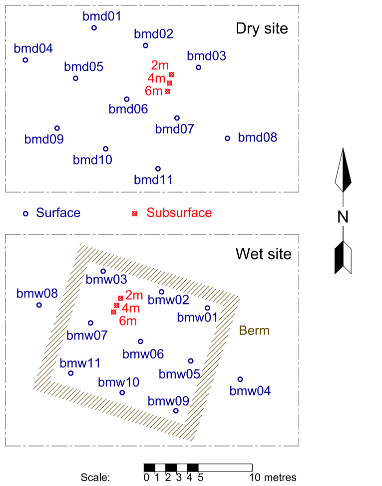

Figure 3. Levelling points used for surveys. Note that the relative position of the two sites to each other is not represented.

4 IN SITU MONITORING RESULTS

Moisture ingress and evaporation are affected by wind, rainfall and temperature. The latter two have been plotted in Figure 4 (a), along with the survey dates and the flooding commencement date. The closest South African Weather Service station is the Vredefort station, 29 km from the field site. The mean annual precipitation (MAP) for the station is 638 mm (Smithers & Schulze 2003). The rain gauge on site recorded 645 mm over the year of monitoring, thus the period was representative of normal rainfall for the area.

The swelling in the wet site was governed by the infiltration of water as the site was flooded and kept submerged for a period of six months (up to the end of September). The surface movement at the location of the tell-tale rods, for purposes of the analysis, was taken as the average across the three beacons closest to the subsurface readings for both the dry site and wet site. Soil movement at each of these points at the dry site and wet site are given in Figure 4 (b) and (c) respectively.

The semi-arid climate with seasonal rainfall is evident from the rainfall record, with a long dry season between May and October (winter months) and concentrated rain spells between December and March (summer months). Table 2 gives the variation in surface movement readings across all beacons at both sites, at the point of maximum swell. It shows high variability of surface movement, especially at the dry site, which would imply highly variable subsurface movement.

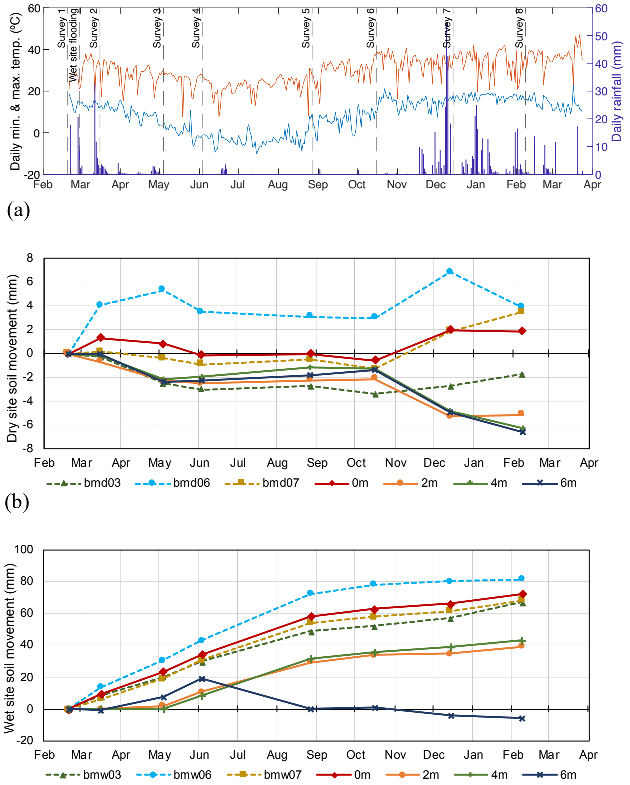

Figure 4. (a) Rainfall and ambient temperature, (b) dry site surveyed soil movement and (c) wet site surveyed soil movement (note different vertical axis scales).

