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Unsaturated Soils

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Sols non saturés
General Report of TC 106
Unsaturated soils

Rapport général du TC 106
Sols non saturés

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ABSTRACT: This general report summarises the contributions on unsaturated soil mechanics submitted to the Discussion Session of TC106 – Unsaturated soils – at the 18th International Conference on Soil Mechanics and Geotechnical Engineering held in Paris in September 2013. The thirty-five papers collected under the framework of unsaturated soil mechanics cover a broad spectrum of problems and procedures at varying scales. Much attention is devoted to issues related to experimental techniques and procedures for hydro-mechanical characterisation of unsaturated soils, with special attention to retention behaviour. Swelling, shrinkage and eventually cracking are the processes which seem to capture most of the attention in view of the performance of engineering systems. A few contributions deal with constitutive and numerical approaches, while only a couple of papers introduce unsaturated soil mechanics into engineering practice. While innovative efforts are mainly addressed to experimental techniques in the laboratory, the most challenging issues in future perspective appear to be related to the assessment of unsaturated geotechnical systems in the field, including contaminated soils and mine tailings, besides more traditional applications dealing with compacted soil structures and soil-atmosphere interaction.

RÉSUMÉ : Ce rapport général résume les contributions sur les sols non saturés soumis à la Session de Discussion du TC106 - Sols non saturés - du 18ème Congrès International de Mécanique des Sols et de Géotechnique tenu à Paris en Septembre 2013. Les trente cinq articles collectés couvrent un large spectre de problèmes et de procédures à des échelles variées. Beaucoup d'attention est portée aux aspects concernant les techniques expérimentales et les procédures de caractérisation hydro-mécanique des sols non saturés, avec une attention spéciale pour les propriétés de rétention. Le gonflement, le retrait et la fissuration sont les problèmes qui semblent les plus étudiés dans les applications géotechniques. Quelques contributions concernent les approches constitutives et numériques, alors que seulement une paire d’articles introduit la mécanique des sols non saturés dans la pratique. Alors que des efforts novateurs concernent principalement l’évaluation des techniques expérimentales dans le laboratoire, le défi des prochaines années semble concerner l’évaluation des systèmes non saturés sur le terrain, incluant les sols contaminés et les résidus, en plus des applications plus traditionnelles liées aux structures en sol compacté et aux interactions entre le sol et l’atmosphère.

KEYWORDS: unsaturated soils, laboratory and field testing, hydro-mechanical behaviour, assessment of geo-engineering systems

1 INTRODUCTION.

The number of papers presented to the Discussion Session organised by the Technical Committee TC106 testifies the interest of the geotechnical community in geo-engineering problems related to unsaturated conditions. Papers on a broad spectrum of aspects of unsaturated soils behaviour, coupled hydro-mechanical processes, laboratory developments, field and laboratory experimental techniques, and geotechnical problems have been submitted. Researchers from all the continents contribute to the session, although most of them come from Europe, Asia – with the relevant participation of Japan with six papers – and North America.

The thirty-five papers submitted to the Discussion Session are summarised in Table 1, where a list of selected keywords tries to provide a first glance on the topics which are capturing most of the attention at present. It appears that lot of effort is addressed to the hydraulic characterisation of unsaturated soils, especially for what concerns experimental techniques and procedures for the description of water retention behaviour, at increasing scale, from the laboratory to the field and possibly the regional scale. It might be argued that the first season of unsaturated soil mechanics, in which the attention has been focussed almost exclusively on the role of suction on the mechanical behaviour of unsaturated soils, has come to an end. The contributions presented to this conference suggest that the mutual influence between the hydraulic history, in terms of both suction and a measure of the amount of water retained in the pores, and the strain history of the soil is considered of paramount importance to understand and describe the peculiar features of geo-engineering problems related to unsaturated soils.

The hydro-mechanical behaviour of compacted soils is still under investigation, both in static and dynamic conditions, together with improvement techniques. Theoretical and constitutive approaches are being evaluated as an extension to unsaturated conditions of approaches previously conceived for saturated soils. Coupled thermo-hydro-mechanical finite element formulations are being consistently used, both to assess the performance of new hydro-mechanical models, and to predict the response of engineering systems.

Problems which have accompanied the history of unsaturated soil mechanics, like constructions on expansive soils or slope stability under rainfall infiltration, still deserve some attention. Studying the conditions leading to cracking and proper tracking and modelling of the cracking process is the present challenge for unsaturated soils undergoing significant volume changes, especially in view of climate changes. To this end, the effects of vegetation on the behaviour of upper soil horizons are being studied, to provide simple but effective models for water balance and vertical displacements, accounting for soil properties. Future challenges also come from fields related to environmental geotechnics, like mine tailings and waste repositories, where unsaturated soil mechanics is starting to be exploited in a consistent way.
<table>
<thead>
<tr>
<th>Authors</th>
<th>Country</th>
<th>Title of the paper</th>
<th>Selected Keywords</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ávila</td>
<td>Peru</td>
<td>Evaluation curves SWCC for tropical Peruvian soils</td>
<td>in situ WRC, tropical soils, modelling</td>
</tr>
<tr>
<td>Sugii et al.</td>
<td>Japan</td>
<td>Measurement of unsaturated ground hydraulic properties using a dynamic state soil moisture distribution model</td>
<td>in situ WRC, hydraulic conductivity</td>
</tr>
<tr>
<td>Maček et al.</td>
<td>Slovenia</td>
<td>Extension of measurement range of dew-point potentiometer and evaporation method.</td>
<td>WRC, dew-point potentiometer, evaporation method</td>
</tr>
<tr>
<td>Reis et al.</td>
<td>Brazil</td>
<td>Determination of soil-water retention curve for a young residual soil using a small centrifuge</td>
<td>WRC, centrifuge technique</td>
</tr>
<tr>
<td>Nishimura</td>
<td>Japan</td>
<td>Application of micro-porous membrane technology for measurement of soil-water characteristic curve</td>
<td>WRC, pressure membrane</td>
</tr>
<tr>
<td>Toll et al.</td>
<td>UK</td>
<td>New devices for water content measurement</td>
<td>resistivity, TDR, water content</td>
</tr>
<tr>
<td>Mendes &amp; Toll</td>
<td>UK</td>
<td>Influence of initial water content on the water retention behaviour of a sandy clay soil</td>
<td>WRC, filter paper, scanning curves</td>
</tr>
<tr>
<td>Fredlund &amp; Zhang</td>
<td>Canada</td>
<td>Combination of shrinkage curve and soil-water characteristic curves for soils that undergo volume change as soil suction is increased</td>
<td>WRC, volume change, property functions</td>
</tr>
<tr>
<td>Mukunoki &amp; Mikami</td>
<td>Japan</td>
<td>Study on the mechanism of two-phase flow in porous media using X-ray CT image analysis</td>
<td>X-ray CT, multiphase flow</td>
</tr>
<tr>
<td>Schaefer &amp; Birichmier</td>
<td>USA</td>
<td>Mechanisms of strength loss during wetting and drying of Pierre shale</td>
<td>shale, residual friction, weathering</td>
</tr>
<tr>
<td>Bai et al.</td>
<td>China</td>
<td>Experimental study on effect of initial moisture content of compressive property of compacted loess like silt</td>
<td>compaction, loess silt, compression</td>
</tr>
<tr>
<td>Mavroulou et al.</td>
<td>UK</td>
<td>Hydro-mechanical properties of lime-treated London clay</td>
<td>lime treated, hydro-mechanical behaviour</td>
</tr>
<tr>
<td>Vázquez et al.</td>
<td>Spain</td>
<td>A simplified model for collapse using suction controlled tests</td>
<td>collapse, modelling</td>
</tr>
<tr>
<td>Byun et al.</td>
<td>Korea</td>
<td>Evaluation of void ratio and elastic modulus of unsaturated soil using elastic waves</td>
<td>elastic waves, void ratio, saturation, suction</td>
</tr>
<tr>
<td>Georgetti et al.</td>
<td>Brazil</td>
<td>Small-strain shear modulus and shear strength of unsaturated clayey sand</td>
<td>bender elements, shear modulus and strength</td>
</tr>
<tr>
<td>Hoyos et al.</td>
<td>USA</td>
<td>Dynamic shear modulus and damping of compacted silty sand via suction-controlled resonant column testing</td>
<td>resonant column, shear modulus, damping</td>
</tr>
<tr>
<td>Zhao et al.</td>
<td>China</td>
<td>Critical state for unsaturated soils and steady state of thermodynamic process</td>
<td>critical state, thermodynamics</td>
</tr>
<tr>
<td>Fathalikhani &amp; Gatmiri</td>
<td>Iran</td>
<td>Numerical study of damage in unsaturated bentonite with 0-stock finite element code</td>
<td>FEM, damage, THM coupling</td>
</tr>
<tr>
<td>Kawai et al.</td>
<td>Japan</td>
<td>Expression of mechanical characteristics in compacted soil with soil/water/air coupled F.E. simulation</td>
<td>coupled FEM, compaction</td>
</tr>
<tr>
<td>Droniuc</td>
<td>France</td>
<td>Etude par la methode des elements finis du comportement des remblais en sols fins compactes</td>
<td>FEM, embankments, compacted soil</td>
</tr>
<tr>
<td>Sakai &amp; Nakano</td>
<td>Japan</td>
<td>Interpretation of the effect of compaction on the mechanical behaviour of embankment materials based on the soil skeleton structure concept</td>
<td>FEM, embankment, compaction</td>
</tr>
<tr>
<td>Makki et al.</td>
<td>France</td>
<td>Effet du retrait du sol sur une maison expérimentale</td>
<td>masonry house, FEM, shrinkage,</td>
</tr>
<tr>
<td>Heyerdahl et al.</td>
<td>Norway</td>
<td>Rainfall-induced collapse of old railway embankments in Norway</td>
<td>stability, railway</td>
</tr>
<tr>
<td>Bajwa &amp; Simms</td>
<td>Canada</td>
<td>Evolution of microstructure during desiccation of oil sands mature fine tailings</td>
<td>tailings, polymer.microstructure</td>
</tr>
<tr>
<td>MacRobert</td>
<td>South Africa</td>
<td>Field capacity and moisture loss during active deposition on tailing dams</td>
<td>tailings, field capacity, moisture loss</td>
</tr>
<tr>
<td>Siemens et al.</td>
<td>Canada</td>
<td>Effect of confining stress on the transient hydration of unsaturated GCLs</td>
<td>GCL, hydration, parametric study</td>
</tr>
<tr>
<td>Mitchell</td>
<td>Australia</td>
<td>Climate change effects on expansive soil movements</td>
<td>climate, expansive soils</td>
</tr>
<tr>
<td>Liu &amp; Yasufuku</td>
<td>Japan</td>
<td>A geotechnical countermeasure for combating desertification</td>
<td>desertification</td>
</tr>
<tr>
<td>Hemmati &amp; Morodeni</td>
<td>France</td>
<td>Etude de la stabilité des pentes non saturées sous les effets de l'infiltration prenant en compte la végétation</td>
<td>slope stability, root water uptake</td>
</tr>
<tr>
<td>Ng et al.</td>
<td>Hong Kong</td>
<td>Soil suction induced by grass and tree in an atmospheric-controlled plant room</td>
<td>vegetation, leaf area index, root area index</td>
</tr>
<tr>
<td>Adem &amp; Vanapalli</td>
<td>Canada</td>
<td>A simple approach for predicting vertical movements of expansive soils using the mechanics of unsaturated soils</td>
<td>expansive soils, vertical displacements</td>
</tr>
<tr>
<td>Ejaaouani et al.</td>
<td>Maroc</td>
<td>Comportement des sols gonflants lors de l’humidification et du séchage</td>
<td>swelling, shrinkage</td>
</tr>
<tr>
<td>Stancia et al.</td>
<td>Romania</td>
<td>Soil chart, new evaluation method of the swelling-shrinkage potential, applied to the Bahlui’s clay stabilized with cement.</td>
<td>swelling, ecologic cement stabilisation</td>
</tr>
<tr>
<td>Auvray et al.</td>
<td>France</td>
<td>Etude de l’impact de l’hygrométrie sur la fissuration d’un sol gonflant</td>
<td>cracking, image processing</td>
</tr>
<tr>
<td>Ávila et al.</td>
<td>Colombia</td>
<td>One-dimensional cracking model in clayey soils</td>
<td>cracking, modelling</td>
</tr>
</tbody>
</table>

Table 1. Synoptic table of contributions to the Discussion Session of TC 106 (WRC = water retention characteristics, FEM = finite element modelling)
2 HYDRAULIC BEHAVIOUR, RETENTION PROPERTIES AND FLOW

Nearly one third of the papers submitted to the Discussion Session deal with retention and flow properties, hence recognising the role of hydraulic state variables on the coupled hydro-mechanical response of unsaturated soil systems.

Characterisation of the retention properties is tackled at different scales. Carrillo-Gil & Carrillo-Acevedo (Peru) summarise 20-years data and models at regional scale for tropical Peruvian soils in the Amazon region (Fig. 1). Both correlations based on soil index properties and experimental data from suction cells are analysed, to provide a general view of the retention properties of three classes of soils (clayey–silty–sandy) coming from five different regions.

When the regional scale is analysed, only results for water content are given, irrespective of hysteresis of the soil water retention mechanisms, of the void ratio and of the hydraulic path, hence disregarding the coupled evolution of volumetric strain and water content. The estimated water retention curves (WRC) typically give wide ranges of water content for given suction, although typical patterns can be identified for the different soil classes (Fig. 2). As it is often the case, silty soils are the most difficult to be uniquely characterised, due to wider differences in plasticity, void ratios and fabric of the silty soils. Differences come from both heterogeneity of the soil properties and different initial void ratios. Attempts to preliminary characterisation of retention properties at the regional scale may be of relevant use for risk mapping. Hopefully, similar databases should be enriched in the future, and possibly re-analysed by means of statistical tools, to fully exploit their potentialities.

Reducing the scale of investigation, Sugii et al. (Japan) analyse a mixed experimental and numerical approach to study the retention and the conductivity properties of an upper unsaturated sandy layer at the site scale, with an infiltrometer scheme typically coming from the field of hydrology. The Authors suggest that simple approximations for the pressure field and moisture distribution upon infiltration may be sufficient to get reasonable estimates for the unknown variables, provided suction is measured at a convenient depth. The comparison between the field data and the results of a laboratory model, replicating the experimental procedure in situ, shows that while the two hydraulic conductivity functions compare well, WRC estimates present significant differences, possibly coming from air entrapment effects.

The latter observation suggests that characterising the hydraulic properties of unsaturated soils still presents open issues, under different viewpoints. On the one hand, faster experimental procedures are sought, in order to allow for reasonable costs – especially in terms of time – of experimental tests. On the other hand, proper characterisation of the hydraulic properties of unsaturated soils need correct interpretation of the multiphase flow process promoted by the different experimental procedures.

For most soils, the determination of retention properties on the whole range of possible suctions usually requires a combination of different experimental techniques, possibly controlling different exchange mechanisms. Maček et al. (Slovenia) combine data from two commercial equipment, to investigate the drying branch of the WRC of different soils. The WRC data are derived from tensiometer readings in an evaporation device for low suctions (0-0.2 MPa), and from relative humidity in the high suction range (>1 MPa), by means of a dew-point potentiometer. By extrapolating the calibration range of the evaporation apparatus, they show how the two data sets may provide a reasonable picture of the whole drying branch of the WRC.

Reis et al. (Brazil) discuss how the experimental determination of the drying branch of the WRC can be speeded up by imposing suction with a small commercial centrifuge, carefully enhanced for WRC testing. Although the original idea dates back more than one century, still the procedure is not common in unsaturated soil testing, and it deserves further attention. In the equipment described by the Authors, the equivalent suction can be controlled either by changing the angular velocity of the centrifuge, or by increasing the lever arm of at constant rotational speed. Four samples could be tested at the same time, after careful assembly and saturation of the set-up. Results obtained in the small centrifuge compare well with data from more traditional testing procedures on both disturbed and remoulded samples of clayey silty sand in the range 0-0.9 MPa, even though small differences between results of the various techniques may be observed, especially in the low suction range. The evidence suggests that careful inspection of the influence of volume changes on the state variables during testing is necessary to interpret correctly data from different testing methodologies. Also, data obtained in the laboratory are seldom the result of the behaviour of the soil samples alone, but they are affected by the whole experimental set-up, materials and procedures adopted. The work presented by Nishimura (Japan), in which a micro-porous membrane is tested in a pressure plate apparatus on different soil types, confirms the latter observation.

Innovative and promising techniques for unsaturated soils now try to exploit electromagnetic properties of multiphase mixtures. Toll et al. (UK) discuss an efficient fast multi-electrodes resistivity system, and present a new combined sensor for suction and water content (Fig. 3), consisting in a coiled TDR, which can be used in conjunction with a high capacity tensiometer for simultaneous measurement of water content and pore water pressure.

Figure 1. Regional scale for hydraulic characterisation of tropical Peruvian soils (Carrillo-Gil & Carrillo-Acevedo)

Figure 2. Proposed WRC ranges for the Huallaga River Watershed: A-sands; B-clays; C-silty-clays (from Carrillo-Gil & Carrillo-Acevedo)
To prevent electrode polarization short pulses and reverse automatic multiplexing to exploit a large number of electrodes. The multi-electrode resistivity system is combined with dimensions in mm (from Figure 3. Schematic of tensiometer housing and coiled TDR - extremely promising for accurate simultaneous monitoring of and water content measurement in a single probe, appears field. The results show that the distribution of the fluid phases in the 3D pore structure in the elaboration of MXCT data (Fig. 5). The work by Mukunoki & Mikami (Japan) includes highlights on one of these aspects, which is the dependence of the retention properties of porous media on the rate of convective flux. The study presented by the Authors was aimed at understanding the mechanism of light non-aqueous phase liquid (LNAPL) migration in sandy soils. To this aim a new testing apparatus was conceived, in which fluid is injected in the sample and the flow is tracked by a micro-focused X-ray computed tomography scanner (MXCT), as schematically depicted in Fig. 4. Both inlet and outlet pressure were measured, as well as the outflowing mass of fluid. The distribution of the fluid phases was analysed taking into account connectivity of the 3D pore structure in the elaboration of MXCT data (Fig. 5). The results show that the distribution of the fluid phases in the soil pore structure following the induced flow will depend on the injection rate. The reason for this dependency is that the local path followed by the injected fluid will change depending on convective (Darcy) velocity, hence on current hydraulic conductivity and hydraulic gradient, kinematic viscosity and interfacial tension.

Figure 3. Schematic of tensiometer housing and coiled TDR - dimensions in mm (from Toll et al.)

The multi-electrode resistivity system is combined with automatic multiplexing to exploit a large number of electrodes. To prevent electrode polarization short pulses and reverse polarity readings are adopted. Calibration tests demonstrate that the proposed equipment gives a maximum error of less than 1% on resistance. The system was used to investigate both drying and wetting of sandy clay samples from a trial embankment. A very well defined resistivity-water content relationship was obtained, with high correlation coefficients, confirming potentialities of this technique both in the laboratory and in the field.

The new sensor proposed in the paper, combining suction and water content measurement in a single probe, appears extremely promising for accurate simultaneous monitoring of the two variables, at least in the laboratory. Provided the interpretation of the data from coiled TDR properly accounted for the geometry of measurement scheme, the new device shows an accuracy for water content determination of ±(0.047–0.075), already by means of a theoretical model based on mixture theory, in the absence of direct experimental calibration. The new probe was tested on different soils, ranging from sand to clay and organic soil, and performed well in most cases against conventional 3 prong TDR device.

It is worthwhile remarking that most of previous experimental data do not allow for an exhaustive picture of the whole WR domain. On the one hand, still most data are collected along drying paths only. On the other hand, for those soils which undergo significant volume change during drying and wetting, the typical data collected just represent a series of pictures of water contents at different void ratios. Information on void ratio at subsequent stages is seldom provided, hence hindering to a certain extent a comprehensive interpretation of the retention behaviour, and generalisation of the laboratory information to different hydraulic paths and history.

Dependence of the WR behaviour on initial state and hydraulic history is discussed by Mendes & Toll (UK) on remoulded sample of sandy clay of low plasticity. Their results confirm that the WR domain is affected by mechanical state parameters especially at low suction values, where the dominant retention mechanism is capillarity. At decreasing water content, the amount of water retained inside the soil mostly depends on the physico-chemical characteristics of the solid phase, and tends to become independent from the actual void ratio, as recently discussed by various authors.

At high water ratios suction changes affect significantly both the hydraulic state and the mechanical state of the soil, depending on the soil fabric and on the shrinkage-swelling properties of the soil. Fredlund & Zhang (Canada) suggest to assist the interpretation of water retention data from conventional equipment – which do not provide usually any information on void ratio – with simple results from conventional shrinkage curves. Careful choice of size and aspect ratio should prevent cracking of the sample during the test, thus allowing to associate a value of void ratio to each suction-water content state. Current degree of saturation can be determined from the pair of data void ratio - water content, hence allowing re-writing the drying branch of the water retention curve in terms of degree of saturation as a function of void ratio too. The procedure suggested may help in better discriminating between changes in water ratio due to retention mechanisms from water expulsion due to changing void ratio, but its applicability is limited to the drying branch of the water retention domain. Also, the consequences of stress state cannot be accounted for, as no stress is applied to the drying sample. The results can therefore be up scaled to the field only for the upper horizon of the soil, where the effects of stress could be disregarded.

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Figure 4. Schematic of MXCT fluid injection testing set-up (from Mukunoki & Mikami)

Figure 5. MXCT images of (a) pore structure, and (b) LNAPL residual distribution (from Mukunoki & Mikami)
3 TESTING AND MODELLING THE MECHANICAL BEHAVIOUR

A lot of effort in the field of unsaturated soil mechanics has been devoted in the past to experimental investigation and theoretical modelling of the mechanical behaviour of unsaturated soils subjected to suction changes. This may be the reason why the contributions dealing with conventional aspects of the mechanical behaviour of unsaturated soils are few, and mostly addressed to specific soils under a limited variety of stress-paths.

The attention is mostly focused on materials used in compacted earth constructions, either to evaluate the performance of the as-compacted materials, or the effects of weathering and of improvement techniques. These contributions are mostly motivated by the need for assessing existing and new embankments under seasonal change of moisture content, and for increasing their resilience to extreme events. More attention is devoted to the investigation of the mechanical behaviour of unsaturated soils under cyclic and dynamic loading, which typically characterise the working conditions of embankments. Moreover, where the seismic risk is high, a necessity is felt for proper criteria to assess embankments after relevant seismic events, which may impair functionality under normal working conditions.

3.1 Compacted natural and treated soils in earth construction

Bai et al. (China) concentrated their attention on the behaviour of a compacted loess like silt. Their study focuses on the influence of compaction water content and compaction energy on the compressibility of the soil. The data show that the compressibility of this soil increases with water content for a given energy. If the compaction energy is changed at a given water content, the minimum compressibility is found to correspond to the density for which the given water content represents the optimum.

Degradation due to wetting and drying on Pierre Shale is analysed by Schaefer & Birchmier (USA), motivated by frequent slope instabilities observed in this formation and extensive problems documented in the construction of dams. The residual shear strength was analysed by means of ring shear apparatus after drying and wetting weathering cycles. Evaluation of physical properties, chemical analysis and Scanning Electron Microscope images completed the evaluation of the effects of weathering. The Authors observed minor changes in mineralogy, but significant changes in the fabric of the material, while no definite conclusions could be reached for residual strength.

To stabilise compacted soils and to reduce the effects of weathering, improvement techniques may be implemented. Mavroulidou et al. (UK) investigate the behaviour of lime treated London clay, for use in stabilisation of roads and pavements, as well as a technique to increase workability during construction. Samples of lime treated London Clay were prepared in the laboratory by static compaction after mixing powder London clay and lime. After curing, with two different techniques, the samples were tested in a triaxial equipment under different controlled saturation conditions. A number of triaxial tests were carried out to assess the effect of lime on the hydro mechanical properties of statically compacted London Clay and lime-treated London Clay samples. Filter paper was used to study the retention behaviour of both the original and the treated soil, in the suction range of interest. In view of the use of the improved material in earth construction, the experimental results suggest that a limited amount of lime could be sufficient to stabilise the soil. The solution is considered preferable to higher amounts of lime, as increasing the lime would decrease the ductility of the soil.

3.2 Response of unsaturated soils to elastic waves

Three contributions analyse the response of unsaturated soils to elastic waves, both as a mean of analysing the state of the soil, and to predict the mechanical behaviour under dynamic loads.

Byun et al. (Korea) use compression and shear waves to analyse the response of a sand and a silty-sand as a function of the degree of saturation. A common pressure plate extractor was modified to apply axial stress and measure elastic wave velocity. Bender elements and piezo-electric disk were mounted in the cell to this aim. The results seem to suggest that for sand the influence of stress level on the elastic stiffness is predominant with respect to the effect of suction. The latter can be better appreciated for the soil containing a higher amount of fines.

Georgetti et al. (Brazil) analyse a compacted clayey sand in the whole range of shear strains, from small strain to failure. Multistage triaxial tests were performed on unsaturated samples kept at constant water content. Suction was measured during the tests by means of axis translation technique. Tests were performed with bender elements to investigate the influence of suction and confining stress on small strain modulus. Also in this case, the small strain modulus appears to be influenced more by confining stress rather than suction, at least in the range investigated. As expected, the effect of suction tends to become more relevant at decreasing confining stress. Nonetheless this result can not be considered a general conclusion, as in this case the soil was investigated in a range of suction where the degree of saturation – hence the state of the soil – hardly changes, keeping around 60.

Dynamic properties of unsaturated sandy silt are studied in a broader sense by Hoyos et al. (USA) with a proximitor-based resonant column device. The device developed by the Authors includes bender elements and allows for testing soils under controlled suction conditions by means of axis-translation technique. Stiffness and damping are investigated, the latter after careful inspection of the influence of suction on the frequency response curves (Fig.6).
The response of the soil is not symmetric with respect to the resonant frequency, which complicates the interpretation of the data in terms of material damping. The logarithmic decay curves were used to the scope. The results confirm once more that suction affects the resonant frequency although to a lesser extent than confining stress. As expected, equivalent viscous damping decreases at increasing suction.

### 3.3 Theoretical and numerical modelling

While recent experimental efforts are dedicated to dynamic and weathering behaviour, still modelling efforts are mostly concentrated on developing constitutive laws for unsaturated soil behaviour under static loads, especially in view of engineering applications. Still a gap appears separating terms of an Instability Index, depending on applied vertical engineering applications. Still a gap appears separating.

Vázquez et al. (Spain) present a simplified model to predict collapse upon wetting, based on oedometer test results on a sandy-silty clay. They summarise their experimental results in terms of an Instability Index, depending on applied vertical stress, giving the amount of expected collapse as a function initial suction and suction change. Although simplified, the models depends on two parameters only, and can thus be suggested for preliminary evaluation of collapse strain when a one-dimensional geometric scheme can be applied in the field.

Zhao et al. (China) discuss an interesting theoretical aspect of unsaturated soil modelling approaches. Starting from the observation that the critical state concept has been acting as a cornerstone in the development of models for saturated soils, they investigate the constraints leading to a thermodynamic consistent definition of critical state for unsaturated conditions. The Authors point out that for unsaturated soils variables including the hydraulic state concur to a proper definition of critical state. A thermodynamically consistent steady state is reached when all the relevant static and kinematic variables, including fluid pressures and volume fractions, reach their asymptotic values, while only further deviatoric strain is observed. Interestingly, they remind that critical state may not be unique, depending on soil fabric, as already observed for saturated soils with a dominant initial structure. Advanced models, accounting for fabric, weathering, degradation usually require a numerical implementation for their evaluation, even before they are adopted to analyze boundary value problems. Fathalikhani & Gatmiri (Iran) present a coupled thermo-hydro-mechanical numerical formulation of a model based on damage theory, developed to analyse the effects of excavation in host geological barriers. In the model damage is treated as a tensorial variable, accounting for the directional crack pattern, while the effects of suction and temperature are assumed to be isotropic. The model, developed on both thermodynamic and micromechanical concepts, is evaluated on a set of literature experimental data on a small scale model of unsaturated bentonite, subjected to a heating and a following relaxation phase. The consequences of suction and temperature changes on damage are investigated during heating and in the relaxation phase. The work highlights once more that interpreting the multiphysics behaviour of soils is far from being straightforward, and that even laboratory tests should be carefully analysed as boundary value problems at a small scale.

Following this line, Kawai et al. (Japan) present a Finite Element model of compaction, using an elastic-plastic model for unsaturated soils. A typical compaction stress history is imposed, including the loading and the unloading stages. Void ratio as well as suction are tracked along the stress path. The simulation highlight that the state which is generally defined as "as-compacted", is the result not only of the loading stage, but also of the following unloading path.

### 3.4 Modelling structures and infrastructures

The dependence of the as compacted state on compaction history complicates the application of models for compacted soils to full scale problems, as the paper by Droniuc (France) suggests. In the latter contribution, a Finite Element analysis of a model embankment made of fine grained soil is presented, and the numerical results are compared to experimental measurements from sensors installed in the model embankment. The Author points out that, besides the choice of a proper hydro mechanical model for the compacted soil, the analysis of an embankment requires a careful investigation of the initial and the boundary conditions. Swelling and shrinkage strains promoted by soil-atmosphere interaction affect the state of the material after compaction, and the heterogeneous profile of suction and water content result in a heterogeneous response of the system to hydro mechanical loads.

Sakai & Nakano (Japan) present a preliminary study on the effects of compaction on the dynamic performance of embankments, motivated by design approach moving towards a performance based concept in high seismic risk countries. Samples of sandy materials, having different grain size distributions, were compacted to different relative compaction degrees, and subjected to constant water content test in a triaxial apparatus. The results show that the density achieved during compaction affects the liquefaction potential of the soil. The experimental data are used to calibrate the model used to perform preliminary analyses of the response of an ideal embankment under seismic action.

Heyerdahl et al. (Norway) introduce another complicating feature in the assessment of infrastructures. Typically, a consistent part of railway embankments are now about 100 years old. In spite of strengthening part of them with modern criteria in recent years, still many of them are still working under conditions which follow their original design. Prolonged rainfall threaten the serviceability of the infrastructure system, by inducing widespread damage of the embankments. In their contribution the Authors provide an overview of damage mechanisms, trying to classify the possible sources of instability. Inadequate performance of culverts, water loads from flooding and slope instability promoted by rainfall infiltration are typically recognised as the main threatening processes. The analysis of one case history, presented in the second part of the paper, confirms that assessing an existing structure, without detailed knowledge of its hydro mechanical state and history, may be problematic, and that exhaustive indication for reliable assessment may not be obtained.

As a whole, it seems that most issues related to the assessment of existing structures and infrastructures concern possible climate changes. In the contribution by Makki et al. (France) the effect of differential settlements on masonry building due to shrinkage promoted by extreme drought is investigated, both on a prototype model and numerically (Fig. 7). In the experimental test, the prototype was supported by jacks, which were selectively removed to simulate differential settlements. A 3D refined model of the masonry construction reproduced well the observed displacements, and allowed evaluating the structural behaviour of the structure under possible action of drought.

Figure 7. Experimental masonry prototype and 3D numerical model (from Makki et al.)
4 EFFECTS OF CLIMATE AND VEGETATION

A consistent number of papers are being published in recent years presenting attempts to evaluate the consequences of increasingly severe climatic conditions. This conference is not an exception, as nearly one third of the papers are broadly related to issues related to this theme.

Liu & Yasufuku (Japan) present a self-watering system of new conception to support superficial vegetation in arid climate. The basic idea is to bury clayey inclusions in coarser soil to exploit their retention properties to store water. The proposed system should be able to regulate the capillary fringe, and reduce evaporation, in turn helping in preventing from salinization. A model test, numerical simulation and design specifications are presented in the contribution. The performance of the proposed system will depend mostly on water retention properties, hydraulic conductivity of the soil, vegetation activity and fresh water availability, but also on the geometrical configuration of the water trap.

Mitchell (Australia) investigates expansive soil movements under climatic impact, for vegetated an non-vegetated areas. The aim of the study is to evaluate the resilience of existing and new structures, and to provide useful revision guidelines for foundation design standards. A simple one-dimensional model averaged with depth is adopted to calculate heave and settlement of expansive soil, subjected to the moisture excess or deficiency predicted for the next half century, summarised by means of a simple moisture index. Hemmatt & Modaresi (France) analyse the stability of slopes under infiltration accounting for vegetation. In their analysis, performed with the aid of a finite element model, both infiltration and evapotranspiration are explicitly accounted for. The latter is described by means of an empirical function giving the evapotranspiration flux as a function of root density and depth. A one-dimensional scheme based on grain size distribution, liquid limit, plasticity index and activity of the clay. With reference to this comprehensive classification, the beneficial stabilising effect of different cement types is evaluated.

A consistent number of papers are being published in recent years presenting attempts to evaluate the consequences of increasingly severe climatic conditions. This conference is not an exception, as nearly one third of the papers are broadly related to issues related to this theme.

Adem & Vanapalli (Canada) discuss a simple approach for vertical displacements of expansive soils. The approach is based on a simple suction-strain relationship, and can be used to predict vertical displacements promoted by suction changes in a one-dimensional scheme. The case of a residential site in the city of Regina, located on a highly expansive clay deposit, is described to suggest how the model can be applied. Suction changes profile were calculated by means of a numerical analysis in which the climatic history during one year was imposed. As the stiffness is assumed to be a function of the degree of saturation, hysteretic water retention behaviour will give different stiffness along drying and wetting path. The latter feature is confirmed among others by Ejjaaouani et al. (Maroc), who present experimental data of a wetting and drying cycle on an expansive clay, and discuss possible source of non-reversible volume change.

Stanciu et al. (Romania) analyse cement stabilization to reduce the swelling and shrinkage potential, as a countermeasure against potential structural damage of structures founded on expansive clay. Comparing different active clays, the Authors propose a unified swelling classification chart, based on grain size distribution, liquid limit, plasticity index and activity of the clay. With reference to this comprehensive classification, the beneficial stabilising effect of different cement types is evaluated.

As a consequence of swelling and shrinkage, cracking may occur in active soils. The issue is of relevant interest for many engineering systems, although cracking occurrence and crack patterns are still difficult to be predicted and characterised.

Avray et al. (France) present a device to analyse cracking evolution as a function of the hydraulic state. The soil investigated is an active mixture of silt and bentonite. Samples with a diameter to height ratio of about 5 were prepared by static compaction and left evaporating in a controlled climatic chamber. The height and the mass of the samples were recorded, together with the crack pattern, which was tracked by photographic imaging. Three samples were analysed during drying. In spite of similar initial conditions and similar vertical strain, significant differences in the crack pattern were observed (Fig. 8). The crack area was found to be higher for the sample that experienced lower lateral shrinkage, while it decreases at decreasing lateral constraint. Although the result is consistent, no reason for different behaviour is evident, as the boundary conditions were identical for the three samples. Nonetheless, it can be observed that cracking is a strongly localised mechanical processes, hence it is dominated by local heterogeneity which may be responsible for different cracking patterns.

More systematic cracking development and evolution were observed by Avila et al. (Colombia), who specifically designed the moulds in order to force repeatable crack pattern. The Authors discuss the stress state in the sample subjected to drying shrinkage, highlighting the role of boundary conditions on the overall behaviour of the soil. For the simple geometrical scheme adopted, the position and the sequence of cracks could be predicted based on a careful simplified stress analysis.

5 SWELLING, SHRINKAGE AND CRACKING

In fine grained soils, multiphysics processes, starting from soil-atmosphere interaction, are accompanied by relevant volume changes, often ending in cracking and degradation. This aspect of the mechanical behaviour is a common issue of various applications in geotechnical engineering, including foundations, liners and mine tailings, to which some of the contributions presented to this session refer.

Avila et al. (Colombia) present experimental data of a wetting and drying cycle on an expansive clay, and discuss possible source of non-reversible volume change. Stanciu et al. (Romania) analyse cement stabilization to reduce the swelling and shrinkage potential, as a countermeasure against potential structural damage of structures founded on expansive clay. Comparing different active clays, the Authors propose a unified swelling classification chart, based on grain size distribution, liquid limit, plasticity index and activity of the clay. With reference to this comprehensive classification, the beneficial stabilising effect of different cement types is evaluated.

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Figure 8. Surface analysis of the crack pattern upon drying on three theoretical identical samples (from Avray et al.)
Sedimentation, consolidation and drying by evaporation are the processes of interest in impoundment of mine tailing containing fine particles. The contribution of MacRobert (South Africa) focuses on the drying behaviour of platinum tailings subjected to one year evaporation. The Author observes that the effects of natural drying in such tailings hardly reached 1 m depth, and that drying of the upper layer prevented further suction development in the lower part of the tailings for a long time. Once more, the contribution confirms that drying is a complicated process, ruled by both the forcing conditions at the boundary and the hydraulic properties of the material.

Bajwa & Simms (Canada) discuss the evolution of the microstructure of oil sand fine tailings by means of scanning electron microscopy and mercury intrusion porosimetry, to better understand the interplay role of desiccation and consolidation. Polymer amended tailings were investigated to try to optimise tailing operation in oil sands. The behaviour pattern turns out to be similar to that of active clays, and drying was found to be more effective in changing the fabric of the oil sand than the addition of the added polymer.

While drying is the most relevant process for mine tailing, wetting dominates the performance of liners in landfill barriers. Siemens et al. (Canada) discuss from a theoretical viewpoint the beneficial effect of confining stress in transient hydration of geosynthetic clay liners (GCL), which enhance the retaining properties of the GCL, in turn increasing its hydration rate.

6 FINAL REMARKS

This report tackled briefly those aspects of unsaturated soil mechanics which were suggested by the contributions submitted to the conference. As such, it is not a comprehensive summary of recent developments. Yet, it provides a broad overview on those aspect which are perceived to be of relevant interest in a present and future perspective.

The overview suggests that unsaturated soil mechanics is still considered as a fundamental branch of soil mechanics, more than a body of knowledge able to provide widely accepted answers to geotechnical design. The most innovative contributions concern experimental set up at different scales. The most challenging open issues seem to be related to existing infrastructures, requiring reliable criteria for their assessment.

7 PAPERS SUBMITTED TO THE SESSION


A simple approach for predicting vertical movements of expansive soils using the mechanics of unsaturated soils

Une approche simple pour prédire les mouvements verticaux des sols gonflants par la mécanique des sols non saturés

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ABSTRACT: The vertical soil movements associated with environmental changes which include climate, vegetation, watering of lawn and soil cover type for a test site with an expansive soil deposit in Regina, Canada were predicted by Ito and Hu (2011) using a combination of soil-atmosphere and soil-displacement models for a period of one year. In this paper, vertical soil movements of the same site were estimated reasonably well considering soil suction changes and associated modulus of elasticity as key parameters in a volume change constitutive relationship that is based on the mechanics of unsaturated soils. The proposed method is referred to as the modulus of elasticity based method (MEBM). The MEBM has been also evaluated earlier for three other case studies with satisfactory results. The results of the study presented in this paper and the other three case studies that were reported in the literature are encouraging for extending the simple MEBM in engineering practice for rational design purposes of both the sub and superstructures constructed in or on expansive soils.

RÉSUMÉ: Les mouvements verticaux des sols associés aux changements environnementaux qui comprennent le climat, la végétation, l'arrosage de la pelouse et le type de couverture du sol pour un site d'essai avec un dépôt de sol gonflant à Regina, Canada ont été prédits par Ito et Hu (2011) en utilisant une combinaison de modèles sol-atmosphère et de déplacement du sol pour une période d'un an. Dans cet article, les mouvements de sol verticaux sur le même site ont été estimés raisonnablement bien en tenant compte de la succion dans le sol et du module d'élasticité associés comme paramètres clés pour une relation constitutive du changement de volume qui est basée sur la mécanique des sols non saturés. La méthode proposée est appelée méthode basée sur le module d'élasticité (MBME). La MBME a également été évaluée précédemment pour trois autres études de cas avec des résultats satisfaisants. Les résultats de l'étude présentée dans ce document ainsi que les trois autres études de cas qui ont été rapportés dans la littérature sont encourageants pour l'application de la MBME pour la conception rationnelle tant des substructures que des superstructures construites dans ou sur les sols gonflants.

KEYWORDS: unsaturated expansive soils, vertical movement, suction, modulus of elasticity, vegetation, climate.

1 INTRODUCTION

Expansive soils shrink and swell in response to alternate dry and wet conditions inducing vertical soil movements. These soil movements cause significant damage to the lightly loaded engineered structures and contribute to economic losses which are greater than all natural disasters combined (Jones and Holtz 1973). The behavior of expansive soils associated with environmental changes hence should be considered when designing buildings, pavements, foundations (shallow and deep), pipelines, retaining walls, earth dams, canal or reservoir linings, and other structures that are constructed in or on expansive soils. Different methods for predicting the volume change behaviour of expansive soils have been proposed in the literature. However, most of the available methods focused on predicting the maximum potential heave at saturation condition. Furthermore, these methods or studies were limited to expansive soils of local regions that cannot be extended universally. Recently, Vanapalli and Adem (2012, 2013), and Adem and Vanapalli (2013) presented and assessed a simple approach for estimating the vertical soil movements of different expansive soils for overcoming these limitations. In this approach, the volume change constitutive relationship developed by Fredlund and Morgenstern (1976) for unsaturated soils was integrated along with the soil-atmosphere model VADOSE/W (Geo-Slope 2007). The analyses of the results demonstrated that the proposed approach, which has been referred to as the Modulus of Elasticity Based Method (MEBM), was capable of reproducing the vertical movement of expansive soils over time in response to the net changes in soil suction within the active zone for three different case studies (Vanapalli and Adem 2012, 2013, and Adem and Vanapalli 2013).

The objective of the present study is to check the validity of the MEBM for an additional test site with an expansive soil deposit in Regina, Saskatchewan, Canada. This site was originally modeled and presented by Ito and Hu (2011). These investigators simulated the Regina expansive clay vertical movements based on the suction data predicted from the soil-atmosphere coupled model by applying one year’s climate data (from 1 May, 2009 to 30 April, 2010). The liquid limit and plastic index values of Regina clay vary from 70 to 94% and from 40 to 65%, respectively. The soil properties required include the soil water characteristic curve (SWCC) and the coefficient of permeability function along with the climate and vegetation data as input for the soil-atmosphere model. The vertical movements of this soil with respect to the changes in suction values were predicted using the soil-displacement model. A general purpose partial differential equation solver (FlexPDE) was used as a tool to solve the governing partial differential equations in the modelling process. Various factors that influence the soil movements such as the climate, vegetation, watering of lawn and soil cover type were considered in the modelling.

In this paper, estimated values of soil suction, volumetric water content and vertical movements of expansive soils at different depths obtained from the MEBM were compared with the published results of Ito and Hu (2011). Due to limitations of space, only the results of soil suction and vertical movements have been presented and discussed in this paper, in addition to providing comparisons with the results of Ito and Hu (2011). There is a good comparison between the results from both the studies.
2 BACKGROUND

2.1 The constitutive relationship for estimating the vertical movements of expansive soils

The volume change behavior of any expansive soil deposit relative to the changes in site conditions can be rationally interpreted by extending continuum mechanics principles in terms of two independent stress state variables of unsaturated soils; namely, matric suction (\( u \)), and net normal stress (\( \sigma \)). In the proposed MEBM, the incremental vertical movement, \( dh \), was related to changes in matric suction neglecting the limited influence of the net normal stress within the surficial active zone as follows:

\[
dh = m_i^2 (u_i - u_{sat})
\]

where, \( m_i^2 = (1 + \mu)/(H(\mu - 1)) \) is the soil structure compressibility modulus associated with a change in suction (\( u \)) where \( H = \) elasticity modulus with respect to change in suction and \( \mu = \) Poisson’s ratio.

To calculate the vertical soil movement for a given site, the soil lateral deformations are negligible. In other words, the \( K_0 \)-loading was assumed in the present study. In the proposed MEBM, the vertical movement, \( dh \), was related to changes in matric suction neglecting the limited influence of the net normal stress within the surficial active zone as follows:

\[
dh = \sum_{i=1}^{n} \frac{m_i^2 (u_i - u_{sat})}{h_i}
\]

where \( n \) is the number of layers.

Oh et al. 2009 studies show that the value of the modulus of elasticity with respect to change in net normal stress, \( E \), varies significantly with soil suction. In the proposed MEBM, the semi-empirical model introduced by Vanapalli and Oh (2010) was used to estimate the modulus of elasticity, \( E \), associated with any value of the soil suction.

\[
E_{\text{sat}} = \frac{1 + \alpha (u_i - u_{sat})}{(P_a/101.3)(S \beta)}
\]

where \( E_{\text{sat}} \) and \( E_{\text{sat}} \) are the soil moduli of elasticity under unsaturated and saturated conditions, respectively, \( P_a \) is atmospheric pressure (i.e., 101.3 kPa), \( S \) is degree of saturation, and \( \alpha \) and \( \beta \) are the fitting parameters.

To calculate the soil structure compressibility modulus, \( m_i^2 \), the modulus of elasticity with respect to change in suction, \( H \), was estimated using the relationship below:

\[
H = E / (1-2\mu)
\]

The relationship between \( H \) and \( E \) may be more complex for soils in a state of unsaturated condition; however, this relationship which is valid for saturated soils has been extended for unsaturated soils in the present study. Similar assumptions were suggested by Geo-Slope International Ltd. for modeling soil heave due to infiltration using SIGMA/W.

2.2 VADOSE/W for estimating the changes in soil suction

Estimation of soil suction changes due to soil water migration (infiltration/evaporation) in the active zone is important in predicting the vertical movement of expansive soils. The computer program VADOSE/W, a product of Geo-studio, was used as a tool to estimate the net changes in soil suction with respect to time and depth (Geo-Slope 2007). The program couples the flow of water, heat and vapor through both saturated and unsaturated soils to provide a direct evaluation of soil water storage and suction. Critical to the formulation of VADOSE/W is its ability to predict actual evaporation as a function of climate data, applied as an upper boundary condition, using the rigorous Penman-Mahal method (Wilson, 1990).

The input parameters required for VADOSE/W include soil properties such as the SWCC and the coefficient of permeability function, climate and vegetation data. The climate data include the daily precipitation, the maximum and minimum daily temperature, the maximum and minimum daily relative humidity, the average daily wind speed and net radiation. The vegetation data include the leaf area index (LAI), the plant moisture limiting point, the root depth and the length of the growing season.

