Proceedings of the XVII ECSMGE-2019
Geotechnical Engineering foundation of the future
ISBN 978-9935-9436-1-3

doi: 10.32075/17ECSMGE-2019-0421

XVII European Conference
ECSMGE 2019
Reykjavik
on Soil Mechanics
and Geotechnical Engineering

An expansive clay for centrifuge modelling

L'argile expansive pour modélisation centrifuge

T.A.V. Gaspar
University of Pretoria, Pretoria, South Africa

S.W. Jacobsz, G. Smit
University of Pretoria, Pretoria, South Africa

A. S. Osman
Durham University, Durham, UK

ABSTRACT: This paper explores the development of an expansive soil which can be repeatedly reproduced for use in centrifuge models to investigate soil-structure interaction problems involving expansive soils. The study considered two materials, namely a sand-bentonite mixture and a naturally occurring, highly expansive clay. For the natural material, two approaches were explored to create a scaled down fissured structure for use in centrifuge testing. The swell behaviour of the two candidate materials was first investigated by means of oedometer tests and then by centrifuge modelling. The centrifuge tests consisted of layers of compacted clay slabs separated by free draining layers. The study revealed that while the sand-bentonite mixture possessed the potential to swell significantly, the time required to do so was impractical for centrifuge studies. It was however found that the approach used to create a scaled down fissured structure in the naturally occurring clay facilitated rapid ingress of water which allowed for significant heave to take place in a much shorter time frame. The results obtained from the centrifuge test conducted on the reworked clay were compared with an empirical heave prediction method and it was found that the swell obtained from the fissured clay matched the predicted heave profile within three hours.

RÉSUMÉ: Cet article explore le développement d'un sol expansif qui peut être reproduit à plusieurs reprises pour être utilisé dans des modèles de centrifugeuses afin d'étudier les problèmes d'interaction sol-structure impliquant des sols expansifs. L'étude a examiné deux matériaux, à savoir un mélange de sable et de bentonite et une argile naturelle très expansive. Pour le matériau naturel, deux approches ont été explorées pour créer une structure fissurée réduite pour des essais en centrifugeuses. Le gonflement des deux matériaux a été investigué par des tests de l'œdomètre suivis par une série de tests de centrifugation. Le test de centrifugation consistait en des couches de dalles d'argile compactées séparées par des couches drainantes. L'étude a révélé que, même si le mélange de sable et de bentonite avait le potentiel de gonfler de manière importante, le temps requis pour le faire n'était pas pratique pour cette présente étude. Il a toutefois été constaté que l'approche utilisée pour créer une structure fissurée réduite dans l'argile naturelle facilitait l'entrée rapide de l'eau, permettant ainsi un soulèvement important dans un délai beaucoup plus court. Les résultats du test de centrifugation effectué sur l'argile retravaillée ont été comparés à une méthode empirique de prévision du gonflement et il a été constaté que le profil de l'argile fissurée correspondait au profil de gonflement prédit en trois heures.

Keywords: swelling clays, expansive soils, unsaturated soils, centrifuge modelling

1 INTRODUCTION

Of all geotechnical engineering problems encountered in practice, perhaps the most costly have been those related to expansive soils. Jones & Holtz (1973) stated how in the United States, damages to structures caused by swelling/shrinking soils amounted to \$ 2.3 billion, more than twice that caused by hurricanes, tornadoes and earthquakes combined. It has since been reported that this value has increased to \$ 13 billion per year (Puppala & Cerato, 2009). The occurrence of these problem soils is by no means confined to the USA, but have resulted in equally devastating consequences around the world, providing the impetus for several symposia and conferences specifically addressed to this topic, from the First Symposium on Expansive clays in South Africa in 1957 to the 7th International Conference on Expansive clays in Dallas, Texas in 1992.