Table 2. Surface measurement variation at point of ultimate swell

Surface movement	Dry site	Wet site
Mean at tell-tales (mm)	2.0	72.1
Mean of all beacons (mm)	–1.8	69.9
Minimum of all beacons (mm)	–7.4	51.6
Maximum of all beacons (mm)	6.8	85.7
Standard deviation	4.2	11.9
Coefficient of variation	–2.38	0.17

4.1 *Discrete layer swell*

The layer deformation of three discrete layers (0–2 m, 2–4 m and 4–6 m, as governed by the depths of the subsurface measurements) was calculated by subtracting the movement of the lower boundary of the layer from that of the upper boundary, thus obtaining the change in layer thickness. The swell or shrinkage, expressed as vertical strain (ε_v) of the layer, is the change in layer thickness (ΔH) divided by the initial layer height (H_0), where $H_0 =$ 2 m for each layer. Positive vertical strain indicates swell, whereas negative strain indicates shrinkage.

$$\varepsilon_v = \frac{\Delta H}{H_0} \tag{1}$$

Figures 5 and 6 show the development of vertical layer strains over time for the dry site and wet site respectively. The vertical strain of the overall expansive profile was determined by dividing the average surface movement at the tell-tales (i.e. ΔH for 0 m) by the assumed thickness of the potentially expansive layer (4.15 m and 7.00 m for the dry and wet sites respectively). Layer swell regimes throughout the soil may be different than what is seen at these single locations, as seen in Table 2.

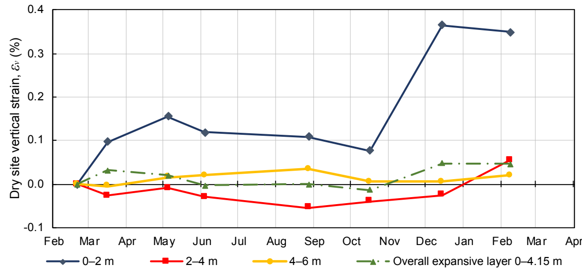

Figure 5. Dry site layer swell under natural moisture conditions.

Ground strains in the dry site were governed solely by the natural moisture conditions of the soil, and therefore the swell magnitudes were lower than those of the submerged wet site. A distinct increase in swell is evident during the wet season, especially in the uppermost layer.

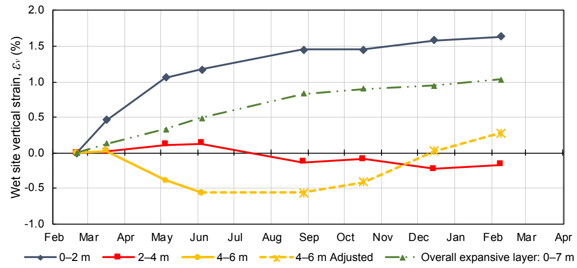

Figure 6. Wet site layer swell under submerged conditions.

The 6 m rod in the wet site appears to have been disturbed between the fourth and fifth surveys (conducted in June and August 2020). The 4–6 m swell profile has therefore been adjusted accordingly, under the assumption that the layer swell remained constant between the surveys, seeing as the layer exhibited a shrinking trend prior to the disturbance and a swelling trend after. The later swelling of the lowest layer may be due to the slow ingress of moisture into the layer due to low hydraulic conductivity. Initially, some compression appears to have occurred under the increased vertical total stress due to porewater entering the upper layers, while swelling only occurred once moisture infiltrated the lower layers of the profile.

The 2–4 m layer lies in a "strongly shattered and slickensided" clay stratum according to the borehole log in Figure 2. The more prominent fissuring in the macro-structure of this layer compared to that of the layers above and below may have resulted in a reduced ability to retain water, which would justify the lower observed swelling strain magnitudes.

4.2 *Soil heave prediction*

The heave occurring at the wet site due to swelling, determined at each depth, was compared to two heave prediction methods which might be used in practice.