The output from VADOSE/W includes modeled data such as temperature, evaporation, suction, and volumetric water content. In the present study, only the modeling results for soil suctions versus time are presented and compared with the published data of Ito and Hu (2011).

3 CASE STUDY: REGINA EXPANSIVE CLAY (ITO AND HU 2011)

The city of Regina, SK, Canada is located on highly expansive clay deposits that exhibit large volume changes as the soil moisture changes. Failures in light infrastructures buried in the soil have increased greatly in recent years, especially in older areas with asbestos cement (AC) water mains (Hu et al. 2008). As a part of a program of study the performance of AC water mains in Regina expansive clay, Ito and Hu (2011) modeled a site located in a residential area with a high water main breakage rate. It includes a park area with thick grass of 100 mm and a wide paved road with 150 mm thick asphalt pavement. Infiltration due to precipitation and park watering and evapotranspiration from the grass were considered to model the soil suction fluctuations for this site.

The results from the Regina test site were used to validate the proposed MEBM in estimating the vertical soil movements over time considering the field condition (vegetated park area and asphalt-paved area). The stratigraphy of the site consists of approximately 6.4 m of highly plastic clay, 1.8 m of elastic silt and 6.8 m of till as shown in Figure 1. The choice of thickness and soil properties for each layer was guided by field observations made by Vu et al. (2007). The climate data obtained from a weather station at the Regina international airport was applied at the vegetative cover over a period of one year (from 1 May, 2009 to 30 April, 2010). Figures 2 and 3 show the SWCCs and the coefficient of permeability functions, respectively, for Regina and other materials used in the numerical modelling. Ito and Hu (2011) provide more details about the soil, the climate, and the vegetation data of the site.

4 RESULTS AND DISCUSSIONS

4.1 Estimation of the soil suctions

The soil profile shown in Figure 1 was modeled using the fully coupled transient analysis with the 2-D software package (VADOSE/W) to estimate the suction changes associated with the environmental changes for a period of one year. Beside the soil properties, the initial and boundary conditions are needed as input data to run the program. The initial conditions for all nodes of the model domain, including pressure and temperature, were derived from implementing a steady-state analysis using the same model. Based on the field suction data measured by Vu et al. (2007), the initial pressure head during the steady-state analysis was set up to be -163.15 m for the top 3 m of the clay layer, -101.97 m for the rest of the clay, -61.18 m for the silt, and -203.94 m for the till. The temperatures of nodes at the lower boundary were set up to be 10°C.
The change in suction, \( H \), was estimated using the relationship below:
\[
\frac{\Delta \psi}{\Delta H} = k_0
\]
where \( \psi \) is the suction, \( H \) is the vertical movement, and \( k_0 \) is the unsaturated hydraulic conductivity. The incremental vertical movement at the mid-layer is given by:
\[
\Delta s = \frac{\Delta H}{\Delta \psi}
\]
where \( s \) is the suction and \( \Delta \) indicates the change in suction and vertical movement.

The soil profile shown in Figure 1 was modeled using the VADOSE/W package (VADOSE/W) to estimate the suction changes associated with the environmental changes for a period of 150 mm and a wide paved road with 150 mm thick asphalt.

The high water retention capability and the low coefficient of permeability of Regina expansive clay layer. The total vertical movement approached minimum values at 3.4 m (which is the active zone depth). According to Azam and Ito (2012), this behavior was attributed to the surface soil layer that was initially at an unsaturated state and imbibed any water available by the infiltration. Likewise, the layer can rapidly lose water under relatively dry conditions.

The results from the Regina test site were used to validate the VADOSE/W output that can be used for predicting the vertical movement of the test site with Regina expansive clay.

4.2 Estimation of the vertical soil movements

To calculate the vertical soil movements at different depths (0, 0.5, 1, 2, 3, and 6 m), the soil profile was divided into several sub-layers up to 6.4 m depth (which is the thickness of Regina expansive clay layer). The total vertical movement of the soil at a certain depth for a given day was computed by adding the vertical movements of all layers up to the considered depth using Equation 2. The soil compressibility modulus, \( m_2 \), was calculated using the Poisson’s ratio (\( \mu = 0.33 \)) and the soil modulus of elasticity in terms of soil suction which was calculated using Equations 3 and 4.

Vanapalli and Oh (2010) suggested the fitting parameters, \( r, \beta, \) equals 2 for fine-grained soil, which was used for Regina expansive clay. The fitting parameter, \( a \), was assumed to be 1/12 in order to provide reasonable comparison between the predicted and the published results of the vertical soil period from 23 June to 12 October as reported in Vu et al. 2007. However, water uptake by mature trees was not included in the modeling.

Similarly to Ito and Hu 2011, and Vu et al. 2007, the vegetation was specified as good grass and the growing season was assumed to start in April and end in October as suggested by Vu et al. (2007). The LAI function for good vegetation with a maximum LAI value of 2 was used as suggested in SoilCover (Unsaturated Soils Group 1996). The root depth of 150 mm was used as suggested and the root distribution was assumed to be triangular. A plant moisture limiting point of 500 kPa and a wilting point of 2500 kPa were used for this simulation.

Mass balance checking was performed on the VADOSE/W run, and the model solved with a total mass balance error of less than 1.5%.

Figure 4 shows the predicted soil suction response to a changing surface boundary over the entire year under the centre of the vegetation cover. The soil suction was found to vary with depth and time. It can be seen that the fluctuations in suctions correlated well with the environmental conditions on the surface boundary. The suction at the ground surface fluctuated widely and these fluctuations reduced with depth. The predicted suctions for this study agreed well with the results of Ito and Hu (2011). The correspondence between the suction values was accomplished using the same meteorological data (e.g., precipitation, temperature), soils properties and initial boundary conditions.

The corresponding suction profiles under the centre of the vegetation cover for various times were also investigated. However, due to limitations of the paper length, suction profiles are not provided in this paper. In general, extreme changes in suction (that vary between 600 and 2500 kPa) occurred at the ground surface. The suction values are typically greater at the surface during relatively dry periods. During infiltration, the suction values decreased at the surface, and it continued to decrease further as water infiltrated to greater depths. The suction fluctuations were predominant at the surface and approached minimum values at 3-4 m (which is the active zone depth).

The water retention capability and the low coefficient of permeability of the Regina clay, especially under unsaturated conditions, impedes the soil suction at higher depths to respond to the variations of the surface boundary. This soil-atmospheric interaction corroborated well with the suction values obtained from Ito and Hu (2011) thereby validating the VADOSE/W output that can be used for predicting the vertical movement of the test site with Regina expansive clay.
movements. The modulus of elasticity under saturated condition, \( E_{sv} = 750 \text{ kPa} \), was suggested based on the results of the conventional oedometer tests conducted by Vu (2003) on Regina expansive clay. The soil movement was predicted as a function of time and depth. Due to limitations of space, the soil movement variations are not provided in this paper. Both shrink and swell behavior were observed in the period of the study. The vertical soil movement has a strong correlation with the predicted suction values. As anticipated, no vertical movement of the soil was observed below the active zone. The vertical soil movements estimated in this study were in reasonable agreement with the results of Ito and Hu (2011), clearly responding to climatic trends and infiltration events. Figure 5 provides a comparison between the vertical soil movements estimated using the MEBM and Ito and Hu (2011) method at different depths and times. The agreement between the results of two methods is reasonable. Some differences observed can be attributed to the differences in predicted soil suction profiles using different methods. This may be also due to the governing equations which were different for both the methods used for estimating the vertical movement.

6 REFERENCES

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Étude de l’impact de l’hygrométrie sur la fissuration d’un sol gonflant

Impact of the hygrometry on the swelling soil cracking

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RÉSUMÉ : Les sols gonflants soumis à des cycles de séchage et d’humidification sont susceptibles de se fissurer dès le premier cycle de dessication. Ces fissures introduisent des modifications du comportement mécanique du matériau. L’objet de cette étude est la présentation d’un dispositif complet de quantification de l’impact des sollicitations hydriques sur les phénomènes de fissuration. Pour cela, plusieurs éprouvettes de sol possédant des caractéristiques et des dimensions identiques ont été soumises à des hygrométries et des succion contrôlées. Les résultats obtenus dans un intervalle de succion allant de 8,5 à 113 MPa, montrent qu’il est important d’évaluer non seulement l’intensité des fissures mais également la surface de retrait radiale du sol, afin d’interpréter correctement les corrélations entre les fissures et les différents paramètres de compactage et de séchage.

Mots clés : sol gonflant, fissure, succion, analyse d’images.

ABSTRACT: Desiccation cracks appear on swelling soils submitted to drying/wetting cycles. These cracks could dramatically modify the mechanical behavior of the materials. This study presents a new device to quantify the impact of the hydric variations on the cracking phenomenon. A series of clayey silt specimen with the same initial compaction parameters was submitted to controlled suction and hygrometry conditions. The results obtained for imposed suctions comprised between 8.5 to 113 MPa, showed the importance of the measurement of not only the crack area, but also the radial shrinkage area for a better understanding of the correlations between crack area and the compaction and desiccation parameters.

Key-Words: Swelling Soil, Crack, Suction, Image Processing.

1. INTRODUCTION

2. MATÉRIAUX ET MÉTHODES
Le sol utilisé dans cette étude est un mélange limon-argile en proportion massique 40 % de limon de Xeuilley et 60 % de bentonite FVO dont les propriétés sont données dans le tableau 1. La courbe de rétention est obtenue par la méthode osmotique pour les succion entre 0 et 8,5 MPa et par la méthode des solutions salines pour les succion supérieures à 8,5 MPa (Figure 1). Le matériau a une teneur en eau à l’optimum Proctor de 26 % avec une densité sèche maximale de 1,47.

| Tableau 1 : Paramètres caractéristiques des matériaux utilisés. |
|---------------|----------------|----------------|----------------|
|               | Normes          | Limon de Xeuilley | Bentonite Mélange |
| $w_1$ (%)     | ASTM, 1998      | 36,8            | 117             | 82,2 |
| $w_p$ (%)     | ASTM, 1998      | 27,6            | 41,2            | 29,5 |
| $I_p$ (%)     | ASTM, 1998      | 9,2             | 75,8            | 52,6 |
| VBS (g/100g)  | ASTM, 2009      | 3,13            | 18,41           | 11,53|
| $\gamma_s$ (kN/m$^3$) | ASTM, 2006 | 26,5            | 25,5            | 25,8 |

2.1. Préparation des éprouvettes et dispositif expérimental
Les éprouvettes de 20 mm d’épaisseur sont préparées par compactage statique dans un moule cylindrique de diamètre 102 mm au fond rainuré. Les rainures du moule sont destinées à empêcher le retrait au cours du séchage. Ces éprouvettes sont ensuite placées dans l’enceinte du dispositif (Figure 2). L’hygrométrie de l’enceinte est imposée par l’intermédiaire des solutions salines.

Le dispositif est placé dans une salle climatisée dont la température est de 20±0,5°C. Ce dispositif permet la mesure continue de la masse, de la hauteur, et de la surface des
fissures déterminées par l’analyse des images obtenues tout au long de l’essai (Figure 2).

![Figure 1 : Courbe de rétention du mélange (E : état initial).](image)

La teneur en eau est déterminée par l’intermédiaire des mesures de masse obtenues par une balance toutes les 10 min. L’évolution de la hauteur de l’éprouvette est mesurée toutes les 8 heures, par un capteur laser de déplacement fixé sur une vitre glissante. Ce capteur peut être déplacé manuellement pour être au-dessus de l’éprouvette lors des mesures. Ces valeurs permettent de déterminer l’évolution de la déformation verticale $\Delta h/h_0$, $h_0$ étant la hauteur de l’éprouvette après compactage.


Un algorithme a été développé afin de déterminer la valeur du retrait moyen à partir du calcul du nombre de pixel noir entourant la surface de sol.

L’aire des fissures à l’intérieur de la surface de l’éprouvette est alors obtenue grâce à l’outil « Analyze Particle » de ImageJ, qui permet le comptage des pixels noirs d’images binaires (Lakshmikantha et al., 2009).

La méthode a été validée de deux manières différentes par comparaison des résultats obtenus sur des images construites numériquement avec une surface de fissures connue, et sur des images d’éprouvettes fissurées dont la surface des fissures avait été déterminée par une méthode manuelle similaire à celle de Peng et al., (2006) (Figure 3). L’écart entre les valeurs obtenues par ces deux méthodes est de 2,5 % pour l’aire de retrait et de 5 % pour l’aire des fissures.

![Figure 3 : Validation de la méthode, a) Image réelle fissurée, b et c) Images obtenues par la méthode d’analyse d’images pour la détermination du retrait et des fissures, d) détermination manuelle de l’aire des fissures et du retrait.](image)

### 3. ESSAIS

Le tableau 2 montre les paramètres et les conditions de séchage des essais présentés dans cet article. La succion initiale des éprouvettes mesurée par la méthode du papier filtre (Fawcett et Collins-George, 1967, ASTM, 1994) est de l’ordre de 8,5 MPa. La succion imposée dans le dispositif d’essai est appliquée par une solution saline saturée (K2CO3). Cette succion correspond à 113 MPa ($w = 7\%$, voir figure 1), et impose un chemin de séchage hydrique aux éprouvettes. L’imposition de ce chemin de séchage induit, dès les premières heures (5 à 9h) de l’essai, l’apparition de fissures de dessiccation. L’équilibre hydrique est atteint au bout de 8 jours.

<table>
<thead>
<tr>
<th>n° essai</th>
<th>h (mm)</th>
<th>w (%)</th>
<th>Densité (kg/m³)</th>
<th>Hygrométrie (%) - MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Essai A</td>
<td>20,0</td>
<td>15,30</td>
<td>1,270</td>
<td>44 - 113</td>
</tr>
<tr>
<td>Essai B</td>
<td>20,0</td>
<td>14,64</td>
<td>1,280</td>
<td>44 - 113</td>
</tr>
<tr>
<td>Essai C</td>
<td>19,9</td>
<td>14,40</td>
<td>1,264</td>
<td>44 - 113</td>
</tr>
</tbody>
</table>

### 4. RÉSULTATS

Dans cette partie, les évolutions de la teneur en eau, des déformations verticales et radiales des éprouvettes, ainsi que de la surface des fissures et du retrait des éprouvettes au cours du séchage sont mesurées et comparées.
4.1. Évolution de la teneur en eau pondérée
Les évolutions des teneurs en eau diffèrent au début du séchage à cause de faibles écarts de la teneur en eau initiale de compactage. Au bout d’un jour de dessiccation, les courbes se superposent. Ainsi, l’évolution de la teneur en eau pondérée est identique pour ces trois essais (Figure 4).

4.2. Impact de l’hygrométrie sur les déformations verticales et latérales
La figure 5 présente les évolutions des déformations verticales mesurées à l’aide du capteur laser au cours du séchage. Les amplitudes des déformations verticales sont très proches pour les trois essais. Les éprouvettes B, C et A présentent des déformations verticales à la fin du séchage respectivement de 1,75 %, 1,98% et 2,08 %. De plus, la cinétique de déformation est la même pour les essais A et C et légèrement plus lente pour l’essai B. Compte tenu des incertitudes de mesure on peut conclure que les déformations verticales sont quasi identiques.

4.3. Évolution de la fissuration
La figure 8 présente les évolutions du Cif* (aire des fissures sur l’aire de la surface des éprouvettes) au cours du séchage pour chaque essai. L’éprouvette B qui présente la plus grande déformation latérale est celle dont la surface est la moins fissurée, tandis que l’éprouvette A avec des déformations latérales plus faibles présente 8 fois plus de fissures que les essais B et C.

Les photos des surfaces des éprouvettes permettent de confirmer les tendances précédemment observées (Figure 7). En effet, la surface de l’essai B après séchage présente des fissures fines et un retrait important, la surface de l’essai A moyen au cours de la dessiccation, obtenu grâce à la méthode d’analyse d’images. Bien que les cinétiques de séchage et les états initiaux soient proches, la cinétique et l’amplitude des déformations latérales diffèrent d’un essai à l’autre (Figure 7). En effet, les déformations latérales à la fin du séchage varient de 0,5% pour l’essai C à 1,5% pour l’essai B et la cinétique de déformation de l’essai B est 2,5 fois plus grande que celle de l’essai C. Afin d’interpréter ces différences, il est nécessaire de compléter ces résultats avec l’évolution du réseau de fissuration.
présente des fissures larges et un retrait plus réduit, et l’essai C présente un retrait faible ainsi que des fissures fines.

Les différences de déformations latérales et d’aires des fissures observées entre les essais A et B, dans la mesure où leur cinétique de séchage, leurs déformations verticales ainsi que leur Ciftot sont similaires peuvent être dues à des différences au niveau de l’interface entre l’éprouvette et les rainures du moules. En effet, une adhésion moins forte entre l’éprouvette et les rainures entraîne un retrait libre plus important et une fissuration plus faible, tout en conservant des déformations verticales, les valeurs de Ciftot et des variations de teneur en eau pondérales similaires. Ainsi, l’adhésion entre l’éprouvette de l’essai B et les rainures pourrait être moins forte que pour l’essai A.

Les différences observées pour l’essai C sont plus difficilement interprétables dans la mesure où à la fois le retrait, le Cif* et le Ciftot sont différents des essais A et B. Ces différences pourraient être attribuées à la densité légèrement plus forte de l’éprouvette C par rapport aux éprouvettes A et B. En effet, Rodriguez et al. (2007) ont également montré que la fissuration et le retrait diminuent lorsque la densité augmente. Des essais supplémentaires sont en cours pour mettre en évidence les paramètres importants intervenant lors d’un chemin de dessiccation.

5. CONCLUSION
Cette étude a permis la mise en œuvre d’un dispositif expérimental complet permettant l’étude de la fissuration et du retrait des éprouvettes subissant des sollicitations hydriques contrôlées. Ce dispositif et la méthode du traitement d’images associée seront utilisés afin d’étudier la fissuration et la cicatrisation des éprouvettes de sols gonflants soumises à des cycles hydriques de séchage-humidification. Plusieurs éprouvettes présentant des conditions de compactage similaires ont été soumises à une même sollicitation hydrique. La cinétique de séchage ainsi que les déformations verticales observées sont similaires d’un essai à l’autre. Cependant, les déformations radiales ainsi que l’aire des fissures observées différent, pointant ainsi la nécessité de mesurer et de contrevenir les évolutions de la teneur en eau, des déformations verticales et latérales, ainsi que du Cif* et du Ciftot afin d’interpréter correctement les résultats.

6. REFERENCES
One-dimensional cracking model in clayey soils

Modélisation unidimensionnel de la fissuration des sols argileux

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ABSTRACT: It is difficult to formulate a general model to capture cracking initiation and evolution, because there are multiple factors that influence these processes. In order to simplify the evaluation and get more insight on the phenomenon, a series of shrinkage tests were performed inducing one-dimensional primary cracking on remolded clay from Bogotá. The results allowed the proposal of a conceptual model that, based on mould shape factor, initial moisture content and evaporation rate, may predict time for crack initiation and identify the location and direction of primary and secondary cracks. Results indicate that boundary conditions play a significant role in cracking evolution, due to the restrictions imposed to the free shrinkage and to the homogeneous evaporation rate. The moulds used for soil desiccation, allowed the induction of predefined cracks when their shape factor values were greater than 1.5. In these cases the model is simple and may be considered as a model of cracking with one degree of freedom. When moulds have a shape factor between 1 and 1.5, the degrees of freedom increase dramatically leading to much more complex crack patterns.

RÉSUMÉ : La fissuration des sols est un phénomène complexe contrôlé par de multiples facteurs qui rendent difficile la formulation d'un modèle qui permette de capturer l'initiation et l'évolution des fissures. Afin de simplifier l'évaluation des facteurs et d'améliorer la compréhension du phénomène, une série d'essais de retrait, induisant une fissuration primaire, ont été effectuées sur de l'argile remoulé de Bogotá. Les résultats ont permis de proposer un modèle simple qui, en fonction du coefficient de forme du moule, la teneur en eau initiale et le taux d'évaporation, peut prédire le moment d'initiation des fissures et identifier l'emplacement et l'orientation des fissures primaires et secondaires. Les résultats indiquent que les restrictions portant sur le libre retrait du sol et l'homogénéité de la vitesse d'évaporation jouent un rôle important dans l'évolution des fissures. Les moules qui ont permis l'induction de fissures prédéfinies étaient caractérisés par un facteur de forme supérieur à 1.5. Dans ces cas-là, le modèle est simple et peut être considéré comme un modèle de fissuration avec un degré de liberté. Lorsque les moules ont un facteur de forme compris entre 1 et 1.5, le degré de liberté s'accroît considérablement, conduisant à des schémas de fissuration beaucoup plus complexes.

KEYWORDS: Cracking, unsaturated soil, shrinkage test, Bogotá clay.

1 INTRODUCTION

Cracking has an important impact on soil behaviour because it affects aspects such as drainage, compressibility and strength. Desiccation processes, conditions of crack initiation, crack evolution and cracking patterns have been studied on different soils under distinct boundary conditions (Marinho, 1994, Miller et al. 1988, Lloret et al. 1998, Kodikara et al. 2000, Yessiller et al. 2000, Vogel et al. 2005, Ávila et al. 2005, Lakshmikantha 2009, Peron et al. 2009, Tang et al. 2010, Lakshmikantha et al. 2012). When drying test on soils are conducted in square or circular moulds, the cracking patterns are in general complex and very difficult to predict. Nevertheless, when the shape of the moulds force a predominantly one-dimensional contraction, cracks tend to appear in a systematic pattern and it is possible to make predictions about the place and direction of the primary, secondary and in some cases, tertiary cracks. One-dimensional conditions lead to a very favourable situation to get some insight in the apparently erratic evolution of the cracking patterns.

This article shows the experimental results obtained in desiccation tests performed on clay samples from Bogotá city where cracking sequences were observed. From these results a simple one-dimensional cracking conceptual model is proposed based on contraction restrictions and soil tensile strength.

This work is a part of a comprehensive research program to study the problems of soil shrinkage and cracking that affect Bogotá city.

2 CRACKING SEQUENCE DURING DESSICATION TESTS ON BOGOTÁ CLAY

Geological and geotechnical characteristics of the subsoil of Bogotá and the effects of surface deformation and cracks on the infrastructures have been discussed by some authors (Lobo-Guerrero et al. 1992, Ingeominas 1996, Ingeominas and Los Andes University 1997, Ávila 1998, Ávila 2003, Ávila et al. 2005). The test described in this paper have been done on samples of Bogotá clay taken between 2 and 4 m depth (wL = 62-65%, PI = 30-35% and Activity = 0.52-0.57). These samples are representative of the layers subjected to desiccation and cracking in many sectors of the urban area.

Although different type of desiccation tests were conducted for the comprehensive evaluation of shrinkage and cracking of this clay, the discussion presented in this paper is basically focused on the cracking sequence observed in the experiments made in the double T shaped moulds shown in Figure 1. These moulds impose restriction to shrinkage and induce systematic cracking patterns. Those patterns are particularly interesting for the analysis of the process of formation and propagation of initial cracks and the subsequent process of cracking.

Reconstituted clay samples were prepared at different initial water content and they were left to dry to an open atmosphere to observe the characteristics of the cracking process evolution. Figure 2 shows three stages of the desiccation process that were observed on three different tests (labeled 1 to 3 in Figure 2) conducted at the same time and under equal atmospheric conditions. First sequence corresponds to a picture taken 4 hours after the initiation of the desiccation process for which moisture contents of the samples ranged between 40% and
41.2\% (stage 1). In all cases cracks initiate at the vertices located in the change of geometry of the moulds.

Figure 1. Small moulds (SM) used in the desiccation tests. (from Ávila, 2004). Dimensions are in mm.

The second sequence represents an intermediate stage and the picture was taken 4h:35min after the beginning of the test. Moisture content ranged between 36\% and 37.2\%. It is clear the complete formation of primary cracks and in all cases their location and form are similar. The volumetric soil contraction can be observed as a separation of the sample from the walls of the moulds.

The third sequence represents the final stage and the picture was taken after 22 h of the test initiation. The primary cracks were completely open and a secondary crack, located near the central part of the sample appears in all samples oriented in parallel direction with respect to the primary cracks. It is remarkable the great similarity observed in the three tests. Note that cracking test repeatability is not frequent due to the multiple variables involved, as previously mentioned.

Similar results were obtained in other tests sequences with different initial water content, as observed in Figure 3. In these tests also tertiary cracks appear in the extremes of all the moulds directed perpendicular to the primary cracks and located in the middle of the extreme areas of the moulds.

During the desiccation process, moisture content of the samples were controlled by weighting them carefully at different times. The relation between initial water content of the samples and water content at initial cracking is presented in Figure 4. It is clear that, the higher the initial moisture content the higher the moisture content at cracking. The relation between both variables implies a greater potential volume change for initially wetter samples due to moisture reduction during desiccation.

3 SIMPLE ONE-DIMENSIONAL CONCEPTUAL MODEL TO EXPLAIN SYSTEMATIC CRACKING

When a soil sample is subjected to a homogeneous drying process, volumetric contraction tends to occur. In free shrinkage conditions, with not friction restrictions in the base or laterally, the sample cracks are not expected because no tensile forces act on it. This condition is sketched in Figure 5 and represents the common case of shrinkage limit tests in which a lubricant is used to reduce friction between the sample and the mould. However, if the sample composition or if the drying conditions are not totally homogenous the sample may crack due to tensile forces generated inside it.

Figure 2. Sequence of cracking of three samples in SM moulds. Primary and secondary cracks are generated in a systematic and homogeneous way in each case.

Figure 3. Homogeneous pattern cracking observed at the final stage of three tests similar to those of Fig. 2 but starting from higher water content.

Figure 4. Relation between initial water content and water content at cracking for SM samples.
Cylindrical shape is convenient to ensure uniform drying and homogeneous contractions, however if the base of the mould is not smooth and friction between the mould and the sample develops during desiccation, nonhomogeneous tensile forces are generated producing complex drying patterns.

Stress changes are generated as a result of the forces induced when the soil tends to shrink but the boundary conditions restrict the free shrinkage. Figure 6 represents a simple picture of the forces that may progress in the different sectors of the sample during the desiccation process. Abu-Hejleh and Znidarcic (1995) and Konrad and Ayad (1997) proposed similar patterns for desiccation cracks formation in clayed soils subjected to one-dimensional consolidation and contraction caused by suction increments. In the central sector (sector 2) action forces tend to occur due to the contraction of the sample and these forces are counterbalanced by the reaction forces generated in the extremes of the sample (sectors 1 and 3) where the reaction walls play an important role in avoiding the contraction of the soil. Primary cracks tend to initiate precisely in the vertices of these reaction walls (points a, f, g or l, in Figure 6) because it is where an important stress concentration occurs. Primary cracks progress in a direction perpendicular to the main action forces, as sketched in Figure 7.

Once the primary cracks have been completely developed, shrinkage continues and new stress conditions appear in the different sectors of the sample. In sector 2 (Fig. 6) action forces are directed to the center of the sample trying to produce contraction or length reductions whereas reaction forces are generated by the friction between the soil and the base of the mould avoiding the sample contraction. As a combination of the action and reaction forces, non-uniform tensile stresses are mobilized along the sample. As it is illustrated in Figure 8 primary cracks appear at the points where mobilized tensile stress equals the tensile strength of the soil (points b and d). In the points a and e the mobilized tensile stress are low because restrictions to contraction are not so strong. On point c some restrictions to shrinkage are produced by the base and sides of the mould and a tensile stress is mobilized but of lower value than stress on points b and d. For that reason sample does not crack at this point.

As soon as primary cracks are completely developed, the sample stress distribution changes drastically and a sketch of the possible distribution is depicted in the lower part of Figure 8. This stress distribution may explain the occurrence of secondary cracks in the middle of the sample (point c) generated by the restriction to shrinkage produced mainly by the base of the mould. In this point mobilized tensile stress equals tensile strength of the sample.

The tertiary cracks shown in Figure 3 could be generated in a similar way than the secondary crack. For that reason the cracks
develop in the middle of the area subjected to tensile stresses. But as previously mentioned, these tertiary cracks are only formed when initial water content of the samples is high, because under these conditions the soil continued to shrink after secondary crack. However the base of the mould constrains that shrinkage producing the new level of cracking.

The model illustrated in Figure 8 is limited to one dimensional shrinkage, this condition is imposed by the shape of the mould where soil desiccation occurs. The shape may be considered by the shape factor (SF) that relates the major to the minor dimensions. For a square or a circle, SF is equals to 1 but for a rectangle SF is greater than 1. Shape Factors greater than about 1.5 tend to impose conditions of one-dimensional shrinkage and cracks appear in a more or less systematic way. For FS lower than 1.5 more than one degree of freedom are present in the shrinkage and cracking sequence, consequently the orientation of those cracks are much more complex to predict (Ávila, 2004).

4 CONCLUSIONS

The prediction of the initiation points and orientation of cracks produced by a desiccation process is in general very complex because many degrees of freedom are present and tensile stresses are mobilized in multiple directions. However under one-dimensional shrinkage that may be imposed to a soil, systematic cracking patterns tend to occur and they may be predicted. The experimental program on small samples prepared under similar conditions and subjected to a common drying atmosphere, showed the repeatability of cracking patterns for primary, secondary and in some cases tertiary cracks.

For laboratory desiccation experiments systematic cracking are commonly observed for moulds that have shape factor (relation length to width) equal or greater than 1.5, for lower values of shape factor the cracking is more complex and difficult to predict.

A simple conceptual model is here proposed to explain why the cracks appear in specific locations and following a particular sequence under the described conditions. This is important for the better understanding of the cracking phenomena in clayey soils, and particularly for the development and validation of numerical analysis of the hydro-mechanical problem of cracking applied to more complex scenarios.

5 REFERENCES


Experimental Study on Effect of Initial Moisture Content on Compressive Property of Compacted Loess Like Silt

Étude expérimentale des caractéristiques de compression des loess compactés

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ABSTRACT: Compacted soil, which widely exists in many sorts of engineering projects, is commonly unsaturated soil; and its physico-mechanical characteristics are great influenced by soil moisture content. To investigate the effect of initial moisture content of the soil on its compressive property, the numbers of oedometer tests are conducted to the specimens with various initial moisture contents and compacted under various compacting energies. The test results show that for the compacted loess like silt, when the compacting energy is same, the compressive modulus decreases with the increased initial moisture content of the specimen, but at the same initial moisture content, the compressive modulus is not monotone increased with the compacting energy increasing, the maximum compression modulus is reached under a particular compaction energy when the water content is exactly the optimum water content corresponding to the compaction energy. Therefore the initial moisture content of compacted loess like silt is one of the most important control indexes for the compaction quality control. To get a high quality of loess like backfill, the initial moisture content of the backfill must be strict controlled when the dry density meets the design requirement.

RÉSUMÉ : Les caractéristiques physico-mécaniques des loess compactés ont un effet direct sur la qualité du compactage de ceux-ci. Elles sont aussi des éléments importants à prendre en compte quand on utilise des loess pour réaliser des fondations et des couches de forme. Des essais en laboratoire ont été effectués pour déterminer la loi de variation des caractéristiques de compactage des loess en fonction des teneurs en eau et d’énergies de compactage différentes. Les résultats des essais montrent que le module de compressibilité est plus grand quand la teneur en eau augmente. Pour la teneur en eau optimale correspondant à une énergie donnée de compactage, le module de compressibilité est alors le plus petit. Par ailleurs, la résistance au cisaillement baisse avec l’augmentation de la teneur en eau. De même, cette dernière change en fonction de l’énergie de compactage et elle est aussi étroitement liée à la teneur en eau optimale correspondant à une énergie donnée. Par conséquent, un contrôle strict de la teneur en eau est très important pour améliorer la qualité de compactage des fondations et des couches de forme réalisées des loess. Par ailleurs, il faut faire attention à l’influence de l’énergie utilisée quand on retient le taux de compactage comme critère pour contrôler la qualité de compactage des loess, et ceci afin de justifier l’application de ce taux.

KEYWORDS: loess like silt, compaction energy, moisture content, compression modulus.

1 INTRODUCTION

Filling technique is widely used in backfill foundation projects of buildings, railways, highways and other subgrades. Usually, backfill refers to the accumulated soil by human activities. The backfill which is compacted in layers is called as compacted backfill. It must be compacted under the certain standard of the material composition, density, water content. The compaction quality of compacted backfill is directly related to the strength and stability of backfill foundation and subgrade (Wang, 2004). With the development of industrialization, the decreasing availability of proper construction sites has led to the increased use of mountain area, where the compacted backfill may be used. Loess like silt is the one of widely used backfill materials.

China has an extensive deposit of loess, which is mainly found in northwestern China. In this region, loess has been widely used as backfill material. The study of machinenical and physical properties of natural loess has been reported by many scholars (Assallay et.al. 1997, Guo et.al. 2000, Xie 2001). However, the study of machenenical and physical properties of compacted loess is still in its begining stage, the relative reports are rare. Therefore, it is necessary to study the mechanical property of compacted loess like silt. This paper reports the experimental results about the compressive property of loess like silt compacted, which may be referred in design and construction for loess like silt compacting.

2 EXPERIMENT

2.1 Material

<table>
<thead>
<tr>
<th>Table 1 Basic properties of tested soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Particle size distribution (%)</td>
</tr>
<tr>
<td>0.25-0.075 mm</td>
</tr>
<tr>
<td>0.075-0.005 mm</td>
</tr>
<tr>
<td>&lt;0.005 mm</td>
</tr>
<tr>
<td>2.7</td>
</tr>
</tbody>
</table>

The soil used in this program is taken from a loess site in Luliang city, Shanxi province, China. The basic properties of the soil are listed in Table 1. According to the Chinese Code, GB/T50 123-1999, Standard for soil test method, it is defined as silt soil.

2.2 Test procedure

The test program are divided into 3 steps as follow:

Step 1, determination of the maximum dry density and optimum moisture content under given compacting energy.

The three groups of remolded loess like silt specimens were compacted under compacting energies of 2684.9 kJ/m³, 1208.2 kJ/m³ and 592.2 kJ/m³, respectively. For each group, there were 5 speciemans made at various initial moisture contents to get a completely moisture vs. density curve, in turn, the maximum
dry density and optimum moisture content under given compacting energy are determined.

Step 2, produce the compacted loess like silt specimens. Three moisture contents of 11.5%, 13.5% and 15.5% were selected to be the initial moisture contents of compacted soil for further experiment. At each moisture content, 4 compacting energies of 671.2 kJ/m³, 1208.2 kJ/m³, 2013.7 kJ/m³ and 2684.9 kJ/m³ were used. In total, there are 12 specimens to be produced.

Step 3, the oedometer test was conducted on each compacting specimen. The compressive pressures was 25, 50, 100, 200, 300, 400, 600, 800, 1600 kPa, respectively, and the corresponding final settlement was recorded. Each loading stopped when the settlement less than 0.01mm/hour.

3 TEST RESULTS AND ANALYSIS

3.1 Maximum dry density and optimum moisture content

![Fig. 1 Moisture content vs. Dry density](image1)

The completely moisture content vs. dry density curve is drown out for compacting energy of 592.2, 1208.2 and 2684.9 kJ/m³ respectively, shown in fig. 1. Table 2 lists the maximum dry density and corresponding optimum moisture content for given compacting energy. Based the testing data, it can be concluded that the maximum dry density is increasing and the corresponding optimum moisture content is decreasing with the increment of compacting energy. This conclusion agrees with that of other scholars.

![Fig. 2 Moisture content vs. Compressive coefficient under same compacting energy](image2)

![Fig. 3 Moisture content vs. Compressive modulus under same compacting energy](image3)

Fig.2 shows the relationships of compressive coefficient and moisture content for 4 different compacting energies. Fig. 3 is the curves of compressive modulus vs. moisture content for 4 different compacting energies.

From figs. 2 and 3, it can be seen that for the compacted loess like silt, the compressive coefficient increases, while the compressive modulus decreases with the increased initial moisture content of the specimen, when the compacting energy is same.

When the compacting energy is smaller, like 671.2 kJ/m³, the compressive coefficient and modulus change with the initial moisture content, but the changes are smaller, the change ratio is less than 0.25. But, when the compacting energy is greater than 671.2 kJ/m³, the changes are obviously, and the change ratio increases with the increment of compaction energy, shown in figs.4 and 5. The maximum change ratio is as high as 4. This may imply that the compressive property of compacted loess like silt is sensitive with the initial moisture content when the compaction energy is greater than 671.2kJ/m³. The greater of the compaction energy is, the more sensitive the soil to initial moisture content.

3.2 Compressive property for same initial moisture content

The oedometer tests are conducted on the compacted loess like soil specimens which were produced with 3 different initial moisture contents and compacted under 4 different compacting energies respectively as described in section 2.2.
3.3 Compressive property for same compacting energy

At the same initial moisture content, the compressive coefficient is not monotone decreasing and the compressive modulus is not monotone increasing with the compacting energy increasing, seen figs. 6 and 7. When the initial moisture content equals to 11.5%, the compressive coefficient is the minimum and the compressive modulus is the maximum at the compacting energy of 2684.9 kJ/m³. When the initial moisture content equals to 15.5%, the compressive coefficient is the minimum and the compressive modulus is the maximum at the compacting energy of 671.2 kJ/m³. Similarly, when the initial moisture content equals to 13.5%, the compressive coefficient reaches the largest value and the modulus dose the smallest value at the compacting energy equal to 1208.2 kJ/m³.

Considering the data in table 2, it can be seen that the maximum compression modulus, meanwhile, the smallest compressive coefficient is reached when the initial moisture content is equal or closed to the optimum moisture content under a particular compaction energy. This emphasises that the initial moisture content is a very important index for obtaining a maximum dry density for given compacting energy. The dry density represents the dense condition and degree of compaction of backfill. Therefore the initial moisture content of compacted loess like silt is one of the most important control indexes for the compaction quality control. To get a high quality of loess like silt backfill, the initial moisture content of the backfill must be strict controlled when the dry density meets the design requirement.

4 CONCLUSIONS

In the present study, the following conclusions can be made about the compressive property of compacted loess like silt.

The maximum dry density increases and the optimum moisture content decreases with the increment of compaction energy.

When the compacting energy is same, the compressive modulus decreases and compressive coefficient increases with the increment of initial moisture content.

When the initial moisture content is same, the compressive modulus decreases and compressive coefficient increases with the increment of initial moisture content.

The authors would like to thank the financial supports of National Natural Sciences Foundation of China (No.51178287) and Natural Sciences Foundation of Shanxi Province (No.2010011029).
6 REFERENCES


Evolution of microstructure during desiccation of oil sands mature fine tailings

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ABSTRACT: The coupling between desiccation and consolidation is a process with important implications for the management of soft soils in general and dewatering of fine grained tailings typical of phosphate, bauxite, and oil sands mining in particular. The management of fine tailings can involve the placement of layers that are allowed to desiccate, and then are subsequently consolidated by burial under fresh tailings. While desiccation does densify the material, it also changes both the strength and volume change behaviour of the subsequently consolidated material. This phenomenon is crucial to the oil sands industry, where regulations mandate that tailings achieve a set undrained strength within 1 year after deposition. To understand the interplay of desiccation and consolidation, the evolution of microstructure of oil sand fine tailings are tracked through different drying and consolidation paths using mercury intrusion porosimetry, and non-biased analyses of Environmental Scanning Electron Microscope images. Preliminary results presented in this paper describe the evolution of microstructure in polymer amended tailings during desiccation. The influence of flocculant dose on the microstructure appears to lessen as desiccation progresses, but the final microstructure retains a more open porosity compared to untreated tailings. Résumé

RÉSUMÉ : Le couplage entre la dessiccation et la consolidation est un processus avec des implications importantes pour la gestion des sols mous en général et le séchage des résidus miniers fins typiques du phosphate, de la bauxite, et des sables bitumineux “oil sands” en particulier. La gestion des résidus miniers fins peut comporter le dépôt des couches pour le séchage, et leur consolidation par enterrément sous les résidus frais. Tandis que la dessiccation densifie le matériau, elle change également la force et le comportement mécanique du matériau consolidé. Ce phénomène est crucial à l’industrie de sables d’huile, où les règlements exigent que les résidus atteignent une force non drainée prédéfinie dans un délai de 1 an après dépôt. Pour comprendre l’effet de la dessiccation et de la consolidation, l’évolution de la microstructure des résidus miniers est étudié pour différents chemins de séchage et de consolidation utilisant la porosimétrie au mercure, et des analyses d’images de microscope à balayage électronique. Les résultats préliminaires présentés dans cet article décrivent l’évolution de la microstructure des résidus modifiés par polymère pendant la dessiccation. L’influence de la dose de floculant sur la microstructure semble diminuer pendant que la progression de la dessiccation, mais la microstructure finale maintient une porosité plus ouverte comparée aux résidus non traités

KEYWORDS: Mature fine tailings, polymer, suction, mercury intrusion porosimetry, SEM, desiccation, microstructure

1 INTRODUCTION

The extraction of oil from oil sands deposit result produces bitumen and tailings. Conventional deposition results in coarse particles (> 74 microns) settling on the beach, while the fine fraction of settles and consolidates extremely slowly, retaining a water content of over 180% after a decade – in this state the tailings are called mature fine tailings (MFT). Due to the extraction process, the clays in the fine fraction are highly dispersed, which results in very low hydraulic conductivity. Because of the volume of MFT produced, impacts include a considerable volume of water is lost to the tailings, and a very large footprint (~200 km² for tailings in the Fort McMurray, Alberta area) and dam constructions costs. In order to accelerate restoration and water reclamation form these tailings, the regulator has imposed new rules, that require an increasing inventory of tailings to achieve specific undrained shear strengths at scheduled time after deposition, the first target being 5 kPa after 1 year.

The new regulations have fostered large-scale experimentation with several techniques to dewater and/or strengthen MFT. One technique is to mix an anionic polymer with MFT and re-deposit the amended tailings in relatively thin lifts (Matthews et al. 2011, Wells et al 2011). The mixing is done in the pipeline, only a few metres from the deposition point. This causes aggregation of the clay particles, and results in decreases in water content by settling down to about 100% water content (50% solids), or even greater. To reach the required 5 kPa shear strength, the geotechnical water content must usually be less than 50% water content. To achieve this, the material may be deposited in thin lifts to facilitate further dewatering due to evaporation, long-term consolidation, or drainage.. However, the relative contributions of desiccation and consolidation to dewatering are not completely understood, and a better comprehension of the relative effects of each process on subsequent dewatering behaviour could contribute to optimizing the overall dewatering process, especially in terms of required layer thickness, and timing of layer sequencing. This paper presents some preliminary data on the microstructure of polymer amended MFT and how it evolves during desiccation. Data on microstructure is obtained using Mercury Intrusion Porosimetry (MIP) and Scanning Electron Microscopy (SEM).

1.1 Mercury intrusion porosimetry (MIP):

MIP finds a pore-size distribution for pores ranging from 0.01 up to 100 microns – while this pore-size distribution might not be the true PSD due to pore accessibility and sample preparation issues, MIP data is known to exhibit strong correlations to permeability, consolidation characteristics, and water-retention behaviour (Simms and Yanful 2005, 2004, Romero and Simms 2008) – it appears to give a good quantitative “fingerprint” of microstructure. For compacted clays, it is known that volume change measured by MIP samples is very close to macroscopic volume change. However, for wetter or slurried clays, it has been shown that MIP only measures a fraction of the total porosity, despite use of a rapid freeze drying technique to dehydrate the samples (Sassinan 2011).

Further details on the methodology of MIP are available in many other references, such as ASTM 4404-10, Simms and Yanful (2004) and Romero and Simms (2008), and are not repeated for reasons of space. All samples were prepared by
freeze-drying, by first cooling pentane in liquid nitrogen, then submerging the soil samples (cubes less than 5 mm in all dimensions) by using either a small strainer, or a miniature tray for very wet samples (MFT with no polymer) in the pentane for 1 minute. Soil samples are subsequently dried under vacuum for 1 hour, prior to the actual MIP test. The porosimeter model was AutoPore IV 9500.

1.2 Scanning electron microscopy (SEM):
We employ backscattered scanning electron microscope (SEM) using a rapid freezing stage (-50 degrees C) before application of vacuum (10^-3 Pa). Grayscale pixel analyses are used to quantitatively compare SEM images.

1.3 Total suction measurement
Total suction measurements used chilled mirror hygrometer (Wenglor WP4PotentiAmeter). Such hygrometers measure the vapour pressure in porous media, by decreasing the temperature in a confined space with the sample, until water condenses on a mirror. Thus the saturated vapour pressure at this controlled temperature is known, which equal to the vapour pressure at the ambient temperature of the sample. The relative humidity is equated to total suction, based on the well-known Kelvin-Laplace equation. The range of this device is theoretically from 0 up to 500 MPa of total suction, but precision is limited to 0.1 MPa. Sample must be extracted from the porous media and placed in a container for insertion in the WP4 device.

1.4 Material and Experimental set up:
MIP and SEM techniques are applied to oil sand mature fine tailings (MFT) amended with different doses of polymer, along with measurement of volume change, desaturation, and total suction in shallow columns (0.30 m in initial height, 0.30 m diameter), exposed to potential evaporation rates of ~ 6 mm /day. Columns were kept on scales, and vertical volume change was estimated by a plumb line dropped on 8 different points on each column. The MFT and polymer were supplied by Shell Canada. The specified MFT is originally at 35 - 40% solid contents (gravimetric water of 135 - 145%). The polymer (Flopam DPR 5285) is mixed into diluted MFT using a paint mixer set to 260 rpm, mixing for 30 seconds – this regime is to reproduce similar mixing conditions to in-pipe mixing that occurs during field trials of polymer-amended MFT deposition at Shell’s Muskeg River Mine. Three columns have three different doses of polymer - 700 ppm of the solid contents, 1000 ppm, 1500 ppm.

2 TEST RESULTS AND DISCUSSIONS

2.1 Basics properties of raw MFT
Table 1 presents some basics characteristics of raw MFT

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Parameters</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water contents (%)</td>
<td>14158</td>
<td>SFR</td>
<td>0.1</td>
</tr>
<tr>
<td>Solid contents (%)</td>
<td>40%</td>
<td>Liquid limit</td>
<td>45</td>
</tr>
<tr>
<td>Density (kg/m³)</td>
<td>1100-1200</td>
<td>Plastic limit</td>
<td>19</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.64</td>
<td>Liquidity index</td>
<td>3.96</td>
</tr>
</tbody>
</table>

These values are in close agreement to other studies of MFT, such as in Jeeravipoolvarn (2005).