Recognising the significant economic implications of this problem soil, numerous full scale (Brackley & Sanders, 1992; Blight, 1984, Meintjes, 1991; Meintjes & Pellissier, 1994) and laboratory studies (Al-Haj & Standing, 2015) have been conducted to understand the mechanisms at play. It is also attractive to investigate expansive soil problems by means of physical modelling and in recent years an increasing number of studies have utilised centrifuge modelling in the study of expansive clays (Amenuvor et al., 2018; Gu et al., 2010; Laporte et al., 2018; Liu & Vanapalli, 2007).

Large parts of Africa are underlain by thick deposits of expansive clay. The worldwide drive towards deployment of renewable energy resources has resulted in a need to construct wind farms in some of these areas. Examples include central Sudan and northern Tanzania. Due to the tight tolerances prescribed by wind turbine manufacturers in terms of allowable foundation movements, it is generally problematic to construct suitable wind turbine foundations on deep expansive soils.

The WindAfrica research project (http://community.dur.ac.uk/wind.africa/), which has recently been funded by the UK Engineering and Physical Sciences Research Council (EPSRC), aims to develop practical guidelines for the design of wind turbine foundations in expansive soils. The project scope includes large scale field testing complemented by centrifuge model tests, numerical modelling and laboratory characterisation of unsaturated expansive soils.

The WindAfrica centrifuge modelling requires the use of an expansive soil that can swell in a reasonable time frame in the centrifuge to allow the effects of various parameters on the performance of the foundation to be studied.

In the small number of centrifuge model studies on swelling clays reported in the literature, swell is generally not induced in-flight due to the considerable amount of time required for water to infiltrate the soil and induce a significant amount of swell. A typical characteristic of expansive clay minerals such as montmorillonite is their extremely low hydraulic conductivity. In spite of this, these materials generally occur in the field in a highly fissured state. It is these fissures that facilitate the migration of water into the soil profile which ultimately produces the observed swell. This presents a difficulty as the highly fissured structure is difficult to capture in the laboratory. If samples are reconstituted from a slurry and compacted to a predetermined density, this fabric will be lost. Similarly, any fissures present in an undisturbed sample would not provide representative field behaviour if the material is used in a scale model as they would likely be too far apart. Failure to capture macroscopic fissures makes laboratory measurements unreliable for time predictions of mechanisms governed by hydraulic conductivity. This point was emphasised by Clayton et al. (1995) where gross overestimations of the coefficient of consolidation were reported for laboratory test methods.

This study describes trials to develop an expansive soil for use in centrifuge modelling with the ability to swell in a reasonable time frame. Development of such a clay has then been used by the authors in a parallel study to investigate the effect of soil expansion on piled foundations (Smit et al., 2019).

2 EXPERIMENTAL PROCEDURE

2.1 *Sample preparation*

The expansive properties of two different materials were considered, namely a mixture containing 70% uniformly graded silica sand and 30% sodium bentonite, as well as a naturally occurring high plasticity clay. Selected material properties are included in Table 1, with the particle size distributions presented in Figure 1.

Table 1. Material properties

Property	Sand-bentonite	Natural clay
Liquid limit (%)	96	92
Plasticity index	75	55
Linear shrinkage (%)	32.5	25.5
Activity	4.41	0.8
Specific gravity	2.69	2.65
In situ bulk density (kg/m^3)	-	1805
In situ moisture content (%)	-	35.03
Unified Soil Classification	CH	CH