The Van der Merwe (1964) method for heave prediction was determined empirically from a dataset of case studies on expansive soils in Southern Africa. The method involves defining the expansive potential of the soil profile, as in Table 1. Based on the clay's expansive potential, a unit heave (PE) is designated for each expansive layer. The proportion of expected surface heave occurring at a given depth (D) is determined by a logarithmic relationship given in Equation 2. The predicted heave is given by Equation 3.

$$D = -k \cdot \log F \tag{2}$$

$$\text{Heave} = PE \cdot \sum_i H_i \cdot F_i = PE \cdot \sum_i H_i \cdot 10^{-D_i/k} \tag{3}$$

$D_i =$ depth below ground surface of layer centroid
$F_i =$ factor accounting for overburden stress
$H_i =$ thickness of layer
$k =$ 20 if D is in ft., thus $k =$ 6.096 if D is in metres
$PE =$ unit heave based on expansive potential

Jones (2017) suggested an adjustment to the Van der Merwe method, where the unit heave factor should take into account the change in water content (Δw) during swelling. Jones proposed that in the absence of laboratory data on Δw, either engineering judgement and knowledge of the site may be used, or the equilibrium moisture content may be estimated as half of the liquid limit of the soil as per Weston (1980). The change in water content is then given by Equation 4, where w_0 is the initial moisture content. A change in moisture content of $\Delta w =$ 14.5% was assumed. Table 3 gives the unit heave for each method.

$$\Delta w = 2LL - w_0 \quad (4)$$

Table 3. Unit heave based on expansive potential

	Medium	High	Very high
PE (Van der Merwe 1964)	0.0208	0.0417	0.0833
PE (Jones 2017)	0.125 Δw	0.250 Δw	0.500 Δw
PE if $\Delta w =$ 0.145 (Jones 2017)	0.0181	0.0363	0.0725

The Weston (1980) method was not considered, as the heave equation raises the liquid limit (LL) to a power of 4.17. Jacobsz & Day (2008) found a 53% to 78% variation in LL across commercial laboratories in South Africa, which brings into question the confidence in the parameter. The magnitude of error associated with slight deviations in LL when using the Weston (1980) prediction method was discussed by Gaspar (2019). Brackley (1980) proposed a method based on PI and soil suction. For a clay element in a fissured profile, greater suctions exist towards the drier edges than in the wetter intact centre. A representative suction is needed to make accurate predictions, of which published guidance is not available (Gaspar 2020).

The progression of wet site heave measurements as swelling occurred over time is plotted in Figure 7. The maximum heave at each depth may be compared to the predicted heave from the described methods. The heave at 4 m is slightly underpredicted by both Jones (2017) and Van der Merwe (1964) for a very highly expansive clay. The heave at 2 m and 0 m are significantly overpredicted by both methods assuming very high expansive potential, and a better fit is obtained with the profiles for medium and high expansive potential. This suggests that the upper strata of the profile are less expansive than what was determined from the initial indicator tests.

Table 4 gives the measured and predicted total heave at the ground surface, and the associated prediction errors. Positive errors are overpredictions. It is worthwhile to note that the methods significantly overestimated the heave, even though the site was fully submerged for several months and would thus be expected to give a worst-case scenario for soil expansion.

Table 4. In situ measured and predicted surface heave.

Expansive potential & method	Heave (mm)	Error (%)
Measured at ground surface	72.1	–
Very high (Van der Merwe 1964)	204.9	184
High (Van der Merwe 1964)	102.5	42
Medium (Van der Merwe 1964)	51.2	–29
Very high (Jones 2017)	178.3	147
High (Jones 2017)	89.1	24
Medium (Jones 2017)	44.6	–38