2.2 Dewatering / desiccation behaviour of polymer amended MFT
As shown in Figure 1, evaporation proceeds at the potential rate of ~ 6 mm /day, until about Day 10. This corresponds to an average water content of 50%, or a solids concentration of 67%. At this water content, polymer amended MFT will have a peak undrained shear strengths in excess of 5 kPa (Matthews et al. 2011). For this relatively short layer thickness (0.30 m initial height), the drying is quite uniform with depth. Total suction values near the (~5 mm) surface increase above 1 MPa at this point (10 days), correlating with the onset of Stage II drying (actual evaporation declines significantly compared to potential evaporation). As described in Wilson et al. (1997), evaporation declines as a function of total suction at the soil surface, the decline becoming significant for total suction in excess of 3 MPa.

Figure 1: Cumulative evaporation in polymer amended MFT columns

Volume change behaviour is shown in Figure 2 (the shrinkage curve) and in Figure 3 (showing relation between degree of saturation and void ratio). Void ratio and degrees of saturation are based on vertical volume change only. Therefore, the initial degree of saturation are somewhat lower (70% initially) than the true value. This low value is also due to large aggregates formed by the polymer, resulting in some significant macroporosity that drains within the first few hours. Figure 3, however, clearly shows when the air entry value (AEV) occurs and the expected subsequent decreasing rate of volume change. Plotting total suction versus water content data from the same samples, and converting water content to degree of saturation using volume change data, a rough water-retention curve (WRC) can be obtained for the MFT with different does of polymer (Figure 4). Generally, the WRC and the volume change behaviour are very similar between the different treatments, with the exception that...
the sample dosed with 1500 ppm of polymer appears to have a higher shrinkage limit. From total suction measurements, the AEV appears to be between 300 and 400 kPa, though this may be biased by osmotic suction.

The appearance of the polymer amended MFT at lower water contents is quite different in comparison to the images for the higher water content samples. At the lower water contents, there is a greater frequency of cracks. However, the difference between samples with different polymer dose is less remarkable than at the higher water content. Grayscale analysis (defining porosity by a range of pixel shade) supports this qualitative judgement.

These results, by themselves, suggest that desiccation does substantially alter the microstructure, similar to other clayey soils (E.G Romero and Simms 2008), and as desiccation progresses the differences in microstructure between samples prepared with different polymer doses become less.

Cumulative pore-size distributions (CPSD) from MIP are shown in Figures 6 and 7. Figure 6 shows the CPSD for the three treatments of polymer amended MFT and untreated MFT at the initial water content (140%). As expected, the treated MFT shows much more porosity in the high pore-sizes than untreated MFT. The treated MFT CPSD are very close together, with the 1500 ppm sample showing somewhat larger pore sizes than the other two treatments. Figure 7 shows how the CPSD changes with desiccation, showing the same trend as the SEM images – there is very little difference between the samples desiccated to 100%. This agrees with the relatively small differences in the water retention curve at high suctions – by the time the AEV is reached (about 80 % water content in Figures 2 and 4), the difference in microstructure and relatively small. However, the MIP data does not apparently explain the slightly higher shrinkage limit of the 1500 ppm treatment.

2.3 Microstructural analysis of polymer amended MFT

SEM tests were carried out adjusting the magnification of SEM device from 0.1 kV to 5 kV and a high vacuum (10⁻⁷ Pa) of the vacuum chamber. A cold stage of -50 °C was applied to all samples for testing. Two samples for each level of polymer dose are shown, for two different water contents of ~100 % and 50% (+/- 5%). Interestingly, while the samples at the higher water content appear to show differences in inter-aggregate porosity, the differences at the lower water content are much less apparent. In the 1500 ppm sample at the higher water content, the shapes of the pores seem more round showing the structure to be more flocculated than that of the other doses.
After desiccation to 50% water content, the amended tailings still show a greater porosity in the larger pores, compared to the raw MFT at 140% in Figure 6.

Figure 6 Mercury intrusion porosimetry test results, showing cumulative pore-size distributions (intrusion and extrusion curves) for amended MFT for a water contents of 140% for 700, 1000, 1500 ppm and raw MFT.

Figure 7 Cumulative pore-size distributions at different degrees of desiccation

3 SUMMARY CONCLUSIONS

Microstructure of polymer amended MFT is studied using MIP and SEM techniques. Both techniques show that the pore-size distributions of the different treatments (polymer doses of 700, 1000, and 1500 ppm) converge with increasing desiccation. This correlates with the very similar water-retention curves of the different treatments, though not with the slightly higher shrinkage limit (58% to 50%) of the 1500 ppm sample. Based on this analyses, it appears that the desiccation behaviour, beyond the initial water release during settling, is relatively insensitive to the range of polymer applied in this study.

Ongoing work is examining the consolidation behaviour of MFT treated with different levels of polymer subsequent to different degrees of desiccation. This information is hoped to assist in optimization of this type of technology for tailings operations in the oil sands.

4 ACKNOWLEDGEMENTS

The authors are greatly thankful to Civil & Environmental Engineering Department, Carleton University, Ottawa, Canada and Shell Canada for the financial support to complete this study.

5 REFERENCES


Jeeravipoolvarn S. (2005); Compression behaviour of thixotropic oil sands tailings, MASc thesis in Geotechnical Engineering; Department of Civil and Environmental Engineering; University of Alberta, Canada


Evaluation of void ratio and elastic modulus of unsaturated soil using elastic waves

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ABSTRACT: The void ratio and elastic modulus of a soil influence the soil’s behavior. In unsaturated soils, soil parameters can be easily affected by the matrix suction and specimen component. The object of this study is to evaluate the elastic moduli and void ratios of unsaturated soils using compression and shear waves. The analytical equation for the void ratio in unsaturated soils is derived from the elastic wave velocities. The volumetric pressure plate extractor (VPPE) is improved for the application of axial load. The compression and shear waves are measured using piezo disk elements and bender elements, respectively, installed on the confining cell. For the preparation of the specimens, sands are mixed with silts of two different volume fractions ($v_f$), 0 and 20%. The specimens are subjected to the axial stresses of 100 or 350 kPa to control the matric suction. The experimental results show that the shear wave velocities in the specimens with $v_f = 20\%$ increase with increasing matric suction, while the shear wave velocities in the sandy soils ($v_f = 0\%$) remained almost constant. The compression wave velocities increase with increasing axial load, while the compression wave velocities remain constant with changes in the matric suction. The void ratio values determined from the elastic wave velocities agree well with void ratio values determined on the basis of the volume of the unsaturated soils. This study suggests that elastic waves can be effectively used to estimate the void ratios and elastic moduli of unsaturated soils.

RÉSUMÉ : L’indice des vides et le module élastique d’un sol influent sur le comportement du sol. Dans les sols non saturés, les paramètres du sol t peuvent être facilement affectés par la succion matricielle et la nature du sol. L’objectif de cette étude est d’évaluer les moduleles élastiques et les indices des vides des sols non saturés en utilisant des ondes de compression et de cisaillement. L’équation analytique pour l’indice des vides dans les sols non saturés est dérivée de la vitesse des ondes élastiques. L’Extraiteur de Plaque de Pression Volumétrique (EPPV) est amélioré pour l’application de la charge axiale. Les ondes de compression et de cisaillement sont mesurées en utilisant des éléments des disques piezoélectriques et des éléments piezocéramiques (‘bender elements’), respectivement, installés sur la cellule de confinement. Pour la préparation des éprouvettes, le sable est mélangé avec le limon avec deux différentes fractions volumiques ($v_f)$, 0 et 20%. Les éprouvettes sont soumises à des contraintes axiales de 100 ou 350 kPa pour contrôler la succion matricielle. Les résultats expérimentaux montrent que les vitesses des ondes de cisaillement dans les échantillons de $v_f = 20\%$ augmentent avec la succion matricielle croissante, tandis que les vitesses des ondes de cisaillement dans les sols sableux ($v_f = 0\%$) sont restées presque constantes. Les vitesses des ondes de compression augmentent avec l’augmentation de la charge axiale, tandis que les vitesses des ondes de compression restent constantes avec les variations de la succion matricielle. Les valeurs de l’indice des vides déterminées par les vitesses des ondes élastiques sont en bon accord avec les valeurs de l’indice des vides déterminées sur la base du volume des sols non saturés. Cette étude suggère que les ondes élastiques peuvent être efficacement utilisées pour estimer les indices des vides et des modules élastiques des sols non saturés.

KEYWORDS: Degree of saturation, Elastic wave velocity, Matric suction, Unsaturated soils, Void ratio.

1 INTRODUCTION

Soils are commonly regarded as fully saturated or completely dry. Experimental studies conducted with unsaturated soils require significantly longer than studies conducted with fully saturated or completely dry soils. Soils are commonly unsaturated near the surface. Soils used in dams or bridge foundations are mostly unsaturated. The stiffness of soils is affected by the amount of air and water that they contain. Since the 1970’s, studies of unsaturated soil mechanics have been conducted (Fredlund et al. 1978, Fredlund and Xing 1994, Vanapalli et al. 1996). As unsaturated soil dries out, the volumetric water content decreases and the matrix suction increases (Fredlund and Rahardjo 1993).

The goal of this study was to characterize unsaturated soils using elastic waves, including compression and shear waves. The elastic waves were measured using bender elements and piezo disk elements. Bender elements and piezo disk elements can change electrical energy into mechanical energy. Both transducers are commonly installed in consolidation cells and field penetration devices for use in seismic investigations (Lee et al. 2008, Lee et al. 2010, Yoon et al. 2010).

In this study, compression and shear wave transducers were installed on the walls of rectangular unsaturated soil characterization cells. This paper presents the background theory related to unsaturated soils, elastic wave velocities, the volumetric pressure plate extractor (VPPE) system, the test procedure, tests results, analyses, and summary and conclusions.

2 BACKGROUND THEORY

2.1 Volumetric change

Matyas (1968) reported that the volume of an unsaturated soils is affected by $u_a - u_w$ and $\sigma - u_w$, where $u_a$ and $u_w$ are the pore air pressure, the pore water pressure and the normal stress, respectively. The term $u_a - u_w$ is the matric suction, and the term...
\( \sigma - u_0 \) is defined as the net normal stress. When the matric suction decreases or the net normal stress increases, the volume of the unsaturated soil decreases.

The volume change of an unsaturated soil can be represented by the net normal stress and the matric suction. The constitutive relationship that takes into consideration of net normal stress and the matric suction is the following (Fredlund and Rahardjo 1993):

\[
dc_v = 3 \left( \frac{1 - 2 \mu}{E} \right) \left( \sigma_{mean} - u_0 \right) + \frac{3}{H} \left( u_n - u_0 \right)
\]

where \( \varepsilon \) is the volumetric strain, \( dc_v \) is the volumetric strain change for each increment, \( \mu \) is Poisson’s ratio, \( E \) is the modulus of elasticity and \( H \) is the modulus of elasticity for the soil structure with respect to a change in matric suction.

2.2 Elastic wave velocities

Physical characterization of soil using shear waves and compression waves is commonly used in the geotechnical field. The physical characteristics of the soil structure can be represented by Young’s modulus (E) and the shear modulus (G). The shear modulus can be determined from the shear wave velocity, using the following equation:

\[
G = \rho \times V_s^2
\]

where \( \rho \) is the density of the soil and \( V_s \) is the shear wave velocity. Young’s modulus, E, can be determined from the shear modulus, G:

\[
E = 2 \cdot G \cdot (1 + \mu)
\]

where is Poisson’s ratio. Poisson’s ratio can be determined from the shear wave velocity \( V_s \) and the compression wave velocity \( V_p \).

\[
\mu = \frac{V_s^2}{V_p^2} - 1
\]

The change in the volume of the soil can be predicted using the modulus of elasticity, with the shear modulus of elasticity and Poisson’s ratio substituted into Equation (1).

\[
\Delta \varepsilon = 3 \left( u_n - u_0 \right) \left( \frac{0.5V_p^2 - V_s^2}{V_s^2} - 1 \right) + \frac{3}{H} \left( u_n - u_0 \right)
\]

where \( \Delta \varepsilon \) is the volumetric strain change for each increment, \( \sigma_{mean} \) is the mean normal stress, \( u_0 \) is the suction at matric suction, \( E \) is the modulus of elasticity, \( H \) is the modulus of elasticity for the soil structure, and \( \mu \) is Poisson’s ratio.

3 LABORATORY TESTING

3.1 VPPE system

In this study, a modified volumetric pressure plate extractor (VPPE) was used. A schematic diagram of the modified system and its peripheral electronics is shown in Figure 1. The modified VPPE system was used to apply axial stresses and measure elastic waves. Axial stresses were applied through the center rod, as shown in Figure 1.

The regulator system on the left side of Figure 1 controls the matric suction and axial stress. The regulator system consists of three regulators. Two of them are used to control the matric suction, and the third controls the axial stress. The settlement was measured by a digital dial gauge attached to the rod that is used to apply axial stress.

The VPPE system also included a load cell to calculate the axial stress. Elastic waves are generated and detected using bender elements and piezo disk elements. The electronics on the right side of Figure 1 are a function generator, a filter amplifier and an oscilloscope, used to generate and detect shear and compression waves. Single sine waves were used for generation and detection of shear and compression waves (Lee and Santamarina 2005, Lee and Santamarina 2006).

3.2 Test procedures

Two types of soil specimens were used for this study. First, a uniform-grain-sized sand with a mean particle diameter of 0.45 mm were used. Second, a sand–silt mixture with a silt volume fraction of 20% was used. The physical properties of the specimens are summarized in Table 1. After partially saturated specimens were placed into the rectangular cell, which was placed on the ceramic plate of the VPPE, the VPPE was closed and matric suction was applied.

Table 1. Physical properties of the soil samples.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>( G_s )</th>
<th>( e_{\text{max}} )</th>
<th>( e_{\text{min}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>2.62</td>
<td>0.82</td>
<td>0.56</td>
</tr>
<tr>
<td>Sand–silt</td>
<td>2.62</td>
<td>0.80</td>
<td>0.42</td>
</tr>
</tbody>
</table>

As matric suction is applied, the degree of saturation of the unsaturated soil changes. Note that vertical axial stress was applied to control the mean normal stress. Elastic waves were continuously measured based on the matric suction. After the matric suction was applied, the soil specimen being tested gradually reached equilibrium. The elastic waves were measured at 1, 4, 9, 16 and 25 minutes and 1, 2, 4, 6 and 24 hours after the application of matric suction. The shear and compression wave velocities were obtained at each degree of saturation. After the maximum matric suction was applied, the matric suction was gradually decreased.
4 RESULTS

The matric suction versus volumetric water content, which is called the soil water characteristic curve (SWCC), is plotted in Figure 2 for the sand specimen.

Figure 2 shows that both the volumetric water content and the degree of saturation gradually decrease as the matric suction increases.

Elastic waves for the sand–silt mixture are plotted in Figure 3. Similar results were obtained for the sand specimen. The elastic wave velocities determined from the measured waves are plotted against the degree of saturation in Figure 4 for the sand specimen and Figure 5 for the sand–silt specimen.

The compression and shear waves for the sand specimen remained almost constant for degrees of saturation from 15 to 85%, as shown in Figure 4. For the sand–silt mixture specimen, the compression wave velocity also remained constant as the degree of saturation decreased from 85% to 7%. The shear wave velocity of the sand–silt mixture, however, increased as the degree of saturation decreased, as shown in Figure 5(b).

5 ANALYSES

The elastic modulus and the shear modulus of each specimen can be estimated using the measured elastic wave velocities. The calculated elastic moduli based on Equations (2) and (3) are plotted in Figure 6 for the sand–silt mixture specimen. From the measured elastic wave velocities and Equation (5), the porosity can be estimated. The calculated wave-based void ratio versus volumetric void ratio is plotted in Figure 7. Figure 7 shows that the wave-based void ratio is similar to the volumetric void ratio.
6 SUMMARY AND CONCLUSIONS

The matric suction applied in the tests described in this paper was controlled using an improved volumetric pressure plate extractor (VPPE) system to simulate unsaturated conditions near the soil surface. The VPPE was improved for the application of the axial load. The elastic moduli and void ratios of the two unsaturated soils tested were determined using compression and shear waves. The compression waves were continuously measured by piezo disk elements and the shear waves were monitored by bender elements. The piezo disk elements and bender elements were installed on the wall of the confining cell. Two types of specimens were used: a sand specimen and a sand–silt mixture specimen with a silt volume fraction of 20%. Axial stresses of 100 and 350 kPa were applied to the sand specimen and the sand–silt specimen, respectively.

The compression and shear wave velocities of the sand specimen remained almost constant for degrees of saturation from 15 to 80%. The compression wave velocity for the sand–silt mixture specimen also remained constant, but the shear wave velocity increased as the degree of saturation decreased. As the applied axial stress increased, the compression wave velocity increased. The void ratios determined from the elastic wave velocities agreed well with the volume-based void ratios. The results of this study suggest that elastic waves can be effectively used to estimate the void ratio and elastic moduli of unsaturated soils.

7 ACKNOWLEDGEMENTS

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8 REFERENCES

Evaluation Curves SWCC for Tropical Peruvian Soils
Évaluation des courbes de rétention d’eau SWCC pour les sols tropicaux péruviens

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ABSTRACT: The geotechnical analysis of this research shows the results of a program conducted in tropical soils of the Peruvian Amazon plain for over 20 years. With this information a database has been developed. It has collected samples and tests that establish the characteristics of these unsaturated soils in all watersheds that shape the Peruvian tropical region, using alternative means of obtaining the characteristic curves and laboratory tests. The results in this paper show geotechnical occurrences that indicate substantial similarity and differences of stress and deformation behavior of Peruvian tropical soils. The presence of these unsaturated soils in this region, where unique features of climate and difficult geological formation are present, makes this information very valuable for engineering design and construction development due to its models applicability to different stress patterns.

RÉSUMÉ : Cet article présente une base de données mise en place avec les résultats d’un programme d’analyse géotechnique mené sur les sols tropicaux de la plaine amazonienne du Pérou depuis plus de 20 ans. Des essais de laboratoire ont permis d’obtenir les SWCC qui établissant les caractéristiques des sols non saturés dans les basins versants qui forment la région tropicale péruvienne. Les résultats présentés ici montrent les événements géotechniques qui indiquent une similarité substantielle et les différences de contrainte et de déformation des sols tropicaux du Pérou. La présence de ces sols non saturés dans cette région, où les caractéristiques uniques du climat et formation géologique complexes sont présents, rend cette information très précieuse pour la conception technique et le développement de la construction en raison de son applicabilité des modèles pour différents schémas de contrainte.

KEYWORDS: tropical unsaturated soils, suction, soil-water curves.

1 INTRODUCTION
Based on recognized and user-friendly models, SWCC have been determined with field tests results and lab soil tropical sampling gathered in the Peruvian Amazonian region for more than 20 years (Carrillo-Gil, 1983). With that feedback, it was possible to elaborate SWCC ranges that represent probable tropical soil behavior as non-saturated geotechnical material. The results from the considered samples for this study were taken from engineering projects and consulting works developed in the Peruvian Amazon region. This area is difficult to access because of its environmental conditions and geomorphology. For those reasons it is very important to come out with Peruvian tropical soils SWCC grouped with distributed ranges for each of the five hydrographic watersheds. This information is very valuable for future relevant projects that are going to be developing in the study region.

2 SITE CONDITIONS IN AMAZON PLAIN
The general geology considers that a large part of the Amazon region has stayed covered during the interglacial periods of the quaternary by an interior sea of shallow water when the level of the oceans had 100 meters above of the existing now (330,000 years ago). It also to fluctuate during several glacial and interglacial periods forming terraces throughout the water courses, dropping to 100 meters below of the original level during the last Glacial Era (17,000 years ago) and remaining in these deep channels the large rivers, between them the Amazon river, raising afterwards to the current level (6,000 years ago). The accomplished studies establish that in the high jungle and in the limits of the low jungle are found many igneous rocks as sedimentary, while in the low jungle prevail residual soils originated by the sedimentary rocks of the tertiary and quaternary and mainly sandstones, shales and clays form them (Fig 1).

Figure 1. High and low jungle location in the Peruvian Map.

The general description of the geomorphology of the Amazon region indicates that the low jungle is substantially flat and as said remains, its height varies between 80 to 400 meters above mean sea level. Due to this small difference of elevation, the rivers flow slowly, getting in the dry station the appearance of lakes. This region of the Amazon plain can be indicated as advanced erosion type. The Amazon Plain is characterize by its
great humidity and soil covered by a dense tropical vegetation (Carrillo-Gil & Dominguez, 1996).

3 SOIL FORMATION

In this study, many researches and recollections of punctual data from the Amazon Plain were validated. The main watersheds from the most important rivers (Amazonas, Marañón Huallaga, Ucayali and Madre de Dios) were considered in order to establish their behavior characteristics in detail (Fig 2). According to the gathered database that consists of 1,318 samples tested in Peruvian tropical soils, forty typical samples were chosen to verify the original rocks that originate residual material. It was found some differences with the mother rock that is generally mention for another regions in South America.

4 CLIMATE CONDITIONS

The regional pluviometric level, where in some cases reach until 4,000 mm annually, depends in the temperature, density, absolute moisture, and other air mass characteristics. The climate acts in the decomposition of the soil and its original material throughout many agents that act in variable conditions and determine a wide quality range of soils. This complexity can be seen in how much different are their properties even though they are located very close among them. However, each of these tropical soils can be very similar to others located at thousands of kilometers of distance (Carrillo-Gil, Carrillo D & Cardenas, 1995).

5 MEASUREMENT

It was necessary to establish measurements conditions as non-saturated soils to evaluate SWCC for each hydrographic determined watershed. Curves were obtained using alternatives methods based on the soil property index and some direct measures by a suction cell prepared especially for these soils. The method for the adjusted estimate is performed using Fredlund & Xing models (1994) and Van Genuchten (1980) because they provide with better adjustment for the available data (Carrillo-Gil, 2008). The suction measurements in the Peruvian Amazon Plain are shown figures 3, 4, 5, 6, and 7.
6 RESULTS ANALYSIS

In general, the considered tropical soils in all Peruvian Amazon plain were clays, limes, and sands that show mixes that generate combine soils in each of the five hydrographic watersheds. For this reason, generalization of the resulted curves is establish only in its format and tendency adopting type A (Clay), B (Clay-Lime), and C (Sand) nomenclature.

The comparison made for each watershed, quality of research soils, geologic origin with respect to the mother rock, and structural heritage evaluated at site have established similarities of SWCC that are shown in figures 8, 9, and 10 for type A, B, and C respectively. These graphs allow the professional to evaluate behavior predictions that in many cases will be used for conceptual projects where suction measurements are difficult to practice (Carrillo-Gil, 2012).

7 CONCLUSIONS

The results analysis from the SWCC showed that Peruvian tropical soils not necessarily show uniformity but a variation and dispersion in accordance with its geological origin and positioning within the great Amazonian rivers. However, considering soil type variations that show wide range, it is found certain similarity of mechanic behavior in accordance with the establish watersheds for this study.

The data of 1,318 samples was review in order to choose 40 tropical soils used in this study to obtain SWCC of similar trends for type A, B, and C evaluated for all watersheds. For type A (clays), it was found the following characteristics: (1) significant water retention (2) high plasticity (3) air entrance value occurs in large suctions (4) suction uncertainty for certain maximum permissible moist content (5) rapid decrease from air entrance value (6) it does not show inflection point, and (7) nule transition zone. For type B (clays-limes), it was found the following characteristics: (1) less plasticity (2) less sudden rupture (3) constant desorption (4) it does not show inflection point (5) easy suction determination for the permissible moist contents, and (6) the air entrance value ranges between values of curves type A & B. For type C (sands), it was found the following characteristics: (1) less water retention (2) sudden rupture until reaching the residual zone (3) it shows inflection point (4) equal suctions for certain moist contents that belong to the transition zone, and (5) the limit effect zone can be very small in some cases, due to the early rupture or air entrance.

It is concluded that the results provide a global vision of the geotechnical characteristics of non-saturated soils from the Peruvian tropic humid. These results will allow in the future the development of new patterns of behavior considering an adequate evaluation and treatment in order to achieve more accurate SWCC. This will concede more numerous and better practical applications for rationalizing special geotechnical materials in many construction projects through out the entire Peruvian Amazon region.

8 REFERENCES (TNR 8)


Finite element analysis of embankments in fine compacted soils

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ABSTRACT: The fine compacted soils (clay or silt) from cuttings can be used in backfill construction, provided they allow optimal compaction and acceptable long-term behavior. Modeling of the mechanical behavior of this unsaturated fine soil is possible by using the finite element method, but numerical models must be validated by experimental work. This paper presents some numerical results from studies on fine compacted soils used in embankment construction subjected to cycles of wetting and drying.

MOTS-CLÉS: sol non saturé, sol fin compacté, remblai, élastoplasticité, analyse aux éléments finis.

KEYWORDS: unsaturated soil, fine compacted soil, embankment, elastoplasticity, finite element analysis.

1 INTRODUCTION

La sensibilité à l’humidification des sols fins argileux ou limoneux a des implications sur le comportement mécanique et hydrique de nombreux ouvrages géotechniques. Dans les travaux de construction de remblais, le problème posé est d’abord celui de savoir si le matériau fin peut être compacté dans des conditions optimales. Puis, on doit s’assurer que le comportement mécanique à long terme permette d’obtenir des déformations acceptables sous les charges mécaniques cycliques dues au trafic, mais aussi sous l’action des cycles de séchage et d’humidification.

L’étude du comportement mécanique des remblais en sols fins compactés montre que la principale source de contraintes mécaniques dans un corps de remblai est son poids propre et fins compactés, donnant lieu à des états de contraintes verticales et horizontales complexes.

Concernant le comportement hydrique des remblais en sols fins compactés, les sollicitations sont diverses : les échanges sol-atmosphère, infiltration par la géo-textile, remontées capillaires par la base du corps du remblai, inondations, induisent des contraintes importantes dans le corps et sur les talus des remblais.

Il existe de nombreux remblais construits, et force est de constater que la plupart d’entre eux ont un comportement satisfaisant du point de vue mécanique : ils assurent un minimum de déformations (ou des déformations acceptables) du point de vue de leur rôle.

Les pathologies constatées sur des remblais en sols fins compactés proviennent principalement des sollicitations hydriques et donc la caractérisation du comportement hydrique de ces matériaux reste essentielle.

De plus, les études de conception des remblais, leur analyse en terme de déformations et encore leur stabilité, passent de plus en plus par l’utilisation de la méthode des éléments fins. Or, la formulation des équations du comportement mécanique et hydrique reste complexe du fait du grand nombre de paramètres (mécaniques, hydriques, d’interaction entre les constituants d’un sol fin), de l’état initial des contraintes et de l’état initial hydrique, ainsi que des conditions aux limites.

Après une courte partie dédiée aux équations du comportement mécanique et hydrique d’un sol fin compacté, cette communication présente quelques aspects pratiques liés à la modélisation par la méthode des éléments fins des sols fins compactés.

Les résultats numériques obtenus sont en accord avec les résultats expérimentaux et montrent la capacité du modèle numérique à prévoir le comportement des sols fins compactés utilisés pour la construction des remblais.

2 MODÉLISATION EN ÉLÉMENTS FINIS

1.1 Les équations du modèle en éléments finis

Pour modéliser numériquement les sols non saturés on suppose souvent que ces sols sont le résultat de la superposition de trois milieux continus, qui, couvrent chacun l’ensemble de l’espace occupé par le sol :
- le milieu continu global (milieu triphasique où on suppose que les mouvements de l’eau et de l’air ont un effet négligeable sur ses propriétés physiques et mécaniques) ;
- le milieu continu air et le milieu continu eau qui peuvent se déplacer l’un par rapport à l’autre à l’intérieur du milieu global et en sortir ou y entrer.

On suppose que les déformations du milieu peuvent être induites par une variation de la contrainte totale (σt+pε) et/ou de la succion (ps), considérées comme variables indépendantes. La loi de comportement est celle proposée par Alonso et al. (1990). C’est une loi de type élastoplastique avec écrouissage. Les contraintes et les déformations doivent satisfaire simultanément les équations d’équilibre et la loi de comportement.
La résolution numérique de ces équations associe la méthode des éléments finis pour discrétiser l'espace et un schéma d'intégration implicite pour discrétiser le temps.

En ajoutant les équations d'équilibre mécanique et les équations qui gouvernent l'écoulement de l'eau et de l'air dans le sol, on obtient un système d'équations décrivant le couplage hydromécanique du système « squelette solide - eau - air ».

En passant par un principe variationnel adapté, on arrive à la forme en éléments finis de ces équations :

\[
\begin{bmatrix}
  R & -C_{uu} & C_{ua} \\
  C_{au} & C_{uu} + K_a \Delta t & C_{aw} \\
  C_{au} & C_{aw} + K_a \Delta t & E_a + K_a \Delta t
\end{bmatrix}
\begin{bmatrix}
  \Delta U \\
  \Delta H_u \\
  \Delta H_a
\end{bmatrix}
= 
\begin{bmatrix}
  C_{uu} H_u - C_{ua} H_u + \Delta F \\
  K_a H_u \Delta t + Q_w \\
  -K_a H_a \Delta t + K_a H_a \Delta t + Q_a
\end{bmatrix}
\]

où \( R \) est la matrice de rigidité, \( \Delta t \) est l'incrément de temps, \( E_a \) est le module de compressibilité de l'air, \( H_u \) et \( H_a \) sont les vecteurs des valeurs de la charge hydraulique et de la charge d'air, respectivement, \( K_u \) et \( K_a \) sont les matrices de perméabilité, \( C_{uu}, C_{aw}, C_{uw}, C_{ua}, C_{au} \) et \( C_{uu} \) sont des matrices de couplage, \( \Delta F \) est le vecteur des forces appliquées, \( Q_w \) est le flux de l'eau et \( Q_a \) le flux d'air, \( S_r \) est le degré de saturation et \( S_{ru} \) le degré de saturation résiduel, \( \gamma_a \) est le poids volumique de l'air, et \( e \) l'indice des vides et \( D \) et \( E \) sont des constantes.

La modélisation proprement dite est faite en utilisant le logiciel de calcul en éléments finis CESAR-LCPC.

1.2 Essai expérimental et résultats numériques

Dans le but d'étudier les transferts hydriques dans les remblais en sols argileux, une expérimentation a été réalisée au Centre d'Expérimentation Routière de Rouen (CER) en collaboration avec l'IFSTTAR, les Laboratoires Régionaux des Ponts et Chaussées d’Aix en Provence et de Toulouse et l’École Universitaire d’Ingénieurs de Lille (EUDIL).

L’argile de Bavent, utilisée pour la construction de ce remblai expérimental, provient d’une carrière de brique de la région de Rouen. Ce matériau a été sélectionné pour trois motifs :

– son utilisation en remblai courant est possible selon les critères du Guide des Terrassements Routiers (GTR) qui fixe les règles de choix et de mise en œuvre des matériaux de remblais routiers en France ;
– une quantité suffisante de ce matériau est disponible en place.
C’est un matériau gonflant.

L’argile de Bavent se présente sous forme d’un mélange à deux couleurs, beige et grise. Un ensemble d’essais d’identification a été effectué dans plusieurs laboratoires impliqués dans l’action de recherche. Les limites de consistance de ce matériau obtenues dans ces laboratoires sont rassemblées dans le tableau 1.

La figure 1 montre la courbe granulométrique de l’argile de Bavent (Alshihabi, 2002). Elle est composée de plus de 85% d’éléments fins (D<80µm). Selon les valeurs des limites de consistance du tableau 1, ce sol peut être classé comme une argile moyennement plastique (A2) ou plastique (A3) selon la classification du Guide des Terrassements Routiers.

La figure 2 montre les résultats d’un essai Proctor normal réalisé au CER de Rouen. La masse volumique sèche à l’optimum Proctor normal vaut \( \rho_{OPN} = 1,71 \ g/cm^3 \). Elle correspond à une teneur en eau optimale de \( w_{OPN} = 19,6\% \).

La figure 3 montre les dimensions des deux parties du remblai. La figure 4 montre la mise en œuvre des deux parties du remblai. Le compactage a été assuré par deux méthodes (pilonneuse et marteau électrique) pour garantir le même compactage au milieu et proche des parois.

Tableau 1. Caractéristiques physiques de l’argile de Bavent déterminées
Les figures 5 et 6 montrent une coupe et une vue en plan du remblai. Elles indiquent l’emplacement de l’ensemble des capteurs dans le remblai.

Les mesures de déplacement pendant une période de plus de 900 jours, avec un cycle hydrique, ont permis de tracer les courbes de déplacement (en mm) en fonction du temps (en jours) (Figure 7). La position symétrique des capteurs aurait dû donner les mêmes courbes de gonflement ou des courbes proches les unes des autres (couples de capteurs S1-S2, S3-S4 et S5-S6). Toutes les courbes ont la même allure. Avant la mise à l’eau, aucune déformation n’a été indiquée par le capteur S5 à la profondeur de 45 cm. Par contre, on a noté un faible gonflement du matériau en S6 et un tassement de l’ordre de 1 mm à proximité de la surface (S1 et S2).

Le comportement de l’argile de Bavent au début des mesures a montré la complexité de son état initial. Le remblai expérimental a été édifié en suivant les mêmes procédures que pour les ouvrages réels. Le matériau a donc subi une forte sollicitation due au compactage. Il a atteint la pression de préconsolidation et l’air dans les pores s’est comprimé. Une fois le compactage achevé, le matériau non saturé a réagi en sens inverse pour décomprimer le volume d’air enfermé dans les pores et il a gonflé.

Avant l’humidification, le sol a été en contact avec l’atmosphère, donc dans des conditions de séchage. Le matériau a donc subi une forte sollicitation due au compactage. Il a atteint la pression de préconsolidation et l’air dans les pores s’est comprimé. Une fois le compactage achevé, le matériau non saturé a réagi en sens inverse pour décomprimer le volume d’air enfermé dans les pores et il a gonflé.

D’autre part, un suivi par nivellement est assuré en trois points de mesure (Figure 6) (Figure 7). La position symétrique des capteurs aurait dû donner les mêmes courbes de gonflement ou des courbes proches les unes des autres (couples de capteurs S1-S2, S3-S4 et S5-S6). Toutes les courbes ont la même allure. Avant la mise à l’eau, aucune déformation n’a été indiquée par le capteur S5 à la profondeur de 45 cm. Par contre, on a noté un faible gonflement du matériau en S6 et un tassement de l’ordre de 1 mm à proximité de la surface (S1 et S2).

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superficielles et se propage ensuite lentement vers les couches profondes à cause de la faible perméabilité de l’argile.

Au cours de l’imbibition, les couches les plus profondes sont restées moins saturées que les couches de surface. D’autre part, le poids volumique augmente avec la profondeur ; c’est pourquoi on observe un gonflement plus important en surface. Le soulèvement atteint 28 mm à 10 cm de la surface (résultat fourni par le capteur S1).

Au début de l’assèchement du remblai, les courbes ont changé de pente et on a remarqué un décalage du début de retrait entre les différentes profondeurs ; comme le séchage s’effectue à l’air, les couches les plus proches de la surface évoluent plus tôt. Les courbes sont restées parallèles et ont varié avec les mêmes pentes.

Les mesures des capteurs positionnés d’une manière symétriques ont les mêmes pentes mais sont décalées. On peut expliquer cette différence par la non-homogénéité du sol, puisque le remblai a été construit avec un sol naturel.

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Figure 7. Gonflement et tassement mesurés par tassomètre à différentes profondeurs, durant le cycle d’humidification-séchage : calculs en éléments finis (courbes EF) et comparaison avec les mesures.

2. CONCLUSION

L’étude de ce modèle réduit pendant deux ans et demi, sous un cycle d’humidification et de séchage, a montré les difficultés et la complexité des conditions initiales dans les ouvrages réels. La non homogénéité du matériau compacté est la source principale de diversité des résultats trouvés. Les effets d’humidification et de séchage sont différents avec la profondeur.

Le remblai relâche ses contraintes totales après le compactage, en cherchant un état d’équilibre. Pendant le cycle, les effets d’humidification et de séchage sont traduits directement par un gonflement puis un retrait dans le matériau. Malgré la différence constatée entre les mesures par tassomètre et par points de nivellement, les résultats dans l’ensemble ont montré une bonne concordance.

Avec des valeurs plus petites d’humidité, on a noté les mêmes variations de la pression interstitielle dans la zone témoin que dans la zone test. Ces valeurs deviennent positives pendant la période d’humidification. Pour la zone expérimentale, on les explique par une saturation du matériau. En revanche, si l’on retrouve ces valeurs positives dans la structure témoin qui n’a pas subi de chargement hydrique, deux possibilités peuvent être imaginées :

- soit, la pression interstitielle positive ne vient pas de l’effet de l’humidification. Tout simplement la pression de l’air dans les pores devient plus importante que la pression de l’eau pour d’autres raisons, comme l’augmentation de la température, par exemple, dans le matériau qui varie pendant cette période entre 18 et 20°C. Dans ce cas, une pression interstitielle positive ne signifie pas forcément une saturation du matériau ;
- soit une variation de l’humidité dans l’air ambiant a conduit à la saturation du matériau de la partie témoin. C’est une possibilité qui est moins acceptable puisque les couches profondes ont également été touchées et présentent les mêmes variations dans la zone test (la séparation entre les parties « remblai expérimental » et la partie « remblai témoin » est faite par une paroi rigide, sans déplacements possibles).

La teneur en eau est le paramètre le plus influencé par l’effet du cycle hydrique :
- avant l’humidification, elle est moins importante près de la surface, augmente vers le milieu de la couche puis diminue en profondeur ;
- pendant l’humidification, elle est plus importante en surface. Elle diminue avec la profondeur ;
- vers la fin d’essai, elle a tendance à revenir vers sa valeur initiale.

On observe que les variations de la teneur en eau sont plus importantes en surface, mais que les mêmes variations se produisent en profondeur, avec un décalage croissant avec la profondeur.

Les premiers résultats obtenus en utilisant le modèle en éléments finis de CESAR-LCPC semblent donner des valeurs de gonflement acceptables, comparables aux valeurs issues des mesures expérimentales, tant pour le gonflement en surface que pour le gonflement en profondeur du remblai. D’autres analyses en éléments finis en cours, concernant la variation de la teneur en eau, du degré de saturation ou encore de la pression interstitielle en fonction du temps permettront de valider le modèle en éléments finis et le rendre utilisable pour la profession.

3. RÉFÉRENCES


Comportement des sols gonflants lors de l’humidification et du séchage

Behavior of swelling soil under cyclic wetting and drying

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ABSTRACT: This article presents the results of wetting and drying laboratory tests conducted on expansive soil. It describes the clays swelling behavior in the wetting process and its shrinkage in the drying process. The results show that the swelling and the shrinking curves do not follow the same path. The characteristic points and shape of the swelling and shrinking curves are qualitatively discussed in the text.

MOTS-CLÉS: gonflement, retrait, adsorption, désadsorption, couche hydratée, diffuse, libre.
KEYWORDS: swelling, shrinkage, adsorption, disadsorption, hydrated layer, diffuse, free.

1 INTRODUCTION

Pour décrire le mécanisme de gonflement et retrait du sol, il est important de connaître et de comprendre la nature et la répartition quantitative de l’eau dans le sol, entre :
- la couche hydratée, qui regroupe les molécules orientées entourant la particule et constitue la frontière avec le reste de l’eau. Suivant la composition chimique de la particule solide et de l’eau, cette couche peut comporter jusqu’à quelques dizaines de couches de molécules dont chacune a une épaisseur de l’ordre de 10⁻⁸ m ;
- la couche diffuse, dans laquelle les cations en solution et la surface chargée négativement des particules solides forment ce que l’on appelle la double couche électrique. La couche diffuse comprend l’eau hygroscopique adsorbée sur la surface des particules solides à partir de l’air du milieu environnant. Sa quantité dépend de l’humidité de l’air et peut varier sensiblement en fonction des conditions atmosphériques ;
- l’eau libre, qui se trouve à l’extérieur de la sphère d’action des forces moléculaires et n’exerce pas d’influence sur le gonflement du sol, mais peut alimenter le gonflement intracrystallin. Elle se divise en eau gravitaire et eau capillaire. L’eau gravitaire possède des propriétés classiques et se déplace dans les pores du sol sous l’action de la pesanteur et de la tension superficielle si elle ne remplit que partiellement les pores du sol, c’est-à-dire quand le sol est un système à trois composants. Lorsque le sol est saturé, il n’y a plus d’eau capillaire et toute l’eau libre se comporte comme l’eau gravitaire.

L’épaisseur de la couche diffuse est variable. Les molécules de la couche diffuse peuvent s’intégrer dans la couche hydratée si le potentiel thermodynamique de la surface de la particule dispersée est supérieur au potentiel électrocinétique de la couche diffuse. Pour les mêmes raisons, la couche diffuse peut absorber une partie des molécules de l’eau libre.

Le déplacement de l’eau liée est impossible quand la teneur en eau du sol est dépassée la capacité maximale d’adsorption du sol. Dans ce cas, chaque particule retient la quantité maximale d’eau, le potentiel à la surface des enveloppes diffuses des particules est nul et il ne se produit pas de migration d’eau.

Dès qu’une partie des molécules d’eau de la couche superficielle du massif de sol commence à s’évaporer, l’équilibre thermodynamique est rompu et il se produit une migration d’eau vers la zone où les couches diffuses sont plus minces jusqu’à ce que l’équilibre soit rétabli dans toute la couche de sol. Les expériences et observations présentées ci-après illustrent les ordres de grandeur de ces phénomènes.

2 VARIATIONS DE LA TENEUR EN EAU D’UN SOL GONFLANT EN LABORATOIRE ET IN-SITU.

Afin de déterminer les relations qualitatives entre les couches hydratée et diffuse et l’eau libre, nous avons réalisé quelques séries d’essais sur des éprouvettes de sol argileux, à l’air libre, dans une cloche en verre contenant un récipient d’eau et à l’étuve. De plus, nous avons observé les variations de la teneur en eau dans un massif de sol, lors de son assèchement par aération dans des conditions naturelles.

2.1 Étude en laboratoire

Procédure d’essai :
Cinq échantillons intacts de sols prélevés sur le site expérimental de Moul El Bergui (région de Safi) ont été testés selon la procédure suivante :
- mesure de la masse initiale de l’échantillon,
- saturation totale ;
- séchage à l'étuve à une température de 105°C ;
- conservation pendant 24 jours à la température constante de 22°C et en atmosphère contrôlée (humidité de l’air 55%) ;
- conservation sous cloche à la température de 20°C avec un récipient contenant de l’eau, jusqu’à la stabilisation de la masse totale de chaque échantillon. L’humidité dans la cloche est passée progressivement de 55 à 100%. La stabilisation a été obtenue au bout de 58 jours ;
- séchage à l’étuve à 105°C pendant 120 heures (5 jours) ;
- séchage à l’étuve à 220°C pendant 72 heures (3 jours).

La masse de l’échantillon a été mesurée à chacune de ces étapes de la procédure d’essai. Leur teneur en eau a atteint 38%.

**Interprétation :**

Nous avons admis que la masse de l’échantillon après séchage à 220 degrés est la masse du sol totalement sec. Cette hypothèse permet de calculer les teneurs en eau successive de l’échantillon au cours du temps (Figure 1). Le sol a une teneur en eau initiale voisine de 20%. La teneur en eau à saturation vaut 38%. Après séchage à l’étude (105°C), la teneur en eau tombe à 1,2 à 2%. L’aération en atmosphère contrôlée produit une teneur en eau finale de 6 à 8%. La réhumidification sous cloche en présence d’eau libre fait remonter la teneur en eau à 15%. Le nouveau passage à l’étuve redonne une teneur en eau de 1,2 à 2%. Nous avons admis que la teneur en eau est nulle après le séchage à 220°C.

Ces données et les informations issues de la littérature nous ont conduits à admettre que la couche hydratée, évaporée entre 105°C et 200°C, correspond à 1,2 à 2% de teneur en eau. Ces données ne permettent pas de distinguer la couche diffuse de l’eau libre.

2.2 **Observations faites sur le terrain :**

Sur le site expérimental d’Ouarzazate (Magnan et al., 2008), lors de l’assèchement du massif de sol saturé (teneur en eau de saturation 35%), il a fallu 35 jours pour l’évaporation de l’eau libre et l’obtention d’un tassement d’environ 4mm. La teneur en eau valait alors 27%. À ce moment, la diminution de la teneur en eau et le retrait du sol s’arrêtèrent et la teneur en eau resta constante et égale à 27% pendant 240 jours. Ceci indique que l’équilibre de la quantité d’eau évaporée et de l’eau adsorbée dans les couches plus profondes s’était établi.

Cet équilibre est celui des couches diffuses. Par conséquent, la teneur en eau libre dans les conditions de cette expérience valait \( w_{dl} = 35 - 27 = 8\% \).

On a déterminé ensuite le volume d’eau évaporé lorsque l’on maintient des éprouvettes de sol durablement à l’air libre à une température élevée (40-43°C). La teneur en eau est tombée à 4%.

Nous avons supposé dans ce cas que la couche hygroscopique liée à la couche hydratée a une épaisseur minimale. La comparaison de ces données avec les résultats présentés plus haut permet de supposer que la couche hydratée correspond à une teneur en eau de 2%. La teneur en eau associée à la couche diffuse est donc égale à 25% et celle de l’eau libre 8%.

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**Figure 1. Variation de la teneur en eau en fonction du temps**
3. ÉTUDE EXPÉRIMENTALE DU RETRAIT ET DU GONFLEMENT

L’étude du mécanisme de gonflement et de retrait du sol argileux lors de l’humidification et de l’aération a été réalisée au moyen de l’appareil de mesure du retrait linéaire développé par le LCPC sur une éprouvette du même sol du site expérimental de Moul El Bergui. (Figure 2).

**Procédure d’essai**


L’humidification est effectuée par injection à la seringue de 2g d’eau à chaque étape, avec mesure du poids total de l’appareil et de l’amplitude du gonflement au cours du temps jusqu’à la stabilisation totale du gonflement.