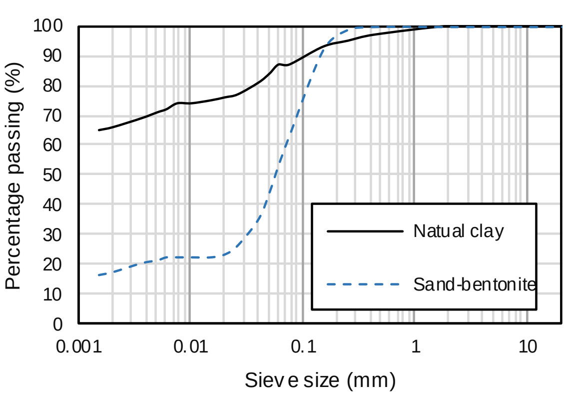

Figure 1. Particle size distribution curves

For tests conducted on the sand-bentonite mixture, the following sample preparation procedure was implemented. Dry sand and bentonite powder at a hygroscopic moisture content of 13.2% were mixed. Once uniformity had been obtained, distilled water was added to bring the mixture to its optimum moisture content of 15.2%. Aggregations of soil exceeding 4.75 mm were broken down and the resulting material was placed in a sealed container in a temperature controlled room for a period of two weeks prior to testing. This allowed for equilibration of moisture to be achieved. Following the equilibration period, samples were prepared at the Proctor density of the material determined according to ASTM D698-12 (2012). This density was targeted due to the fact that at lower densities, sand-bentonite mixtures have been shown to exhibit collapse upon inundation rather than heave (Dineen et al. 1999). The targeted density was achieved by means of static compaction. The static compaction method was selected as it has been reported to reliably produce multiple samples at similar densities (Booth, 1976; Whitman, 1970).

For testing of the undisturbed clay, two approaches were explored to achieve a scaled-down fissured structure. The first approach involved breaking down lumps of intact clay at the in-situ moisture content by hand such that the resulting aggregations passed the 4.75 mm sieve. The resulting broken-down material was then statically compacted to its in-situ bulk density at the in-situ moisture content. The second approach to breaking down the intact clay involved grating the clay to a uniform size rather than performing the breaking down by hand. An illustration of the two soils prior to compaction is presented in Figure 2.

2.2 *Oedometer tests*

As an initial indication of the magnitude of swell and the time required for swell to be achieved, swell tests were conducted under load in the oedometer on the different material types according to ASTM D4546-96 (1996). Initial tests were conducted at an overburden pressure of

50 kPa due to the fact that free-swell tests, i.e. swell tests under no additional overburden provide variable results. This would have resulted in difficulties when comparing the different sample preparation methods.

Figure 1. Broken vs grated clay structure prior to compaction

Due to the time required for the sand-bentonite mixture to achieve full swell, only the swell test under 50 kPa overburden was needed to obtain an estimate of the magnitude of swell as well as the time required to achieve full swell. For the natural clay, swell tests were conducted at 50 kPa and 100 kPa overburden pressure.

2.3 *Centrifuge tests*

In addition to the oedometer swell tests, centrifuge tests were conducted using the 150 g-ton geotechnical centrifuge at the University of Pretoria (Jacobsz et al. 2014) to examine the expansive properties of the candidate materials. The models presented here involved the testing of five 50 mm clay slabs separated by drainage layers comprising of a needle punched non-woven geotextile. In later tests, a geotextile-sand combination was used to increase the rate of infiltration between the slabs of soil.

Once the model had achieved the target centripetal acceleration of 30 g and initial settlement had stopped, water was introduced into the strongbox via two water wells adjacent to the clay layers. Miniature extensometers and digital image correlation (DIC) were utilised to monitor the swell in each layer, with moisture content probes being used to monitor the ingress of moisture into the system. A diagram of the model setup has been included in Figure 3.

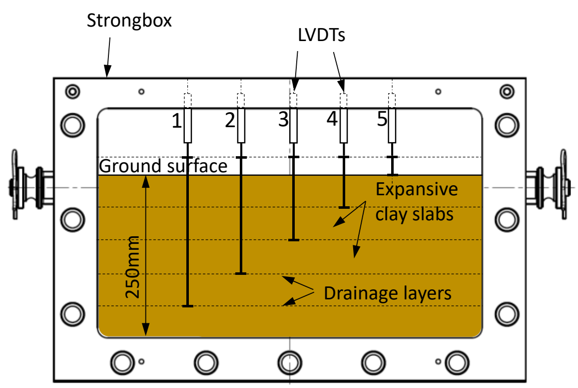

Figure 3. Centrifuge model

3 RESULTS

3.1 *Oedometer swell tests*

The results presented in this section served to provide an initial estimate of the time required for swell to take place, as well as the total amount of swell that could be expected upon inundation. Figure 4 presents the results of a swell test conducted for the sand-bentonite mixture under an overburden pressure of 50 kPa. From this result, it can be seen that while the soil mixture did produce a significant amount of swell, the time required for swelling to occur amounted to several weeks.