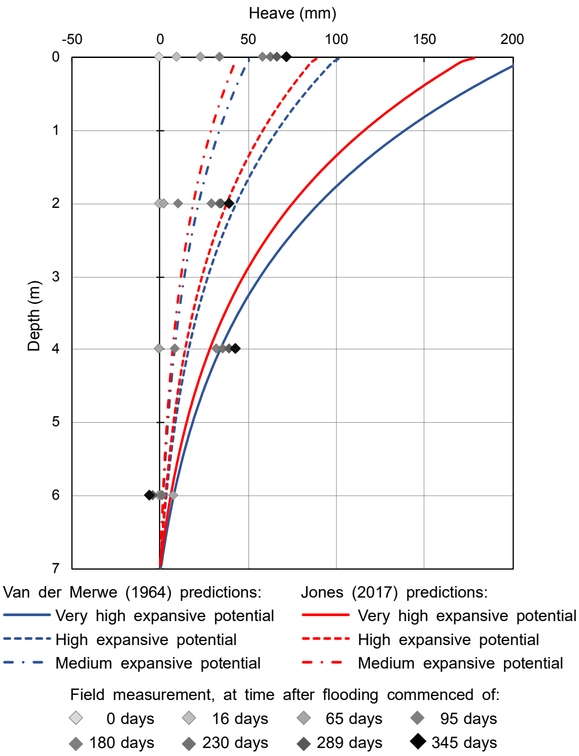

Figure 7. Wet site predicted and measured in situ heave profile.

5 CENTRIFUGE TEST

5.1 *Experimental setup*

Centrifuge modelling of the greenfield swell was used for comparison against the field results. Five 50 mm thick slabs of highly expansive clay were prepared and separated by geotextile drainage boundaries, so as not to restrict lateral flow (Gaspar 2020; Gaspar et al. 2019). The structure of the profile on site was emulated by grating the clay and statically compacting it. This preparation procedure allowed for some degree of fissuring to be maintained in the slabs, increasing the rate of water infiltration.

The model was tested at a centrifugal acceleration of 30 *g* (i.e. scale factor $N =$ 30) in the geotechnical centrifuge at the University of Pretoria (Jacobsz et al. 2014). The model thus corresponds with a 7.5 m thick prototype expansive clay profile, similar to the conditions at the field wet site. Water was introduced from the bottom of the model through a remotely controlled solenoid valve, allowing swell to take place in-flight. Flooding of the model took 30 minutes, which corresponds with a prototype time of 18.75 days (see Table 5). The centrifuge model layout is illustrated in Figure 8.

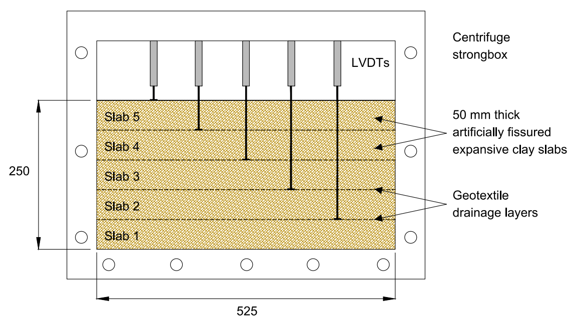

Figure 8. Centrifuge model layout (after Gaspar 2020; Gaspar et al. 2019).

5.2 *Test results*

All results in this paper from the model study are presented in terms of the prototype scale. The applicable centrifuge scaling laws are given in Table 5 (after Wood 2003). Caicedo et al. (2006) found that the time scaling law for one-dimensional consolidation applied to swelling of expansive soils.

Table 5. Applicable centrifuge scaling laws for centrifugal acceleration g-level of N (after Wood 2003)

Parameter	Scaling law (model/prototype)
Distance and displacement	$1/N$
Time during infiltration	$1/N^2$
Time during consolidation/swelling	$1/N^2$
Strain	1

The swell (vertical strain) was calculated using the same approach as that of Section 4.1. The vertical strain of the uppermost three layers is given in Figure 9. The progression of prototype heave at each depth is given in Figure 10, along with the predicted profiles for very high expansive potential as discussed in Section 4.2. The test and predicted maximum surface heave, and associated errors, are given in Table 6.

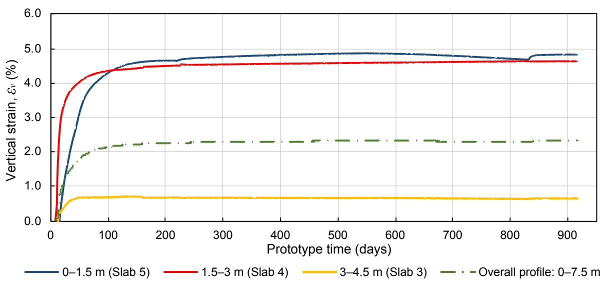

Figure 9. Centrifuge test prototype layer vertical strain over time.