Ensuite, on effectue l’injection d’eau suivante et ainsi de suite jusqu’à la saturation totale du sol et la stabilisation complète du gonflement à la dernière étape.

À la fin du processus de gonflement, l’injection d’eau a été stoppée et l’éprouvette a été soumise à une aération naturelle à l’air libre dans le même appareil. Des mesures de masse et de déformation de retrait ont été effectuées pendant cette phase.

**Résultats des essais et interprétation**

Les résultats de ces essais de gonflement et retrait du sol sont présentés sur la figure 3.


La pression de gonflement créée par la formation des couches diffuses ne peut provoquer instantanément des déformations de gonflement car les liaisons structurelles dans le sol s’y opposent:

- les liaisons rigides (de cimentation, de cristallisation et de condensation);
- les liaisons plastiques (van-der-Waals, de coagulation, colloïdales, intermoléculaires).

Sur la courbe de gonflement de la figure 3, on peut définir les points et sections caractéristiques suivantes:

- section 0-1 : début de l’adsorption d’eau par la macrostructure du sol, dépassement des forces de van-der-Waals, rupture des liaisons rigides;
- section 1-2 : début de l’adsorption d’eau par la microstructure du réseau cristallin des particules, dépassement de la résistance des liaisons plastiques entre les particules, intensification de la formation des couches diffuses autour des particules et du gonflement du sol;
- section 3-4 : décroissance de l’intensité d’adsorption de l’eau par la microstructure du réseau cristallin des particules ;
- section 4-5 : saturation des couches internes de séparation des phases dans le réseau cristallin des particules. Le point 5 marque la fin de l’adsorption d’eau due à la saturation des surfaces internes du réseau cristallin.

Sur la courbe de retrait à l’air de la figure 3, on peut définir les points et sections caractéristiques suivantes :

- section 5-4' : début de l’évaporation de l’eau libre dans les macropores sans modification des couches diffuses, avec un retrait d’ampleur limitée ;
- section 4'-3' : début de la désadsorption dans la macrostructure du sol et sur les surfaces de séparation des phases dans le système discret. Début de la désadsorption dans les microstructures du réseau cristallin des particules, retrait du sol ;
- section 3'-2' : désadsorption intense dans la macrostructure et la microstructure, retrait intense du sol par suite de la diminution de l’épaisseur des enveloppes diffuses, augmentation de la concentration des sels dans la solution interstitielle ;
- section 2'-1' : apparition de liaisons moléculaires empêchant le retrait du sol ;
- section 1'-0' : apparition de liaisons rigides accompagnant la diminution continue de la teneur en eau du sol. Fin du retrait.

Sur la figure 3, on note la différence des isothermes d’adsorption et de désadsorption, car l’adsorbant retient fortement les substances adsorbées.

L’énergie dépensée pour le gonflement direct du sol n’est pas égale à l’énergie produite par la déformation inverse (retrait). Pendant le processus de retrait, se produit une certaine dissipation d’énergie liée au frottement interne dans le matériau. La dissipation de l’énergie se produit tout lors des déformations plastiques que lors des déformations visqueuses. Dans le premier cas le frottement interne est supposé proportionnel à l’amplitude de la déformation et est appelé frottement sec ou de Coulomb. Dans le second cas, la dissipation est appelée viscosité et est proportionnelle à la vitesse de déformation (frottement liquide). Habituellement, dans les milieux réels, la dissipation d’énergie existe sous ces deux formes.

La partie de l’énergie de déformation dissipée par viscosité provoque dans le sol deux processus conduisant à la diminution de la résistance du matériau à la déformation :
- une réorganisation irréversible de la structure du sol ;
- le réchauffement du milieu, qui conduit, en règle générale, à la diminution de sa résistance à la rupture (Goldštejn, 1971).

Du point de vue de la thermodynamique, au cours de l’écoulement, toute l’énergie est pratiquement dépensée pour le réchauffement du matériau. Si le processus est lent, la chaleur obtenue est dissipée dans le milieu et l’augmentation de la température peut être imperceptible. Le processus peut alors être considéré comme isotherme.

4. CONCLUSION

Les expériences réalisées permettent de tirer les conclusions suivantes :

1. Les processus de gonflement et de retrait du sol doivent toujours être analysés conjointement.
2. Les déformations de gonflement et de retrait des sols sont aux aspects opposés d’un même processus qui se déroule dans le sol au cours du temps en fonction du degré d’humidification ou d’assèchement, ainsi que de la charge appliquée.
3. Lorsque les particules d’argile sont mises en contact avec de l’eau, qui possède son propre champ de force, il se produit une concentration et adsorption des molécules d’eau sur la couche superficielle des particules sous l’action des forces d’attraction moléculaires.
4. Dans la nature, l’adsorption n’est pas réversible car le milieu dispersé retient solublement l’eau, par exemple du fait des modifications chimiques qui se produisent lors de l’adsorption. Le phénomène de retenue de la substance adsorbée par l’adsorbant lors de la désadsorption provoque une hystérésis d’adsorption et traduit la non coïncidence des isothermes d’adsorption et de désadsorption.
5. La structure des systèmes dispersés des sols argileux doit être divisée en deux types principaux :
   - les structures thixotropes de coagulation,
   - les structures cristallisées par condensation.
6. La couche diffuse joue le rôle principal dans le processus de gonflement et de retrait des sols.
7. Les molécules d’eau de la couche diffuse peuvent se mêler à la couche hydratée et adsorber une partie des molécules de l’eau libre.
8. Les études réalisées en laboratoire et in situ ont permis d’établir les pourcentages d’eau de la couche hydratée et de la couche diffuse ainsi que de l’eau libre.
10. Pendant le retrait du sol, il se produit un dégagement d’énergie irréversible, dû au frottement interne du matériau.
11. Dans tous les processus d’humidification et de séchage d’échantillons de sols, pour des variations importantes de la teneur en eau, la formation de la couche diffuse, la déformation de gonflement et la déformation de retrait ont approximativement un caractère linéaire. Ces déformations sont non linéaires dans les parties initiale et finale des courbes qui représentent les déformations de gonflement et de retrait en fonction de la teneur en eau.

5 RÉFÉRENCES

Numerical study of damage in unsaturated bentonite with θ-stock finite element code

Étude numérique d'endommagement pour les milieux poreux non saturés avec le code des éléments finis θ-stock

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ABSTRACT: A coupled thermohydromechanical damage model of Arson and Gatmiri (THHMD model) in an unsaturated quasi-brittle rock mass is briefly indicated in this paper. The model is based on the use of independent state variables (net stress, suction, and thermal stress). The approach has been mixed thermodynamic and micromechanical theories. The stress-strain thermodynamic relations have been derived from the free energy, which has been written as the sum of damaged elastic deformation energies and of residual strain potentials; moreover, the damage rigidities have been computed by applying the Principal of Equivalent Elastic Energy for each stress state variable. Damage has been assumed to have an isotropic influence on air and heat flows, through the inelastic component of volumetric strains. The influence of damage on liquid water and vapor transfers has been accounted for by introducing internal length parameters, related to specific damage-induced intrinsic conductivities. The THHMD model has been implemented in θ-Stock Finite Element code. A numerical study is conducted on the impact of the thermal loading on the response of the unsaturated bentonite.

RÉSUMÉ : un modèle d’endommagement introduit par Arson et Gatmiri (THHMD model) pour un milieu non saturé fragile est brièvement présenté dans cet article. Ce modèle est développé en utilisant des variables d’état indépendantes et en combinant les approches thermodynamique et micromécanique. La relation de déformation-contrainte a été obtenue en dérivant l’énergie libre qui a été considéré comme la somme de l’énergie de déformation élastique endommagée et le potentiel de déformation résiduelle. La rigidité du milieu endommagé est calculée par le principe de l’énergie équivalant élastique (PEEE). L’influence de l’endommagement sur les flux de l’air et de la chaleur est considéré isotrope. L’impact de l’endommagement sur le transfert de l’eau et de la vapeur a été introduit en utilisant les paramètres de la longueur interne qui affecte la conductivité intrinsèque endommagée. Ce modèle a été implanté dans le code des éléments finis θ-Stock. Des études numérique et paramétrique sont menées afin de clarifier les effets du chargement sur la réponse de la bentonite non saturé.

KEYWORDS: Damage, Finite element method, Poromechanics, Thermohydromechanical coupling.

1 INTRODUCTION

Damage modeling has become a crux point in the study of the Excavation Damaged Zone (EDZ) (Martino and Chandler, 2004, Mertens and Bastiaens 2004). In the context of nuclear waste storage, cracking effects have to be accounted for in the constitutive laws of non-isothermal unsaturated porous media (Gens et al. 1998). The geological barriers, often made of quasi-brittle material like granite or clay-rock, undergo damage during the excavation phase. Hydro-mechanical interaction may occur in the neighborhood of the engineered barrier, which is generally constituted of unsaturated compacted clay.

Most of the existing damage models proposed to unsaturated porous media are formulated by means of an effective stress, defined in same way as Bishop’s stress (Arson and Gatmiri 2008.a). These framework are not satisfactory to represent some aspects of the behavior of unsaturated soils, like wetting collapse (Houlsby 1997, Fredlund and Morgenstern 1977). Alternatively, the THHMD model involves independent state variables (net stress, suction and thermal stress), in order to emphasize the role of suction rigidity. Waste is a heat source which can generate traction, and thus cracks. After a brief presentation of the theoretical frame in the first part of this paper, the numerical parametric study which is inspired from a laboratory test is performed in order to determine the main parameters controlling the generation of damage in THHMD model.

2 OUTLINE OF THE MODEL

2.1 Damage representation

It should be mentioned that in this paper, presentation of the main concepts of damage is just for help us to deduce and adapt the deductions to the theoretical concepts of damage.

In the following, the damage variable Ω will be defined as crack density tensor expressed in a principal base:

$$\Omega_{ij} = \sum_{k=1}^{3} n_k \delta_{ij}$$  \hspace{1cm} (1)

Stress and damage will be assumed to have the same principal directions. Physically, the damaged behavior of the RVE is modeled by three meso-cracks representing three main families of fissures. Each meso-crack is characterized by a direction $n_k$ (normal to the crack plane) and a volumetric faraction $d_k$.

2.2 Phenomenological approach

Following the modeling approach of Gatmiri (Gatmiri and Arson 2008.b), it has been assumed that thermal and capillary phenomena are isotropic. Assuming each strain contribution may encompass an elastic (e) and an inelastic (d) part leads to:

$$de_{ij} = (de_{M}^{e} + de_{M}^{d}) + \frac{1}{3}(de_{Sv}^{e} + de_{Sv}^{d})\delta_{ij} + \frac{1}{3}(de_{T}^{e} + de_{T}^{d})\delta_{ij}$$ \hspace{1cm} (2)
M, S and T subscripts refer to a thermodynamic conjugation to net stress, suction and thermal stress, respectively. The free energy breakdown used by Dragon and his coworkers (Drago et al. 2000), for damaged dry materials, is generalized and extended to multiphase media:

\[
\Psi_e (\varepsilon_{M_1} , \varepsilon_{SV} , \varepsilon_{TV} , \Omega_{ij} ) = \frac{1}{2} \varepsilon_{M_1} \cdot D_{ijkl}(\Omega_{ij} ) \cdot \varepsilon_{M_1} + \frac{1}{2} \varepsilon_{SV} \cdot \beta_s (\Omega_{ij} ) \cdot \varepsilon_{SV} \\
+ \frac{1}{2} \varepsilon_{TV} \cdot \beta_T (\Omega_{ij} ) \cdot \varepsilon_{TV}
\]

(3)

The first three terms are the mechanical, capillary and thermal degraded elastic energies respectively. They depend respectively on damage mechanical, capillary and thermal rigidities \( \beta_s (\Omega_{ij} ) \) and \( \beta_T (\Omega_{ij} ) \). The second three terms are residual strain potentials, which quantify the remaining openings due to cracks after unloading. The derivation of the free energy \( \Psi_e (\varepsilon_{M_1} , \varepsilon_{SV} , \varepsilon_{TV} , \Omega_{ij} ) \), provides the whole stress/suction relations. The damage stress \( \gamma_d \) and the damage evolution function, writes:

\[
\gamma_d (\Omega_{ij} , \Omega_{ij} ) = \frac{1}{2} \frac{\partial \Psi_e (\Omega_{ij} )}{\partial \varepsilon_{M_1}} \cdot \varepsilon_{M_1} + \frac{1}{2} \frac{\partial \Psi_e (\Omega_{ij} )}{\partial \varepsilon_{SV}} \cdot \varepsilon_{SV} + \frac{1}{2} \frac{\partial \Psi_e (\Omega_{ij} )}{\partial \varepsilon_{TV}} \cdot \varepsilon_{TV} \\
+ \frac{1}{2} \frac{\partial \Psi_e (\Omega_{ij} )}{\partial \varepsilon_{M_1}} \cdot \varepsilon_{M_1} + \frac{1}{2} \frac{\partial \Psi_e (\Omega_{ij} )}{\partial \varepsilon_{SV}} \cdot \varepsilon_{SV} + \frac{1}{2} \frac{\partial \Psi_e (\Omega_{ij} )}{\partial \varepsilon_{TV}} \cdot \varepsilon_{TV}
\]

(4)

The damage evolution function is assumed to depend on the tensile strains that develop the skeleton. As in many models (Dragon et al. 2000, Homand-Etienne et al. 1998, Shao et al. 2005) (among others), a very simple damage evolution function is used:

\[
f_d (\gamma_d , \Omega_{ij} ) = \frac{\gamma_d}{\gamma_d} + \gamma_d - \gamma_d \cdot \Omega_{ij} \\
C_0 - C_1 \Omega_{ij} \Omega_{ij}
\]

(5)

\( C_0 \) is the initial damage-stress rate that is necessary to trigger damage. \( C_1 \) controls the damage increase rate. The damage evolution law is computed by an associative flow rule (Arson and Gatmiri, 2010).

2.3 Micro-mechanical approach

The elastic components of the strain tensor are determined by computing the damaged rigidities \( D_{ijkl}(\Omega_{ij} ) \) and \( \beta_s (\Omega_{ij} ) \). Damaged Stress state variables are defined (damaged net stress, damaged suction and damaged thermal stress), by using the forth-order operator of corbeidos and Sidoroff (1982) (noted \( M_{ijkl}(\Omega_{ij} ) \)):

\[
M_{ijkl}(\Omega_{ij} ) \sigma_k = (\delta - \Omega_{ij} )^{-1/2} \sigma_j (\delta - \Omega_{ij} )^{-1/2}
\]

(6)

The Principle of Equivalent Elastic Energy is applied on the three elastic potentials \( \frac{1}{2} \varepsilon_{M_1} \cdot D_{ijkl}(\Omega_{ij} ) \cdot \varepsilon_{M_1} \), \( \frac{1}{2} \varepsilon_{SV} \cdot \beta_s (\Omega_{ij} ) \cdot \varepsilon_{SV} \) and \( \frac{1}{2} \varepsilon_{TV} \cdot \beta_T (\Omega_{ij} ) \cdot \varepsilon_{TV} \). The final expressions of the damaged rigidities are:

\[
D_{ijkl}(\Omega_{ij} ) = M_{ijkl}(\Omega_{ij} )^{-1} ; \quad \beta_s (\Omega_{ij} ) = \frac{\gamma_s \beta_s}{[\delta - \Omega_{ij} ]^{1/2} \delta_j} \\
\beta_T (\Omega_{ij} ) = \frac{\gamma_T \beta_T}{[\delta - \Omega_{ij} ]^{1/2} \delta_j}
\]

(7)

\( \gamma_s \) and \( \gamma_T \) are the mechanical, capillary and thermal rigidities in the intact state, respectively.

2.4 Moisture Transfer laws

The details of the modeling of isothermal transfers in porous media may be found in (Gatmiri & Arson 2008b).

Liquid water and vapour transfers are assumed to be diffusive. Hydraulic conductivity is modeled by a second-order permeability tensor \( K_{wij} \):

\[
K_{wij} = k_T (T) G_s (\gamma_{ij} ) K_{int}(\gamma_{ij} , \Omega_{ij} )
\]

(8)

Only the intrinsic water permeability \( K_{int}(\gamma_{ij} , \Omega_{ij} ) \), depending on porosity \( n \), and thus on the behavior of the solid skeleton, may be influenced by damage. A specific crack related component \( k_{2ij} \) is introduced in order to model the influence of damage on liquid water transfer:

\[
K_{int}(\gamma_{ij} , \Omega_{ij} ) = k_{w0}^{10 \nu e r} \cdot \delta_{ij} + k_{2ij} (n^{\nu e r} , \Omega_{ij} )
\]

(9)

The intrinsic permeability related to fracturing is thus a function of the crack densities, \( d^k \):

\[
k_{2ij} (n^{\nu e r} , \Omega_{ij} ) = \frac{2 \nu_{w0}}{12 \mu_w} \cdot \pi^{2/3} A^2 \lambda \gamma_{ij} \sum_{k=1}^{\infty} \left( \delta^k \right)^{5/3} (\delta_{ij} - n_{w0})
\]

(10)

\( \gamma_w \) and \( \mu_w \) are the volumetric weight and the dynamic viscosity of liquid water respectively. \( b \) is the characteristic dimension of the REV and plays the role of an internal length parameter (Arson and Gatmiri, 2010).

3 NUMERICAL RESULTS OF UNSATURATED BENTONITE

3.1 The Simulation

The numerical simulation has been inspired by the laboratory test of Pintado (Pintado et al. 2002), and the simulation has been performed by damage model integrated in 0-Stock Finite Element code (Gatmiri and Arson 2008b). In the Pintado laboratory test, a thermal source is installed between two cylinder-shaped bentonite samples with diameters of 38mm and heights of 76mm which are both wrapped in isolate foam. The bottom being maintained at a constant temperature (Gatmiri et al. 2010). The calculations are performed through axial symmetry. The initial saturation degree \( S_{w0} \) is equal to 0.63 like in the experiment conditions. After a heating period of one week, a relaxation period of seven weeks is observed. All of the imposed boundary conditions are given in Figure 1.
magnitude of damage force in a specific height decreases in comparison with the previous time.

This is because the damage is affected by different parameters: In one side, heating changes stress distribution which is the generator of damage; in the other hand, there is extraction of solid skeleton at the zones near the thermal source, and there is contraction of it at the further zones, which leads closure of openings and reducing of generated stresses. Further, the suction increase is equivalent to a compressive loading, since it diminishes damage stresses. Therefore, we can say that at first, factors generating damage forces induce cracks; afterward, in the following times damage force reduction factors beget to drop damage force which has been produced at former times.

Table 1. Parameters of the five Van Genuchten water retention curves

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<th>α (VG)</th>
<th>n (VG)</th>
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<td>1.2</td>
</tr>
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<td>B</td>
<td>1.87E+08</td>
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</tr>
<tr>
<td>C</td>
<td>2.87E+08</td>
<td>1.429</td>
</tr>
<tr>
<td>D</td>
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</tr>
<tr>
<td>E</td>
<td>3.87E+08</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Figure 1. Bentonite heating test. Boundary conditions.

Figure 2. Element number 22.

Figure 3. Five used Van Genuchten water retention curves

Figure 4. Evaluation of damage in all specimens at the whole testing time

Though the heat source is shut down at the time of one week, the magnitude of damage at the height of about 0.0585 to 0.0665 meters above the bottom of samples is increased at second week in comparison with one week. The increase of damage in mentioned part of the sample even after shutting down the heat source can be discuss as below: It explained before that in the loading phase, there is extraction of solid skeleton at the zones near the thermal source, and there is contraction of it at the further zones. After removing thermal loading, the distribution of temperature in all elements of upper part of sample has got relatively uniform; therefore, the zones with extraction of skeleton get smaller, also the zones with contraction of skeleton extract. The extraction of this part causes the pores get bigger and so that the crack opening rises in this part.

Figure 5, illustrates the damage parameter versus temperature for an element which is located near the heat source (element number 22 which is shown in Figure 2). The samples can be divided into two upside part and downside parts, and the trends of drying or wetting are investigated. With this strategy, during the heating period the upside part of the sample loses its water while the downside part of the sample gets wetter. At the top part of the samples, using the water retention curves shown in Figure 3, it can be concluded that specimen E has the highest amount of moisture and samples D, C, B, A, respectively get drier. Since the chosen element is in up part of the sample, according the above arguments, the specimen A is the drier.
sample. It can be clearly seen in Figure 5, that this sample behaves more brittle in comparison with other samples.

In order to investigate the trend of water permeability, the variation of this parameter due to saturation degree, and damage is drawn respectively in Figures 6 and 7, for an element (No.22) near the heater.

Figure 5. Variation of damage parameter respect to the temperature for all specimens for element 22.

As it shown in Figures 6 and 7 water permeability reduces from A toward E. This is why the specimens get dry from A to E. First that there is no crack in samples, and pores are in their initial size, increasing of saturation degree leads to water permeability reduction. After occurrence of cracks, pores get larger. This factor prompts to grow the permeability, but graphs trend shows that the saturation degree reduction conquers this factor. Therefore, the coupling of these two parameters causes falling trend in water permeability. After removing thermal loading, the magnitude of damage almost remains constant, while by reversing water direction in samples, saturation degree rises and therefore permeability increases.

4 CONCLUSIONS

Theoretical framework of a damage model of Arson and Gatmiri dedicated to non-isothermal unsaturated porous media and formulated in independent state variables (net stress, suction and thermal stress) is presented. The damage model has been implemented in $\theta$-stock Finite Element code. A parametric study on initial damage is then performed to assess the influence of the Excavation Damage Zone (EDZ) on the response of the nuclear waste repository during the heating phase. Different parameters such as suction and thermal stress effect on generation of damage. It is observed that water permeability is mostly affected by the variation of saturation degree of the specimen. Overall, the trends meet the theoretical expectations.

5 REFERENCES


Combination of Shrinkage Curve and Soil-Water Characteristic Curves for Soils that Undergo Volume Change as Soil Suction is Increased

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ABSTRACT: The soil-water characteristic curve, SWCC, is commonly used for the estimation of unsaturated soil property functions, USPF, in geotechnical engineering practice. The indiscriminate usage of the estimation techniques for unsaturated soil property functions can lead to erroneous analytical results and poor engineering judgment. Essentially all estimation procedures for unsaturated soil property functions, USPFs, make the assumption that the soil will not undergo volume change as soil suction is increased. The evaluation of the correct air-entry value has a significant effect on the estimation of subsequent USPFs. This paper describes how the SWCC laboratory results can be properly interpreted with the assistance of a shrinkage curve. Laboratory data sets are then used to illustrate how the test data should be interpreted for high volume change soils.

RÉSUMÉ : La courbe des propriétés hydriques des sols, SWCC, est communément utilisée afin d’estimer les fonctions des propriétés des sols non saturés, USPF, en géotechnique. L’usage abusif des techniques d’estimation des fonctions des propriétés des sols non saturés peut mener à des résultats analytiques erronés et à un mauvais jugement au point de vue ingénierie. Toutes les procédures d’estimation pour les fonctions des propriétés des sols non saturés, USPF, font l’hypothèse que le sol ne subira aucun changement de volume avec une augmentation de la succion. L’évaluation d’une valeur d’entrée d’air correcte a un effet significatif sur l’estimation des subséquents USPF. Cet article décrit comment les résultats de laboratoire des SWCC peuvent être correctement interprétés avec l’utilisation d’une courbe de retrait. Des résultats d’essais en laboratoire sont utilisés pour illustrer comment les données d’un essai doivent être interprétées pour des sols subissant d’importants changements de volumes.

KEYWORDS: soil-water characteristic curves, shrinkage curve, volume change, soil suction, unsaturated soil property functions.

1 INTRODUCTION

The soil-water characteristic curve, SWCC, provides vital information for applying unsaturated soil mechanics in engineering practice. Much of the information regarding the use of SWCC originated in soil physics and agriculture-related disciplines. With time, information regarding the use of the SWCC has been embraced for geotechnical engineering applications (Fredlund, 2002; Fredlund and Rahardjo, 1993).

A common difficulty arises when large volume changes occur in the soil as soil suction is increased. Sludge material and slurry material may be deposited at water contents above the liquid limit of the material. The material is deposited in ponds and allowed to dry in order to increase its shear strength. The geotechnical engineer may be called upon to undertake numerical modeling simulations of the drying process.

2 SIGMOIDAL EQUATIONS FOR SWCCS

There are several sigmoidal type equations that have been proposed to mathematically describe the water content versus soil suction relationship (e.g., van Genuchten, 1980; Fredlund and Xing, 1994). The S-shaped sigmoidal equations have the appearance of being able to fit SWCC data regardless of the measure that is used to represent the amount of water in the soil (e.g., gravimetric water content, volumetric water content, or degree of saturation). The Fredlund and Xing, (FX), (1994) SWCC equation can be used to illustrate the usage of a sigmoidal equation for various designations of water content. The FX (1994) equation uses a correction factor that allows all SWCCs to go to zero water content as soil suction goes to 1,000,000 kPa. The FX (1994) equation is first written in terms of gravimetric water content and can then be used to best-fit the SWCC.

\[
w(\psi) = w_s \left[ 1 - \frac{1}{\ln(10^6/\psi)} \ln \left( \exp(1)+\left(\frac{\psi}{\psi_r}\right)^{n_s}\right) \right] \]

where: \(w(\psi) = \text{gravimetric water content at any specified suction, } \psi; w_s = \text{saturated gravimetric water content}; \psi_r = \text{residual soil suction}; a_s, n_s, \text{ and } m_f = \text{the fitting parameters for the SWCC equation. Equation 1 is written for the gravimetric water content designation; however, the equation could also be best-fit volumetric water content or degree of saturation versus soil suction. The gravimetric water content SWCC can be used in conjunction with a shrinkage curve to more accurately interpret the parameters required for the estimation of unsaturated soil property functions.}

The degree of saturation versus soil suction can be computed by combining Eq. 1 with the shrinkage curve data. The volumetric water content versus soil suction SWCC is also required to obtain the water storage coefficient for the soil. The volumetric water content must be related to the instantaneous overall volume of the soil mass in order to obtain the correct value for numerical modeling purposes. Volume change of the overall soil specimen can be taken into consideration if a “shrinkage curve” is measured. The shrinkage curve is generally measured under conditions of zero net normal stress.
3 USE OF A SHRINKAGE CURVE

The entire shrinkage curve, (i.e., the plot of total volume (or void ratio) versus gravimetric water content), from an initially saturated soil condition to completely oven-dry conditions is of value for the interpretation of SWCC data. As saturated clay soil dries, a point is reached where the soil starts to desaturate. Upon further drying, another point is reached where the soil dries without significant further change in overall volume. The corresponding gravimetric water content appears to be close to residual soil suction.

The shrinkage curve can be experimentally measured from initial high water content conditions to completely dry conditions. A digital micrometer can be used for the measurement of the volume at various stages of drying as shown in Figure 1. Brass rings can be used to contain the soil specimens (i.e., the rings have no bottom). The rings with the soil are placed onto wax paper and dried through evaporation. The dimensions of the specimen are appropriately selected such that cracking of the soil is unlikely to occur during the drying process. The initial dimensions selected for the shrinkage curve specimens used in this study were a diameter of 3.7 cm and a thickness of 1.2 cm.

The mass and volume of each soil specimen can be measured once or twice per day. Four to six measurements of the diameter and thickness of the specimen were made at differing locations on the specimens. It has been observed that as the specimen diameter begins to decrease, with the specimen pulling away from the brass ring and the rate of evaporation increases.

The “shrinkage curve” can be best-fit using the hyperbolic function proposed by Fredlund et al., (1996, 2002). The equation has parameters with physical meaning and is of the following form:

$$e(w) = a_{sh} \left[ \left( \frac{w}{b_{sh}} \right)^{c_{sh}} + 1 \right]^{D}$$

where: $a_{sh}$ = the minimum void ratio ($e_{min}$), $b_{sh}$ = slope of the line of tangency, (e.g., $= e / w$ when drying from saturated conditions), $c_{sh}$ = curvature of the shrink-age curve, $w$ = gravimetric water content, $G_s$ = specific gravity and $S$ = degree of saturation.

Once the minimum void ratio of the soil is known, it is possible to estimate the remaining parameters required for the designation of the shrinkage curve. The minimum void ratio the soil can attain is defined by the variable, $a_{sh}$. The $b_{sh}$ parameter provides the remaining shape of the shrinkage curve. The curvature of the shrinkage curve commences around the point of desaturation is controlled by the $c_{sh}$ parameter.

4 DEGREE OF SATURATION

The degree of saturation of the soil can be written as a function of gravimetric water content (as a function of suction) and void ratio (as a function of gravimetric water content).

$$S(w) = \frac{w G_s}{a_{sh} \left[ \left( \frac{w}{b_{sh}} \right)^{c_{sh}} + 1 \right]^{D}}$$

The degree of saturation can be further written as a function of gravimetric water content and the equation for the shrinkage curve, both which are functions of soil suction.

5 RESULTS ON REGINA CLAY

The effect of volume change on the interpretation of SWCCs was studied for Regina clay. The laboratory test results are presented and show significance of overall volume change on the interpretation of the SWCC.

The air-entry value, AEV, for Regina clay was determined from the degree of saturation SWCC remained a constant around 2500 kPa. An empirical construction procedure involving the intersection of two straight lines on a semi-log plot was used to determine a single number associated with the AEV remained constant. (This was confirmed by the experimental results). The “$w$ Break” on the gravimetric water content SWCCs were then compared to the air-entry value for the soil. The ratio of AEV to $w$ Break was used as a measure of the
The effect of volume change on the interpretation of the correct air-entry value for the soil.

Shrinkage curves and soil-water characteristic curves were measured on Regina clay. Slurry Regina clay was prepared at a gravimetric water content slightly above its liquid limit. The shrinkage curve results are presented in Figure 2. The void ratio of Regina clay decreases as water evaporates from the soil surface. The clay begins to desaturate near its plastic limit. The best-fit parameters for the shrinkage curve are $a_b = 0.48$, $b_b = 0.17$, and $c_b = 3.30$. The specific gravity of the soil was 2.73.

Figure 3 shows the gravimetric water content, $w$, plotted versus soil suction for Regina clay was preloaded at 196 kPa. Its initial water content was 53.5%. The high water content specimen showed that a gradual break or change in curvature around 50 kPa. The curvature is not distinct and does not represent the true air-entry value of the material. The gravimetric water content SWCC was best-fit with the FX (1994) equation and yielded the following parameters; that is, $a_f = 140$ kPa, $n_f = 0.87$, and $m_f = 0.72$. Residual suction was estimated to be around 200,000 kPa. It is necessary to use the shrinkage curve to calculate other volume-mass soil properties and properly interpret the SWCC results for the true AEV.

Figure 2. Shrinkage curve for several samples of Regina clay.

The best-fit shrinkage curve equation can be combined with the equation for the FX (1994) equation for the SWCC. The resulting plot of degree of saturation, $S$, versus soil suction is shown in Figure 4. The results show that there is a distinct air-entry value for Regina clay is about 2,500 kPa. The true air-entry value was also found to be similar for all preconsolidated Regina clay samples. The degree of saturation SWCCs must be used to estimate the AEV of the soil and subsequently the calculation of the unsaturated hydraulic conductivity function. The degree of saturation also indicates that residual condition can be more clearly identified as being at a suction of about 200,000 kPa (i.e., residual degree of saturation of 20 percent).

Several other SWCC tests were performed on the Regina clay; each test starting with soil that had been preconsolidated from slurry to differing applied pressures. Figure 5 shows the gravimetric water content versus soil suction plot for a soil preconsolidated to 6.125 kPa. The FX (1994) fitting parameters are $a_f = 18.0$ kPa, $n_f = 0.88$, $m_f = 0.76$ and $h_r = 800$ kPa.

Figure 6 shows the gravimetric water content versus soil suction plot for a soil preconsolidated to 49.0 kPa. The FX (1994) fitting parameters are $a_f = 90.0$ kPa, $n_f = 1.10$, $m_f = 0.70$ and $h_r = 2000$ kPa. Figure 7 shows the gravimetric water content versus soil suction plot for Regina clay preconsolidated to the highest pressure of 392 kPa. The FX (1994) fitting parameters are $a_f = 120.0$ kPa, $n_f = 0.84$, $m_f = 0.70$ and $h_r = 2000$ kPa.

6 INTERPRETATION OF THE REGINA CLAY RESULTS

The difference between the break in the gravimetric water content SWCC and the true AEV for Regina clay is expressed as \[\text{AEV}/(\text{Break in curvature on } w \text{ SWCC})\]. The volume change of the soil is once again expressed as the change in void ratio, $\Delta e$, divided by $(1 + e)$ and all void ratio values are determined from the shrinkage curve.

The horizontal axis of Figure 8 shows that the Regina clay soil specimens changed in volume by 65% to 150% as soil suction was increased to residual suction conditions. At 70% volume change, the true AEV is 60 times larger than the break in curvature indicated by the gravimetric water content SWCC. Also at 120% volume change, the true AEV is 129 times larger than the break in curvature indicated by the gravimetric water content SWCC. The laboratory test results clearly indicate the significant influence that volume change as soil suction increases has on the interpretation of the data.
The laboratory SWCC test results on Regina clay illustrate the need to separate gravimetric water content SWCC into two components. Part of the change in water content is due to a change in volume while the soil remains saturated. The other part of the change in water content is associated with a change in degree of saturation.

The proposed estimation procedure based on the SWCC and the saturated hydraulic conductivity makes the assumption that the reduction in hydraulic conductivity with suction needs to be accommodated in an independent manner.

Figure 7. Gravimetric water content versus soil suction for Regina clay preconsolidated to 392 kPa.

Figure 8. Difference between the break in the gravimetric water content SWCC and the Air-Entry Value for Regina clay.

The estimation of the permeability function with respect to a change in suction can now be considered as having two components; one component due to a change in void ratio and the other components due to a change in the degree of saturation. Further research should be undertaken to verify that the unsaturated soil property functions can indeed be estimated for all types of material by using the interpretation procedure suggested in this paper.

7 CONCLUSIONS AND RECOMMENDATIONS

Changes in the volume of the soil specimens as soil suction is increased can significantly affect the interpretation of the SWCC. This paper presents a procedure that can be used to independently consider the effects of volume change (where the soil remains saturated) from the desaturation of the soil specimen. The effects of volume change are shown to be significant, resulting in erroneous calculations of the permeability function for a soil.

8 REFERENCES

Small-strain shear modulus and shear strength of an unsaturated clayey sand

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ABSTRACT: This paper presents and discuss results of laboratory tests with a compacted clayey sand. Two sets of tests were performed to analyze the shear strength and the small-strain shear modulus ($G_s$) of this soil. The first set consisted of triaxial compression tests performed on saturated specimens and multistage triaxial compression tests with controlled suction on unsaturated specimens, which were sheared under constant water content. The second set consisted of tests to measure the $G_s$ of specimens under constant suction using the bender elements technique. The results allowed observing the development of suction during shearing and influence of suction and confining stress on the shear strength and on the small-strain shear modulus of the tested soil. A planar failure envelope could nicely fit experimental data of shear strength. The influence of net confining stress on the small-strain shear modulus reduced as the suction increases. The shear modulus tended to increase non-linearly with suction, seeming to reach an asymptotic value.

RÉSUMÉ : Cet article présente et discute les résultats des essais de laboratoire sur un sable argileux compacté. Deux séries d’essais ont été effectuées pour analyser la résistance au cisaillement et le module de cisaillement en petites déformations ($G_s$) de ce sol. La première série comprenait des essais de compression triaxiaux réalisés sur des échantillons saturés et des essais de compression triaxiale avec succion contrôlée sur des échantillons non saturés, qui ont été cisaillés en teneur en eau constante. Les essais de la deuxième série ont servi à mesurer le $G_s$ d’échantillons sous succion constante en utilisant la technique des bender éléments. Les résultats obtenus ont permis l’observation de la succion pendant le cisaillage du sol. Ils ont aussi permis d’observer l’influence de la succion et de la contrainte de confinement sur la résistance au cisaillement et le module de cisaillement du sol testé. Une enveloppe de rupture plane pourrait bien représenter les données expérimentales de résistance au cisaillement. L’influence de la contrainte de confinement sur le module de cisaillement diminue avec l’augmentation de la succion. Le module de cisaillement a tendance à augmenter de façon non linéaire avec succion, semblant atteindre une valeur asymptotique.

KEYWORDS: Unsaturated soil, shear modulus, bender elements, shear strength.

1 INTRODUCTION

Unsaturated soils are present in large areas worldwide, especially in tropical and subtropical zones. Soil water in unsaturated soil is under suction, which is known to influence the mechanical and hydraulic properties of the soils. For instance, it is known that shear strength increases with suction up to some limit value and that hydraulic conductivity tends to decrease with suction by many orders of magnitude, depending on the level of suction considered. Several laboratory techniques have been developed or adapted to evaluate the suction influence on key properties of the soils, such as the axis translation technique (Hilf 1956) used to control soil suction in triaxial and consolidation tests. In particular, triaxial compression tests on unsaturated samples are used not only to evaluate shear strength but also to estimate soil stiffness through the analyses of stress-strain curves. However, these stiffness measurements face many drawbacks as pointed out by Atkinson and Sallfors (1991). An attempt to overcome such difficulties is using local displacement transducers (Jardine et al. 1984) and bender elements (Dyvik and Madshus 1985) in conventional laboratory apparatuses. Specifically, bender elements technique has increasingly called the attention of researchers over the past decades. This technique intends to be a simple and accurate method to measure the small-strain shear modulus of the soil (Dyvik and Madshus 1985). When compared to other methods of obtaining the small-strain shear modulus, bender elements technique provided good agreement or slightly overestimated values in tests performed by Youn et al. (2008) and Hoyos et al. (2008).

This paper presents test results performed to investigate the shear strength and the shear modulus at small strains of a compacted sandy soil formed under tropical environment. The shear strength was obtained through triaxial compression tests, which were conventional consolidated-drained for saturated soil and constant water content tests for unsaturated soil. The small-strain shear modulus was measured using bender elements in both saturated and unsaturated soil at different suction and under different isotropic confining pressures.

1 SOIL PROPERTIES AND SPECIMENS PREPARATION

The soil used in this investigation is a clayey medium to fine sand of colluvial origin. It was under the action of pedogenetic processes typical of tropical environments, where periods of intense rain and elevated temperature induce severe weathering process. As a result, bases and silica are removed, remaining the more resistant minerals such as quartz. Resulting clay fraction is composed by iron and aluminum oxides and kaolinite. The soil presents liquid and plastic limits of 32 and 16%, respectively. The fine to medium sand fraction is 67% and the clay content is 28%. According to the Unified Soil Classification System it is a SC soil. Standard Proctor parameters are maximum dry density of 1.80 g/cm³ and optimum water content of 13.8%.

The triaxial compression and bender element tests were performed with dynamically compacted specimens molded with
13.8% of water content and relative compaction of 95% of maximum dry density from Standard Proctor.

The soil-water retention curve for such compaction condition is shown in Figure 1. Plate funnel and filter paper techniques were used to obtain the experimental data on drainage path. The model proposed by Fredlund and Xing (1994) was fitted to experimental data, providing Equation 1 to represent the soil-water retention curve.

\[ S_r = 33.6 + \frac{100 - 33.6}{\ln \left[ \frac{\psi}{3.62} \right]^{0.32}} \]  

where \( S_r \) is the saturation degree and \( \psi \) is the matric suction. This is a typical curve of sandy soil, with low air-entry value and large desaturation for suction variation between 4 and 10 kPa. The relative low dry density after compaction also contributes to these features.

2 TESTING PROCEDURES

2.1 Shear strength

In this experimental program, triaxial compression tests were performed to investigate the shear strength of the soil. The tests comprised shearing five saturated specimens under consolidated-drained (CD) condition. Additionally, three constant water content (CW) triaxial tests were carried out on unsaturated specimens, which were previously led to some known suction. The multistage technique (Ho and Fredlund 1982) was chosen for these latter experiments in order to reduce their duration, as suction equilibrium is a known time consuming process.

In the CD tests, the specimens were saturated by backpressure until the parameter B was at least 0.95. Next, the soil was isotropically consolidated and then sheared at a strain rate of \( 10^{-6}/s \), which is lower than the calculated from the consolidation stage. The effective confining pressures were of 50, 100, 150, 300 and 500 kPa.

For the CW tests, the specimens went through a pre-testing procedure, which consisted of reducing the compaction-induced suction to zero by capillary rising, and then imposing target suction via axis translation technique in auxiliary chambers. In the CW tests, the soil was consolidated maintaining constant suction \( (u_t - u_a) \), where \( u_t \) and \( u_a \) are respectively pore-air and pore-water pressures, and increasing the net confining pressure \( (\sigma_3 - u_t) \) at each stage. The net confining pressures were of 50, 150, 300 and 500 kPa, and target initial suctions were of 15, 40 and 100 kPa. At the end of consolidation, pore-water pressure was allowed to equilibrate by closing the water drainage valve. Then, suction at the beginning of shearing was computed from the difference between pore-air and water pressures. Maintaining pore-water undrained and pore-air pressure controlled, shearing was carried out at a strain rate of 6.7 x \( 10^{-6}/s \) until axial strains of about 5%, where stress approximately leveled off. In the fourth stage, the specimen was allowed to shear up to larger strains. The strain rate of CW tests was selected based on data of triaxial compression tests on different soils types gathered by Fredlund and Rahardjo (1993) and also on tests presented by Georgetti and Vilar (2010) in the same soil.

2.2 Small-strain shear modulus

The experimental program also comprised tests to measure saturated and unsaturated soil shear modulus. Saturation and suction imposition prior to testing were carried out following the procedure described in 3.1. During the tests, the specimens were consolidated in triaxial cells with bender elements embedded in the top caps and base pedestals. Different levels of isotropic stress were applied keeping saturation or constant suction. This paper focus on part of these experiments in which the soil were confined in steps from 10 to 500 kPa of isotropic effective stress or net stress, for the saturated and unsaturated specimens, respectively. At the end of each confining step, shear waves were transmitted through the soil by the bender elements. Input signals were single sine pulses with a voltage of 14 V and frequencies that ranged from 1 to 50 kHz. Such wide range of frequencies allowed selecting the pulses with minor signal interferences of near-field effects. The small-strain or maximum shear modulus of the soil was then calculated by Equation 2.

\[ G_a = \rho V_s^2 \text{ with } V_s = L/t \]  

where \( \rho \) is the density of the soil; \( V_s \) is the shear wave velocity; \( L \) is the wave path length, taken as the distance between the tips of source and receiver bender elements; and \( t \) is the shear wave travel time. The shear wave travel time was estimated as the first arrival of the received signal, more specifically, as the time interval between the transmitted and the first major deflection of the received signal. Besides being straightforward, the first arrival has been recommended as a reliable method for calculating the shear velocity (see Chan 2010 and Clayton 2011, for instance).

Four tests were carried out following the above procedure, with suction ranging from zero (saturated soil) to 100 kPa.

3 RESULTS AND DISCUSSIONS

3.1 Shear strength

From the triaxial compression tests with both saturated and unsaturated soil, the maximum deviator stresses were used to obtain shear strength parameters. It was possible to fit two shear strength envelopes: one, up to approximately 200 kPa of stress and other considering the stresses larger than 200 kPa. The corresponding shear strength parameters were effective cohesion \( (c') = 10 \text{ kPa} \) and effective friction angle \( (\phi') = 31^\circ \), and \( c' = 0 \) and \( \phi' = 33^\circ \), respectively.

Regarding constant water content triaxial compression tests in the unsaturated compacted soil, typical stress-strain and suction-strain curves are shown in Figures 2 and 3, respectively. Figure 2 shows that shear strength increased at each stage and small elastic strains were recovered when the specimen was axially unloaded to beginning the next stage of testing. The visual inspection of specimens and the format of the stress-strain curves indicate that no distinct failure plane was formed and therefore is reasonable to use the multistage test to evaluate the shear strength of this soil. Figure 3 reinforces a tendency of suction variation that was already observed by Georgetti and Vilar (2010): after an initial decrease, suction tended to increase for the lower net confining stresses and remained approximately constant for the larger confining stresses. Rahardjo et al. (2004)
also noticed suction reduction for axial strains until about 3% and smaller changes as shearing progressed after that. The authors performed CW tests on a compacted residual soil from the Jurong sedimentary formation using a suction range that is above the range chosen for the tests presented in this paper. As pointed out, target initial suctions used in this experimental program varied between 15 and 100 kPa. However, after consolidation, the suction was allowed to vary and suctions at failure ranged from 2 to 43 kPa.

3.2. Small-strain shear modulus

From the tests performed with bender elements, the shear wave velocities were calculated and plotted against the wave path length (L) to wavelength (λ) ratios. The wavelength was estimated from the relation between V_s and the frequency of the input signal. According to Sanchez-Salinero et al. (1986) and other authors, the L/λ is useful to select signals with reduced near-field effects. An analysis of the results of the saturated and unsaturated bender element tests indicated that the shear wave velocity tended to remain constant when L/λ is larger than 3. Therefore, the wave velocity in each stress condition was taken as the average velocity for L/λ ≥ 3.

The influence of isotropic confining stress and suction on the small-strain shear modulus of the soil is shown in Figures 5 and 6. Both figures readily indicate that an increase in any of these variables is able to rise G_s, which ranged from 78 to 468 MPa. In Figure 5, it can be noticed that potential curves (Equation 4) nicely fitted the experimental data of G_s versus confining stress.

The parameters obtained from the fits can be seen in Table 1.

\[ G_s = a (\sigma_2 - u_s)^b \]  \( (4) \)

where G_s is in MPa, (σ_2 - u_s) represents either effective or net isotropic stress in kPa, and a and b are empirical fitting parameters.

In Table 1, some aspects of the influence of suction on the small-strain shear modulus can be observed. The parameter a increased with increasing suction. Moreover, the constant b decreased when suction increased, which means that the net confining stress has more influence on the soil under lower suctions than on the soil under higher suctions.

The small-strain shear modulus variation with suction is shown in Figure 6, where a hyperbolic function (Equation 5) better suited the experimental data in comparison to the potential fit. Table 2 shows the parameters of Equation 5 for each confining stress.

\[ G_s = G_{s,sat} + \frac{u_s - u_a}{m + n (u_s - u_a)} \]  \( (5) \)

where G_{s,sat} is the small-strain shear modulus of the saturated soil, m and n are empirical fitting parameters, G_s and G_{s,sat} are in MPa, (u_s - u_a) in kPa.