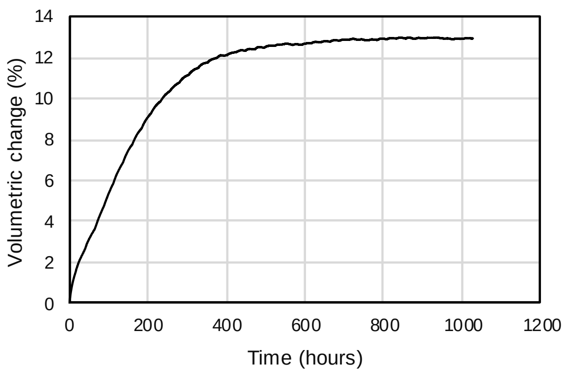

Figure 4. Swell under 50 kPa (sand-bentonite mixture)

Figure 5 and Figure 6 present the results of swell tests conducted on the natural clay at overburden pressures of 50 kPa and 100 kPa respectively. The results illustrate a comparison of tests conducted on undisturbed samples as well as on material broken down and compacted to the in-situ bulk density as described in Section 2.2. The significant differences in total swell between the undisturbed and reworked samples can be attributed to the fact that the undisturbed samples had higher initial moisture contents compared to the reworked samples.

Figure 5 and Figure 6 illustrate that the natural clay produced total heave on par with that achieved by the sand-bentonite mixture in a tenth of the time. Furthermore, the results illustrate that the two methods used to break the clay down before compaction produced similar swell characteristics (magnitude and duration required to achieve swell). For this reason, it was decided that for the centrifuge test conducted on the natural clay, the material would be grated to a uniform size prior to compaction, rather than breaking it down by hand. In addition to producing the desired swell characteristics, the 'grating' approach is significantly faster which led to less moisture being lost during sample preparation.

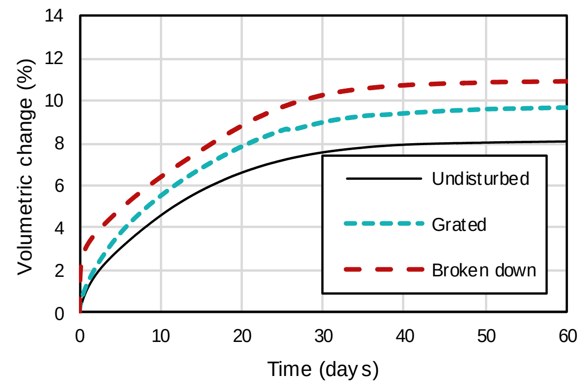

Figure 5. Swell under 50 kPa (natural clay)

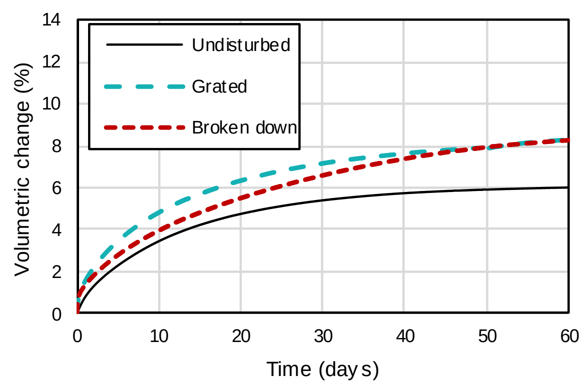

Figure 6. Swell under 100 kPa (natural clay)

3.2 Centrifuge tests

3.2.1 Sand-bentonite mixture

Tests were initially performed with only a layer of geotextile separating successive clay slabs. Figure 7 illustrates that after approximately 20 hours the total measured heave for all five layers amounted to only 0.5 mm (0.2%). It was suspected the slow rate of swell could be due to compression of the geotextile layers inhibiting moisture ingress. For this reason, the test was stopped after 20 hours and drainage between clay slabs was improved by adding a clean uniformly graded sand to form geotextile-sand-geotextile drainage layers. The results of the centrifuge model with modified drainage have been included in Figure 8.