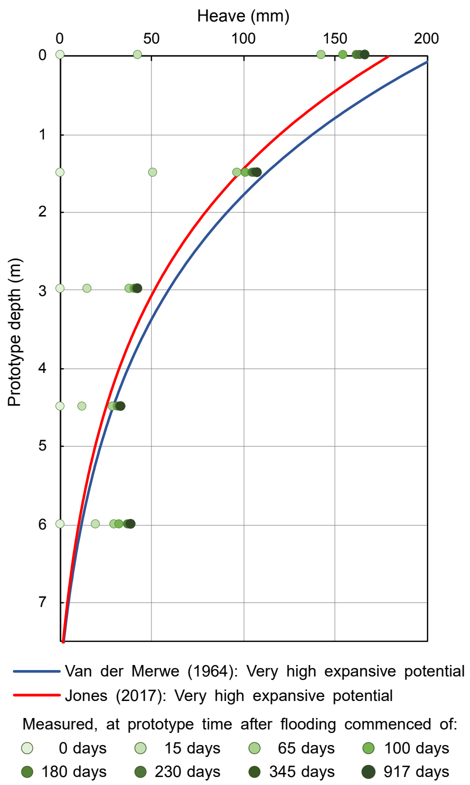

Figure 10. Centrifuge test prototype predicted and measured heave.

Table 6. Centrifuge test prototype measured and predicted surface heave, and prediction error

Expansive potential & method	Heave (mm)	Error (%)
Measured at ground surface	168.2	–
Very high (Van der Merwe 1964)	209.1	24
Very high (Jones 2017)	181.9	8

The modelling of a fissured structure and inclusion of geosynthetic drainage layers allowed rapid ingress of water into the clay layers, allowing swell to occur within a reasonable time frame in the centrifuge. Each layer had reached 95% of its ultimate swell within 5 hours in model time, corresponding with 188 days in prototype time. It is worthy to note that the lower layers reached ultimate swell before the upper layers, due to the fact that water was introduced from the bottom of the model. Also note that some slippage of the LVDT at 6 m may have occurred. Figure 10 and Table 6 show a good correlation at all depths between the centrifuge prototype heave and both prediction methods for very high expansive potential.

5.3 *Comparison against field results*

The centrifuge prototype results were compared to the in situ field results from the wet site. It should be noted that the mechanisms of infiltration in the field and the centrifuge model are different. Introduction of water from below forced all layers to reach maximum swell potential, which may not be true in reality due to evaporation, permeability and drainage conditions. The superior control over infiltration in the laboratory, where an unlimited supply of water was available to flow into a closely fissured soil mass, represents an extreme case of swell. Using four surface swell measurements, Gaspar (2020) found that boundary conditions had insignificant effects on measured swell.

Figure 11 shows the vertical strain of the uppermost layer and of the overall profile for both datasets over time. The top layer and overall profile of the centrifuge prototype exhibited swell magnitudes more than double those in the field. In addition, the progression of swell in situ was more gradual than the increase and flattening of the swell progression in the centrifuge.

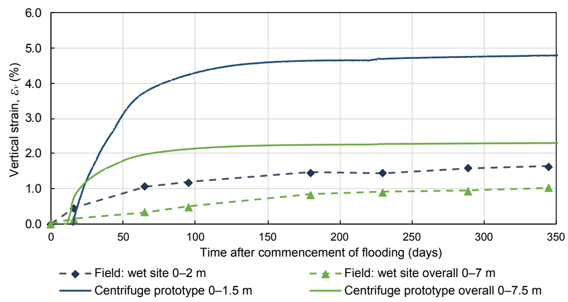

Figure 11. Comparison of in situ and centrifuge swell over time.