Data from saturated and unsaturated tests were gathered in the space [(σ_1 - σ_3)/2], [(σ_1 + σ_3)/2 - u_s] and (u_s - u_a). Thus, points of maximum ordinates of the Mohr circles and the associated suction at failure were plotted in Figure 4. It is possible to notice that the points are approximately distributed along a plane surface. Taking this into account, the experimental points were fitted by a planar failure surface according to the proposition of Fredlund et al. (1978), resulting into a nice surface fitting with determination coefficient (r^2) of 0.99. The obtained shear strength parameters were transferred to the τ (σ - u_s) and (u_s - u_a) space, resulting in a τ of 8 kPa, φ of 32° and θ, the friction angle with respect to suction, equal to 27°. The planar shear strength envelope, expressed in Equation 3, shows an internal friction angle that is quite close to the value derived from the saturated shear strength envelope. The friction angle with respect to suction was lower than φ and the cohesion obtained from the three-dimensional fit was approximately the same value obtained from the saturated envelope.

\[ \tau = 8 + (\sigma - u_s) \tan 32° + (u_s - u_a) \tan 27° \]  \( (3) \)

where τ is the shear strength and (σ - u_s) is the net normal stress.
As can be seen, the experimental data is nicely fitted by a non-linear relationship between small-strain shear modulus and the variables suction and confining pressure. The non-linearity between shear modulus and net confining stress or soil suction was also observed for sands by Nyunt et al. (2011) who used, however, different fitting equations to represent these relationships.

The hyperbolic fit suggests that $G_s$ tends asymptotically to a limit value, which for the range of suction tested, seems to be close to the shear modulus associated to the suction of 100 kPa. However, tests at larger suction should be performed to confirm this behavior.

This behavior was probably associated to the small range of soil suction registered at failure, since some non-linearity has been reported in the literature to affect the relationship between shear strength and suction.

The wave path length to wavelength ratio was used to evaluate the interference of near-field effects on the shear wave velocities and resulting shear modulus. The small-strain shear modulus tended to increase non-linearly with net confining stress and soil suction. The increasing of shear modulus with net confining stress was more significant as suction was reduced. The shear modulus has increased with suction, seeming to approach an asymptotic value.

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6 REFERENCES


Étude de la stabilité des pentes non saturées sous les effets de l’infiltration prenant en compte la végétation

Study of the stability of unsaturated slopes under the effects of infiltration taking into account the vegetation

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RÉSUMÉ : L’effet de l’extraction d’eau par des racines sur la stabilité des pentes est étudié. Les variations de la teneur en eau et de la succeion dans le sol influencées par l’extraction d’eau par des racines ont été calculées par la méthode des éléments finis. La profondeur d’influence de la variation de la succeion pour la stabilité est calculée en fonction des paramètres hydromécaniques du sol. On peut voir que l’effet de la variation de la succeion sur la stabilité des pentes et par conséquent l’effet de l’extraction d’eau par des racines est limité à cette profondeur.

ABSTRACT: In this work the effect of root water uptake on slopes stability is studied. Variations of soil water content and soil suction influenced by root water uptake are calculated using finite elements method. The depth of influence of suction variation on slope stability is calculated as a function of soil hydro-mechanical properties. It can be seen that the effect of variations of soil suction on slope stability is a function of soil properties and therefore root water uptake effect on slope stability is limited to this depth.

Mots-clés : stabilité des pentes, extraction d’eau par des racines, sol non saturé

Key words : slope stability, root water uptake, unsaturated soil

1 INTRODUCTION

L’infiltration d’eau dans un sol non saturé influence la stabilité des pentes. L’infiltration d’eau a deux effets : d’une part, elle augmente le poids du sol humide et d’autre part elle réduit la succeion dans le sol, ce qui entraîne une réduction de la résistance au cisaillement du sol. Les variations de degré de saturation et par conséquence la succeion dans la profondeur du sol peuvent être déterminées en fonction du taux d’infiltration et des paramètres hydriques du sol.


2 CADRE THÉORIQUE

L’infiltration d’eau et l’extraction d’eau par des racines sont présentées très brièvement dans cette partie.

2.1 Infiltration

L’infiltration d’eau dans un sol partiellement saturé peut être décrite par l’équation de Richards :

\[
\frac{d(\rho_\theta)}{dt} = -\nabla(\rho_\theta KH) + f \tag{1}
\]

où \( \theta = S_r \) est la teneur en eau volumique, \( K \) est la conductivité hydraulique (perméabilité) du sol en fonction du degré de saturation et \( \Delta H \) est le gradient de la charge hydraulique. \( f \) est le terme d’extraction qui peut être relié à l’extraction d’eau par des racines. L’équation (1) en fonction de \( \theta \) et \( H \), peut être également écrite en fonction de la pression d’eau \( p_w \) et du degré de saturation \( S_r \) :

\[
\frac{nS_r d\rho_{sw} + dS_r}{\rho_w dp_w} \frac{dS_r}{dt} = -\nabla \left( \frac{K}{\rho_w g} \nabla (p_w) + K \right) + f \tag{2}
\]

La perméabilité d’un sol non-saturé est une fonction de sa perméabilité à l’état saturé et du degré de saturation. L’expression suivante est adoptée dans cette étude :

\[
K = k_s \left( \frac{S_r - S_{sat}}{1 - S_{sat}} \right) \tag{3}
\]

où \( k_s \) et la perméabilité du sol saturé et \( S_{sat} \) est le degré de saturation résiduel. On adopte également l’expression de la courbe de rétention d’eau proposé par van Genuchten (1980) :

\[
S_r(p_w) = S_{sat} + (1 - S_{sat}) \left[ 1 + \left( \frac{\alpha_p}{\rho_w g} \right)^m \right]^{-n} \tag{4}
\]

où \( \alpha_s \), \( m \) et \( n \) sont des paramètres du matériau.
2.2 Extraction d'eau par des racines

La plupart des modèles d’extraction d’eau sont dérivés du modèle macroscopique de Feddes et al. (1978). Les différences résident en général dans la méthode de détermination des paramètres. Le terme d’extraction \( f \) dans l’équation (2) est donné pour chaque profondeur en fonction de l’extraction maximale et une fonction réductrice \( \alpha \) :

\[
    f = \alpha(S_f) f_{\text{max}}
\]

Dans le modèle 2D proposé par Indraranta et al. (2006) utilisé par Hemmati (2009) et Hemmati et al. (2011), le terme d’extraction d’eau maximale est défini par :

\[
    f_{\text{max}}(r, z, t) = G(\beta) F(T_p)
\]

où \( G(\beta) \) est la fonction de distribution de densité des racines et \( F(T_p) \) est la fonction de distribution de transpiration potentielle sur la profondeur \( z \) et \( T_p \) est la transpiration potentielle.

3 MODÉLISATION DE L’EXTRACTION D’EAU PAR DES RACINES SUR UNE PENTE PARTIELLEMENT SATURÉE

Une pente partiellement saturée avec des racines distribuées sur une profondeur de 1 mètre à partir de la surface du sol est modélisée (Figure 1). L’épaisseur de la couche modélisée est de 10 mètres et la nappe phréatique est considérée sur la base de la pente parallèle à la surface. La pression d’eau sur la surface est donc d’environ -100 kPa avec une distribution initialement linéaire dans la profondeur. La courbe de rétention d’eau est présentée sur la Figure 2 (courbe rouge). Les variations du degré de saturation et de la succion sont présentées sur la Figure 3. Les variations dans le profil de la succion montrent l’effet de l’extraction d’eau par des racines qui pourra se traduire par une augmentation de la résistance au cisaillement du sol. La variation du degré de saturation peut influencer la stabilité des pentes seulement jusqu’à une certaine profondeur, ce qui est étudié dans la section suivante.

4 ZONE D’INFLUENCE DE NON-SATURATION

Dans les sols non saturés le facteur de sécurité d’une pente infinie est influencé par la succion dans le sol (Figure 4) :

\[
    F = \frac{\tau’}{W \sin \theta} = \frac{W \cos \theta + f_w (p_w - p_a)}{W \sin \theta} \tan \phi’
\]

où \( p_a \) et \( p_w \) sont respectivement la pression d’air et la pression d’eau, \( \phi’ \) est l’angle de frottement interne du sol. Le paramètre \( f_w \) est fonction du degré de saturation du sol. Il peut être également exprimé en fonction du rapport de l’angle de frottement interne à l’état non-saturé et celui à l’état saturé.

Dans cette étude, nous nous sommes intéressés à étudier l’effet de l’infiltration ou de l’évapotranspiration sur la stabilité des pentes dans les sols partiellement saturés. Ainsi on se limite aux cas qui ne sont pas stables dans un état saturé. Le glissement de pente est supposé être dû à l’infiltration de l’eau depuis la surface du sol et la zone de glissement est considérée être dans un niveau supérieur et suffisamment loin de la nappe phréatique. Différentes courbes de rétention d’eau (Figure 2) et
différents angles de frottement interne pour différents angles de talus sont considérés. Les propriétés hydrauliques des sols sont considérées proches de celles des sols argileux. Trois valeurs de l’angle de frottement sont étudiées: 20°, 25° et 30°. Comme on s’intéresse à l’étude de l’instabilité due à la saturation, nous étudions les cas qui ne sont stables que dans l’état non saturé. Par exemple pour un sol ayant un angle de frottement interne de 20°, les angles de talus de 21° (Figure 5), 23° (Figure 6), 25° (Figure 7) et 27° (Figure 8) sont étudiés qui sont tous instables dans le cas saturé. Le coefficient de sécurité diminue par l’augmentation de la teneur eau (réduction de la succion du sol). Pour chaque valeur d’angle de frottement, trois courbes de rétention (Figure 2) sont étudiées en considérant trois valeurs différentes de \( \alpha \): 0,3, 1,25 et 2,2. A titre d’exemple, les résultats de calcul sont présentés pour un angle de frottement interne de 20°.

Les figures 5 à 8 représentent les isovaleurs de facteurs de sécurité (F.S.) calculés à différents profondeurs et différents degrés de saturation. Pour un degré de saturation donné, la profondeur donnée par l’isovaleur de F.S.=1 représente l’épaisseur maximale d’une couche stable. Celle-ci peut également se traduire en profondeur d’une zone dans laquelle la stabilité peut être influencée par les variations de degré de saturation. On constate que pour un degré de saturation et un angle de talus donnés, l’épaisseur de la couche stable diminue lorsque le paramètre \( \alpha \) augmente. A titre d’exemple on considère un talus avec \( \theta = 21° \) et \( S_0 = 0,5 \). L’épaisseur de la couche stable pour \( \alpha = 0,3 \) (Figure 5-a) peut atteindre jusqu’à environ 3 mètres, tandis que pour \( \alpha = 2,2 \) (Figure 5-c), elle atteint 0,5 mètre. Les épaisseurs de ces couches stables peuvent se traduire en profondeur d’une zone dans laquelle la stabilité peut être influencée par le degré de saturation.
Evidemment pour un angle de frottement interne donné, l’épaisseur de la couche stable diminue avec l’angle de talus, ce qui peut être constaté en comparant les Figure 5-a, Figure 6-a, Figure 7-a et Figure 8.

Les résultats montrent que la profondeur d’influence de la non-saturation sur la stabilité des pentes est fonction de la courbe de rétention d’eau du sol. Cette influence est plus significative pour des courbes de rétention correspondant à une valeur de $\alpha_s$ plus petite.

On peut conclure que les paramètres les plus importants pour la stabilité d’un talus à l’état non-saturé sont les paramètres hydriques, i.e. la courbe de rétention d’eau du sol. Ces paramètres contrôlent également l’effet de la succion engendrée par des racines sur la stabilité de talus. Cependant la contribution mécanique des racines en tant qu’élément de renforcement est indépendante de ces paramètres.

5 CONCLUSION

L’influence de la variation de la succion due à l’extraction d’eau par des racines sur la stabilité des talus est étudiée. Les résultats de la modélisation présentent les variations du profil de saturation et de la succion. Les variations de la succion sont plutôt limitées à la zone racinaire.

Les isovaleurs de facteurs de sécurité pour différentes courbes de rétention d’eau permettent d’évaluer la profondeur d’influence de la non-saturation. Pour un angle de talus donné, en fonction de courbe de rétention d’eau du sol, l’épaisseur d’une couche stable peut être fortement influencée par le degré de saturation dans le cas des $\alpha_s$ faibles. Le degré de saturation joue un rôle moins important sur l’épaisseur de la couche stable pour des valeurs de $\alpha_s$ élevées. La zone d’influence de non saturation peut être donc calculée en fonction des paramètres hydriques du sol, et indépendamment de la cause de variation de la succion. Les résultats montrent que l’importance de la végétation, autrement dit de la non-saturation, sur la stabilité des pentes dépend fortement des paramètres hydriques du sol. Dans la plupart des cas, la zone d’influence de non-saturation est limitée au premier mètre de profondeur. Par conséquent, la présence de la végétation peut renforcer la stabilité d’un talus uniquement pour des glissements superficiels.

6 REMERCIEMENTS

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7 RÉFÉRENCES


Rainfall-induced collapse of old railway embankments in Norway

Influence des precipitations sur l'instabilité d'anciens remblais ferroviaires en Norvège

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ABSTRACT: During the second half of year 2011 heavy rainfall caused dramatic damage on railway foundations in Southern Norway. Most of the railway foundations affected are old, generally constructed between 1850 and 1950 at a time when construction work was done manually, and soil materials from cuts in local natural deposits were utilized for construction of nearby embankments. Quality of culverts and embankments therefore does not correspond to modern construction standards. As a consequence the embankments do not perform well during intense and prolonged rainfall as demonstrated in the fall of 2011. In some cases, low capacity of old culverts caused rise in water levels upstream of embankments, followed by internal erosion and ended in complete destruction of embankments. However, the collapse of several embankments was not related to this factor. The geotechnical behaviour of some of the collapsed embankments, constructed of sandy, silty and clayey material was studied, and the results for one of these are presented in this paper. Unsaturated flow properties were taken into account to explain the behaviour.

RÉSUMÉ: Au cours de la seconde moitié de l’année 2011, suite à d’intenses précipitations, un certain nombre de voies ferrées et leurs fondations ont été endommagées dans le Sud de la Norvège. La plupart des ouvrages touchés étaient anciens et avaient été construits dans les années 1850-1950, à une époque où les travaux se faisaient à la main et où les matériaux employés pour la construction des remblais étaient prélevés sur des chantiers à proximité. La qualité de ces ouvrages ne correspond donc pas aux standards actuels de construction. Par conséquent, ils ne sont pas toujours en mesure de résister à de fortes et longues périodes de pluie, comme cela s’est démontré à l’automne 2011. Dans certains cas, la faible capacité des drains a entraîné la montée du niveau d’eau en amont des digues, provoquant l’accélération de l’érosion interne et, au final, la destruction complète de la structure. Dans certains cas néanmoins, le phénomène s’explique par d’autres causes. Le comportement géotechnique de certains des ouvrages ayant été détruits, lesquels sont généralement construits sur des sols sableux, silteux et argileux, a été étudié et est présenté dans cet article. Les écoulements non saturés ont été pris en compte pour expliquer les phénomènes observés.

KEYWORDS: railway, stability, embankment, culvert, rainfall, retention curve, permeability, grain size distribution

1 INTRODUCTION
The railway infrastructure in Southern Norway suffered serious damage during the second half of 2011. The damages, often leading to complete destruction of sections of the railway, were caused by prolonged and intense rainfall. Monthly rainfall of >150% of normal values were observed, with estimated periods of 30-50 years. NGI assisted on a number of occasions where railroad infrastructure was damaged. Cases were distributed over a large geographical area and on most major railway lines in Southern Norway. In this paper, typical damages are discussed. A case study of a failed railway embankment is also presented, with special emphasis on the geotechnical behaviour.

2 OLD RAILWAYS AND DAMAGES
2.1 Some features of existing railway infrastructure
Railway lines constructed between 1850 and 1950 constitute the major part of present railway infrastructure in Norway. Some modernization as widening of embankments has been done, some culverts have been renovated, new ballast types introduced, however, much of the original substructure, 50-150 years old, remains more or less unaltered. Documentation from the construction phase shows that the principal of mass balance along the railway was used. Manual labour and mass transport by wheel barrows prevented long distance transport, and prohibited the use of materials from e.g. remote stone quarries. Soil from cuts was placed in nearby embankments; hence, materials as clay, silt and sand are today encountered in the railway embankments. Culverts passing under the railway were typically constructed by dry masonry of rectangular hewn blocks of stone. Some culverts have been modernized as part of maintenance or after damage, often by inserting plastic pipes into old stone culverts. This reduces the cross section, and flow capacity may be reduced, although smoother surface partly compensates for reduced cross section. Inspection however shows that even after operating times of >100 years many culverts are in surprisingly good condition.

2.2 Types of damages
In the following description of damages, direct hit on the infrastructure from landslides initiating outside the railway is excluded. The discussion is limited to damages on railway infrastructure due to extraordinary rainfall in the second half of year 2011. Observed damages are categorized in a few main groups: 1. Damages related to culverts. 2. Damages related to flooding. 3. Damages related to embankment slope failure.

2.3 Damages related to culverts
Embarkment collapses caused by inadequate performance of the culverts is a well-known phenomenon along existing railways. Depending on the soil type in the embankments, complete destruction of embankments may occur surprisingly quickly. In pioneering days of railway engineering, culverts...
were not designed by detailed analysis of discharge from upstream catchments (although planning was thorough, including years of field surveying and observations). However, over long time periods and including possible climatic change with increased rainfall, it is no surprise that the flow capacity of the culverts may be exceeded from time to time. Human activity, as urban development or construction of highways, may also result in changed drainage patterns along the railway line, increased discharge and reduced concentration time for the catchment. "Wear and tear" of the constructions and insufficient maintenance may reduce flow capacity of culverts. Maintenance may fail to detect damage or blockage of inlet or outlet, or collapse of culverts inside embankments. One problem has been that culverts are not extended when embankments are widened, which may result in burying of inlet or outlet.

In 2011 damage was frequently caused by flash floods following intense short-duration rainfall. High discharge may result in upstream damming of water due to the culvert being unable to transport the discharge from the catchment, unless the water finds alternative ways underneath or through the embankment (which in turn destabilized several embankments constructed of sand during the same period). Damming of water will increase pore-water pressures in the embankment and lead to collapse of a construction which needs drained conditions to remain stable. Overtopping of embankments is particularly disastrous, especially for embankments constructed of materials that are easily eroded, as sand and silt. Embankments may experience rapid and total destruction under such conditions. A photo showing an example of complete destruction of the embankment around a culvert is shown in Fig. 1. Note that the embankment is mainly constructed of relatively fine, sandy soil.

Figure 1. Example of total destruction of embankment around culvert near Ål, Bergensbanen railway line (Oslo to Bergen).

Settlements of the underground due to the weight of the embankment may be considerable on soft marine clay, and also result in deformation of the embankment. Settlements increase under the centre, and decrease towards the foot of the embankment. Horizontal sliding of culverts may also be caused by horizontal soil pressures within the embankment. Displacements result in the opening of gaps between stone blocks. Sprinkling of soil from the embankment may result in cavities in the embankment. Gaps may also result in water leaving the culvert, finding new flow paths through the embankment, causing internal erosion. Large deformations may lead to collapse and internal blocking of the culvert. Vegetation transported during flooding may block the inlet of culverts temporarily, which may not easily be detected during an intense rainstorm. Landslides in the side terrain can block the culvert and cause upstream damming.

For flash-flood events it is a problem that even regular and frequent inspection of the railway during a critical situation may be insufficient to detect incipient collapse of embankments. Collapse of the embankment around a culvert may occur between two consecutive inspections due to short-term intense scour. For small catchments concentration times are short, and the distance between existing meteorological stations does not give sufficient information to forecast flash-flood events along the railway lines (even when weather radar images are used as supplement). In some cases, trains, unable to stop when a collapsed section of the railway line was encountered without any warning, have spectacularly passed the collapsed section of the embankment on rails hanging in the air, as a suspended bridge. In other cases, suspected embankment collapses have been reported by the engine driver, who noticed unusual behaviour of the train. The weight of the train was the remaining load necessary to initiate the collapse, which may have occurred as rapid liquefaction of saturated soil volumes.

Improved maintenance, redesign and reconstruction of culverts may reduce problems in the future. Improved design of culverts with built-in safeguards (increased cross section, double pipelines etc.) is possible, but costly judged from normal maintenance budgets. Modernization normally is restricted to already known problem areas. This will keep existing culverts mainly unaltered and still contributing to a high future risk.

2.4 Damage related to flooding

A second type of damages is caused by flooding in large rivers and lakes along the railway. Settlements, local slope failures of embankments, erosion along embankments and deposition of fines are typical results of general water rise in rivers or lakes. Slopes typically collapse when external water levels normalize.

During 2011 flooding was primarily a problem along the Dovre line, the main railway line between Oslo and Trondheim, and traffic was cut in periods. Due to the nature of flooding in large rivers and lakes, these situations generally are less dramatic than sudden destruction of embankments at culverts. Water levels rise comparatively slowly (when compared to flash floods in small catchments), which allows evaluation of the situation as it develops. Railway lines are normally resilient to such events, and complete collapse will normally not occur. For regional flooding the situation may be monitored as flooding develops, and associated risk for train traffic be evaluated. The Norwegian national system for flood warnings is well developed, and flooding the situation may be monitored as flooding develops, and associated risk for train traffic be evaluated. The Norwegian national system for flood warnings is well developed, and flooding in large rivers and lakes should come as no surprise. Based on regional warnings and weather forecasts mitigating actions may ideally be well planned (e.g. reduced speed, temporary closure of train traffic). The National Railroad Administration also has introduced three alert levels for these situations, based on weather forecasts.

2.5 Damages related to embankment slope failure

A third type of damage relates to slope failures in embankments. Some embankments collapsed due to increased supply of water in dikes upstream of the embankment. However, several embankments collapsed where there were no culverts, no dammed water upstream of the embankment, and where no flooding occurred. Some collapses appeared rather enigmatic at the first glance, and are interesting from a geotechnical point of view. One slope failure occurred on an embankment elevated about 5-6 m above a flat terrain, while another occurred on an 8-10 m high fill across a ravine. For these embankments water should ideally not be able to invade the construction, however, this is exactly what happened. One case is discussed in more detail in this paper.
3 CASE: TOMTER EMBANKMENT FAILURE

A slope failure at a railway embankment at Tomter occurred on 23rd December 2011. The line was not physically cut off, but train traffic was stopped until inspection by geotechnicians from NGI was made the following day. A cross section is shown in Fig. 2. Field survey was somewhat hindered by low temperatures and a thin frozen crust that had developed through the preceding night. It was found that the embankment consisted of a bottom layer of clay on top of the natural terrain (marine clay), followed by a layer with high sand and silt content. Above this layer there was a 1 m thick layer of sand and gravel, added in a general uplift of the track around 1950-60, and at the top crushed rock ballast. In spite of the elevation above the surrounding terrain, the sandy/silty layer was observed to be very moist, and appeared almost liquefied. The water content \( w \) of a bag sample was measured to 17.5 %. In situ saturation rate of the sandy/silty layer may not be determined from a bag sample (in situ density is unknown and water is lost during sampling), but this indicates that the sandy/silty layer may in fact have been close to full saturation prior to the slope failure.

![Cross-section of failed embankment at Tomter, Østfold county railway line. Geometry of slope failure 23rd December 2011 is indicated. Natural soil below embankment is marine clay.](image)

Results from grain size distribution (GSD) analysis of the sample are summarized in Table 1 and Fig. 3. The moist layer described in field as a sandy/silty layer is characterized as sandy, silty, gravely and clayey material, according to terminology and grain size limits defined in Norwegian Geotechnical Association (1982). The soil is well graded.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gradation number ( C_n = D_{90}/D_{10} )</td>
<td>101.1</td>
</tr>
<tr>
<td>Clay content ( D &lt; 0.002 ) mm (%)</td>
<td>6.9</td>
</tr>
<tr>
<td>Silt content ( 0.002 &lt; D &lt; 0.006 ) mm (%)</td>
<td>21.4</td>
</tr>
<tr>
<td>Sand content ( 0.006 ) mm (&lt; D &lt; 2 ) mm (%)</td>
<td>54.2</td>
</tr>
<tr>
<td>Gravel ( 2 ) mm (&lt; D &lt; 60 ) mm (%)</td>
<td>17.5</td>
</tr>
</tbody>
</table>

An empirical curve for the grain GSD is also shown in Fig. 3. The curve was fitted by using a five parameter equation for unimodal GSD (Fredlund et al. 2000), see Eq. 1. In Eq. 1, the percentage of particles \( P_p \) passing a certain sieve size is given as a function of the particle diameter \( D \) (mm). The parameter \( a_p \) is related to the breaking point of the curve, \( n_p \) is related to the steep part of the curve, \( m_p \) is related to the shape of the curve in the fines region, \( d_i \) is related to the fines content and \( d_i \) is the minimum allowable particle size. The value of \( d_i \) was chosen based on the grain size data. The other parameters are result of statistical optimization. For the resulting curve the \( R^2 \) value is 99.15%. Final parameters are shown in Table 2. Also shown in Fig. 3 is the logarithmic PDF (probability density function) for the sample, which is the result of differentiating the GSD-curve. The PDF will correctly represent the most frequent particle size when first taking the logarithm of the particle size, see Eq. 2 (Fredlund et al. 2000), in which \( P(D) \) is the logarithmic PDF. For the analyzed sample the most frequent particle sizes are found in the sandy fraction, with a peak at 0.3-0.4 mm. This corresponds with the laboratory description of the soil, where the first adjective (“sandy”) nominates the largest mass fraction.

![Results from laboratory GSD analysis, empirical GSD function and logarithmic PDF (Fredlund et al. 2000).](image)

![Cross-section before failure. Runout. Geometry of landslide scarp.](image)

### Table 2. Parameters for empirical GSD curve (Fredlund et al. 2000).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Sample Tomter A</th>
</tr>
</thead>
<tbody>
<tr>
<td>( a_p )</td>
<td>0.6133</td>
</tr>
<tr>
<td>( n_p )</td>
<td>0.8357</td>
</tr>
<tr>
<td>( m_p )</td>
<td>1.4909</td>
</tr>
<tr>
<td>( d_i )</td>
<td>0.7612</td>
</tr>
<tr>
<td>( d_i ) (mm)</td>
<td>0.0005</td>
</tr>
</tbody>
</table>

4 ANALYSIS OF EMBANKMENT FAILURE

The railway embankment that collapsed at Tomter on 23rd December 2011 is used to illustrate the geotechnical behaviour of old railway embankments. The seepage module Seep/w of the geotechnical software Geo-Studio 2007 (Geo-Slope International 2007) was used for flow analysis of the embankment. The routine in Seep/w for predicting the water retention curve from GSD data (Aubertin et al. 2003) was used for layer B. For layers A and C, \( k_h \) curves are used to represent typical properties for these layers and are not discussed further here. Hydraulic conductivity functions and saturated permeability for layers A, B and C are shown in Fig. 3 and Table 3, respectively. The top layer D (crushed rock ballast) is assumed very permeable and completely drained, and is excluded from the seepage analysis.

The hydraulic conductivity curves show the well-known effect that less permeable clay (when saturated) is more permeable than coarse-grained soils for high matric suction.
Profile P1 (left) and P2 (right) as indicated in Fig. 5.

Parameters and the effective stress principal (Terzaghi 1943) may be used. Stability calculations for the embankment with positive pore-water pressures from top of layer B result in critical values the safety factor (~1.0) for realistic choices of shear strength parameters. In Fig. 7 results are shown for 75-90% of hydrostatic pore-water pressure distribution from top of layer B. It is underlined that strength parameters were not measured. Stability analyses were done by the limit equilibrium method using the software package GeoSuite Toolbox (ViaNova Systems 2007).

The soils in the embankment prevent water from being drained from the construction. The situation probably varies through the year, and factors not taken into account in the analyses may improve or worsen the situation. Additional water may be transported along the embankment from other areas, on top of the clay layer which may be deformed by settlements, or by capillary suction in layer B. The particular worry for this kind of slope stability problem is that there are normally not clear any precursors to the failure and destabilization of the embankment is not easily observed.

5 CONCLUSIONS

A: Under-dimensioned, damaged or blocked culverts may result in rapid destruction of old railway embankments during flash-floods, which may occur more frequent in the future as a result of climate change. The problem may be addressed by improved maintenance/inspection, or by redesign/modernization of the drainage systems. B: Slope failures may occur in old embankments constructed of clay, silt, sand and gravel without clear precursors to failures. Analyses indicate that slope stability may be critical also without unusual weather conditions. There seems to be a need for improved research on the geotechnical behaviour of such embankments.

6 ACKNOWLEDGEMENTS

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7 REFERENCES

Dynamic shear modulus and damping of compacted silty sand via suction-controlled resonant column testing

Propriétés dynamiques d'un sable limoneux par des tests en colonne de résonance sous aspiration contrôlée.

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ABSTRACT: Dynamic properties of unsaturated soils, particularly shear modulus and material damping, play a fundamental role in the analysis/design of critical geotechnical infrastructure resting on unsaturated ground, or made of compacted unsaturated soils, when subjected to static and dynamic loads. This paper introduces a proximitor-based resonant column device with self-contained bender elements suitable for testing soils under controlled-suction conditions via the axis-translation technique. A series of suction-controlled resonant column and bender element tests were conducted on several statically compacted samples of silty sand under net stresses and suction states ranging from 50-400 kPa. Particular attention was devoted to the influence of suction over the frequency response curves and cyclic hysteretic stress-strain loops. The results confirm the influence exerted by the stress/suction history experienced by the soil, in terms of dynamic shear modulus and damping.

1 INTRODUCTION

Dynamic properties of unsaturated soils, particularly shear modulus and material damping, play a fundamental role in the analysis/design of critical geotechnical infrastructure resting on unsaturated ground, or made of compacted unsaturated soils, when subjected to static or dynamic loads. Most conventional soil testing techniques, however, cannot capture this very small-strain behavior and thereby considerably underestimate the true soil stiffness. Several efforts have been reported since the early 1980’s to study the effects of capillarity and saturation on small-strain stiffness of unsaturated soils via resonant column (RC) or bender element (BE) testing, including Brull (1980), Wu et al. (1984), Qian et al. (1991), Marinho et al. (1995), Picornell and Nazarian (1998), Cabarkapa et al. (1998), Cho and Santamarina (2001), Mancuso et al. (2002), Inci et al. (2003), Kim et al. (2003), Mendoza et al. (2005), Cabarkapa and Cuccovillo (2006), Vassallo et al. (2006), Sawangsumrit et al. (2008, 2009), Ng et al. (2009), and Khosravi et al. (2010).

The BE technique has proved a feasible way to investigate unsaturated soil stiffness at very small shear strain amplitudes. However, there is a great need for assessing the suitability of this technique, particularly for unsaturated soils, as compared to more fully-standardized procedures such as the resonant column and simple shear test methods.

This paper introduces a suction-controlled proximitor-based resonant column apparatus which features self-contained BEs for the simultaneous testing of soils under both techniques. Particular attention is devoted to the influence of suction over the frequency response curves and cyclic hysteretic stress-strain loops. The results highlight the critical influence exerted by the stress/suction history experienced by the soil in terms of both dynamic shear modulus and damping.

2 RC/BE DEVICE: MAIN FEATURES

The model THS-100 resonant column cell features a reinforced acrylic chamber of 1000 kPa confining pressure capacity. The bottom pedestal, for samples of 70-mm diameter, features a full set of HAEV (5-bar) ceramic disks, as well as one BE crystal (receiver) for shear-wave velocity readings: Figure 1(a). The top cap features a full set of coarse porous stones, for uniform pore-air pressure application/control, and also one BE transmitter. An electrical servo motor actuator is used for the application of torsional loads with +/– 2.33 kN-m (peak) capacity, and 300-Hz frequency range.

The input torque is measured in pfs (percent of full scale) units, with 100 pfs equivalent to a 10 kN-m torque. Mounted on an internal floating frame, thus allowing for large vertical deformations, the actuator includes a servo amplifier for closed-loop control of torsional loads, and one proximitor mounting acting as the internal angular displacement transducer: Figure 1(b). A model PCP-15U pressure panel is used for direct control of pore-air pressure through the top cap, with dual pressure regulators/gauges for precise measurement/control of matric suction, $s = u_a (u_w = 0)$.

Figure 1. THS-100 resonant column cell with self-contained BEs.
3 TEST SOIL AND PERFORMANCE VERIFICATION

The soil material used in this work classifies as silty sand (SM) according to the USCS: 70% sand and 30% silt. The coarse fraction has particle sizes between 0.5-1.2 mm. The passing No. 40 sieve fraction has liquid limit, LL = 26.4%, and plastic limit, PL = 22.2%. Samples were statically compacted into a 70-mm diameter, 130-mm height, compaction split mold via a triaxial loading frame. Each sample was prepared in three lifts, at a constant displacement rate of 1.0 mm/min, to a target void ratio, e = 1.0, and dry unit weight, $\gamma_d = 13.13 \, \text{kN/m}^3$. The initial water content of 26% corresponds to an average degree of saturation of 72% and initial matric suction of 20 kPa, according to the soil-water retention curve (Hoyos et al. 2011).

Calibration of the proximitor-based RC device was first accomplished by conducting resonant column tests on a 9.5 mm (0.375 in) diameter, stainless aluminum rod, which also yields the polar moment of inertia of the entire drive system. The test yielded expected values for torsional stiffness of the aluminum rod, k = 26.4 GPa, and polar moment of inertia of the drive system, $I_o = 737.8 \, \text{kg-mm}^2$. Performance verification testing was then carried out through a comparative analysis of results from proximitor-based RC tests and accelerometer-based RC tests (ASTM 1993) on identically prepared samples of SM soil.

Figure 2(a) shows a full set of frequency response curves obtained from compacted SM soil in the accelerometer-based RC device. The specimen was subject to different input-voltage amplitudes ranging from 0.25 to 5 Volts, thus generating a family of curves with different resonant frequencies and peak accelerometer outputs, from which shear modulus G and shear strain amplitude $\gamma$ can be calculated. All tests were performed under constant 40 psi confinement. Soil softening (degradation) is manifested by the so-called backbone curve. Likewise, Figure 2(b) shows a full set of frequency response curves obtained from an identically prepared sample of SM soil tested in the proximitor-based device. In this case, however, the specimen was subject to different input-torque magnitudes ranging from 1 pfs (0.1 kN-m) to 10 pfs (1 kN-m). Peak shear strain fractions $\gamma$ (cm/cm) can be readily assessed from each test. All tests were also performed under a constant 40 psi confinement.

Results show that a 1-pfs input torque in the proximitor-based apparatus induces a similar response as a 0.25-Volt input signal in the accelerometer-based apparatus, which is typically used to ensure shear strain levels below a threshold limit $\gamma_t$. It can also be observed that a 10-pfs input torque induces a higher degree of soil softening than the maximum 5-Volt input signal in the accelerometer-based device. The scope of the present work, however, is limited to linear (low-amplitude or small-strain) stiffness response of unsaturated soils; therefore, a 1-pfs input torque (0.1 kN-m) was adopted for all subsequent suction-controlled tests performed in the proximitor-based RC device, as described in the following section.

4 RC/BE TEST PROCEDURES AND SOIL RESPONSE

A series of RC and BE tests were simultaneously conducted on identically prepared samples of SM soil. Each sample was tested under constant matric suction, s = 50, 100, 200, or 400 kPa, induced via axis-translation technique; and four different net confining pressures, $(p - u_a) = 50, 100, 200, \text{and } 400 \, \text{kPa}$. The soil was first isotropically compressed to a target confining pressure, p = 50 kPa. Pore-air pressure $u_a$ was then gradually increased (soil drying) to the pre-established value of suction, while the net confining pressure was kept constant at 50 kPa by simultaneous and equal increases of the external confinement.

Pore-air pressure $u_a$ was maintained constant until no further change in water volume from within the soil (less than 0.035 ml/day) was observed, at which point pore-fluids equalization was considered complete. A 36-hr equalization time (1.5 days) was found suitable for all suction states. Equalization stage was finally followed by a constant-suction ramped consolidation to the target values of net confining pressure. All RC tests were conducted by sweeping the entire input-torque frequency scale until obtaining a thorough frequency response curve, typically between 50 and 250 Hz. The peak torsional vibration was then completely cut off to record the free-vibration decay curve.

Figure 3(a) shows a family of typical frequency response curves obtained from suction-controlled RC tests on SM soil under constant matric suction, s = 50 kPa, and net confining pressures, $(p - u_a) = 50, 100, 200, \text{and } 400 \, \text{kPa}$. Likewise, Figure 3(b) shows typical curves under constant matric suction, s = 200 kPa. It can be readily observed the critical influence that suction has on soil response under resonance, with a significant rightward shift of all curves for higher suction state, s = 200 kPa. This can be directly attributed to the expected increase in effective stress and, hence, rigidity (stiffness) of soil skeleton at higher suctions. The level of net confinement, however, has a more pronounced effect than matric suction. It can also be observed that the half-power points, that is, frequencies on each side of the frequency response curves corresponding to a shear strain of 0.707($\gamma_{\text{max}}$), become less apparent as suction increases, i.e., the frequency response is less symmetric about the resonant frequency. Hence, the assessment of material damping using the half-power bandwidth method becomes less reliable at higher suction states.
5 DYNAMIC SHEAR MODULUS AND DAMPING

Figure 4 shows the variation of small-strain shear modulus $G_{\text{max}}$ (from both RC and BE tests) with net confining pressure, and for different suction values. The trends confirm those shown in Figure 3: suction is observed to have a significant influence on soil stiffness, though not as pronounced as that of net confining pressure. The BE technique yields $G_{\text{max}}$ values reasonably close to those obtained via resonant column testing. Solid lines in Figure 4 represent best-fit power regression functions of the form, $G_{\text{max}} = A(p - u_a)^B$. Constant $A$ represents the value of $G_{\text{max}}$ (MPa) at net confining pressure, $(p - u_a) = 1$ kPa; while constant $B$ is the slope of the best-fit curve, which represents how susceptible soil stiffness is to changes in net confinement $(p - u_a)$.

During bender element testing, the first arrival of shear-wave was taken as the point of zero crossing after the first inflection of the received signal, which corresponds to the first arrival of the shear-wave based upon experimental and numerical studies (Lee and Santamarina 2005). The travel distance is taken as the tip-to-tip distance $L$ between bender elements, hence the shear-wave velocity is computed as $V_s = L/t$, where $t =$ travel time. Knowing $V_s$ and the total mass (bulk) density of the specimen $\rho$, the small-strain shear modulus can be determined as $G = \rho V_s^2$.

Figure 5 shows the change in small-strain damping ratio $D_{\text{min}}$ (from RC tests) with matric suction, for different net confining pressures. Damping is calculated from logarithmic decay curves using: $D_{\text{min}} = (1/2\pi)\log(Z/Z_o)$; where, $Z_o$ = peak amplitude of the first free-vibration cycle, and $Z_n$ = peak amplitude of the nth cycle. The trends confirm those in Figure 3, with lower damping (higher stiffness) at higher matric suction. In general, material damping tends to be overestimated by the half-power bandwidth method: $D_{\text{min}} = (1/2)(f_2 - f_1)/f_r$; where $f_r =$ resonant frequency.

The main focus of the present work has been on small-strain stiffness of compacted silty sand. The cyclic behavior of soils, however, is nonlinear and hysteretic; consequently, the shear modulus and material damping are heavily strain dependent. Figure 6 shows the cyclic hysteretic stress-strain loops from two SM soil samples subjected to a cyclic 10-pfs input torque (1 kN-m) at matric suctions, $s = 50$ kPa (thinner trace) and $s = 200$ kPa (thicker trace), respectively; both under the same net confining pressure, $(p - u_a) = 200$ kPa. Equivalent viscous damping could also be evaluated from the area enclosed by the cyclic hysteretic loops. Therefore, the loops further substantiate the trends shown in Figure 5, with smaller areas enclosed by the cyclic hysteretic loops, and lower shear strains induced by the same cyclic shear stress, with increasing matric suction.

6 CONCLUDING REMARKS

Suction-controlled resonant column tests on compacted SM soil shows that the newly implemented proximitor-based RC device is suitable for testing soils under controlled suction states via axis-translation technique. Test results underscore the influence of soil suction over the frequency response curves, logarithmic decay curves, cyclic hysteretic stress-strain loops, and the small-strain stiffness properties of compacted SM soil. Lower material damping (higher stiffness) is observed at higher suction states. In general, material damping tends to be overestimated by the half-power bandwidth method. Simultaneous suction-controlled bender element tests produced $G_{\text{max}}$ values reasonably close to those from resonant column tests.

The general trends observed in this research effort are similar to those previously reported for a more limited range of test variables (e.g., Kim et al. 2003, Sawangsuriya et al. 2009, Ng et al. 2009). The time frame and scope of the present work did not contemplate investigating the effects of initial void ratio, stress history, hydraulic hysteresis, or the impact of net normal stress and/or suction history on the normalized $G/G_{\text{max}}$ and $D/D_{\text{min}}$ response of SM soil. The authors are currently embarked on a more thorough research effort to gain further insight into all these dynamic aspects of unsaturated soil behavior, including
RC/BE testing at matric suction states near the air-entry value of the test soil.

**Figure 5.** Damping response of SM soil at different net confinements

**Figure 6.** Cyclic hysteretic shear stress vs. shear strain loops from SM soil samples subjected to a cyclic 10-pfs input torque (1 kN-m).

7 REFERENCES

Expression of mechanical characteristics in compacted soil with soil/water/air coupled F.E. simulation

Expression des caractéristiques mécaniques des sols compactés par une simulation couplée sol/eau/air par éléments finis

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ABSTRACT: Results obtained in a lab compaction test are difficult to apply to the design and compaction control at an actual geotechnical engineering site. This is attributed to the fact that the mechanism for compaction has not been explained using the principles of soil mechanics. The main theme of this study is interpretation of the compaction mechanism with unsaturated soil mechanics. Here, static compaction tests were simulated with soil/water/air coupled finite element analysis code DACSAR-MP. Consequently, the shape of the compaction curve was successfully expressed. Moreover, the effects of compaction on compressibility and permeability of compacted soil could be reasonably explained. Additionally, shear deformation was applied to the specimen obtained from static compaction simulations in soil/water/air coupled analysis. The relationships between shear strength and compaction curve showed good agreement with the actual behavior.

Résumé : Les résultats des essais de compactage obtenus dans un laboratoire sont difficilement applicables à la conception et au contrôle de compactage dans des sites géotechniques d’ingénierie réels. Ceci peut être attribué au fait que le mécanisme du compactage n’a pas été expliqué à partir des principes de base de la mécanique des sols non saturés. Dans ce cadre, nous avons simulé le compactage statique avec de la terre, de l’eau ou de l’air doublé d’une analyse par éléments finis, code DACSAR-MP. Ainsi, nous avons pu mettre à jour avec succès la forme de la courbe de compactage. De plus, nous avons réussi à expliquer scientifiquement les effets du compactage sur la compression et la perméabilité des sols compactés. En outre, les essais de cisaillement ont été simulés sur l’éprouvette obtenue à partir de la simulation d’un compactage statique couplée sol/eau/air. Les relations entre la résistance au cisaillement et la courbe de compactage sont parfaitement en accord avec le comportement réel.

KEYWORDS: Compaction, Unsaturated soil, Soil/water/air coupled simulation

1 INTRODUCTION

Most onshore earth structures, such as the earth dam or embankments and river levees, are constructed by compaction. It is generally known that the maximum dry density appears at the optimum water content under constant compaction load. This relationship is called compaction curve. In previous research, the characteristics of compacted soil were compared with the compaction curve (see Fig. 1). Water content at the extremal values of mechanical characteristics was found to be different from the optimum water content of the compaction curve. This means that increase in dry density does not directly influence increase in shear strength or a decrease in compressibility and permeability. Moreover, in-situ tests (e.g., Proctor: 1933) and/or experimental construction are actually needed for construction of compacted earth structures since compaction effects are dependent on the method of compaction. In this study, the relationship between characteristics of compacted soil and the compaction curve is expressed using soil/water air coupled F. E. Simulation to clarify the compaction mechanism and the characteristics of soil induced by compaction.

2 NUMERICAL MODELS IN SOIL/WATER/AIR COUPLED ANALYSIS

In this study, the soil/water/air coupled F. E. analysis code, known as DACSAR-MP, is used for simulations. The numerical models applied to DACSAR-MP are detailed as follows:

2.1 Constitutive model for unsaturated soil

![Figure 1. Characteristics of compacted soil with compaction curve (Kuno: 1974)](image)

![Figure 2. Yield surface of constitutive model for unsaturated soil](image)
The constitutive model proposed by Ohno et al. (2007) is used. The effective stress for unsaturated soil is defined as:

\[ \sigma' = \sigma^\text{sat} + p_s \mathbf{I} \]  

(1)

\[ \sigma'^\text{sat} = \sigma - p_s \mathbf{I}, \quad p_s = S_s S' \]  

(2)

\[ s = p_s - p_a, \quad S_s = S_s - S_s' \]  

(3)

Here, \( \sigma' \) is the effective stress tensor, \( \sigma^\text{sat} \) is the net stress tensor, \( \mathbf{I} \) is the unit tensor, \( s \) is suction, \( p_s \) is suction stress, \( p_a \) is pore-air pressure, \( p_w \) is pore-water pressure, \( S_s \) is degree of saturation, \( S_s' \) is effective degree of saturation, and \( S_s' \) is degree of saturation at \( s \to \infty \). The yielding function is expressed as:

\[ f(\sigma', \zeta, \epsilon') = MD \ln \frac{p'}{p'^\text{sat}} + D \frac{q}{p'} - \epsilon'_0 = 0 \]  

(4)

\[ \zeta = \exp\left[ (1 - S_s) \ln a \right], \quad MD = \frac{\lambda - \kappa}{1 + \epsilon_0} \]  

(5)

\[ p' = \sigma' - \zeta \mathbf{I}, \quad q = \frac{\epsilon'}{3}; \quad s = s' p' \mathbf{I} = 1 - A' \]  

(6)

Here, \( \epsilon'_0 \) is plastic volumetric strain, \( M \) is \( q/p' \) in critical state, \( D \) is dilatancy coefficient, \( a \) and \( \epsilon_0 \) are shape parameters expressing increase in yield stress due to desaturation, and \( \lambda \) and \( \kappa \) are compression and expansion index, respectively. The yield surface expressed by Eq. (4) is illustrated in Figure 2. The following elasto-plastic constitutive model is obtained from Eq.(4) and the associated flow rule.

\[ \sigma' = D : \epsilon' - C : \epsilon_s \]  

(7)

Here, \( D \) is elasto-plastic stiffness matrix, \( \epsilon' \) is strain tensor, \( C \) is the tensor expressing change in stiffness due to desaturation.