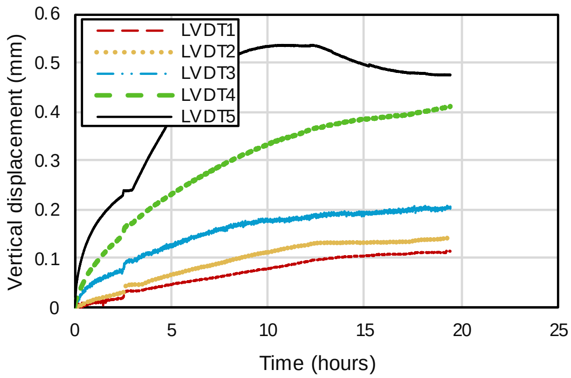

Figure 7. Vertical displacement per layer for sand-bentonite mixture (geotextile drainage only)

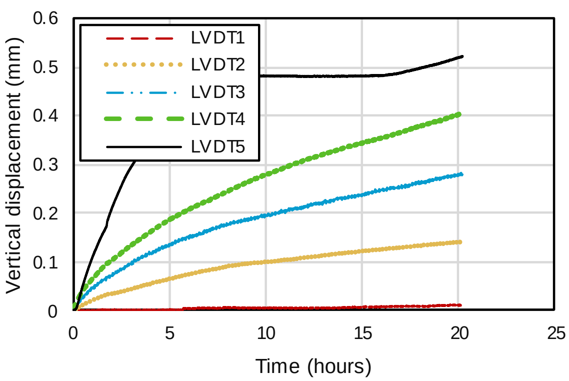

Figure 8. Vertical displacement per layer for sand-bentonite mixture (geotextile sand drainage)

The results presented in Figure 8 illustrate almost precisely the same trend shown in Figure 7. For a period of 20 hours, the total measured swell remained at approximately 0.5 mm, illustrating that drainage paths were not cut-off when only geotextiles were used. The slow rate of swell observed for both test configurations was simply due to the low hydraulic conductivity of the sand-bentonite mixture. At the end of the 20 hour period presented in Figure 8, the centre slab was retrieved and several moisture content readings were taken at different positions to determine the moisture variation in the slab. These measurements illustrated that significant increases in moisture content (~10%) were only obtained on the outer 10 mm of the slab cross-section. Moisture content measurements closer to the centre revealed only a 1.3% increase in moisture from the slab's as-compacted state. These results illustrate that, upon inundation in the centrifuge, moisture ingress is confined to the outer portion of the slabs which then inhibits any signifant heave from occuring in a practical time period.

3.2.2 Natural clay

Figure 9 illustrates swell as a function of time for the centrifuge test conducted on natural clay. Similar to what was observed from the oedometer swell tests presented in Section 3.1, Figure 9 illustrates a significantly higher rate of swell compared to that observed for the sand-bentonite mixture. After 20 hours, the total measured swell amounted to 13 mm (5.2%), and continued to steadily increase for the remainder of the test. The practical implications of the amount of swell obtained from the centrifuge model was assessed by means of an empirical method proposed by Van der Merwe (1964). Being so widely used in engineering practice in South Africa, this approach to heave prediction was used to compare the measured swell in the centrifuge model to what would typically be expected at full scale for this soil type. According to Van der Merwe (1964), the soil classifies as having a very high potential expansiveness based on its plasticity index and clay fraction.