Figure 12 shows the associated ultimate heave measurements at every depth. In the lower profile (3 m and below) both the centrifuge prototype and field ultimate heave measurements compared well with the predicted profiles for very high expansive potential. In the upper profile (above 3 m), the centrifuge prototype exhibited significantly greater heave than the field test site, and the predictions for very high expansive potential correlated well with the centrifuge prototype, whereas the predictions for medium to high expansive potential were a better fit for the field test. In the centrifuge test, the same clay and structure could be simulated throughout the model, and thus similar expansive behaviour was evident with depth. The expansive behaviour of the in situ soil is more variable with depth, despite the fact that the expansive profile comprises of a fairly uniform fissured alluvial clay deposit throughout.

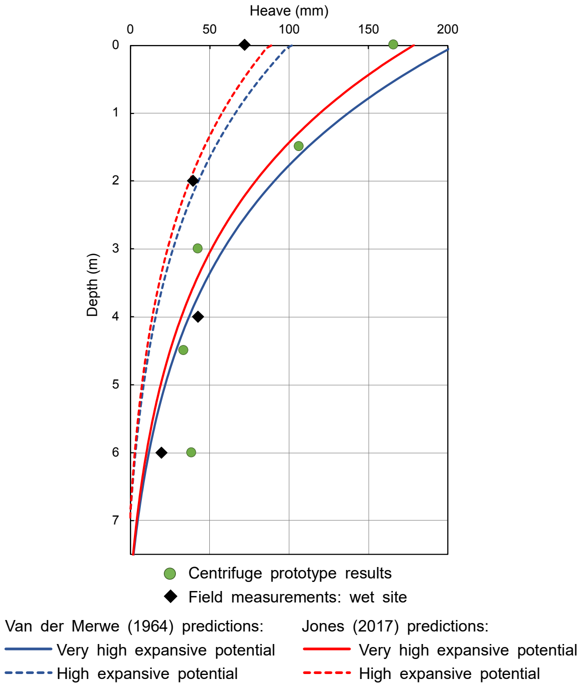

Figure 12. Centrifuge prototype and in situ maximum heave measurements, compared with predicted profiles.

6 CONCLUSIONS

The following conclusions were drawn from this study:

- In situ swelling and shrinkage of an unsaturated expansive clay profile showed a distinct correlation with seasonal climatic conditions.
- The expansive behaviour of the natural soil profile (based on empirical prediction methods) was observed to be variable, despite the fact that the composition, origin and structure of the material showed little variation with depth.
- Simulating a fissured macro-structure in the centrifuge model allowed in-flight swell to occur within a reasonable time frame. Swelling occurred more gradually in situ than in the centrifuge test prototype.
- The lower 4 m of the centrifuge prototype profile showed a similar swell response to that of the in situ profile, whereas the upper 3 m exhibited more than twice the vertical strain than that of the field site.
- Empirical methods by Jones (2017) and Van der Merwe (1964) overpredicted ultimate surface heave within 8% and 24% respectively of the centrifuge prototype results, assuming very high expansive potential. Assuming medium to high expansive potential, the respective methods predicted ultimate surface heave within –29 to 24% and –38 to 42% of the in situ monitoring results.

7 ACKNOWLEDGEMENTS

The authors would like to express their gratitude to the UK Engineering and Physical Sciences Research Council (EPSRC) Global Challenges Fund for the financial support under the WindAfrica project, Grant Ref: EP/P029434/1.

Stakeholders from Yellowstar Mining are thanked for their assistance and the use of facilities. Thanks to all contributors from the Universities of Cambridge, Durham and Pretoria for the laboratory work and installation of field instrumentation.

8 REFERENCES

Brackley I.J.A. 1980. Prediction of soil heave from suction measurements. *Proceedings: 7th Regional Conference for Africa on Soil Mechanics and Foundation Engineering, Accra, Ghana,* 1, 159–166. ISSMFE.

Caicedo B., Medina C. and Cacique A. 2006. Validation of time scale factor of expansive soils in centrifuge modelling. *Proceedings: 6th International Conference on Physical Modelling in Geotechnics, Hong Kong, 4–6 August 2006*, 273–277.

da Silva Burke T.S., Jacobsz S.W., Elshafie M.Z.E.B. and Osman A.S. 2021. Measurement of pile uplift forces due to soil heave in expansive clays. *Submitted*.