### 3 ANALYTICAL CONDITIONS

The objective of compaction is compressing soil mass with draining air. In this study, compaction is defined as compression and rebound of unsaturated soil under drained air and undrained water conditions, and the static compaction test is simulated with soil/water air coupled F.E analysis. Figure 3(a) shows analytical mesh. One-dimensional geometric condition is assumed, and undrained water for all boundaries and drained air for upper boundary conditions are provided. Figure 4 shows the loading condition. Table 1 summarizes the material parameters for simulations and Figure 5 shows soil water retention characteristic curves (SWRCC). The SWRCC model proposed by Kawai et al. (2007) is used here. A void ratio of 0.85 and water content of 10 to 28% are provided for initial conditions. Initial suction is set according to the primary wetting curve. Moreover, shear deformation shown as Figure 3(b) is applied to the specimen obtained from static compaction simulations to examine shear strength of compacted soil (Simple shear simulation).

### 4 SIMULATION RESULTS AND DISCUSSION

#### 4.1 Static compaction simulation

Figure 6 shows changes in the void ratio of element 3 under 800kPa compaction load. The yield stress, the folding point of compaction line, is found to depend on water content. Since pore-water is not drained during compaction, the degree of saturation increases with compression due to loading, and the degree of saturation decreases with rebound due to unloading (Figure 7). This behavior is more remarkable on the specimen with higher water content. Figure 8 shows the relationship between suction and the degree of saturation during compaction. According to SWRCC, the increase of saturation due to loading means the wetting process and suction decreases. On the other hand, the decrease of saturation due to unloading creates increase in suction in accordance with the drying process. Consequently, suction changes are more remarkable on the specimen with higher moisture because it shows a relatively bigger change in the degree of saturation (Figure 9). Suction after compaction is greater than before compaction for all specimens. This means that compaction contributes to increase in stiffness of the soil. The specimen with lower moisture shows higher suction after compaction. The changes in pore-air and pore-water pressure are shown in Figures 10 and 11 respectively.
The pore-air pressure of 98kPa indicates atmospheric pressure. Air permeability increases with decrease in the degree of saturation. Therefore, air can be drained easily on the specimen with lower moisture, while air pressure increases due to air entrapment on the specimen with higher moisture. Air pressure remains the same even after compaction due to drainage difficulty in the specimen with higher moisture. When air remains the same even after compaction due to drainage entrapment on the specimen with higher moisture. Air pressure increases due to air saturation. Therefore, air can be drained easily on the specimen with lower moisture. On the other hand, degree of saturation totally. Uniform distribution appears on the specimen compacted under 400kPa compaction load are introduced here. Figure 13 shows distributions for the degree of saturation after compaction. The state quantities on the specimen compacted under 400kPa compaction load are expressed well here. Distributions for the degree of saturation during compaction.

Figure 15. Distribution of void ratio on same dry density

Figure 16. Water content – consolidation yield stress relation

and the decrease in the optimum water content with increase in the compaction load are expressed well here. Distributions for state quantities on the specimen compacted under 400kPa compaction load are introduced here. Figure 13 shows distributions for the degree of saturation after compaction. The specimen with higher moisture shows higher degree of saturation totally. Uniform distribution appears on the specimen with lower moisture. On the other hand, degree of saturation tends to increase gradually when it approaches the air-drained boundary. In the region over the optimum water content (about 24%), high degree of saturation appears only around the air-drained boundary since air permeability is fairly low. Figure 14 shows distributions of void ratio. There are different tendencies depending on whether it is under or above the optimum water content.
content, similar to degree of saturation. Permeability of compacted soil shown in Figure 1 is defined from stable flow on compacted soil under certain hydraulic gradient. Since lower permeability appears on the drier and denser specimen, the minimum permeability should appear under the optimum water content. However, the minimum permeability actually appears in the region over the optimum water content. It is assumed that this tendency can be attributed to distribution of degree of saturation and void ratio around the optimum water content shown in Figures 13 and 14. Figure 15 shows distribution of void ratio within the specimen compacted to dry density of around 1.55 (g/cm³). When we construct the embankment, dry density is specified for measurement standards after track maintenance work. However, from Figure 15 it is found that the distribution of void ratio varies according to compaction load and water content, even on the specimen with the same dry density.

Figure 16 shows the relationship between water content and yield stress of the compacted specimen in the unsaturated state, \( p'_{y} = \zeta p'_{sat} \). Arrows in the figure indicate the optimum water contents of compaction curves. The yield stresses of the specimens compacted under 800 and 1600 (kPa) in the region with low-water content are overestimated due to characteristics of the constitutive model. Consequently, the peaks for yield stresses appear in the region that is little drier than the optimum water content (shown in Figure 19), and the simulation results agree with the experimental behavior.

4.2 Simple shear simulation on compacted specimen

Figure 17 shows the relationship between shear strain and shear strength on the specimen obtained from static compaction simulations in simple shear simulations. Figure 18 shows elastic shear modulus read from the specimen in Figure 17. The arrows in the figure indicate the optimum water contents of compaction curves where it is found that elastic shear modulus depends on dry density. However, the peaks of shear strength appear in the region that is little drier than the optimum water content (shown in Figure 19). This is attributed to the yield surface. Figure 20 shows the stress paths during shear. In the figure, the initial yield surfaces are drawn. The stress path reaches to the dry side of the yield surface first and is then bound for the critical state with strain softening. The size of the yield surface depends on plastic volumetric strain and degree of saturation. The former factor is related to dry density and the latter factor is related to water content. Additionally, the initial stress state after compaction depends on suction stress calculated with suction and degree of saturation shown in Eq.(2). Consequently, the maximum shear strength appears in the region that is drier than the optimum water content, as is generally known.

5 CONCLUSIONS

Static compaction and simple shear simulation were conducted with soil/water/air coupled F. E. analysis code, applying the constitutive model for unsaturated soil, DACSAR-MP. Consequently, the shape of the compaction curve and the characteristics of compacted soil could be reasonably expressed. This proves that ‘compaction’ can be defined by the initial and boundary condition problem on unsaturated soil.

6 REFERENCES

A Geotechnical Countermeasure for Combating Desertification

Une mesure géotechnique pour lutter contre la désertification

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ABSTRACT: A self-watering system and the design method are proposed. The self-watering system, which can collect and store all kinds of water, comprised of the simple ground is much efficient to support surface vegetation. The system is designed by installing soil structures into original sandy ground. Finer soils or artificial materials can be used as the materials of the soil structures. The system can continually raise the ground water to a certain depth in the sandy ground using the capillary force. Moreover, it can minimize the evaporation from the system, which provides the potential to prevent salinization. To design the system practically, information like soil water retention curve, hydraulic conductivity and other information such as planting density and weather condition are needed. The self-watering system works under the condition of no extra energy input.

RÉSUMÉ : Un système d’auto-arrosage et le procédé de conception sont proposés. Le système est conçu en installant des couches de sol fin dans un sol sableux d’origine. Les sols fins ou des matériaux artificiels peuvent être utilisés comme les matériaux de la structure du sol. Le système peut élever continuellement l’eau du sol à une certaine profondeur dans le sable à l’aide de la force capillaire. En outre, il peut minimiser l’évaporation du système, ce qui offre la possibilité de prévenir la salinisation. Pour concevoir ce système en pratique, des informations comme la courbe de rétention d’eau du sol, la conductivité hydraulique et d’autres informations telles que la densité de plantation et les conditions météorologiques sont nécessaires. Le système d’arrosage fonctionne dans la condition de non apport d’énergie supplémentaire.

KEYWORDS: unsaturated soil, desertification.

1 INTRODUCTION

Desertification is the degradation of land in arid, semi-arid, and dry sub-humid areas due to various factors: including climatic variations and human activities (UNCCD 1994). The rapid expansion of desertification has resulted in serious environmental deterioration, economic loss, locally unsteadiness political situation and social upheaval. Because of the serious situation of desertification, prevention of the degradation of land becomes key issue. Among existed countermeasures, greening is considered to be one of the most effective methodology which can protect the biodiversity threaten by desertification, minimizing cost and providing positive multifunction. In the application of the methodology, the vegetation is the core. However, in order to fulfill the requirement of the growth of the plants, available water resource is the one of the important limitation. In arid land, groundwater is usually used as one of important water resources. Therefore, the technical methodology is suggested, which use groundwater to fulfill the requirement of the growth of the plants. However, there are numerous barriers to its implementation. One of these is that the costs of adopting sustainable agricultural practices sometimes exceed the benefits for individual farmers, even while they are socially and environmentally beneficial. Another issue is the simplicity of the technique to be acceptable by local people. From the geotechnical and geoenvironmental point of view, any technique should solve the issues such as, mechanism of raise of the groundwater up to the root zone of the plants, prevention of the salinization of the ground and design of the system that can sustainably provide the water to the plants.

In arid or semi-arid area, which characterized by lack of available water, water is one of the main limitations to the growth of plant. Frequently, capillary fringe is too deep to be used by plant in these areas. The self-watering system has been proposed. The self-watering system, which is designed to collect and store all kinds of water, comprised of the simple ground is much efficient to support surface vegetation. A self-watering system and the design method are proposed. The system is designed by installing soil structures into original sandy ground. Finer soils or artificial materials can be used as the materials of the soil structures. The system can continually raise the ground water to a certain depth in the sandy ground using the capillary force. Moreover, it can minimize the evaporation from the system, which provides the potential to prevent salinization. To design the system practically, information like soil water retention curve, hydraulic conductivity and other information such as planting density and weather condition are needed. The self-watering system works under the condition of no extra energy input.

2 SELF-WATERING SYSTEM

The soil layer often provides a medium to plant for its requirement of rooting, water and nutrient. The water flow has effect on physical property, such as consistence, strength of aggregates, aeration and temperature of soil, which is relevant to the growth condition of plant. The most direct effect of the water condition of soil is that it influences the growth of plant. The root of plant can absorb the amount of water to fulfill its need for transpiration and the amount of solute for its mineral nutrient. The transpiration water disappears in atmosphere as vapor condition. Finally, a water flow moves through soil towards to root.

In arid or semi-arid area, which is characterized by lack of available water, water is the main limitation to the growth of plant. Frequently, capillary fringe is too deep to be used by plant in these areas. The self-watering system, which is designed to collect and store all kinds of water, comprised of the simple ground is much efficient to support surface vegetation. The design target of this system is to setup an equivalent condition between the storage capacity of water and rate of usage.

The maximum water content can be held in soil before it drains downwards is field capacity, $b_f$, which is water content when drainage ceases. Field capacity is closely correlated to the volumetric water content retained in soil at -33 kPa of suction (Richards and Weaver, 1944). The capillarity storage capacity (CSC) in unit area of a soil layer can be determined by integrating its volumetric water content over its thickness, $m$, it can be described by
The overlying horizontal layer yields total water in plate. However, not all of this stored water can be absorbed by plants. The minimum water content the plant requires not to wilt is permanent wilting point, $\theta_{pwp}$, which is defined as the water content at -1500 kPa of suction. Evaporation can also reduce water content of soil to residual condition, e.g. this value can be generally considered as zero for sandy soil. However, the evaporation processes mainly influence the area near ground surface. Considering the capillary enhancement system is buried at certain depth in the ground, the stored water within the system is only removed by plants. The available capillary storage capacity (ACSC) can be written as follows

$$ACSC = \sum_{z} \left(f(\theta) \right)dz$$  \hspace{1cm} (1)

where, $f(\theta)$ is mathematic function for wetting branch of soil water retention curve, $z$ is elevation above a vertical datum.

The self-watering system is formed at the interface of hydraulically dissimilar unsaturated soil layer where a fine soil layer overlies an original relatively coarse soil ground at the certain height. Under natural unsaturated conditions, the retention characteristic at the interface between the two kinds of soil layers allows the capillary water flow from coarse layer into fine layer. Ground water or irrigation water continually enters into the fine layer until the hydraulic equivalent is achieved. The water will be suspended and stored within the fine soil layer. The evaporation and transpiration will break the hydraulic equilibrium of the system. Then a new dynamic hydraulic equivalence will be setup subsequently.

There are several designed functions of the self-watering system. First of all, the system can continuously supply water to fulfill the requirement of growth of the plants. Secondly, the system can absorb and store the water that comes from various resource, such as, atmosphere (precipitation, dew), surrounding ground or ground water. Thirdly, it can minimize the quantity of evaporation of the water in the system. Fourthly, the system works without extra energy input. Based on the designed functions, the self-watering system is proposed.

Figure 2 shows a conceptual diagram for the self-watering system located in sandy ground. As shown in the figure, two types of the self-watering system are proposed. The left side of the figure is the system in ‘T’ type. The right side of the figure is the system in suspended type. Both the two types of the self-watering system are made from installing fine soil layer in sandy ground. The ‘T’ type fine soil layer consists of plate part and pillar part. The plate part is horizontally buried in sandy ground. The main function of plate part is to store capillary water. Therefore, the design of this part should be large enough to reach the required storage quantity. The pillar part is vertically inserted down to the ground water level in sandy ground. The main function of pillar part is to absorb water by capillary force. Therefore, the design of this part should be large enough to assure the rate of supply to the plate part. For a self-watering system in suspended type, it contains only a plate part. The function of the plate concludes both functions of plate part and pillar part of the ‘T’ type system.
conductivity of finer soil and coarser soil respectively, and $k_f$ is larger than $k_c$.

3 VERIFICATION OF SYSTEM

Based on the design method mentioned above, this section will give out a design case by using the k-7 soil and the k-8 soil. In here, the k-7 soil and k-8 soil are used to simulate the coarser soil and the finer soil respectively. Both model test and numerical simulation are performed to verify the design.

3.1 Model Test

As shown in Figure 4, the model test is conducted within a steel chamber (300 cm × 300 cm × 25.5 cm). Figure 5 shows the schematic view of the setup of the model test. The left side of the figure shows the setup of the ‘T’ type system. The right side of the figure shows the setup of the suspended type system. According to the soil water retention curve of the k-7 soil (Figure 3), the water entry value is around 12 kPa. Therefore, at critical condition, the plate part of the ‘T’ type system is laid at 120 cm above the water level. Based on the observation of evaporation test, the maximum effective depth of evaporation is assumed as 40 cm, and then the thickness of the plate part is determined as 130 cm. The width of the pillar part is set as 10 cm, the ratio of radius is set as 1:3, and then the size of the ‘T’ type system can be determined. Considering the effect of interaction of the interface, the height of suspended type of the system is laid at 100 cm above the water level. Based on the same assumption of the maximum effective depth of evaporation, and then the thickness of fine layer is determined as 150 cm. In order to keep the consistency with the ‘T’ type system, the width of the finer layer is set as 30 cm. Figure 6 shows the grain size distribution of soils used in this study.

3.2 Numerical Verification

Figure 7 shows the FEM mesh of the numerical model. As shown in the figure, the model set-up follows the experiment work described above, in which the upper boundary is atmospheric condition. The lower boundary was assigned to condition of constant saturated water content. The FEM column was discretized uniformly, except the area, where fine layer exits, was discretized in to small triangular mesh. A number of observation points were located at different elevations as the location of water content sensor in physical model.

Figure 8 shows the comparison between test result and numerical result. As shown in the figure, several observations can be obtained. First of all, the self-watering system in both ‘T’ type and suspended type can absorb the water from the bottom and store the water in higher part of the system. Secondly, there is a good consistency between the test result and numerical result. It means that the numerical method can be used as one tool to predict the unsaturated water flow in the self-watering system.
4 CONCLUSIONS

In this paper, the self-watering system has been proposed. The design methodology has also been given out. Based on the verification of the system, the conclusions can be drawn as follows,

1) Based on the information of the follows, the soil water retention curve and unsaturated hydraulic conductivity of sandy ground, the maximum influence depth of evaporation, the target planting density and effective depth of roots of the plants, the monthly quantity of evaporation, transpiration, capillary water and irrigation, the self-watering system can be designed.

2) The self-watering system works under the condition of no extra energy input. Even though, the system can raise the ground water up to the certain depth of the unsaturated sandy ground. The system can absorb and store some quantity of water. The system can minimize the quantity of evaporation of the water in the system.

3) Numerical method is proven to be a useful tool to predict the unsaturated water flow within the system.

4) Both the two types of the self-watering system can fulfill the requirement of designed functions. However, in the case study, the efficiency of water supply of 'T' type system is higher than that of the suspended type system. Because the pillar part in the 'T' type system has higher efficiency of absorbing water. It can be predicted that when the critical height of the suspended system decreases, the opposite tendency will occurs.

5 ACKNOWLEDGEMENTS

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6 REFERENCES


Extension of measurement range of dew-point potentiometer and evaporation method

Extension de gamme de mesure de potentiomètre de point de rosée et méthode d’évaporation

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ABSTRACT: There are numerous methods for measuring soil water retention curve (SWRC). With tensiometer device it is possible to measure suction up to 85 kPa, but above this point measurements are not possible due to cavitation of water. To measure higher suctions other methods should be used. One of very popular devices is dew-point potentiometer WP4-T (Decagon Devices), which enables suction measurements above 1000 kPa. If high capacity tensiometers from HYPROP evaporation method device and dew-point potentiometer are used for suction measurements, the measurements are only possible for low suction range between 0 and approximately 100 kPa and for high suction range between 1000 and 300000 kPa. Between 100 and 1000 kPa suction could be measured by other methods or some extension of these two methods should be used. This paper presents SWRC measurements with HYPROP and WP4-T devices for different soils, the extension of suction range for both devices and comparison between measured suctions in the extended measurement range.

RÉSUMÉ : Il y a de nombreuses méthodes pour mesurer la courbe de rétention d'eau de sol (SWRC). Il est possible de mesurer la succion jusqu'à 85 kPa avec le dispositif de tensiomètre, mais au-dessus de ce point les mesures ne sont pas possibles en raison de cavitation d'eau. Pour mesurer des suctions plus élevées d'autres méthodes devraient être utilisées et l’un des dispositifs très connus est le potentiomètre de point de rosée WP4-T (les appareils Décagone), qui permet des mesures de succion au-dessus de 1000 kPa. Si des tensiomètres haute capacité de l’instrument HYPROP, la méthode d'évaporation et le potentiomètre de point de rosée sont utilisés pour des mesures de succion, les mesures sont seulement possibles pour la gamme de succion basse entre 0 et environ 100 kPa et pour la gamme de succion élevée entre 1000-300000 kPa. Entre 100 et 1000 kPa la succion pourrait être mesurée par d'autres méthodes ou une certaine extension de ces deux méthodes devrait être utilisée. Cet article présente des mesures SWRC avec des dispositifs HYPROP et WP4-T pour des sols différents, l'extension d'une gamme de succion pour les deux dispositifs et la comparaison entre des suctions mesurées dans la gamme de mesure prolongée.

KEYWORDS: soil suction, dew-point potentiometer, evaporation method, soil water retention curve.

1 INTRODUCTION

Soil suction and the soil water retention curve (SWRC) influence many engineering properties of unsaturated soils and their behaviour. Therefore, accurate measurements of soil suction are important for modelling soil behaviour (Fredlund and Rahardjo, 1993). Due to large differences in soils and their SWRC different measuring techniques were developed and each of them has its own limitations. A good review of measuring techniques can be found in Tarantino et al. (2008).

Tensiometer was developed by Richards (1928) and it measures matrix suction between 0 and 85 kPa. Suction measurements above 85 kPa are not possible due to cavitation of water inside the tensiometer. Special high capacity tensiometer, which enables suction measurements up to 1500 kPa, was developed by Ridley (1993). This is done by using deaired deionised water, small water volume behind porous cap, smooth surfaces, as well as special materials and preconditioning techniques.

In HYPROP evaporation method device (UMS) a continuous SWRC is obtained by simultaneously and continuously measuring suction with two high capacity tensiometers installed at different heights of the soil sample and weight change during drying. A typical work range of HYPROP device is from 0 to slightly above 100 kPa.

For soil suction measurements between 1000 and 300000 kPa a potentiometer WP4-T (Decagon Devices) could be used (Operator’s manual, 2005). A dew-point potentiometer measures relative humidity of air above soil sample and soil suction is calculated trough Kelvin equation. This measurement technique is an indirect measurement of total suction. In the range of 100 and 1000 kPa the soil suction could be measured using other methods or some extension of these two methods should be performed. Using the potentiometer WP4-T the accuracy of the measuring method can be increased, if average of data are used instead of a single point measurement. With this approach total suction as low as 300 kPa can also be measured. In case of HYPROP device the suction range can not be extended, but single value of suction could be estimated from air entry value (AEV) of tensiometers porous ceramic caps.

In this paper the HYPROP device and potentiometer WP4-T were used for soil suction measurements and an attempt was made to connect the results within the grey zone, where the limitations of both methods exist.

2 SOIL WATER RETENTION CURVE

Soil water retention curve (SWRC) is defined as relationship between the water content and the soil suction and it can be divided in 3 characteristic zones (Bardner, 1965) (Figure 1):

1. Capillary saturation zone: where soil is fully saturated. Changes in water contents result in volume deformations without any decrease in the degree of saturation.

2. Desaturation zone: When the matrix suction exceeds the air entry value (AEV) of the tested soil, the degree of saturation decreases rapidly. Hysteresis between wetting and drying curve is typical for the desaturation zone.

3. Zone of residual saturation: in this zone water content can be changed only by vapour transport (Bishop, 1960). The beginning of this zone is residual suction ($s_r$), which is
defined as a cross-section of two logarithmic lines as shown in Figure 1 (Fredlund and Xing, 1994).

Shape of SWRC is defined by suction at AEV and at residual suction and is typical for type and density state of the soil. Zapata et al. (2000) showed among others the influence of soil index properties on the shape of SWRC. Kawai et al. (2000) showed the importance of void ratio and Vanapalli et al. (1999) showed the influence of soil structure on SWRC. If good and representative SWRC is to be measured, all these things should be considered.

3.1 HYPROP evaporation method device

The evaporation method is frequently used method for measuring both the SWRC and the suction permeability curve. The method is based on the measurements of suction using tensiometers installed at different heights inside the soil specimen simultaneously with measuring the specimen weight changes due to the evaporation of water from the specimen. Due to large number of measurements a continuous SWRC is obtained. After the simplified evaporation method (Schindler, 1980) only the average weight and suctions at two points are measured. Due to short time interval the spatial and temporal nonlinearity are negligible. Therefore, two assumptions can be made:

- there exist quasi steady state conditions, which means that flux and hydraulic gradient are constant over the time interval, and
- the linear decreasing of water content and linear decreasing soil suction. This means that soil suction in the middle of the specimen is an average suction measured by tensiometer and that water content in the middle of the specimen is the same as the average water content.

Schindler and Müller (2006) had shown that these two assumptions are valid only if the evaporation rate is constant. If the evaporation rate is decreasing, the suction profile is not linear. Peters and Durner (2008) studied the error made by these two assumptions in the final clearly non linear zone and showed that errors made by linear approximation are negligible. Therefore, two assumptions can be made:

3.1.1 HYPROP device preparation

High capacity tensiometers for the HYPROP device are saturated and preconditioned by cycles of deairing at vacuum (app. 92 kPa) and by the applying normal atmospheric pressure. As the deionised water is used, due to the small water volume (app. 92 kPa) and by the applying normal atmospheric pressure. As the deionised water is used, due to the small water volume and the special preconditioning, suction over 400 kPa can be measured with HYPROP device (Schindler et al., 2010). At the tests described in this paper only suctions up to 150-200 kPa could be reached by same preconditioning.

3.1.2 Sample preparation

A sampling steel cylinder of known weight and volume was pushed into the undisturbed or in the laboratory prepared, compacted sample. The overlapping soil along the ring’s rim was cut by a sharp knife. Special care should be taken not to smear the pores at the surfaces, as this would increase AEV of the top soil surface. The HYPROP device uses sampling ring with a height of 5 cm and a diameter of 8 cm (Figure 2).

The specimen in the ring is then saturated by immersing it in water. Volume changes are prevented by porous stones on both ends and with the weight of 10 kPa applied on top of the specimen. Better saturation can be achieved when the specimen is saturated under vacuum.

3.1.3 Measurement

Into a saturated soil specimen two boreholes are drilled and in these two boreholes the tensiometers from HYPROP device are installed. The saturated specimen with the HYPROP device is put on a balance and the measurements start. The suction on both tensiometers and the weight change of the specimen are recorded simultaneously every 10 minutes. In the first stage when the water tension in tensiometers is increasing, the readings are in good correlation with the soil suction at the location of the tensiometer. In the second stage the cavitation inside the tensiometer appears and the tensiometer readings are more or less constant. Due to upward tensiometer direction only a small amount of water is drown into the soil specimen. In the 2nd stage the soil suction is higher than the water tension measurements. When the suction in the soil increases over the AEV of the porous cap, air comes into the tensiometer and the water tension inside the tensiometer collapses. When suction in both tensiometers collapses, measurements are finished and the water content and the dry density of the specimen are measured using the standard procedures. The duration of the whole test is between 1 and 2 weeks.

3.1.4 The extension of measurements

The basic idea for extending the measurement range is to use the ceramic cap AEV (Schindler et al., 2010). At this point the air comes to tensiometer and the water tension rapidly collapses to 0 kPa. The soil suction should be the same as AEV of the tensiometer’s ceramic cap. If this assumption is valid, an interpolation by high order polynomial functions of suction between stage 1 and this point can be performed (Figure 3).

By applying this procedure to both tensiometers the measured data can be extended to higher suctions (up to 800 kPa).

Unlike Schindler et al. (2010) only the average suction of both tensiometers at the point of tension collapse of the top
tensiometer is used (Figure 3) and not the whole interpolation function.

Figure 3. Principle of extending the soil suction range using the AEV of the tensiometer’s ceramic porous cap (Schindler et al., 2010).

3.2 Dew point potentiometer WP4-T

Vapour pressure methods are ideal for measuring suctions at the dry end of SWRC. The suction is measured when the equilibration of vapour pressure above the soil specimen is achieved. If vapour pressure is measured, the suction can be calculated using the Kelvin equation:

\[ \psi = \frac{\rho w}{M_w} R T \ln \left( \frac{p}{p_0} \right) \]  

where \( \psi \) is total suction, \( R \) is gas constant (8.314 J/mol), \( T \) is temperature (K), \( M_w \) is molar mass of water (18 g/mol), \( \rho_w \) is water density, \( p \) is vapour pressure and \( p_0 \) is saturated vapour pressure, \( p/p_0 \) is relative humidity.

Potentiometer WP4-T measures vapour pressure through the dew point temperature. This is done by cooling a mirror the reflectance of which is changed when dew appears. From the measured mirror temperature, vapour pressure above the sample can be calculated and from the measured air temperature above the sample saturated vapour pressure can be calculated.

Detailed description of the dew point device can be found elsewhere (Leong et al., 2003, Campbell et al., 2007).

Figure 4. Schematic cross section through dew-point potentiometer WP4-T (Bulut et al., 2002).

3.2.1 Specimen preparation

When measuring suction with vapour pressure methods, dry density and structure have little effect on the suction value (Campbell and Gardner, 1971, Thakur et al., 2006, Birle et al., 2008). Therefore, only disturbed specimens were used. All samples were air dried in and then sieved through a 2 mm sieve. A known amount of water was added to the 5-10 g of soil; soil-water mixture was homogenously mixed and then it was compacted in stainless steel cup forming a single specimen. Prepared specimen was sealed and then rested for 24 hour or more to ensure that the water content and the suction equilibrate.

To measure the whole SWRC, specimens with water contents from the air dried soil to the soil in soft consistency were prepared. The procedure is similar to that described by Campbell et al. (2007).

3.2.2 Measurement

Before the regular measurements started, potentiometer WP4-T had been calibrated each day by using standard solution of 0.5 M KCl. In case a deviation from the suction of 2.22 MPa for more than 100 kPa was recorded, the device was cleaned and recalibrated. In case of smaller deviations, a deviation was used to correct readings at low suctions.

All measurements were done in continuous mode for about half an hour or longer and the measurements were recorded via Hyperterminal program. When the vapour equilibration was observed, an average of measurements after equilibration was calculated.

The vapour equilibration is more important at low suctions (<1 MPa), as it can be seen from the diagrams in Figures 5 and 6.

Figure 5. Suction measurements with potentiometer WP4-T in continuous mode. Black horizontal lines are average suction of the last measurements.

Figure 6. Comparison between SWRC obtained with potentiometer WP4-T from the first measurements and the average value after equilibration.

The resolution of the measurements is 100 kPa and this should be acknowledged, but with this measurement technique the error due to vapour equilibration is eliminated. With this technique good measurements can be performed at as low as 300 kPa, which is significantly lower than 1000 kPa as reported.
4 EXPERIMENTAL RESULTS

SWRC was measured with both methods on 29 soil samples. Three samples belonged to the group of clean sands with less than 5% fines (0.063 <mm), marked by SP-SW symbol, six samples contained 5-12% fines (SP-SM), seven samples were silty sands (SM), eight of them were clays (CH or CL) and five samples were mixed soils with fines content between 40 and 70% (SC/CH). The last mentioned samples were undisturbed, all other were artificially prepared in the laboratory.

Results of some grain size distributions are presented in Figure 7 and the SWRC measurements of these samples are presented in Figure 8. Initial conditions in HYPROP device of specimens shown in Figure 8 are presented in Table 1.

Table 1. Initial conditions in HYPROP device of specimens shown in Figures 7 and 8.

<table>
<thead>
<tr>
<th>classification</th>
<th>w (%)</th>
<th>( \rho_d ) (t/m³)</th>
<th>e</th>
<th>Sr (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SW-SP</td>
<td>15.5</td>
<td>1.80</td>
<td>0.528</td>
<td>81</td>
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<tr>
<td>SM</td>
<td>15.3</td>
<td>1.91</td>
<td>0.440</td>
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<td>96</td>
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<td>CH</td>
<td>91.0</td>
<td>0.740</td>
<td>2.80</td>
<td>92</td>
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</table>

At some samples double porosity was observed when measurements with both methods were put together (Figure 9). The first desorption could be correlated to soil structure or capillary drying and the second is due to drying of fine particles. This behaviour could be natural behaviour or it is inherent due to laboratory compaction.

5 RESULTS AND DISCUSSION

From Figure 8 it can be observed that both measurements correlate well. It can be also observed that HYPROP device measurements are sufficient only for sands with relatively small amount of silty fines (< 20%). In case of mixed soils, where the behaviour is defined by fines or in case of silts and clays HYPROP device measurements are not sufficient and in this case potentiometer WP4-T should be used as well.
Due to large volume deformations of such soils the measurement of shrinkage limit is advisable. Shrinkage limit \( w_s \) is marked by horizontal line in Figure 8.

From potentiometer WP4-T measurements nothing is known about the behaviour at low suction range, as it could be observed that the shape of SWRC changes at low suctions. This behaviour could be measured by HYPROP device.

Figure 9. The observed double porosity of some silty sands.

From the HYPROP device measurements an average suction at the tension collapse of the upper tensiometer was determined for all samples. This suction value was compared with the estimated suction from the potentiometer WP4-T measurements at the same water content. A relative difference between both methods was calculated and they were compared to the zone on SWRC (Table 2, Figure 10). An average relative difference is approximately 30%, and is higher if HYPROP measurements in stage 2 or if the soil has double porosity.

Table 2. Estimated suction and water content at tension collapse and the relative error to the measurements with WP4-T.

<table>
<thead>
<tr>
<th>classification</th>
<th>zone (from-to)</th>
<th>suction (kPa)</th>
<th>w (%)</th>
<th>RDiff suction (%)</th>
<th>RDiff w (%)</th>
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<td>7</td>
</tr>
<tr>
<td>CH</td>
<td>1-1</td>
<td>631</td>
<td>30.9</td>
<td>1</td>
<td>0</td>
</tr>
</tbody>
</table>

RDiff – relative difference, bold – SWRC presented in Figure 8, italic - SWRC presented in Figure 9, * - observed double porosity.

6 CONCLUSION

The main purpose of the paper was to demonstrate that the two simple and cheap methods for measuring soil suction in the laboratory are useful when the extensions of their measuring ranges are made and that the comparisons between both measurement methods were found to be good.

However, both methods have their drawbacks. If potentiometer WP4-T is used, the osmotic suction can influence the results. In this case the osmotic suction should be measured.

When the HYPROP device is used, the AEV of the tested soil is important. When the AEV of the soil is higher than the AEV of the ceramic cap, the described assumptions are not necessarily valid. Therefore, good engineering judgment is still needed.

The proposed extensions give good and reliable data. Differences between both methods are small and are in the range of expected differences even for measurements with the same devices.

An extension of the working range of HYPROP device gives less accurate predictions if the zone on SWRC is changed during stage 2 or if the soil has double porosity.
REFERENCES


Field capacity and moisture loss during active deposition on Tailings Dams

Capacité au champ et perte d'humidité pendant le dépôt actif des résidus

MacRobert C.
University of the Witwatersrand

ABSTRACT: A common method to manage tailings in semi-arid environments is to self-impound the waste as it dries. This research investigated the degree of in-situ drying of platinum tailings. Following sedimentation and drying a steady state developed. This was marked by gravimetric water contents varying within a narrow range related to the materials field capacity. Low water contents indicative of significant suction were only recorded following 6 months of dormancy. Liquidity indices indicated that during normal operation only the outer 50 m dried sufficiently to impound the waste stream.

RÉSUMÉ : Les résidus miniers dans des environnements semi-arides sont souvent mis en dépôt et font prise. Cet article présente l'étude du degré de séchage in situ des résidus de platine. Suite à la sédimentation et au séchage, un état permanent d'hygrométrie est atteint. Cet état est très proche de la capacité au champ du matériau. Les indices de liquidité montrent que pendant l’opération, un séchage suffisant pour retenir l’écoulement des résidus, s’effectue seulement sur une profondeur externe de 50 m.

KEYWORDS: Tailings, Reference Evapotranspiration, Moisture Loss, Field Capacity, Strength Gain.

1 INTRODUCTION

The self-impoundment of mine tailings is dependent on whether the geotechnical behaviour enables strength gain within realistic time frames. In semi-arid environments this is aided by the drying effect of evaporation.

This paper presents results of research carried out on the in-situ drying behaviour of platinum tailings. This was done by monitoring gravimetric water contents following successive field depositions on two back-to-back tailings dams over eleven months. The rate of drying is correlated with Reference Evapotranspiration with the extent of drying illustrated to be controlled by field capacity. Liquidity indices are presented to illustrate the strength gain that occurs.

1.1 Test Work

Test work was carried out on two back-to-back facilities; Dam 1 a 100 ha conventional upstream spigoted facility (raised at 2.3 m/year) and Dam 2, a 100 ha waste rock impoundment filled via a series of spigots (raised at 4.6 m/year). Two separate processing plants supplied similar tailings; the South Plant to Dam 1 and the North Plant to Dam 2.

Sampling took place every 50 m along a 400 m test section on Dam 1 with access by a specially constructed catamaran drawn by a steel cable. On Dam 2 sampling took place every 50 m along a shorter 200 m test section accessed via scaffold from the pool wall. In both cases the test section ran from the spigot points to the pool in the interior. Table 1 details the raw data obtained during the study.

Test depositions were scheduled to deposit 400 mm of material on each test strip at similar cycle intervals. However the depth of material deposited was not uniform due to the inherent beaching behaviour. This resulted in only the outer 100 m having a similar rate of rise of 2.5 m/year on both test strips. On Dam 1 less material was deposited past 100 m whereas on Dam 2 more material was deposited, resulting in rates of rise of 1.2 m/year and 4.0 m/year respectively for these sections.

<table>
<thead>
<tr>
<th>Activity</th>
<th>Raw data obtained</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beach sampling via bulk samples, grab samples, and auger samples</td>
<td>Particle size distributions, particle specific gravities, gravimetric water content, calibrated gypsum block suction tests and triaxial permeability tests.</td>
</tr>
<tr>
<td>Site climatic data</td>
<td>A-Pan evaporation, rainfall and daily minimum and maximum temperatures.</td>
</tr>
<tr>
<td>South African Weather Service, Mokopane Station</td>
<td>Daily temperature, wind speed and relative humidity.</td>
</tr>
<tr>
<td>Historical monitoring and design data</td>
<td>Atterberg limits, evaporative drying tests, filter paper suction tests</td>
</tr>
</tbody>
</table>

2 ANALYSIS OF RESULTS

2.1 Rate of Moisture Loss

The results following the sampling of three depositions on Dam 2 were analysed to determine the rate of moisture loss during sedimentation and drying to steady state.

The rate of sedimentation was determined by linear regression using water contents determined from slurry densities during deposition and grab samples following cessation of deposition. The density of the slurry varied considerably with the water content on average 94 % with a standard deviation of 33 %. Sedimentation was observed to be complete within 65 hours (7 hour standard deviation) with a
final water content of 41 % (1.3 % standard deviation). The water released during sedimentation is available for recovery.

Similarly the rate of drying was determined by linear regression using the water contents following sedimentation, surface samples recovered during the drying stage, and the average steady state value. Figure 1 shows a composite graph of the sedimentation, drying and steady state curves with respective raw data at 50 m along the beach. The rates of drying for the respective depositions along the beach are given in Table 2.

During winter as the evaporative energy is lower, more moisture may be lost through seepage with the opposite being the case during summer. It is also likely that this water bleeds up to the surface as the material consolidates and is recovered. This is illustrated by the k – values being slightly higher during winter and lower during summer but a longer study would be required to quantify this variation.

Figure 2 illustrates the time required for the sedimentation step and then, using the average k – value, the number of days to reach steady state based on daily ET0 values. Such relationships can be used to optimise the safe development of tailings dams.

2.4 Steady State

The steady state after drying was investigated by analysing auger samples taken at 200 mm intervals to a depth of 1 m increasing to 2.5 m as the study progressed. Within the time frame of sampling following drying of each test deposition no trend of water content with time was observed. Rather values varied from sample date to sample date within a narrow distribution. To investigate this steady state various laboratory tests were done. This was supplemented by computer analysis and predictive modelling. Table 3 summarises these results which were used to assess the steady state.

<table>
<thead>
<tr>
<th>Table 3. Geotechnical Parameters</th>
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<tr>
<td>Parameter</td>
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<tr>
<td>----------------------------------</td>
</tr>
<tr>
<td>Air Entry Value</td>
</tr>
<tr>
<td>Peak Dry Density (kg/m³)</td>
</tr>
<tr>
<td>Porosity</td>
</tr>
<tr>
<td>Field Capacity, at 33 kPa (Miller &amp; Donahue, 1990)</td>
</tr>
<tr>
<td>Saturated Hydraulic Conductivity (m/s)</td>
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<tr>
<td>Liquid Limit</td>
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<tr>
<td>Plastic Limit</td>
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<tr>
<td>Particle Specific Gravity</td>
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<tr>
<td>Grading Parameters (µm)</td>
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<tr>
<td>Dam 1, D10</td>
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<tr>
<td>Dam 1, D60</td>
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<tr>
<td>Dam 2, D10</td>
</tr>
<tr>
<td>Dam 2, D60</td>
</tr>
<tr>
<td>Residual Volumetric Water Content, at 1500 kPa (van Genuchten, 1980)</td>
</tr>
<tr>
<td>Field Capacity, Water Content at a Hydraulic Conductivity of 10⁻¹¹ m/s³ (Meyer &amp; Gee, 1999)</td>
</tr>
<tr>
<td>Dam 1</td>
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<tr>
<td>Dam 2</td>
</tr>
</tbody>
</table>

The mean water content for the entire 11 month data set from both dams was 27 % (Standard deviation of 6 %). This
corresponded to the air entry value. Assuming a normal distribution 98 % of the water contents were between 41 % and 14 %. These values corresponded to the average settled water content and the water content below which asymptotic suctions developed.

With depth the mean water content remained constant at 27 %. The variance on the other hand was 50 % for the top 400 mm, 35 % from 400 mm to 1000 mm decreasing to 20 % at 1500 mm and then remaining constant. Large dispersion in the upper layer is due to this being the freshly deposited layer. During deposition water seeps into the underlying layers and is then drawn up during evaporative drying. This process is reflected in the variance below the freshly deposited layer. The decreasing dispersion with depth reflects the decreasing influence of evaporation. The constant variance below 1500 mm suggests this is the limit of evaporative influence.

To explore the controlling effect of field capacity on the degree of moisture loss the following null and alternative hypothesis were tested:

H₀: Water contents at each sampling point have a different population mean to the field capacity mean
H₁: Water contents at each sampling point have the same population mean to the field capacity mean

The two-tailed t-test with unequal variances was used to test the hypothesis. The variances were assumed to be different as the field samples were taken at various stages of drying whereas field capacity has a narrower variance. Field capacity values predicted by the modified Kovács method (Aubertin et al, 2003) for each dam were used. It was assumed that these values better represented the grind differences.

Figure 3 and 4 illustrate the distributions of water content at each position along the Dam 1 and 2 beaches for the entire study. Results of the hypothesis testing are included along the bottom.

These results suggest that during active deposition the steady state water contents are controlled by field capacity with only partial suctions developing. Seepage into the beach during deposition rises to replenish deficits preventing suctions greater than field capacity developing. Closer to the pool the distributions were observed to be in equilibrium with the phreatic surface due to the greater portion of saturated values. This observation was more pronounced on Dam 1 than on Dam 2, presumably due to the fact that phreatic surfaces become more depressed along longer beaches.

Prior to test work on Dam 1 the test section was left dormant for 6 months during high evaporative conditions over spring and summer. Water contents obtained during the baseline sampling showed extensive drying had taken place. The mean water content for the upper 1000 mm was 21 %. Based on the quartile ranges 75 % of the values were below the air entry value of 27 %. And 25 % of the values had water contents indicative of large suctions being below 15 %. This suggests that during active deposition sufficient moisture is available to replenish deficits. Only after long dormancies is this moisture expended and air dry conditions reached.

2.5 Strength Gain

The impact of this limitation on drying to strength gain during active deposition was investigated by calculating the liquidity indices based on the average Atterberg limits. The liquidity indices were then divided into three categories: less than 0 (i.e. above the plastic limit), between 0 and 1 (i.e. between the plastic and liquid limit) and greater than 1 (i.e. above the liquid limit). These categories are plotted in Figure 5 and 6 respectively to sampling position relative to the final elevation at the end of test work.

On Dam 1 it is apparent that only the outer section reached a state of high shear strength (Bovis 2003), with 48% of the samples having a liquidity index less than 1 and 34% less than 0. However, 75 % of the water contents at this position were lower than the maximum liquid limit. After 50 m the proportion of liquidity indices less than 1 was on average 15% for all samples having a liquidity index less than 1 and 34% less than 0. However, 75 % of the water contents at this position were lower than the maximum liquid limit. After 50 m the proportion of liquidity indices less than 1 was on average 15% for all sampling points, being slightly higher at 50 m and decreasing towards the pool. Thus the majority of the interior is prone to fail under shear (Holtz & Kovacs 1981) although it was able to support a man and prevent the auger hole collapsing.

Baseline sampling on Dam 2 indicated that it had not gained significant strength, with only 30 % of the samples at the head...
of the beach having a liquidity index below 1 and the average for the remainder of the beach being 5%. This is attributed to the high rate of rise prior to the test deposition. Analysis at the head of the beach within the first two test depositions indicated that 51% of the samples had a liquidity index below 1 with 75% of the samples below the maximum liquid limit; as was the case for Dam 1 at this position. This drying front also appeared to extend roughly 1 m below the first deposition yet insufficient strength gain appeared to have occurred during the shorter sampling window of the final deposition.

3 CONCLUSION

This paper reported on extensive test work carried out over an 11 month period on two back-to-back platinum tailings facilities. The following conclusions are drawn from the analysis presented:

1. The beach acted as a natural thickener with the tailings slurry settling within 65 hours from a water content of on average 95% to 41%. This water is available for recovery.
2. Following sedimentation water is lost through evaporative drying. The gravimetric water content decreased at a higher rate during summer and a lower rate during winter. The k – value or ratio between water content loss and reference evapotranspiration per day was not constant for all depositions. The k – value was lower during lower evaporative periods. This suggests that seepage and bleed water may contribute a larger portion of moisture loss during lower evaporative periods. A longer study is required to quantify this relationship.
3. After the drying stage a steady state developed. This was marked by water contents varying from sample date to sample date within a narrow distribution. No apparent trend with time was observed. This narrow distribution was found to reflect the seepage of water into upper layers during deposition and capillary rise during drying. Closer to the edge of the dam this distribution was defined by the materials field capacity as sufficient moisture was available to prevent further suctions developing. Only after long dormant periods was this source of moisture observed to deplete. Closer to the pool the water contents were observed to be saturated with the closer phreatic surface preventing any suctions developing. This observation was more pronounced on Dam 1 presumably as the phreatic surface became more depressed along the longer beach.
4. This limitation on drying during active deposition on strength gain was investigated. Only at the outer position did liquidity indices show that substantial shear strength developed. Liquidity indices for the remainder of the beach indicated that the material was prone to fail under shear. This observation was made for all test depositions where the rate of rise for the outer sections was 2.5 m year⁻¹. The baseline conditions on Dam 2 did not exhibit this strength gain at the beach head due to the 4.6 m year⁻¹ rate of rise thus the requirement of a waste rock impoundment.

4 ACKNOWLEDGEMENTS

The author wishes to acknowledge Anglo American Platinum Mogalakwena Mine, Fraser Alexander Tailings, SRK Consulting and the South African Weather Bureau for making this study possible.