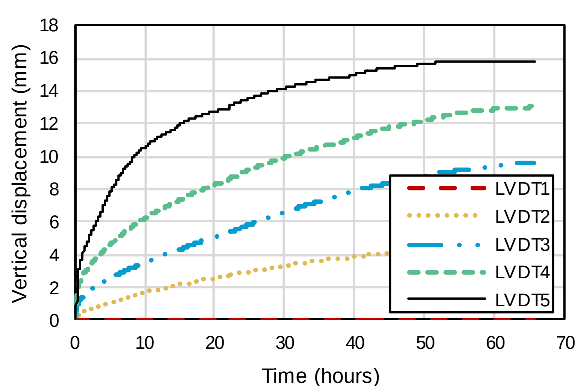

Figure 9. Vertical displacement per layer for natural clay

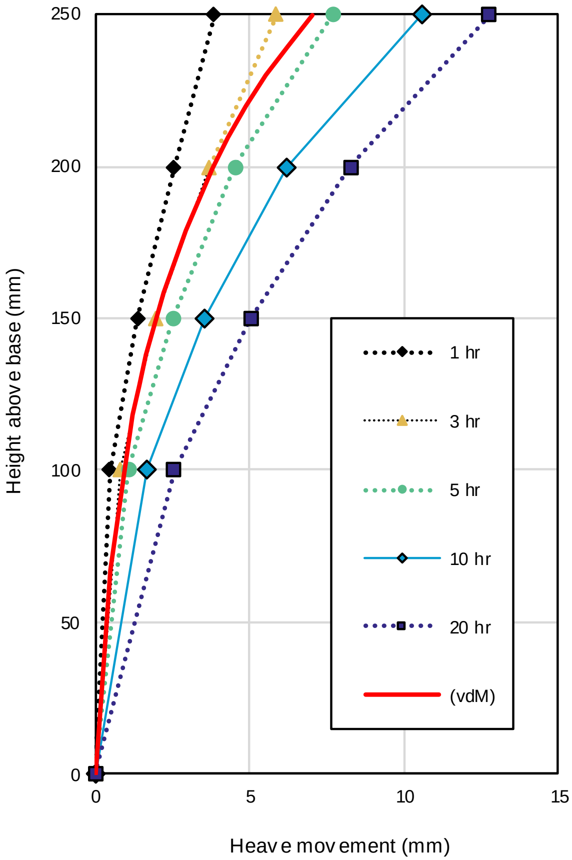

Figure 10. Measured/predicted heave along model depth

Figure 10 illustrates the amount of swell measured along the depth of the centrifuge model at various instances in time. Also included in these results is the Van der Merwe heave prediction for the soil profile. From Figure 10 it can be seen that the swell profile measured throughout testing closely resembles the empirically predicted heave profile. It is also useful to note that the swell predicted by Van der Merwe (1964) was achieved within three hours of testing.

4 CONCLUSIONS

The development of an expansive clay for centrifuge modelling was explored. The study considered a sand-bentonite mixture and a reworked naturally occurring clay. While the sand-bentonite mixture was found to possess significant swell potential, the time required for a notable amount of heave to take place was deemed impractical for centrifuge modelling.

A preparation method whereby a naturally occurring clay was grated at its in-situ moisture content and then statically compacted back to its in-situ bulk density was found to produce samples with favourable swell characteristics (i.e. a significant amount of swell in a short time period). Results of the centrifuge swell test conducted on a natural clay were compared with an empirical swell prediction model proposed by Van der Merwe (1964) and it was found that the reworked clay profile achieved the Van der Merwe heave prediction within three hours. It can therefore be stated that the method used to create a scaled fissured structure can be used to obtain swell magnitudes typically encountered in practice in a short time period.