Das B.M. 2006. *Principles of geotechnical engineering*. 6th ed. Thomson Learning.

Diop S., Stapelberg F., Tegegn K., Ngubelanga S. and Heath L. 2011. *A review of problem soils in South Africa*. Council for Geoscience Report Number: 2011-0062.

Fredlund D.G., Rahardjo H. and Fredlund M.D. (2012). *Unsaturated Soil Mechanics in Engineering Practice*. John Wiley & Sons.

Gaspar T.A.V. 2019. A comparison of swell prediction methods to centrifuge testing. *7th African Young Geotechnical Engineers Conference, Cape Town, South Africa, 7–10 October 2019.* ISSMGE.

Gaspar T.A.V. 2020. *Centrifuge modelling of piled foundations in swelling clays*. PhD thesis. University of Pretoria.

Gaspar T.A.V., Charlton T.J., Osman A.S., Augarde C.E., Coombs W.M., Moghaddasi H., Jacobsz S.W., Smit G., da Silva Burke T.S., Murison R.A., Kearsley E.P., Biscontin G., Haigh S.K., Al-Haj K.M., Elshafie M.Z.E.B., Elarabi H., Elsharief A.M., Zein A.K.M, Abdelatif A.M.O. and Rimoy S.P. 2022. Investigating the design of piled foundations for wind turbines in swelling clays. *Proceedings: 20th International Conference on Soil Mechanics and Geotechnical Engineering, Sydney, Australia, 1–5 May 2022.* ISSMGE.

Gaspar T.A.V., Jacobsz S.W., Smit G. and Osman A.S. 2019. An expansive clay for centrifuge modelling. *Proceedings of the 17th Regional European Conference on Soil Mechanics and Geotechnical Engineering, Reykjavik, Iceland, 1–6 September 2019.* ISSMGE.

Jacobsz S.W. and Day P.W. 2008. Are we getting what we pay for from geotechnical engineering laboratories? *Civil Engineering,* 8–11.

Jacobsz S.W., Kearsley E.P. and Kock J.H.L. 2014. The geotechnical centrifuge facility at the University of Pretoria. *Proceedings: 8th International Conference on Physical Modelling in Geotechnics, Perth, Australia, 14–17 January 2014*, 169–174. CRC.

Jones D.E. and Holtz W.G. 1973. Expansive soils – the hidden disaster. *Civil Engineering*, 43(8), 49–51.

Jones G.A. 2017. An empirical preliminary prediction of heave. *Journal of the South African Institution of Civil Engineering*, 59(4), 64–66.

Jones L.D. and Jefferson I. 2012. Chapter C5 – Expansive soils. *In: Institution of Civil Engineers: Manuals series*. 413–441. ICE Publishing.

Miao L., Wang F., Cui Y. and Shi S.B. 2012. Hydraulic characteristics, strength of cyclic wetting-drying and constitutive model of expansive soils. *Proceedings: 4th International Conference on Problematic Soils, Wuhan, China*, 303–322.

Nelson J.D., Chao K.C., Overton D.D. and Nelson E.J. 2015. *Foundation engineering for expansive soils*. John Wiley & Sons.

Smithers J.C. and Schulze R.E. 2003. *Design Rainfall and Flood Estimation in South Africa*. WRC Report 1060/1/03, Water Research Commission, Pretoria, RSA, 156.

Van der Merwe D.H. 1964. The prediction of heave from the plasticity index and percentage clay fraction of soils. *The Civil Engineer in South Africa*, 6(6), 103–107.

Weston D.J. 1980. Expansive roadbed treatment for Southern Africa. *Proceedings of the 3rd International Research and Engineering Conference on Expansive Clays*, 1, 339–360.

Williams A.A.B., Pidgeon J.T. and Day P.W. 1985. Problem Soils in South Africa – State of the Art. *The Civil Engineer in South Africa*, 27(7), 367–377.

Wood D.M. 2003. *Geotechnical modelling*. CRC press.