5 REFERENCES


ABSTRACT: This paper presents the results of tests carried out on a full scale model house built on jacks to simulate the different settlement distribution of its foundations during a period of drought. Results of observations made during several tests are presented. A numerical analysis of the behaviour of the experimental house was realized. The three-dimensional finite element analysis using the FEM code CESAR-LCPC uses volumetric elements for the concrete blocks, the mortar joints and the concrete of the footing and of the wall ties, whereas the reinforcing bars are modelled using one-dimensional elements, superimposed to the volumetric mesh. Three analyses were performed, in order to match the conditions of the experiments: the supports were suppressed under one fourth, then the half, then another fourth of the house. The calculations gave results close to the measurements.

RÉSUMÉ : Cet article présente les résultats d’essais réalisés sur un modèle en vraie grandeur de maison en maçonnerie, construite sur des vérins pour simuler différentes distributions de tassement de sa fondation lors d’une période de sécheresse. Les résultats des observations faites lors de plusieurs essais sont présentés. Une modélisation numérique du comportement de la maison expérimentale a été effectuée. Pour le calcul tridimensionnel en éléments finis au moyen du logiciel CESAR-LCPC, on a utilisé des éléments de volume pour les parpaings, les joints en mortier et le béton des semelles et des chaînages, tandis que les armatures étaient représentées par des éléments linéiques superposés au maillage volumique. Pour représenter les conditions de l’étude expérimentale, trois calculs ont été effectués, en enlevant tous les appuis sous un quart de la maison, puis sous la moitié de la maison, puis sous un autre quart de la maison. Les calculs ont donné des résultats proches des mesures.

MOTS-CLÉS : maison individuelle, maçonnerie, retrait du sol, instrumentation, vérin hydraulique, modélisation, élément fini.

KEYWORDS: Single house, masonry, soil shrinkage, instrumentation, hydraulic jack, modelling, finite element.

1 INTRODUCTION

Les dommages produits par les effets de la sécheresse sont particulièrement fréquents dans les maisons individuelles à un seul niveau ou avec un étage, construites sur des fondations superficielles. Ces dommages résultent de la combinaison d’un aléa, le tassement produit par le retrait d’un sol argileux, et de la vulnérabilité de la construction, c’est-à-dire son incapacité à résister aux forces et moments créés par la perte partielle d’appui sous les fondations. Pour progresser dans la compréhension de ces dommages, la construction d’une maison expérimentale, que l’on pourrait soumettre à des cycles de retrait et gonflement du sol de fondation, mais aussi renforcer et retester ensuite, a été décidée. Ce travail a été effectué dans le cadre du projet ANR ARGIC (2007-2009 : Analyse du retrait-gonflement et ses incidences sur les constructions, sous la coordination du BRGM) et de l’opération de recherche du LCPC qui porte sur l’effet de la sécheresse sur les constructions (2006-2010). Pour maîtriser le calendrier de ces recherches, il a été décidé de concentrer l’étude sur le fonctionnement de la maison elle-même, sans dépendre du comportement au cours du temps d’un sol argileux sous les fondations, par nature aléatoire. La maison a été construite hors sol, dans un hall d’essai du Centre d’expérimentation routière du Centre d’Études Techniques de l’Équipement Normandie-Centre à Rouen. Elle est connue sous le nom de « Station MISS » (Maison Individuelle Soumise à la Sécheresse). Le choix a été fait de commander à un maçon la construction d’une maison de dimensions standard, en suivant les prescriptions des normes (DTU) applicables à la construction des maisons individuelles. La seule particularité de cette maison est que les semelles de fondation ont été construites sur des appuis métalliques que l’on peut remplacer par des vérins dans les zones où l’on veut modifier les conditions d’appuis des fondations. Des mesures des mouvements verticaux des fondations, des déformations des chaînages et des efforts dans les vérins ont été organisées, ainsi qu’un suivi visuel de la formation des fissures dans les murs et les fondations. Cet article présente la construction du dispositif expérimental et les résultats des premiers essais. Il présente aussi l’analyse numérique en éléments finis effectuée au LCPC en utilisant le module MCNL du code de calcul CESAR-LCPC et compare les résultats des calculs avec les observations faites sur la maison expérimentale.

2 ÉLÉMENTS BIBLIOGRAPHIQUES

Différents auteurs ont travaillé sur le fonctionnement mécanique des constructions en maçonnerie pour comprendre leur comportement sous différentes sollicitations, telles que la compression uniaxiale, la traction uniaxiale, le chargement biaxial et le cisaillement (Fouchal 2006, Hendry 2001). Ces sollicitations élémentaires se combinent dans la réponse mécanique de l’ouvrage en maçonnerie et contribuent à ce que l’on peut appeler la « résistance » de la construction. Ces auteurs sont arrivés à la conclusion que les discontinuités inhérentes à la maçonnerie, c’est-à-dire les interfaces entre les briques et le mortier, fragilisent son comportement et que la rupture des structures en maçonnerie est essentiellement due à la différence des propriétés mécaniques des briques et des joints (Hilsdorf 1969). Nous avons choisi de modéliser séparément les parpaings et les joints comme des éléments de volume ayant un comportement mécanique élastoplastique, de façon à pouvoir identifier précisément la position des zones les plus sollicitées.
dans la structure tridimensionnelle de la maison. Ce choix donne plus de précision sur la distribution des contraintes dans la maçonnerie, mais nécessite un grand nombre d’éléments. Les joints, qui ont une épaisseur faible par rapport aux parpaings, sont souvent assimilés à des interfaces dont la caractérisation mécanique se réduit à la description du frottement entre deux parpaings, en utilisant la loi de frottement de Coulomb (Subash et al 1996). Sinon, ils sont considérés comme des volumes déformables (c’est le cas dans notre étude). Lorsque cette approche est adoptée, il est nécessaire de déterminer expérimentalement les caractéristiques mécaniques des constituants, pour les prendre en compte dans le modèle de calcul. Abdou et al. (2008) sont partis d’essais de cisaillement effectués sur des assemblages briques/mortier pour développer un modèle d’interface pour les joints. Ils ont aussi développé un modèle élastoplastique avec endommagement pour décrire le comportement des briques. Ces modèles sont implémentés dans le code de calcul en éléments finis CAST3M. Pour distinguer la rupture en traction de la rupture en compression, ils ont développé un critère à plusieurs surfaces : la traction est prise en compte en considérant deux critères de Rankine, tandis qu’un critère de Hill est utilisé en compression. Les caractéristiques mécaniques nécessaires dans le cas des briques sont le module d’Young, le coefficient de Poisson, les résistances en traction et compression et l’énergie de fissuration. Pour les joints soumis à une traction, on utilise les rigidités normales et tangente, l’énergie de fission et la résistance à la traction. Pour les joints soumis à un cisaillement, on utilise la cohésion, l’angle de frottement et l’énergie de cohésion. Fouchal et al. (2007) ont utilisé un modèle d’adhésion pour modéliser l’interface entre les briques et les mortiers, qui permet de coupler les conditions de contact unilatérale, de frottement et d’adhérence entre deux solides déformables. Ce modèle estimplémenté dans le code de calcul LMGC90. Les caractéristiques mécaniques d’interface sont représentées par les raideurs normales et tangente, l’énergie de cohésion, le coefficient de frottement et la viscosité. Nous avons ainsi modélisé les comportements des matériaux à comportement non linéaire (élastique linéaire – plastique). Lesurs caractéristiques mécaniques sont le module d’Young, le coefficient de Poisson et les résistances en compression et en traction. Nous avons utilisé le calcul du module MCNL du code de calcul en éléments finis CESAR-LCP, qui résout les problèmes de comportement mécanique non linéaire. Pour cette modélisation, nous avons considéré les conditions réelles des essais réalisés sur la maison expérimentale : même géométrie, mêmes conditions aux limites et chargement identique à l’expérience. Pour ce qui concerne les caractéristiques mécaniques des matériaux, faute de disposer de mesures sur les matériaux de la maison, nous avons utilisé des caractéristiques de matériaux qui ressemblent (parpaings et joints), trouvées dans la littérature (Hendry 2001, Gabor 2002 Lemaître 1988).

3 CONCEPTION DU DISPOSITIF EXPÉRIMENTAL
La maison expérimentale a été conçue pour permettre la répétition des essais et la comparaison des comportements de la structure avec et sans plancher. Un plan rectangulaire de 10,65m x 8,65m de dimensions extérieures (Figure 1), divisé en quatre secteurs supposés indépendants les uns des autres (Figure 2), a été adopté. Les murs extérieurs comportent des baies (portes-fenêtres) et des fenêtres. Seul le gros œuvre de la maison a été exécuté. Les charges correspondant à la toiture ont été remplacées par des massifs de béton répartis sur les murs pignons et de façade. La maison est divisée en quatre secteurs, qui permettent de tester deux fois chaque modalité structurelle. La maison repose par l’intermédiaire de sa semelle sur une série de supports mécaniques amovibles, fixés sur un support rigide en béton armé scellé en place. L’ensemble repose sur une plateforme peu déformable. Pour la réalisation des essais dans un secteur de la maison, on vient remplacer les appuis mécaniques de ce secteur par des vérins hydrauliques et produire les tassements ou soulèvements de la semelle de la maison à l’aide de ces vérins hydrauliques.

![Figure 1. Vue d’ensemble de la maison expérimentale (MISS)](image)

![Figure 2. Secteurs d’essai.](image)
été coffrés et coulés en place, avec quatre armatures HA10 chacun. Le plancher haut est destiné à supporter les surcharges simulant la présence d’une charpente et d’une couverture. Il est constitué de bastaings posés à champ et de dalles de plancher en bois. Il est lesté à 7,5 t sur la façade et 2,85 t sur le mur pignon, avant la réalisation des essais.

3.2 Protocole d’essai

L'essai consiste à abaisser progressivement les vérins sous une partie de la semelle de fondation, en observant le comportement de la structure de la maison. Cette opération est prévue secteur par secteur, l’essai sur un secteur pouvant être répété si nécessaire sur le secteur symétrique. Pour le secteur choisi pour l’essai, la procédure expérimentale commence par la substitution des appuis mécaniques de la maison par des vérins. On abaisse ensuite les vérins en suivant une courbe de tassement parabolique (tassement maximal du côté extérieur). Les vérins sont d’abord abaissés pour les deux premiers appuis à partir du coin de la construction. Cette action a pour effet de permettre un tassement de la semelle de fondation de la maison. L’opération se poursuit en autorisant progressivement un tassement différentiel parabolique sur toute la longueur des semelles. Les valeurs caractéristiques retenues sont 1/1000ème, 1/500ème et 1/250ème. Les tassements autorisés sont imposés tous les mètres, ce qui est la distance entre deux vérins hydrauliques. Le temps d’observation et de suivi entre deux phases est en général de 24 heures. Le critère d’arrêt retenu est l’apparition de fissures dans les murs d’au moins 2 mm de largeur.

4 MODÈLE NUMÉRIQUE

Nous avons choisi une représentation tridimensionnelle de la structure de la maison, dans laquelle les parpaings, les joints et le béton sont modélisés par des éléments volumiques à huit nœuds, tandis que les chaînages horizontaux et verticaux sont représentés par des éléments linéiques superposés au maillage tridimensionnel. Comme déjà indiqué, chaque élément de volume est caractérisé par cinq paramètres : la masse volumique, le module d’Young, le coefficient de Poisson, la résistance en compression et la résistance en traction. La figure 3 présente les deux maillages de la maison : le maillage des éléments de volume à huit nœuds, qui représentent le béton des fondations, les parpaings et les joints et le maillage des éléments linéiques, qui représentent les armatures présentes dans les chaînages. Le maillage comporte 111 643 éléments, qui se divisent en : 108 194 éléments à 8 nœuds (246 98 pour les parpaings, 31 552 pour les joints, 51 944 pour les poutres en béton et le plancher), 34 499 éléments linéiques à 2 nœuds pour les armatures. Les propriétés des trois types de matériaux des éléments de volume sont données dans le tableau 1. Les propriétés des éléments linéiques des armatures sont données dans le tableau 2. Trois calculs, avec trois séries de conditions aux limites, ont été effectués, pour reproduire les conditions des trois essais décrits dans la première partie de cet article : suppression des appuis sous le secteur 1, suppression des appuis sous les secteurs 1 et 4, suppression des appuis sous le secteur 2.

4.1 Modélisation de l’essai sur le secteur 1

L’essai sur le secteur 1 consiste à abaisser les vérins hydrauliques qui portent la semelle de fondation sous le coin de la maison sans plancher. Pour le calcul, cette condition équivaut à l’absence d’appui sous cette partie de la semelle. Cet état, imposé par étapes dans l’étude expérimentale, mais sans que cela provoque de désordres dans la structure de la maison, a été imposé directement dans le calcul. On observe un enfoncement de la partie de la maison privée d’appuis et des mouvements latéraux du haut des murs de façade. La valeur calculée du déplacement vertical maximal (sous l’angle de la maison) vaut 3,4 mm, ce qui est proche de la valeur observée (3,2 mm). La figure 4 compare les tassements calculés et observés sous les semelles de fondations.

| Tableau 1. Propriétés mécaniques des matériaux pour le calcul |
|-------------|----------------|-----------------|----------------|----------------|
| Masse volumique (kg/m³) | Module d’Young (MPa) | Coefficient de Poisson (-) | Résistance en compression (MPa) | Résistance en traction (MPa) |
| Béton        | 2500         | 40000          | 0,2            | 20             | 2              |
| Joints       | 1500         | 6000           | 0,2            | 6              | 1              |
| Parpaings    | 1000         | 4000           | 0,1            | 4              | 0,5            |

| Tableau 2. Propriétés mécaniques des armatures pour le calcul |
|-------------|----------------|-----------------|----------------|----------------|
| Masse volumique (kg/m³) | Module d’Young (MPa) | Coefficient de Poisson (-) | Aire de la section droite (mm²) |
| Armatures   | 7850          | 210000         | 0,3            | 157            |

Les aires de la section droite de l’armature des fondations (151 mm²) et de l’armature des chaînages verticaux et horizontaux (157 mm²) sont très proches. Une valeur unique a été choisie pour les deux.

c. Figure 4. Essai sur le secteur 1 – Tassement de la semelle (mm)

La résistance en compression des éléments de maçonnerie est plus importante que leur résistance à la traction, qui est parfois négligée. Nous avons examiné les contraintes de traction produites dans les éléments en maçonnerie (éléments volumiques) et les efforts normaux dans les éléments linéiques et les avons comparés à la résistance à la traction introduite dans le calcul pour déterminer les zones où des désordres peuvent apparaître, comme des fissures. Ces contraintes dépassent la
résistance à la traction du béton (2 MPa) dans certaines zones de béton armé (semelles) d’extension limitée, où elles atteignent 3,41 MPa. Les valeurs maximales sont de 1,28 MPa dans les joints (pour une résistance à la traction de 1 MPa) et 43 MPa dans les armatures (effort normal de traction / aire de la section droite), ce qui reste en dessous des résistances à la traction des armatures. Les déformations du béton armé sont contenues par l’élasticité des zones contiguës, de sorte que l’on ne peut voir apparaître aucun désordre ni fissure dans la semelle, ce qui est compatible avec le résultat de l’étude expérimentale. Il faut noter que la valeur du moment fléchissant dans les éléments linéaires est de l’ordre de 5 N.m. Cette valeur faible ne dépasse pas la résistance des éléments.

4.2 Modélisation de l’essai sur le secteur 1 et 4

L’essai sur les secteurs 1 et 4 consiste à abaisser les vérins hydrauliques sous la moitié de la maison (côté sans plancher). Pour le calcul, cette condition équivalent à l’absence d’appuis sous cette partie de la semelle. Cette configuration a été appliquée directement dans le calcul, alors que l’essai comportait plusieurs phases. La figure 5 compare les tassements calculés et mesurés des semelles. Les tassements calculés et observés ont des valeurs proches. Le calcul a été effectué en suivant les phases de l’expérimentation, dans laquelle on a déchargé d’abord le secteur 1, puis on a replacé des supports sous ses fondations mais sans les ramener à la position initiale (le tassement de quelques millimètres a été conservé). Pour le calcul des secteurs 1 et 4, nous avons gardé comme état initial de déformation et de contraintes l’état final du calcul précédent (secteur 1). Ceci explique pourquoi les tassements calculés des secteurs 1 et 4 ne sont pas symétriques, alors que le maillage et les matériaux le sont.

4.3 Modélisation de l’essai sur le secteur 2

L’essai sur le secteur 2 consiste à autoriser un déplacement de la semelle de fondation sous un quart de la maison du côté du plancher, et à bloquer les déplacements calculés pour le chargement des secteurs 1 et 4 (créer des appuis avec des déplacements initiaux, car la maison n’a pas été ramenée à son état initial avant de réaliser l’essai sur le secteur 2), tout en tenant compte des contraintes produites dans la structure après le chargement des secteurs 1 et 4 comme état initial de calcul pour le secteur 2. La figure 6 compare les tassements calculés et mesurés. Le tassement maximal calculé vaut 4,8 mm, alors que la mesure a donné 5,6 mm. On observe que les murs du secteur 2 sont sensiblement moins sollicités que ceux du secteur 1 et 4. Les efforts normaux de traction dans la chaîne des secteurs 1 et 4 augmentent. Par contre, les efforts normaux de traction dans le secteur 2 sont faibles.

5 CONCLUSION

La modélisation tridimensionnelle en éléments finis de la maison expérimentale testée au CER de Rouen a été effectuée en suivant une démarche naturaliste. Nous avons reproduit et décrit la géométrie et les propriétés mécaniques de chacun des matériaux utilisés pour la construction d’une maison courante. La modélisation de la suppression des supports des fondations sous un ou deux secteurs de la maison a produit des résultats encourageants en termes de tassements et de déformations de la structure en maçonnerie de cette construction. Les amplitudes des déformations dépendent des modules attribués aux matériaux, pour l’essentiel les parpaings, le mortier et le béton des semelles de fondation. Les conclusions opérationnelles que l’on peut tirer de cette étude ne sont pas issues du seul calcul : la plus importante est que la construction de maisons conformes aux règles françaises actuelles (DTU) semble suffisante pour leur donner une bonne résistance aux mouvements de retrait des sols dus à la sécheresse. Le calcul confirme cette bonne tenue d’une structure en maçonnerie avec des semelles de fondation en béton armé et des chaînages verticaux et horizontaux régulièrement espacés.

6 RÉFÉRENCES

Hydro-mechanical properties of lime-treated London Clay

Propriétés hydromécaniques de l’argile de Londres traitée à la chaux

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ABSTRACT: This paper investigates the effect of lime on the hydromechanical properties of untreated London Clay (a high plasticity clay) and lime-treated and London Clay samples, using two different curing methodologies, and two different dosages of hydrated lime beyond the Initial Lime Consumption level. They were then subject to a number of triaxial tests, to investigate the effect of the above factors on the properties and behaviour of the treated soil in the saturated and partially saturated state. Soil water retention curves were also determined and the favourable effect of the lime on the volumetric stability of the soil demonstrated.

RÉSUMÉ : On étudie l’effet de la chaux sur les propriétés hydromécaniques de l’argile de Londres, à partir d’une série d’essais triaxiaux. On décrit l’effet du dosage en chaux, des conditions de cure et de la saturation partielle. Les résultats expérimentaux indiquent que la chaux a réduit la compressibilité et a amélioré la résistance de cisaillement du sol. Cependant les éprouvettes traitées à la chaux ont affiché un comportement avec radoucissement en grandes déformations, contrairement aux éprouvettes d’argile de Londres qui ont affiché un comportement avec écouissage positif (durcissement). Ce comportement était particulièrement prononcé dans le cas des éprouvettes à dosage en chaux élevé. Dans la suite on mesure la succion et les courbes de retentissement d’eau à l’aide de la méthode du papier filtre, démontrant l’effet favorable de la chaux sur la stabilité volumique du sol.

KEYWORDS: lime treated London Clay; hydro-mechanical properties; triaxial testing; soil water retention curve

1 INTRODUCTION

Lime treatment has been extensively used to improve the engineering properties of clay soils; namely to increase the workability of high plasticity soils during construction, and for the stabilisation of roads and pavements (capping layers, sub-bases and subgrades). With these applications in mind, research on lime-stabilised clays has historically focused mainly on properties such as plasticity, CBR or Unconfined Compression Strength (UCS). There is however limited information in the international literature based on triaxial testing that can be used to describe the constitutive behaviour of lime-treated soils. There is also lack of information on the properties of these soils in the partially saturated state, although they are typically compacted and hence, partially saturated. This paper investigates the mechanical properties of a high-plasticity clay (London Clay) and the effect of factors such as lime percentage and curing methodology through CD triaxial tests. Results for the soil water retention curve of this soil are also presented.

2 MATERIALS AND EXPERIMENTAL PROCEDURES

2.1 Materials

The soil used in this study was London Clay taken from an excavation at Westminster Bridge in the city of London and depths corresponding to B2 stratigraphic unit (King, 1981). The soil was air-dried at an average temperature of 22°C and a relative humidity of 60% for a month and pulverised. Figure 1 shows the particle size distribution of the portion of the soil passing the BS 425 μm sieve. X-ray diffraction (XRD) tests showed 50% I1lite, 26% Montmorillonite, 15% Kaolinite and 9% Chlorite (relative % of each clay mineral with respect to clay fraction).
Commercially available hydrated lime was used after its suitability for soil stabilisation has been established. Chemical analysis on the lime sample carried out in duplicate showed that the relative proportion of calcium hydroxide to calcium oxide was 4.88:1.00. The lime was mixed with clay in dry condition. Plasticity tests were performed on London Clay mixed with lime at percentages of lime of 0%-8% by dry unit of soil respectively, for mellowing periods of 1 and 24 hours respectively. These showed no change in the plasticity characteristics of the lime-treated soil beyond 4% of lime addition (see Fig. 2). Hence this was considered to be the minimum necessary lime percentage for treating this clay. The percentage was confirmed by initial consumption of lime test results (see Fig. 3).

2.2 Specimen preparation

For the preparation of untreated London Clay samples the clay powder was thoroughly mixed with water to achieve a water content of 25.5% (the Proctor optimum) and left to hydrate in sealed bags for 72 h. For the preparation of lime treated samples, dry London Clay and hydrated lime powders were thoroughly mixed and then the required amount of water was added (27% and 32% i.e. dry and wet of Proctor optimum for the lime treated soil). Static compaction was selected as the best way of exerting sufficient control over the compaction process of a clayey soil. In this experimental investigation both types of specimen were compacted at the same target dry density of 1.43 g/cm³, corresponding to the maximum standard Proctor dry density of the London Clay soil. The compaction of the triaxial specimens (76 mm height and 38 mm diameter) was conducted in split-moulds of the appropriate dimensions. The soil was placed in the mould in six equal layers and compressed at a monotonous displacement rate of 1mm/min until the required height was reached. The loading ram was then held in contact with the soil for another 5 minutes to reduce the rebound upon unloading. A similar method was adopted for the compaction of the specimens for the SWRC tests. These were compacted in standard oedometer cutting rings of 75 mm diameter and 20 mm height used as compaction moulds. After compaction two different methods of curing for the lime-treated specimens were used, namely water curing and air curing. In the first method the specimen was left in the mould to cure in contact with water for the whole curing period. In the air curing method the specimen was wrapped in several layers of cling film and stored in controlled environmental conditions for the specified curing period. To complete saturation after curing, back-pressure saturation was applied regardless of the curing technique.

2.3 Triaxial testing

To assess the effect of cementation, indicative sets of different triaxial testing results will be shown. These were performed on specimens of London Clay and the corresponding lime-treated London Clay specimens prepared and tested at a variety of different conditions. All saturated specimens were sheared drained after isotropic consolidation, following a $q/p = 3$ path. For the saturated lime-treated specimens results based on two different curing methods (air or water curing), and two different percentages of lime will be shown. For partially saturated specimens, results from four tests will be shown: a) two compacted specimens of a treated (4% lime, air cured) and an untreated soil respectively sheared as compacted (UU test); and (b) two compacted specimens of a treated (4% lime, air cured) and an untreated clay respectively, that were brought to a 300 kPa suction equilibration before testing and subsequently isotropically consolidated under a net stress of 200 kPa and sheared drained following a $q/p = 3$ path, maintaining a constant cell pressure and a constant suction of 300 kPa. Axis translation was used to control the suction during testing. The reason for showing results from two different test types was to demonstrate that the effects of cementation were similar, irrespective of the testing conditions.

2.4 Filter paper testing

The filter paper used in the present research to measure matric suction ("contact" filter paper technique) was Whatman No.42 filter paper with a calibration formula according to Chandler and Gutierrez (1986). The soil specimen was placed between two Perspex disks. Filter papers were placed on each side of the specimen, between the soil and Perspex disk interfaces. The soil specimens were then tightly wrapped in multiple layers of cling film and sealed bags and left in an insulated environment for one week at a time. After this period the filter papers were carefully removed and their water content was determined. Subsequently, the soil specimens were left for air-drying until the new target water content was achieved. They were then wrapped again for the new moisture content measurement to be performed one week later.

3 RESULTS AND DISCUSSION

3.1 Triaxial testing results

Figures 4 and 5 show indicative comparative stress-strain relationships and volumetric strains of London Clay and lime-treated London Clay specimens (saturated and partially saturated specimens respectively) prepared and tested under various conditions. It can be seen that for all types of tests the London Clay specimens show a strain hardening behaviour for both saturated and partially saturated samples irrespective of the suction level, although there is an increase of strength with increasing suction as expected. Conversely the lime-treated soil shows a strain softening behaviour irrespective of the mode of curing and testing, and this is consistent with the breakage of the cementation bonds. The lime-treated soil became increasingly stiffer, stronger but also more brittle with the increase in lime percentage and also when it was air cured as opposed to water cured. Although the three sets of tests in Figure 5 are not directly comparable due to the different preparation and/or testing procedures adopted, it can still be seen that for the same net stress of 200 kPa, the strength of the lime-treated specimens also increased with suction. In the partially saturated state the brittle behaviour and strain softening of the lime treated specimens was particularly pronounced. Whereas this is the expected behaviour of a partially soil due to the effect of suction in this instance the behaviour is presumably due to the combined effect of suction and the breakage of cementation bonds. Note that for all lime-treated samples (saturated or partially saturated) dilation is clearly observed after, rather than before the peak stress (especially in the 6% air cured samples), i.e. the extra component of strength is not due to the dilatancy as it would be in the case of a particulate material; instead peak strength is mobilised well before the maximum rate of dilation. As dilation only happens upon softening, this could be related to the breakage of cementation bonds and is consistent with the typical behaviour of soft rocks (Vaughan 1993).
Figure 6 shows the stress path plots in the q:p’ plane together with the peak (applicable to lime treated soil only) and critical state lines. As for 4% lime the values of M for the lime treated and untreated London Clay were very close irrespective of the mode of curing, it was concluded that both types of soil converged on the same Critical State line in the q:p’ plane with M=0.88 (see Fig. 6), i.e. a critical state angle of friction $\phi'=22.5^\circ$, consistent with values reported in the literature for London Clay. This implies that this lime content does not appear to have modified the frictional properties of the material. For 6% lime, M and consequently the critical state friction angle were slightly higher (1 and 25.4° respectively), presumably due to the formation of a greater amount of cementing material (due to pozzolanic reactions induced by the surplus of lime above the ILC) coating the particles. The collective characteristics of the soils from the shearing stage are shown in Table 1. The compressibility behaviour of the soils was difficult to assess fully due to the limited range of isotropic compression pressures. The results were complemented with data from K₀ compression using equipment that achieved a range of confining pressures up to 2000 kPa. Even so, full destructuration of the material did not occur. For the ranges of confining pressure considered the increase in stiffness upon lime treatment was very considerable (for instance for the 4% water-cured soil $\lambda=0.05$ whereas for the untreated soil $\lambda=0.14$). Consequently, during compression lime-treated samples maintained for the most part higher specific volumes $\nu$ than untreated samples (although the latter started with higher $\nu$ due to swelling).

![Figure 4. Indicative triaxial testing results (saturated soil): (a) q:ε sö results (b) ε aust results](image)

![Figure 5. Indicative triaxial testing results (unsaturated soil, tests performed at a net stress of 200 kPa): (a) q:ε sö results (b) ε aust results](image)

![Figure 6. Drained triaxial testing (saturated soils): Stress paths and Critical State and peak state lines in the q:p’ plane](image)

<table>
<thead>
<tr>
<th>Soil</th>
<th>$c'$</th>
<th>$q'_p$</th>
<th>$\phi'$</th>
<th>M</th>
</tr>
</thead>
<tbody>
<tr>
<td>untreated</td>
<td>0</td>
<td>N/A</td>
<td>22.5</td>
<td>0.88</td>
</tr>
<tr>
<td>4% lime water cured</td>
<td>38</td>
<td>26.5</td>
<td>22.5</td>
<td>0.88</td>
</tr>
<tr>
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<td>39</td>
<td>30.7</td>
<td>22.5</td>
<td>0.88</td>
</tr>
<tr>
<td>6% lime, air cured</td>
<td>170</td>
<td>39.9</td>
<td>25.4</td>
<td>1</td>
</tr>
</tbody>
</table>
3.2 Filter paper testing results

Figure 7(a) shows the variation of the gravimetric water content with suction. There is an apparent higher overall water retention capacity of the treated soil due to the higher initial water content however it can be seen that the rate of water loss with suction of the lime treated soils is not much different compared to the respective untreated sample, despite the fact that lime has changed the soil structure. Despite the expected change in the nature of the soil (mineralogy and size / specific surface) after treatment, at higher suctions where adsorptive phenomena predominate, the differences between treated and untreated soil are not clear, which is difficult to explain. For the untreated soil it is expected that compaction conditions do not affect the results so much, as at low saturation adsorptive forces gradually predominate and the effect of soil structure appears to have little influence on the SWRC. However it would be expected that the water retention of the chemically treated soils should have been different to that of the untreated soil, due to the change in the composition and specific surface area of the soil (related to the adsorptive forces) brought about by the lime treatment. This is not noticeable in the results of treated sample compacted dry of optimum; there is however some indication that this happens to some extent for the lime-treated soils compacted at higher water content (those show a slightly steeper desorption slope, implying faster desaturation) which could perhaps be attributed to the fact that water facilitated further pozzolanic reactions due to enhanced ion migration, and hence further alteration of the microstructure. As for untreated soils, the lime-treated sample compacted dry of optimum, showed a lower water retention capacity compared to the respective sample compacted dry of optimum due to the more open structure. As with untreated soils, the deformability of the lime-treated samples compacted wet of optimum is higher (see Fig 7c), however the lime treated samples showed overall much lower volumetric strains with respect to the untreated soil, especially for the higher lime content as a result of cementation. Overall it can be clearly noted that cementation considerably affected the strain related quantities (the void ratio and the volumetric strain) due to the increased stiffness.

4 CONCLUSIONS

A number of triaxial tests and filter paper tests were carried out to assess the effect of lime on the hydromechanical properties of statically compacted London Clay and lime-treated London Clay samples respectively. The results showed that the lime-treated soil became increasingly stiffer, stronger but also more brittle with the increase in lime percentage and also when it was air-cured as opposed to water-cured. The strain softening and stiffness degradation at narrow strain ranges was even more pronounced for partially unsaturated air-cured lime-treated London Clay soil was due to the combined effect of lime and suction. It appears that water curing and lower percentages of lime could in fact be more beneficial as they increase sufficiently the stiffness and strength of the soil without resulting in very brittle behaviour and abrupt strain softening within the range of strains of relevance for engineering design. The effect of the lime on the water retention capacity of the material was found to be less pronounced but a considerable reduction in volumetric changes with suction change was noted.

5 ACKNOWLEDGEMENTS

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6 REFERENCES

Influence of initial water content on the water retention behaviour of a sandy clay soil

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ABSTRACT: In order to model the impact of climate changes on the infrastructure for the built environment, such as embankments and cuttings, an understanding of the water retention behaviour is required. To this end, a series of filter paper tests were carried out on remoulded samples of a sandy clay material of medium plasticity. The soil samples were prepared with different initial water contents and dynamically compacted. Filter paper tests were performed to determine the soil water retention curve (SWRC) from a saturated state. Other subsamples were wetted or dried to reach different water contents for testing. The obtained results were then compared with the SWRC. The drying tests showed typical behaviour of scanning curves, however the wetting curves showed untypical behaviour, where the curves appeared to overlap each other with no clear pattern. The observed behaviour from laboratory samples can be extrapolated to field conditions, where future climate change will have a major impact on the water retention behaviour on earth structures, which will have implications for the geo-mechanical behaviour.

RÉSUMÉ : L'impact du changement climatique sur les constructions géotechniques, comme les remblais/déblais, doit être appréhendé. Dans cette optique, une série d'essais par la méthode du papier filtre a été réalisée sur des échantillons reconstitués d'une argile sableuse à plasticité moyenne, afin d'analyser les conséquences sur la courbe de rétention d'eau du sol. Le matériau a été préparé avec des teneurs en eau initiales différentes afin d'obtenir des échantillons par compactage dynamique. Ces échantillons ont été utilisés soit directement pour effectuer la méthode du papier filtre, soit leur teneur en eau a été modifiée après compactage. Les résultats obtenus ont alors été comparés avec la courbe de rétention d'eau. Les tests en séchage montrent un comportement classique sur les courbes de transitions, alors que ceux en humidification montrent un comportement atypique puisque les courbes se croisent sans donner une tendance claire. A partir des résultats expérimentaux, une extrapolation peut être réalisée concernant le comportement des ouvrages géotechniques, où les changements climatiques futurs auront des répercussions sur le comportement hydromécanique du sol.

KEYWORDS: Filter paper, SWRC, Scanning curves

1 INTRODUCTION.

Earth structures (i.e. road embankments, railway embankments, earth dams and flood defences) can fail when pore water pressures increase significantly (and soil suction drops) following intense rainfall or flooding. With predicted changes in climate patterns, such failures are likely to become more frequent with significant economic implications. The 4th Assessment Report of IPCC (IPCC, 2007) states: “Continued greenhouse gas emissions at or above current rates would cause further warming and induce many changes in the global climate system during the 21st century that would very likely be larger than those observed during the 20th century”. The increased global warming will affect climate patterns, with longer and drier summers followed by wetter winters with more intense storms predicted for UK and northern Europe. To model the impact of these changes in climate on earth structures requires an understanding of the water retention behaviour.

The water retention behaviour can be characterized by determining the soil water retention curve (SWRC) for a specific soil. A SWRC is defined from the relationship between water content and suction. The water content can be expressed either as gravimetric water content, w, volumetric water content, θ or even degree of saturation, S. A SWRC is typically S-shaped and is hysteretic (Figure 1), meaning that for a given water content, higher suctions can be obtained when following a drying path than following a wetting path. In some cases a soil may not follow a continuous path from a totally dried or totally wet state. It is very common to find soils in an intermediate state when the direction of water content change is reversed. These intermediate stages are known as scanning curves. In Figure 1 two kinds of scanning curves are presented as simple examples: an ascending scanning curve where the initial condition was reached while following a drying path and was subsequently wetted and a descending scanning curve where the intermediate stage starts on the wetting path of the SWRC and the material gradually dries until the drying path of the SWRC is reached. In reality any point between the primary wetting and drying paths can exist, but by following wetting or drying the curve will eventually converge with one of the primary paths of the SWRC.
The SWRC is dependent on various physical, chemical, mineralogical and mechanical properties. One factor that influences the SWRC is the initial water content. Many studies have been put forward in recent years on the effect of the initial water content on SWRC, mainly considering SWRCs following drying paths. Vanapalli et al. (1999) observed the influence of compaction in soil samples at different levels (at optimum, wet and dry of optimum water contents). Where, the SWRC following a drying path was determined for samples dry of optimum, it was found to be steeper than the SWRC at optimum or wet of optimum. The attributed reason was that the samples tested dry of optimum have a highly aggregated macrostructure, thus resembled the behaviour of coarser material. However, the micro structure governs the SWRC wet of optimum (as soils are unlikely to be aggregated in this condition). It has been observed that at high suctionals all SWRCs seem to converge to a single curve. Studies have been conducted by Marinho and Chandler (1993), Ng and Pang (2000), Birle et al. (2008), among others, that have shown that the SWRCs are greatly influenced by the initial water content.

In this paper a series of suction measurements using filter paper tests is presented that were carried out on remoulded samples of a sandy clay soil. Soil samples were prepared with different initial water contents (10%, 13%, 15%, 20% and 22%) and dynamically compacted that were then used to perform the filter paper tests. Other samples were wetted or dried to reach the other water contents for testing. The results obtained were then compared with the soil water retention curve (SWRC) for this material drying from a saturated state.

2 MATERIALS AND METHODS

2.1 Material properties

The soil material used in this study was glacial till sourced from a stock pile in County Durham, UK. From the particle size distribution shown in Figure 2, the soil material is classified as well graded sandy clay. As for the index properties, the liquid and plastic limits were found to be 43.3% and 23.7% respectively, meaning a plasticity index of 19.6.

![Figure 2. Particle size distribution curve of the sieved material.](image)

Due to the variability in sample preparation, observed in preliminary tests, the soil material was sieved to a maximum particle size of 2.80mm to remove occasional gravel sized particles. The resulting compaction curve is shown in Figure 3, where the optimum water content was found to be 15.5% at a maximum dry density of 1.719 Mg/m³.

2.2 Sample preparation

Samples with size 100mm in diameter and 200mm in height were dynamically compacted after preparation at 5 different water contents: 10%, 13%, 15%, 20% and 22%. Subsequently, subsamples were trimmed down to discs with 55mm in diameter and 20mm in height.

For testing purposes, discs with similar water content were later dried or wetted to other water contents (e.g. subsamples at an initial water content of 15% were dried to 10% or 13% and wetted to 20% or 22%). The drying procedure used was air drying, while the wetting procedure was conducted inside an humidifying chamber. In both cases, after the subsamples had reached the target water content, they were sealed off for a period of at least 5 days for water content homogenization. Detailed information on these procedures can be found in Mendes (2011).

![Figure 3. Compaction curve obtained for the sieved material.](image)

2.3 Filter paper technique

The filter paper technique was used to determine the soil suction. This technique can measure soil suction either by vapour flow (non-contact filter paper – total suction) or by liquid flow (contact filter paper – matric suction). This measurement is achieved by letting the soil-filter paper system to reach equilibrium. When this equilibrium is reached the measurement of soil suction can be determined. The major advantages of the filter paper technique are the wide measuring range, from 0 up to 30MPa, and low cost. The major drawbacks, however, are the long term equalization period (5 to 14 days) and the quality of the measurement, which is dependent on the experience of the user and also on the calibration curve that relates the water content of the filter paper to suction.

The method for the filter paper technique used in this work was adapted from Bulut et al. (2001) for the measurement of soil matric suction. Three filter papers, in this case Whatman 42, were placed in intimate contact between two sample discs of similar water content, as shown in Figure 4. The outermost filter papers were used to prevent contamination from soil particles and the middle filter paper was used for the measurement. The whole setup, filter papers and disc samples, was sealed with electrical tape to prevent contact between the filter papers and air and placed inside a glass jar. The jar was later wrapped in plastic film, coated in paraffin wax and submerged in a water bath at 25°C for an equalization period of 14 days. After the equalization period, both sample discs and filter paper were quickly removed from the glass jar in order to determine the filter papers’ wet mass to a level of accuracy of 0.0001g. These were later oven dried for determination of the water content. The water content of the filter paper was then used to determine the corresponding suction, by means of a calibration curve, associated with the known water content of the sample discs.

Based on the work of Noguchi et al. (2011) it was found that the calibration curve that gave a best match for this particular soil was that obtained by van Genuchten (1980) in the form of equation (1).
3 RESULTS AND DISCUSSION

The suction measurements obtained for the different water contents, along with the soil water retention curve (SWRC) following the primary drying path obtained by Noguchi et al. (2011), are presented in Figures 5 to 8. The primary drying SWRC obtained by Noguchi et al. (2011) was obtained from samples prepared initially at 25% of water content, close to a fully saturated state. As shown in Figures 6 or 8, the primary drying SWRC has the typical shape of a bimodal function.

3.1 SWRCs following drying paths

Figure 5 shows the best fit curves obtained for the SWRCs at different initial water contents following drying paths. It is clear from Figure 5 that the obtained curves are initially lower than the primary drying curve obtained by Noguchi et al. (2011) for a specimen prepared at 25% water content. However, later, at around 10-11% of water content, or 1500-2000 kPa of suction, it can be observed that the curves converge to the primary drying SWRC. This suggests that the SWRC of soils compacted at lower water contents follow drying paths that are very like the behaviour of scanning curves.

Figure 6 shows the matric suction SWRC following drying paths in terms of the degree of saturation. Due to changes in methodology, volumetric measurements were only obtained in tests for initial water contents of 20% and 22%. Comparing them with the primary drying curve obtained by Noguchi (2011) the two curves 20% and 22% initially fall under, but later converge with the primary curve.

These results show many of the features identified in the conceptual model for drying proposed by Toll (1995). As suggested by Vanapalli et al. (1999), there is a higher resistance to desaturation (flattening of the SWRC) with decreasing initial water content.

3.2 SWRCs following wetting paths

The behaviour of the SWRCs obtained following wetting paths, however, shown untypical behaviour. As is observed from Figure 7 the SWRCs that followed a wetting path moved towards the primary drying curve, rather than towards the primary wetting curve. Although the primary wetting curve was not determined, the impression is that the behaviour of the wetting SWRCs seems different to that expected. The SWRCs seem to cross the primary drying curve in an ascending form, where the SWRC obtained from 10% of water content was the first to cross at 300 kPa of suction followed by the SWRC for the water content of 13%, 15% and so on.

Similar results were observed in the matric suction – degree of saturation relationships for the SWRCs show in Figure 8. The lack of tests where volumetric measurements were obtained was not sufficient to fully understand the behaviour of the SWRCs that followed a wetting path. However, a general trend of the SWRCs was observed in Figure 8 where the SWRCs overlapped each other. However, it has to be remembered that samples compacted at lower water contents will have different soil fabrics. It seems this is more significant in affecting the wetting behaviour than the drying.
4 CONCLUSIONS

A study is presented of the influence of the initial water content on the water retention behaviour of a sandy clay soil. Using the filter paper technique, soil water retention curves (SWRCs) were obtained for samples with different initial water contents (10%, 13%, 15%, 20% and 22%). SWRCs following drying and wetting paths were obtained for the different initial water contents and compared with the primary drying curve for a sample prepared at a water content wet of optimum.

It was found that the drying curves tended to merge around 11% (equivalent to a suction of 1500-2000kPa) converging to the primary drying curve. However, the SWRCs that followed wetting paths showed atypical behaviour tending to intercept the primary drying curve at high water contents / low values of suction. This was also shown by the matric suction – degree of saturation relationship with the SWRCs intercepting the primary drying curve. This might lead to the view that the paths followed by the SWRCs were different to what might be expected. However, this behaviour can be explained by the difference in fabric of samples prepared dry of optimum water content.

5 ACKNOWLEDGEMENTS

The authors would like to acknowledge Dr. Sieffert and Dr. Liu for their assistance in translation of the abstract.

6 REFERENCES


ABSTRACT: Climate change effects on expansive soil movements are quantified using the Thornthwaite Moisture Index (TMI). The TMI is calculated from the moisture deficiency and surplus, both related to rainfall, and the potential evapotranspiration which is derived from temperature. The predicted temperature increase and rainfall reduction in 2030 and 2070 are used to derive the TMI of an area. In this way, values of TMI at present, in 2030 and 2070 are derived for Adelaide, Melbourne, Perth and Sydney. Established relationships between TMI and the depth and magnitude of soil suction changes for sites with and without the presence of trees, and the relationships between soil movement and soil suction changes, are used to predict the increase in soil movement for a site. A specific example is given for Melbourne for a site without trees and with a group of trees. It is shown that a significant increase in predicted soil movement is expected with climate change. Therefore a continuing revision of footing design standards would be required in order to cater for the effects of climate change.

KEYWORDS: climate change, expansive soil, Thornthwaite Moisture Index, tree effects

1 INTRODUCTION

There is considerable scientific evidence that emissions from economic activity are causing changes to the earth’s climate (Stern 2007). For example, in south-eastern and south-western Australia, current predictions indicate that generally, the 2030 and 2070 temperatures are expected to increase by about 1°C and 3°C respectively, and the 2030 and 2070 winter and spring rainfall is predicted to decrease significantly (CSIRO 2007).

Should these predictions prove accurate, these temperature increases and rainfall reductions are expected to cause increased changes in soil moisture content from those at present. As well, soil moisture changes due to trees become more adverse during periods of hotter and drier weather, when the tree demands more soil water than is available from rainfall. As expansive soils have the potential for undergoing significant movement with soil moisture changes, the effect of climate change on soil moisture changes needs to be predicted so that provisions can be made to effectively respond to the challenges of climate change when dealing with expansive soils.

Notwithstanding that the current predictions could prove to be inaccurate, present day geotechnical engineers are faced with the challenge of dealing with climate change in order to cater for the possibility that the current predictions suggest, may actually eventuate.

2 CLIMATE CHANGE PREDICTIONS

CSIRO (2007) has made predictions of climate change for Australia, and a summary of the predictions for changes in temperature and rainfall from present day averages for Adelaide, Melbourne, Sydney and Perth in 2030 and 2070 is shown in Table 1 for the emission scenarios as defined in CSIRO (2007).