5 ACKNOWLEDGEMENTS

The authors gratefully acknowledge the financial support from the UK Engineering and Physical Sciences Research Council (EPSRC) Global Challenges Fund under the Wind Africa project, Grant Ref: EP/P029434/1

6 REFERENCES

Amenuvor, A.C., Li, G-W., Hou, Y-Z. & Chen, W. 2018. Shrinkage Cracking in Physical Model of Undisturbed Expansive Clay Slope subjected to Wet-Dry Cycles, *Proceedings of the 7th International. Conference on Unsaturated Soils, Hong Kong.*

Al Haj, K. M. A., Standing, J. R. 2016. Soil water retention curves representing two tropical clay soils from Sudan. *Géotechnique* 66(1), 71–84.

ASTM D698-12e2. 2012. Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12 400 ft-lbf/ft^3 (600 kN-m/m^3)), ASTM International, West Conshohocken, PA.

ASTM D4546-14e1. 2014. Standard Test Methods for One-Dimensional Swell or Collapse of Soils, ASTM International, West Conshohocken, PA.

Booth, A.R. 1976. Compaction and preparation of soil specimens for oedometer testing. *Soil specimen preparation for laboratory testing. ASTM STP 599. American Society for Testing and Materials*, 216-228.

Brackley, I.J.A., Sanders, P.J. 1992. In situ measurement of total natural horizontal stresses in an expansive clay. *Géotechnique. 42(3)*, 443-451.

Blight, G.E. 1984. Power station foundations in deep expansive soil. *International Conference on Case Histories in Geotechnical Engineering*. Missouri University of Science and Technology, USA.

Clayton, C.R.I., Matthews, M.C., Simons, N.E. 1995. *Site investigation: A handbook for engineers*, Oxford, GB. Blackwell Science.

Dineen, K., Colmenares, A., Ridley, A.M., Burland, J.B., 1999. Suction and volume changes of a bentonite-enriched sand. *Proceedings of the Institution of Civil Engineers: Geotechnical Engineering.*

Gu, X.W., Zhang, W.M., Xu, G.M. 2010. Earth pressure at rest of expansive soil against retaining wall. *Physical Modelling in Geotechnics*,443-448.

Jacobsz, S.W., Kearsley, E.P., Kock, J.H.L. 2014. The geotechnical centrifuge facility at the University of Pretoria. *Physical modelling in Geotechnics: Proceedings 8th ICPMG* (Eds: Gaudin, C. & White, D.), 169-174. CRC Press.

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

Laporte, S., Siemens, G.A., Beddoe, R.A. 2018. Physical modelling of roads in expansive clay subjected to wetting-drying cycles. *Physical Modelling in Geotechnics*, 175-178.

Liu, Y., Vanapalli, S. 2007. Influence of Lateral Swelling Pressure on the Geotechnical Infrastructure in Expansive Soils. *Journal of Geotechnical and Geoenvironmental Engineering ASCE*. 143(6), 1-19.

Meintjes, H. 1991. A case history of structural distress on heaving clay: Colinda Primary School. *Proceedings of the tenth regional conference for Africa on soil mechanics and foundation engineering*. 99-140.

Meintjes, H., Pellissier, J. 1994. An experimental pile in deep residual expansive clay. *Proceedings of the 13th International Conference on Soil Mechanics and Foundations Engineering.* 487-492.

Puppala, A.J., Cerato, A.B. 2009. Heave Distress Problems in Chemically-Treated Sulfate-Laden Materials. *GeoStrata*. 10(2), 28-32.

Smit, G., Jacobsz, S.W., Gaspar, T.A.V., Osman, A.S. 2019. Centrifuge modelling of pile pull-out tests in expansive soil. *Proceedings of the XVII ECSMGE*, Reykjavik, Iceland

Van der Merwe, D. H., 1964. The prediction of heave from the plasticity index and percentage clay fraction of soils, *The Civil Engineer*.

Whitman, R.V., Roberts, J.E., Man, S. 1960 One Dimensional Compression and Wave Velocity Tests, and Responses of Soil to Dynamic Loads, Report 4, Publ. 106, Soil Engineering Division, Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology, Cambridge, Mass., 1960.