Table 1: Climate Change Predictions from CSIRO (2007)

<table>
<thead>
<tr>
<th>Season</th>
<th>Adelaide</th>
<th>Melbourne</th>
<th>Sydney</th>
<th>Perth</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2030</td>
<td>2070</td>
<td>2030</td>
<td>2070</td>
</tr>
<tr>
<td>Summer</td>
<td>+0.9</td>
<td>+3.0</td>
<td>+1.0</td>
<td>+3.1</td>
</tr>
<tr>
<td></td>
<td>+1.0</td>
<td>+3.1</td>
<td>+0.9</td>
<td>+2.9</td>
</tr>
<tr>
<td>Autumn</td>
<td>+2.8</td>
<td>+0.8</td>
<td>+2.7</td>
<td>+0.9</td>
</tr>
<tr>
<td></td>
<td>+0.9</td>
<td>+2.9</td>
<td>+1.0</td>
<td>+0.8</td>
</tr>
<tr>
<td>Winter</td>
<td>+0.8</td>
<td>+2.4</td>
<td>+0.7</td>
<td>+2.2</td>
</tr>
<tr>
<td></td>
<td>+0.7</td>
<td>+2.6</td>
<td>+0.8</td>
<td>+0.7</td>
</tr>
<tr>
<td>Spring</td>
<td>+0.9</td>
<td>+3.0</td>
<td>+0.9</td>
<td>+2.9</td>
</tr>
<tr>
<td></td>
<td>+2.9</td>
<td>+3.3</td>
<td>+0.9</td>
<td>+2.9</td>
</tr>
</tbody>
</table>

Rainfall %

<table>
<thead>
<tr>
<th>Season</th>
<th>Adelaide</th>
<th>Melbourne</th>
<th>Sydney</th>
<th>Perth</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2030</td>
<td>2070</td>
<td>2030</td>
<td>2070</td>
</tr>
<tr>
<td>Summer</td>
<td>-0.5</td>
<td>-1.4</td>
<td>-0.5</td>
<td>-0.5</td>
</tr>
<tr>
<td></td>
<td>-1.4</td>
<td>-0.5</td>
<td>-2.5</td>
<td>-0.5</td>
</tr>
<tr>
<td>Autumn</td>
<td>-1</td>
<td>-4</td>
<td>-2</td>
<td>-3</td>
</tr>
<tr>
<td></td>
<td>-3</td>
<td>-0.9</td>
<td>-2.9</td>
<td>-1.0</td>
</tr>
<tr>
<td>Winter</td>
<td>-6</td>
<td>-19</td>
<td>-4</td>
<td>-12</td>
</tr>
<tr>
<td></td>
<td>-19</td>
<td>-4</td>
<td>-12</td>
<td>-5</td>
</tr>
<tr>
<td>Spring</td>
<td>-8</td>
<td>-23</td>
<td>-7</td>
<td>-21</td>
</tr>
<tr>
<td></td>
<td>-23</td>
<td>-7</td>
<td>-17</td>
<td>-9</td>
</tr>
</tbody>
</table>

Notes on Table 1: 2030 predictions are A1B emission scenario 50 percentile; 2070 predictions are A1F1 emission scenario 50 percentile

The predictions given in CSIRO (2007) are relative to 1990, however in this paper they are considered to be relative to long term average temperatures and rainfall at the present time. It can be seen from Table 1 that generally, the 2030 and 2070 temperatures are expected to increase by about 1°C and 3°C respectively. Table 1 also indicates that the 2030 and 2070 winter and spring rainfall is predicted to decrease significantly.
The magnitudes of the predicted changes in temperature and rainfall for 2030 and 2070 are expected to cause a decrease in soil moisture content from that experienced at present, thus leading to higher soil shrinkage on expansive soil sites.

In order to quantify the effect of climate change predictions on the magnitude of soil moisture changes, it is necessary to first establish the relationship between climate and seasonal rainfall and temperature. This is undertaken in this paper using the Thornthwaite Moisture Index.

3 THORNTHWAITE MOISTURE INDEX

Thornthwaite (1948) proposed an empirical method for estimating potential evapotranspiration as a climatic factor. Thornthwaite’s method enables potential evapotranspiration (PE) to be estimated using only monthly average temperature data (I) for a particular location, and a simply derived adjustment factor, which is applied to correct for latitude and month length by Equation (1)

\[ PE = 16 \left( \frac{10I}{a} \right) \times \text{Latitude correction factor} \]  

where  

\[ a = 0.000000675I^3 - 0.000771I^2 + 0.017921I + 0.49239 \]

In Equation (1), the heat index (I) is given by Equation (2)

\[ I = \sum_{i=1}^{12} i, \text{ where } i = \left( \frac{236.7}{25.1} \right)^{0.514} \]  

By enabling estimation of monthly potential evapotranspiration and balancing this with monthly rainfall, Thornthwaite (1948) made it possible to estimate seasonal moisture deficiencies and surpluses.

A moisture deficit is defined as the amount by which the net monthly potential evapotranspiration exceeds monthly rainfall during a period of zero soil moisture storage. Similarly, a moisture surplus is the amount by which the net monthly rainfall exceeds potential evapotranspiration when soil moisture storage is at capacity (defined as being 100 mm water); this surplus is assumed to drain as runoff. Soil moisture recharge occurs during months when the net moisture balance is positive and soil water storage is below capacity at the beginning of the month, while soil moisture depletion occurs when the net moisture balance is negative, but the soil water storage at the beginning of the month is non-zero. During the spring, soil moisture is rapidly utilized by vegetation, particularly annual grasses, which quickly mature, then die off once the soil moisture has been depleted.

The Thornthwaite Moisture Index (TMI) is calculated from the derived moisture deficiency and surplus by Equation (3).

\[ TMI = \frac{100(\Sigma \text{Surplus}) - 60(\Sigma \text{Deficit})}{\Sigma PE} \]

Determination of the TMI based on present day monthly temperature and rainfall averages for Perth is outlined in Table A1 of Appendix A. It is shown that for Perth at the present time, the TMI = +16.

The TMI is used to categorize the climate of a particular location. A higher positive value of TMI corresponds to a wetter climate, while a greater negative number is associated with a more arid climate. Therefore climate change predictions of higher temperatures and lower rainfall will result in a smaller TMI than at present for a particular locality.

4 PREDICTED CHANGES IN TMI

The author (Mitchell 2011,2012) used the seasonal temperature rise and general rainfall decline predictions (Table 1) for Adelaide, Melbourne, Sydney and Perth for 2030 and 2070 and applied them to the current climate averages to yield new average TMI estimates for each city in 2030 and 2070. An example for Perth for 2070 is given in Table A2 of Appendix A, and the results for all four cities are shown in Table 2.

<table>
<thead>
<tr>
<th>City</th>
<th>TMI</th>
<th>Present</th>
<th>2030</th>
<th>2070</th>
</tr>
</thead>
<tbody>
<tr>
<td>Adelaide</td>
<td>-19</td>
<td>-26</td>
<td>-34</td>
<td></td>
</tr>
<tr>
<td>Melbourne</td>
<td>-8</td>
<td>-14</td>
<td>-24</td>
<td></td>
</tr>
<tr>
<td>Perth</td>
<td>16</td>
<td>6</td>
<td>-14</td>
<td></td>
</tr>
<tr>
<td>Sydney</td>
<td>43</td>
<td>27</td>
<td>3</td>
<td></td>
</tr>
</tbody>
</table>

It is seen from Table 2, the predicted TMI for each city examined decreases with time, indicating increasing aridity.

From a geotechnical perspective, predictions of a lower TMI and hence an increase in the severity of desiccation of the soil profile could potentially lead to an increase in the magnitude of expansive soil movements for regions of Adelaide, Melbourne, Sydney and Perth characterised by highly expansive soils. This is quantified in the next section.

5 RELATIONSHIPS BETWEEN EXPANSIVE SOIL MOVEMENTS AND TMI

By the Australian standard AS2870-2011, the characteristic surface movement \( y_s \) of a site is determined from the design soil suction changes and the soil instability index \( I_m \) over the depth of design soil suction change \( h = H_s \) by Equation (4) and Figure 1 (the surface soil suction change is \( \Delta u_s = 1.2 \text{ pF} \) by AS2870 - 2011).

\[ y_s = \sum_{h=0}^{H_s} I_p \Delta u_s h \]  

Figure 1. Design soil suction extremes for a “normal” site by AS2870-2011

Equation (4) and Figure 1 are for “normal” sites such as sites that are not affected by trees. When considering tree effects, one recommended method of AS2870-2011, considers an additional soil suction as shown in Figure 2. This leads to an additional soil movement due to trees \( y_{t_{\text{max}}} \) by Equation (5).

\[ y_{t_{\text{max}}} = \sum_{h=0}^{H_s} I_p \Delta u_s h \Delta h \]  

The value of \( y_{t_{\text{max}}} \) is corrected for the distance of the tree from the footing of a building to give the actual tree effect \( y_t \).
Technical Committee 106 / Comité technique 106

Figure 2. Design soil suction extremes for a site with a group of trees by AS2870-2011

By AS2870-2011, the parameters \( H_s \), \( H_t \), and \( \Delta u_{base} \) increase with a decreasing Thornthwaite Moisture Index for the site as shown in Figure 3 for \( H_s \) and \( H_t \) (\( H_t \) is shown for a tree group) and in Figure 4 for \( \Delta u_{base} \) (shown for a tree group). As the magnitude and depth of soil suction changes increase with decreasing TMI associated with a drying climate, by Equations (4) and (5), expansive soil movements will increase.

Figure 3. Relationships between \( H_s \) and \( H_t \) and TMI by AS2870-2011

For the predicted future decrease in TMI of a location shown in Table 2, the increased depths of soil suction changes \( H_s \) and \( H_t \) and the increased soil suction change \( \Delta u_{base} \) can be determined from Figures 3 and 4 respectively, and the predicted increase in soil movement can then be calculated from the soil suction changes by Equations (4) and (5).

Figure 4. Relationship between \( \Delta u_{base} \) and TMI by AS2870-2011

6 EXAMPLE

The author (Mitchell 2011, 2012) has illustrated the implications of a predicted decrease in TMI by determining the soil movement at present, in 2030 and in 2070, for a site in Melbourne for a group of trees where \( y_{t_{max}} \) from Equation (5) is equal to \( y_{c} \) for a soil profile assumed for simplicity to be uniform clay of \( I_{pt} = 2.5\% \).

By Table 2, the TMI at present, and as predicted in 2030 and 2070, are respectively -8, -14, and -24. By Figure 3, the values of \( H_s \) corresponding to these TMI values are respectively 2.3 m, 3.0 m and 4.0 m, and the \( H_t \) values are respectively 3.6 m, 4.1 m and 4.5 m, and by Figure 4, \( \Delta u_{base} = 0.43, 0.46 \) and 0.55 respectively.

Table 3 summarises the results of the predicted soil movement by Equations (4) and (5) for the Melbourne site.

<table>
<thead>
<tr>
<th>TMI</th>
<th>( y_{s} ) (no trees)</th>
<th>( y_{s} + y_{t} ) (with trees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Present</td>
<td>35 mm</td>
<td>65 mm</td>
</tr>
<tr>
<td>2030</td>
<td>45 mm</td>
<td>75 mm</td>
</tr>
<tr>
<td>2070</td>
<td>60 mm</td>
<td>95 mm</td>
</tr>
</tbody>
</table>

It can be seen from Table 1 that a significant increase in predicted soil movement is expected with climate change. Therefore a continuing revision of footing design standards would be required in order to cater for the effects of climate change.

7 CONCLUSIONS

This paper has shown how climate change effects on expansive soil movements can be quantified using the Thornthwaite Moisture Index (TMI). This is because the TMI is calculated from the moisture deficiency and surplus, both related to rainfall, and the potential evapotranspiration which is derived from temperature, so that predicted temperature increases and rainfall reductions in 2030 and 2070 can used to derive the future TMI of an area. Values of TMI at present, in 2030 and 2070 were derived for Adelaide, Melbourne, Perth and Sydney. Established relationships between TMI and the depth and magnitude of soil suction changes for sites with and without the presence of trees, and the relationships between soil movement and soil suction changes, are used to predict the increase in soil movement for a site. A specific example is given for Melbourne for a site without trees and with a group of trees. It is shown that a significant increase in predicted soil movement is expected with climate change. This implies that a continuing revision of expansive soil movement predictions and footing design standards would be required in order to cater for the effects of climate change.

8 REFERENCES


### APPENDIX A

#### TABLE A1: DETERMINATION OF THE TMI FOR PERTH, AT PRESENT

<table>
<thead>
<tr>
<th>Month</th>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>Jun</th>
<th>Jul</th>
<th>Aug</th>
<th>Sep</th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rainfall</td>
<td>9</td>
<td>13</td>
<td>19</td>
<td>46</td>
<td>123</td>
<td>182</td>
<td>173</td>
<td>135</td>
<td>80</td>
<td>55</td>
<td>22</td>
<td>14</td>
<td>871</td>
</tr>
<tr>
<td>Ave. temp ((t))</td>
<td>24.3</td>
<td>24.7</td>
<td>22.8</td>
<td>19.9</td>
<td>16.7</td>
<td>14.0</td>
<td>13.3</td>
<td>13.4</td>
<td>14.9</td>
<td>17.0</td>
<td>20.3</td>
<td>22.7</td>
<td></td>
</tr>
<tr>
<td>Heat index ((i))</td>
<td>10.95</td>
<td>11.23</td>
<td>9.95</td>
<td>8.09</td>
<td>6.21</td>
<td>4.75</td>
<td>4.40</td>
<td>4.45</td>
<td>5.22</td>
<td>6.38</td>
<td>8.34</td>
<td>9.88</td>
<td>89.86</td>
</tr>
<tr>
<td>Latitude Corr. factor</td>
<td>1.21</td>
<td>1.03</td>
<td>1.06</td>
<td>0.95</td>
<td>0.91</td>
<td>0.84</td>
<td>0.89</td>
<td>0.95</td>
<td>1.00</td>
<td>1.12</td>
<td>1.15</td>
<td>1.23</td>
<td></td>
</tr>
<tr>
<td>Potential evap. ((PE))</td>
<td>137.4</td>
<td>120.8</td>
<td>106.2</td>
<td>72.8</td>
<td>49.4</td>
<td>32.2</td>
<td>30.8</td>
<td>33.4</td>
<td>43.3</td>
<td>62.9</td>
<td>91.6</td>
<td>122.1</td>
<td>903</td>
</tr>
<tr>
<td>Water balance</td>
<td>-128.4</td>
<td>-107.8</td>
<td>-87.2</td>
<td>-26.8</td>
<td>73.6</td>
<td>149.8</td>
<td>142.2</td>
<td>101.6</td>
<td>36.7</td>
<td>-7.9</td>
<td>-69.6</td>
<td>-108.1</td>
<td></td>
</tr>
<tr>
<td>Storage</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>73.6</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>92.1</td>
<td>22.4</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Deficit</td>
<td>128.4</td>
<td>107.8</td>
<td>87.2</td>
<td>26.8</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>85.7</td>
<td>435.9</td>
<td></td>
</tr>
<tr>
<td>Surplus</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>123.4</td>
<td>142.2</td>
<td>101.6</td>
<td>36.7</td>
<td>0</td>
<td>0</td>
<td>403.9</td>
<td></td>
</tr>
</tbody>
</table>

\(a = 1.970\)

**TMI = 16**

Notes on Table A1: Water balance = Rainfall – PE; Storage = previous month’s storage + water balance for month (= 0 if result is negative, and =100 if result ≥ 100 mm); Deficit = |water balance| – storage\(_{\text{previous month}}\) (if storage for month = 0); and Surplus = water balance + storage\(_{\text{previous month}}\) – 100 (if storage for month = 100).

#### TABLE A2: DETERMINATION OF THE TMI FOR PERTH, PREDICTED FOR 2070

<table>
<thead>
<tr>
<th>Month</th>
<th>Jan</th>
<th>Feb</th>
<th>Mar</th>
<th>Apr</th>
<th>May</th>
<th>Jun</th>
<th>Jul</th>
<th>Aug</th>
<th>Sep</th>
<th>Oct</th>
<th>Nov</th>
<th>Dec</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rainfall</td>
<td>7.9</td>
<td>11.4</td>
<td>16.7</td>
<td>40.5</td>
<td>108.2</td>
<td>142.0</td>
<td>134.9</td>
<td>105.3</td>
<td>58.4</td>
<td>40.2</td>
<td>16.1</td>
<td>12.3</td>
<td>694</td>
</tr>
<tr>
<td>Ave. temp ((t))</td>
<td>27.2</td>
<td>27.6</td>
<td>25.5</td>
<td>22.6</td>
<td>19.4</td>
<td>16.3</td>
<td>15.6</td>
<td>15.7</td>
<td>17.8</td>
<td>19.9</td>
<td>23.2</td>
<td>25.6</td>
<td></td>
</tr>
<tr>
<td>Heat index ((i))</td>
<td>12.99</td>
<td>13.28</td>
<td>11.78</td>
<td>9.82</td>
<td>7.79</td>
<td>5.98</td>
<td>5.60</td>
<td>5.65</td>
<td>6.84</td>
<td>8.10</td>
<td>10.21</td>
<td>11.85</td>
<td>109.9</td>
</tr>
<tr>
<td>Latitude Corr. factor</td>
<td>1.21</td>
<td>1.03</td>
<td>1.06</td>
<td>0.95</td>
<td>0.91</td>
<td>0.84</td>
<td>0.89</td>
<td>0.95</td>
<td>1.00</td>
<td>1.12</td>
<td>1.15</td>
<td>1.23</td>
<td></td>
</tr>
<tr>
<td>Potential evap. ((PE))</td>
<td>174.6</td>
<td>154.0</td>
<td>130.8</td>
<td>87.4</td>
<td>57.8</td>
<td>35.0</td>
<td>33.3</td>
<td>36.1</td>
<td>51.6</td>
<td>75.7</td>
<td>112.8</td>
<td>153.2</td>
<td>1102</td>
</tr>
<tr>
<td>Water balance</td>
<td>-166.7</td>
<td>-142.5</td>
<td>-114.0</td>
<td>-47.0</td>
<td>50.4</td>
<td>107.0</td>
<td>101.6</td>
<td>69.2</td>
<td>6.8</td>
<td>-35.5</td>
<td>-96.7</td>
<td>-140.9</td>
<td></td>
</tr>
<tr>
<td>Storage</td>
<td>0</td>
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<td>0</td>
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\(a = 2.427\)

**TMI = -14**

Notes on Table A2: From Table 1 for Perth in 2070, predicted monthly rainfall decrease below present day averages is 12% for summer (i.e. December to February), 12% for autumn (i.e. March to May), 22% for winter (i.e. June to August) and 27% for spring (i.e. September to November). Also from Table 1, the predicted average monthly temperature increase over present day averages is 2.9°C for summer, 2.7°C for autumn, 2.3°C for winter, and 2.9°C for spring.
Study on mechanism of two-phase flow in porous media using X-ray CT Image Analysis

Etude sur le mécanisme de transfert biphasé dans les milieux poreux par l'imagerie aux rayons X

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ABSTRACT: The objective of this study is to understand the mechanism of light non-aqueous phase liquid (LNAPL) migration in sandy soil. This paper introduces a newly developed test apparatus for measuring flow injection of micro-focused X-ray computed tomography (MXCT) scanners, and an image analysis technique. The pore scale and its distribution in the specimen are then evaluated. Only those pore structures trapping LNAPL are extracted, and cluster analysis is applied to each. This image processing technique allows quantitative evaluation of pore scale and pore structures trapping LNAPL.

RÉSUMÉ: L'objectif de cette étude est de comprendre le mécanisme de migration du LNAPL dans un sol sablonneux. Dans cet article, la technique récemment mise au point d'analyse de scanner et d'images par appareil de test d'injection de flux micro-focalisé de tomographie par rayons-X assisté par ordinateur est décrite. L'objectif est d'évaluer l'échelle de porosité et sa répartition dans l'échantillon. Seules les structures poreuses formant les LNAPL sont extraites et une ‘analyse de cluster est appliquée à chaque structure poreuse. Cette technique de traitement par image a permis une évaluation quantitative de l'échelle et de la structure poreuse du LNAPL.

KEYWORDS: Multiphase flow, ground contamination, Image Analysis, X-ray CT

1 INTRODUCTION

Multiphase flow is an issue in the study of ground contamination by light non-aqueous phase liquids (LNAPLs), where quantitative parameters regarding particle shape and pore structure are often desired. The basic mechanics of LNAPL migration were developed along with the theory of unsaturated flow mechanics. In recent studies, van Genuchten and Mualem models have been used to deduce the relative permeability coefficient of LNAPLs in porous media (Mualem, 1976; van Genuchten, 1980), allowing visualization of the degree of saturation distribution for LNAPLs by numeric simulation (e.g., EPA, 1997). However, LNAPLs flow though pore structures, potentially polluting groundwater due to residual LNAPLs in pores. Characteristics of the pore structure of sand, such as pore size, shape, spatial distribution, and connectivity, are therefore important, and should be evaluated in three dimensions to understand the mechanism of LNAPL capture. Recent technologies such as X-ray computed tomography (CT) scanners have made possible nondestructive evaluation of pore diameter and distribution in the study of fluid behavior in sandy soil (Wildenschild et al., 2002; Altman, 2005; Wildenschild et al., 2005; Al-Kharusi & Blunt, 2007; Mukunoki et al., 2010).

New techniques for image processing and analysis of X-ray CT images are thus now required (Mukunoki et al., 2011). The objective of this study is to understand the mechanism of LNAPL migration in sandy soil. This paper introduces a newly developed test apparatus for injecting fluid for micro-focused X-ray computed tomography (MXCT) scanners, and an image analyzing technique proposed by the authors for evaluating pore scale and distribution. Pore structures trapping LNAPLs are then visualized by cluster analysis, allowing static evaluation of pore scale and structure. Specifications of the micro-focused X-ray CT scanner used in this study are in Mukunoki et al., (2011).

1 EXPERIMENTAL METHODS

Figure 1 shows the developed fluid injection test apparatus. An aluminum mold holds the sand sample to be scanned. The inner diameter of this mold is only 10 mm, making gravity effects negligible. Fluids are injected using two syringe pumps with a minimum constant flow velocity of 0.1 cm³/h. The developed experiment consists of injecting fluids into a 14.61 cm³ sample. Mass outflow is measured using a precision balance, and the injection rate is controlled through syringes and a syringe pump that allow slow rates. Table 1 shows specifications of the fluids tested. To allow discrimination of LNAPL and water phases using micro CT, iodine potassium was added to the water. This KI water has a density of 1.25 g/cm³, but its kinematic viscosity is almost the same as that of pure water. The LNAPL used is
2 IMAGE PROCESSING ANALYSIS

2.1 Marker-controlled watershed processing

X-ray CT images are composed of voxels, called CT-values, which are proportional to the material density. To separate pore space and soil particles in the image, it is necessary to define a threshold value for the CT values. Figure 3 shows an X-ray CT image of Toyoura sand that includes KI solution and LNAPL. That image also includes magnetic sand, so four kinds of CT-value exist in the CT image of Figure 3(a): soil particles, solid-phase magnetic sand particles, KI solution, and liquid-phase LNAPL. Knowing the number of materials in the CT images allows fixing areas of magnetic sand particles, soil particles, LNAPL, and pore space, as well as identifying mixels where two or more materials occupied a single voxel. Watershed processing, which is based on CT-value gradient changes between materials, was applied only to mixel areas in what is called the marker-controlled watershed (MCW) method. Figures 3(b)–(d) show the MCW process. Figure 3(b) is the CT-value profile obtained from lines in the CT image in Figure 3(a). Clarification of the materials in the CT image allows defining the CT-value range for each material without knowing CT-values for the mixel. To evaluate the gradient in the mixel areas, Figure 3(c) shows a profile of the gradient obtained from Figure 3(b). Finally, the coordinate in the CT image with maximum gradient was defined between different materials, and watershed processing was applied to the CT image. This CT-value thresholding technique does not lose spatial geometry information, and is an objective approach for distinguishing the CT threshold values.

2.2 Image analysis technique for connectivity of pore structure based on mathematical morphology

Mukunoki et al. (2011) proposed an image processing technique that uses mathematical morphology (Soille, 2002) to evaluate the 3-D distribution of pore scale in dry, sandy soil (a two-phase soil particle and air mixture). We used this method to evaluate the volume and diameter of pore water, and showed that LNAPL existed in pore structures. Moreover, connectivity of the pore structure affects flow behavior, and so should be considered in this study. If continuous structures could be isolated as part of a pore structure, a cluster labeling method would give the number of isolated LNAPL blobs and their connectivity. The authors applied the evaluation method of the 3D distribution of pore scale, and used this method to find a circle with minimum diameter as shown in Figure 4. This method successfully separated entire pore structures, making it possible to evaluate spatial distribution of the LNAPL volume.

3 RESULTS AND DISCUSSION

3.1 LNAPL injection to the sand with saturation of 100%

Intrinsic permeability \( k \) of the tested sand was \( 1.75 \times 10^{-11} \text{ m}^2 \) for Case 1 and \( 1.57 \times 10^{-11} \text{ m}^2 \) for Case 2, as obtained from the injection test. In this study, the injection speed was a parameter for this test. Figure 5 shows the difference between inlet and outlet pressures, and the mass of fluid collected at the outlet for Case 1. The injection pore volume (PV) at the break-through
point was 0.65 for Case 1 and 0.67 for Case 2. The saturation degree of LNAPL was measured from the injection volume when we observed the LNAPL in the plate at the outlet position. Values were 80.1% for Case 1 and 83.1% for Case 2. These similar values indicate that the prepared samples both had similar pore structures at the initial condition.

3.2 KI solution injection into sand polluted by LNAPL

Figure 6 shows the volume profile of LNAPL recovered by KI solution injection. As shown in Figure 6, the injection pore volume at the breakthrough point was 0.51 for Case 1 and 0.56 for Case 2, indicating that the entry pressure for Case 1 was observed earlier than Case 2. To evaluate this phenomenon, the capillary number may help in the following equation (1):

$$ Ca = \frac{v_w d_{al}}{\gamma_w} $$

where $v_w$ is Darcy’s velocity, $\nu$ is kinematic viscosity, and $\gamma$ is the interfacial tension. Subscript w means “wetting.” The capillary number $Ca$ indicates the ratio of viscosity and interfacial tension, and when $Ca$ is less than $10^{-6}$ multiphase flow should be dominated by capillary pressure (Mayer & Miller, 1993). In other words, a low $Ca$ number may cause fingering flow in porous media. In this study, $Ca$ for Case 1 was $1.57 \times 10^{-6}$ and $Ca$ for Case 2 was $3.14 \times 10^{-7}$. LNAPL saturation degrees after the KI solution injection test in Case 1 and Case 2 were 21.5% and 9.5%, respectively, confirming that $Ca$ greater than $10^{-6}$ caused less volume of residual LNAPL.

3.3 Evaluation of pore structure using the MCW method

Figure 7 shows MXCT images of sand after injecting LNAPL (Figure 7(a)) and KI solution (Figure 7(b)) in Case 1. X-ray CT images are generally 256-level grayscale, with black areas having the least density and white areas the greatest. Figure 7(a) shows that LNAPL was trapped, and its shape was different. Also, the black shape of LNAPL in Figure 7(b) was more circular than as seen in Fig 7(a). These LNAPL blobs are residual LNAPL trapped due to KI solution injecting. Multi-threshold processing with the MCW method was applied to a CT image with $512 \times 512 \times 512$ cubic voxel area. The 4-color MXCT image shown in Figure 8(b) was created from Figure 8(a).

Based on 4-color CT images, only pore structure and residual LNAPL could be extracted from the CT images in Figure 8(b). The porosity obtained from 3D CT images was 40.7% for Case 1 and 38.5% for Case 2. In fact, the porosity calculated from the mean dry density of the sand sample was 39.4%, and the difference should be negligible (<1% error), confirming that multi-threshold processing with MCW gave reasonable values for porosity. Residual LNAPL after injecting the KI solution was 16.7% for Case 1 and 12.6% for Case 2. Measured saturation of residual LNAPL from the mass in the outlet plate was 4.8% less than the obtained value from MCW for Case 1, and 3.1% greater than that for Case 2.

Figures 9(a) and 9(b) show 3D CT images of pore structure and LNAPL blobs isolated with MCW processing. Pore structure and LNAPL blobs were thus successfully extracted from soil samples in three dimensions without disturbance.

3.4 Cluster analysis of pore structure

Figures 10 shows histograms of pore size in Case 1 and Case 2. Frequency in vertical axis means the normalized value by total voxel number of pore structure. Maximum pore size in Case 2 was slightly greater than in Case 1, as shown in Figure 10. There were 1317 pore elements with LNAPL in Case 1 and 1023 in Case 2. Table 3 summarizes saturation degrees of LNAPL for Case1 and Case 2 before and after KI solution injecting obtained from X-ray CT images treating MCW processing. Initially, the sand sample for Case2 had only 3% greater saturation degree of LNAPL than Case 1. However, residual saturation degree of LNAPL for Case 2 was less than...
half for Case 1. Figure 11 shows 3D MXCT images of pore structure isolated by mathematical morphology and cluster analysis for Case 1 and Case 2. Cluster analysis of continuous pore structure such as in soil materials allows defining pore elements, allowing for their statistical analysis. In Figure 11, the red cluster with greatest volume has a more complex shape than the blue cluster with small volume, which is shaped like balls. There were 99 LNAPL clusters isolated in Case 1 and 156 in Case 2, implying that fingering flow with capillary force caused by LNAPL remained in wide area in the sample for Case 1. On the other hand, Case 2 with the capillary number (Ca) twenty times greater than Case 1 decreased the capillary pressure more than Case 1 as Mayer & Miller, (1993) discussed; and hence, it can be concluded that the fingering flow with capillary force was restrained.

3.5 Factor of LNAPL trapping

If LNAPL migrated in a single direction, water should be almost completely flushed out. However, pore structure was distributed in three dimensions, so pore structure connectivity and a three-dimensional bottleneck effect should be considered to evaluate residual LNAPL in the soil. In the case of sandy soil, small changes in capillary pressure in excess of the entry pressure significantly increased or decreased saturation. Factors for trapping LNAPL are thus related to the connectivity of pore structures and drastic changes in capillary pressure with bottleneck effect. Besides, Ca would be related to the trapping distribution in the sand as shown in Fig. 11 (a) and (b).

4 CONCLUSIONS

Sandy soil specimens were scanned by micro-focused X-ray CT scanners, and spatial distribution of pore structures with LNAPL were visualized and quantitatively evaluated by a newly developed image processing technique using marker-controlled watershed, a mathematical morphological method, and cluster analysis. Key conclusions are as follows:
1) The developed thresholding technique worked well, and allowed objective definition of locations with changing greatest CT values without histogram analysis.
2) Factors for trapping LNAPL are related to the connectivity of pore structures and drastic changes in capillary pressure with bottleneck effect.
3) The capillary number (Ca) more than $10^{-5}$ reduced the capillary effect and residual LNAPL would be distributed in local position. Meanwhile, the less Ca had dominant flow with capillary effect so LNAPL was trapped in wide area of sample.

5 ACKNOWLEDGEMENTS

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6 REFERENCES

EPA/600/R-97/102 Environmental Protection Research Laboratory
Soil suction induced by grass and tree in an atmospheric-controlled plant room

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ABSTRACT: Vegetation is commonly found on soil slopes and landfill covers worldwide. Although mechanical contribution of roots has been considered in some stability calculations, any effects of suction induced by root-water uptake on the reduction of water permeability and the increase of shear strength are generally ignored. Due to a lack of experimental study, relationships among induced soil suction, atmospheric conditions (such as radiant energy) and plant characteristics (such as leaf area index (LAI) and root area index (RAI)) are not well understood. In order to improve our fundamental understanding of soil-water-plant-atmospheric interaction, this study measures and compare induced suction and its distribution by grass (*Cynadon dactylon*) and tree (*Schefflera heptaphylla*) planted in completely decomposed granite (silty sand) in an atmospheric-controlled plant room. By monitoring the responses of induced suction in each vegetated soil for two weeks, the effects of the two different plants on suction distributions are highlighted and discussed. Observed suction induced is interpreted in conjunction with measured interception of radiant energy by plant leaves through LAI and also measured plant root characteristics through RAI.

RÉSUMÉ : La végétation est généralement présente sur les pentes du sol et les couvertures d'enfouissement partout dans le monde. Bien que la contribution mécanique des racines ait été prise en compte dans certains calculs de stabilité, les effets de la succion induite par la racine sur la réduction de la perméabilité à l'eau et sur l'augmentation de la résistance au cisaillage soient généralement ignorés. En raison d'un manque d'études expérimentales, les relations entre la succion du sol induite, les conditions atmosphériques (tels que l'énergie rayonnante) et les caractéristiques de la plante (comme l'indice de surface foliaire (ISF) et l'indice de surface de racine (ISR)) ne sont pas bien compris. Afin d'améliorer notre compréhension fondamentale de l'interaction sol-eau-plante-atmosphère, cette étude mesure et compare la succion du sol et sa distribution par l'herbe (*Cynadon dactylon*) et l'arbre (Schefflera heptaphylla) plantés dans un sol en granite complètement décomposé (sable limoneux) dans une chambre atmosphérique contrôlée. En surveillant les réponses de la succion induite dans chaque sol pendant deux semaines, les effets des deux plantes différentes sur les distributions de succion du sol sont mis en évidence et discutés. La succion du sol observée est interprétée avec l'interception de l'énergie rayonnante mesurée par ISF et aussi les caractéristiques des racines mesurées par ISR.

KEYWORDS: vegetation, suction, unsaturated soil, leaf area index, root area index.

1 INTRODUCTION

Many soil slopes and landfill covers worldwide are covered with vegetation. While there are engineering needs to search for sustainable and environmentally friendly slope stabilisation methods, shallow slopes in particular, vegetation has been considered to be an alternative, through mechanical and hydrological processes (Greenwood et al. 2004). Mechanical root reinforcement has been researched to be beneficial to slope stability (Greenwood et al. 2004). On the contrary, research on the effects of suction due to root-water uptake on the reduction of water permeability and the increase in shear strength is relatively scarce. Due to the lack of rigorous and systematic research, effects due to plant-induced soil suction are generally ignored when analyzing transient seepage and stability of a vegetated soil slope.

As far as plant-induced suction is concerned, some studies reported in literature mainly focused on responses of soil moisture/suction due to the presence of grass species (Ng et al. 2012). Investigation on the effects of other plant species such as tree on root-water uptake and induced changes of soil moisture/suction are rather limited.

It has been well-recognized that the amount of transpiration and root-water uptake of any plant species strongly depend on characteristics of plant leaf and root and atmospheric parameters such as radiant energy. Correlations between intercepted radiant energy and leaf area index (LAI) have been commonly used to interpret plant-induced changes of soil moisture (Monsi and Saeki 1953). LAI is a dimensionless index defining the ratio of total one-sided green leaf area to projected area of a plant on soil surface. For a plant having a higher LAI, a larger leaf area is available to intercept radiant energy for transpiration. While root-water uptake always takes place below ground, this common approach may be less direct because LAI is an index reflecting characteristics of plant leaf above ground. Further investigation on relationship between induced suction and other more relevant plant characteristics such as root area is needed.

To improve our understanding on the interaction among soil, water, plant and atmosphere, a test program was conducted to measure the magnitude and distribution of induced suction in vegetated soil. Two plant species, namely Schefflera heptaphylla (tree) and Cynadon dactylon (grass), which are commonly found in Hong Kong and other parts of Asia including Vietnam, India and Malaysia, were selected for investigation and comparison. Three test boxes with and without vegetation were purpose-built and they were monitored for two weeks under identical, controlled and constant atmospheric conditions.
2 EXPERIMENTAL SET UP AND TEST PROCEDURES

2.1 Test boxes and instrumentation

In this study, three test boxes were designed and manufactured. Two of them were planted with grass (denoted as test box G) and with tree (denoted as test box T), whereas one was left bare as a control (denoted as test box B). Figure 1 shows the overview of the setup of the three boxes in a room, where air temperature, radiant energy, relative humidity and potential evaporation rate were controlled at 22.3±1°C, 2.1±1 MJ/m², 43.7 % and 5±0.2 mm/day, respectively. Each test box has a cross-section dimension of 300 mm x 300 mm and a depth of 350 mm. At depths of 30, 80, 140 and 210 mm below soil surface in each box, miniature-tip tensiometers were installed to measure negative pore-water pressure or suction ranging from 0 to 90 kPa. In order to quantify soil suction induced by tree transpiration in box T, bare soil surface around the tree stem was covered with a plastic sheet to minimize soil evaporation. Similarly, soil surface in bare box B was also covered for fair comparison. In box G, since soil surface was fully vegetated with grass, it was thus not covered with the plastic sheet and exposed to atmosphere during testing. At the bottom of each test box, there were nine drainage holes with diameter of 5 mm each for free drainage during testing (not shown in the figure).

![Figure 1. Overview of the three-purpose-built boxes B, G and T in an atmosphere-controlled room](image)

To allow for photosynthesis, a fluorescent lamp was used and it was mounted on top of each vegetated test box. The lamp emitted a constant daily radiant energy (R) of 2.1 MJ/m². In total, 16 measurements of R, were made along soil surface of boxes B and T using quantum sensors. In box T, any radiant energy difference between the applied and measured radiant energy is equal to the energy intercepted by tree leaves. It should be noted that this calculation neglects (1) reflected radiant energy at each individual leaf surface due to low albedo (0.10 – 0.15; Taha et al. 1988) and (2) radiant energy used to heat up air due to low air density (Blight 2004). Energy distribution could not be measured in box G because soil surface was fully covered with grass where quantum sensor (which has limited size) was difficult to be placed on soil surface for measurements.

2.2 Soil type and selected plant species

Completely decomposed granite (CDG), which is commonly found in Hong Kong, was used. Results from sieve and hydrometer analysis reveal that the gravel, sand, silt and clay contents of CDG are 19, 42, 27 and 12 %, respectively. Based on the measured particle-size distribution and Atterberg limit, CDG may be classified as silty sand (SM) according to the Unified Soil Classification System. Each test box was compacted with silty sand of which the targeted dry density and water content by mass were 1496 kg/m³ (i.e. degree of compaction of 80 %) and 12%, respectively.

In this study, a grass species (Cynodon dactylon) and tree species (Schefflera heptaphylla) were selected for investigation. The grass species is commonly known as Bermuda grass, which is a warm-season grass widely cultivated in warm climates of the world. In box B, seeds of Bermudagrass were distributed uniformly on soil surface and they were allowed to germinate and grow for 10 months in the atmosphere-controlled room. After growing for 10 months, the average lengths of grass shoot and depth were found to be 90 and 110 mm, respectively. The LAI of grass is estimated to be 2.2.

For box T, a mature tree, Schefflera heptaphylla, which has a shoot height of 900 mm and root depth of 240 mm (50 % longer than grass root), was transplanted to the centre of the box. The tree had a canopy diameter of about 220 mm (73% of the width of the box T) and the shape of the canopy is spindle – shaped. The LAI of the tree is determined to be 4.6. In both boxes G and T, fertiliser was not added to prevent osmotic suction induced by changes of salt concentration in soil (Krahn and Fredlund 1972).

2.3 Test plan and procedures

After preparing all the three test boxes (B, G and T), they were tested in the atmospheric-controlled room. In each box, rainfall with intensity of 100 mm/hour and duration of one hour was applied on box surface using a calibrated rainfall sprinkler system as shown in Figure 1. This applied rainfall event is equivalent to the return period of 10 years of rainfall in Hong Kong (Lam and Leung 1995). Throughout the entire rainfall event, all drainage holes at the bottom of each test box were opened to allow for free downward drainage. After rainfall, soil surfaces in test boxes B and T were covered with laminated plastic sheet, whereas that for grass box G was left exposed. Each test box was then monitored for two weeks and any suction changes at the depths of 30, 80, 140 and 210 mm were recorded continuously. All drainage holes at the bottom of each box remained open during the monitoring period.

3 OBSERVED TREE ROOT CHARACTERISTICS

In order to investigate tree root characteristics such as root area index (RAI) and its distribution within the root zone, the tree was removed from box T after testing. An image analysis was then conducted on tree root system using an open source program, Image J (Rasband 2011). RAI is an index normalising total root surface area for a given depth range \( \Delta h \) (assumed to be 10 mm in this study) by circular cross-section area of soil (on plan), whose diameter is defined as the furthest distance between two ends of root. It should be noted that RAI of grass was not measured because the diameter of fine roots was much smaller than the accuracy of the image analysis.

![Figure 2. The measured distribution of RAI along root depth of the tree](image)

Figure 2 shows the measured distribution of RAI along root depth of the tree. Maximum RAI of 0.74 is found near soil surface. The RAI decreases almost linearly to less than 0.03 at the root depth of 240 mm. Obviously, RAI can vary differently from species to species. While the observed linear RAI profile of the tree in this study is found to be similar to that measured in sweet gum (Simon and Collison 2002), it is different from other tree species, black willow, where non-linear RAI profile was observed (Simon and Collison 2002). In addition to plant species, RAI can also be affected by soil density. Laboratory study carried out by Grzesiak (2009) showed that when soil is denser, plant roots are found to be less uniform along depth. Obviously, this is because an increase in soil density would
increase mechanical resistance for plant roots to penetrate through soil. As a result, more roots would concentrate (i.e., higher RAI) in shallower depths and the number of roots would decrease with an increase in depth. The observed RAI profile in this study is useful for interpreting suction induced in vegetated soil with the tree. This is because RAI indicates surface area of roots available for root-water uptake, which would affect the magnitude of suction induced in the soil. Any relationship between RAI and suctions induced in box T is discussed later.

4 EFFECT OF TREE LEAF ON ENERGY DISTRIBUTION

Figure 3 compares the measured horizontal distributions of radiant energy received on soil surface ($R_s$) for boxes B and T. As expected, the soil surface for bare box B receive almost all the energy (i.e., $R_s = R_i$) uniformly along the width of the box. On the contrary, the measured distribution of $R_s$ is different in box T. It can be seen that at the centre of the box, the measured $R_s$ is minimum. This is because of the substantial interception of incoming radiant energy by tree leaves. The maximum percentage of energy interception (i.e., $(R_i - R_s) / R_i$) is estimated to be about 50%. Moreover, measured values of $R_s$ are found to increase when the distance is further away from the tree stem on both sides of the box. This is because there are smaller number of tree leaves, and hence amount of interception, away from the tree stem (see Figure 1). At the two edges of the box, the measured values of $R_s$ are found to be close to the applied radiant energy of 2.1 MJ/m² (see Figure 3). This is expected because energy measurements were made outside the canopy of the tree and thus no radiant energy can be intercepted. Similar distribution of intercepted radiant energy was also found by Buler and Mika (2009) for an apple tree of which the canopy was also spindle-shaped. It is obvious that energy distribution is strongly dependent upon the canopy shape of a tree species. For a given apple tree, Buler and Mika (2009) showed that distributions of intercepted energy depended on the shape of canopy (i.e., V – shape and Hybrid – shape). Measured interception of radiant energy in this study is used to interpret plant-induced soil suction later since the magnitude of energy intercepted by tree leaves would affect amount of tree transpiration and hence root-water uptake.

Although the amount of radiant energy interception was not determined for box G due to the full coverage of grass surface, it could be estimated based on LAI of grass (i.e., 2.2) using Beer-Lambart Law, which has been widely used for estimating intercepted radiant energy for various plant species including grass (Kiniry et al. 2007). The law states that amount of light intercepted through grass leaf increases with an increase in leaf area exponentially (Monsi and Saeki 1953). For a given LAI and assuming that the extinction coefficient (i.e., a measure of the absorption of light) to be 1.1 based on the thickness of grass shoot (Kiniry et al. 2007), the percentage of radiant energy interception by grass is estimated to be more than 90%. In other words, a large portion of radiant energy would be intercepted by grass shoot for transpiration, whereas only a limited amount of it would fall on bare soil surface (i.e., less than 10%) for evaporation. This implies that for a given incoming radiant energy in the atmospheric-controlled room, suction induced in box G would be attributed to grass transpiration mainly.

5 COMPARISON OF SUCTION PROFILES WITH AND WITHOUT VEGETATION

Figure 4 compares measured distributions of induced suction along depth in all three test boxes, B, G and T during two weeks of testing. The measured root depths of grass and tree are depicted for reference. The measured initial distribution of suction in bare box B is found to be nearly uniform, suggesting that the hydraulic gradient is about one. In other words, water flow in these two boxes was mainly driven by the gravity, seeping towards bottom drainage holes. When grass was present, suctions recorded at depths within the root zone in box G are slightly higher than those in deeper depths because of root-water uptake. For box T, the measured suction was higher than those in the other two boxes, particularly at 30 and 140 mm depths. As compared to bare box B, the observed higher suctions retained in the two vegetated boxes G and T were likely attributed to the changes of water retention ability of soils, due to the presence of plant roots (Scanlan and Hinz 2010).
After two weeks of testing, suctions recorded at all depths in bare box B increase by nearly the same amount of 6 kPa, suggesting that there was again unit gradient downward flow. The downward water flow during the monitoring period in box B was expected because surface evaporation was greatly minimised by covering the soil surface using the plastic sheet (see Figure 1). On the contrary, uniform suction distributions are not observed in both vegetated boxes G and T. It can be seen that there are significant increases in soil suctions at all depths in these two boxes, particularly within plant root zones in shallower depths. This is not surprising because hydraulic gradient near soil-atmosphere interface is much higher than that in deeper depths. Measured suctions at all depths in box G are higher than those in Box B. When grass is present, the peak induced suction due to grass root-water uptake (i.e., 50 kPa) is found to be six times higher than that in bare box B (i.e., 8 kPa) at 30 mm depth. Although measurements at 140 and 210 mm depths were made outside the root zone of grass, suctions at these depths are also influenced by the root-water uptake happened in shallow depths. As identified by Ng et al. (2012), vertical influence zone of induced suction in soil vegetated with grass could be as deep as four times of grass root depth.

For box T, suctions recorded at all depths are higher than those in the bare box B. When the tree is present, the peak suction induced at 30 mm depth (i.e., 67 kPa) is found to be almost eight times higher than that in bare box B (i.e., 8 kPa). This is attributed to tree transpiration by intercepting 50% of incoming radiant energy (see Figure 3) and also soil evaporation by receiving remaining energy fallen on soil surface. It is important to recognize that the observed distribution of suction in box T is consistent with that of RAI shown in Figure 2. The amount of suction induced is found to increase with an increase in RAI proportionally. For a tree having a higher RAI, this means that a larger surface area of tree roots is available for root-water uptake to induce higher suctions. When compared to box G, it can be seen that suctions recorded at all depths in box T are about 34% higher (see Figure 4). This is likely because the root diameter and root depth of the tree are larger and deeper than those of grass, respectively. As compared to short, fine grass roots, the characteristics of tree roots are more favorable for root-water uptake. At deeper depths of 140 and 210 mm, the measured suctions in box T are expected to be higher than those in box G. This is because suction measurements at these two depths were made within the tree root zone in box T, but those in box G were outside the root zone of grass.

6 SUMMARY AND CONCLUSIONS

In this study, a series of laboratory tests were carried out to explore the effects of two different plant types (grass and tree) on induced suction and its distribution in compacted silty sand in an atmospheric controlled plant room. Measured induced suction in vegetated soils were compared and correlated with energy distribution and plant characteristics.

After two weeks of testing, peak suctions (i.e., 50 kPa and 67 kPa) induced in silty sand vegetated with grass and tree are found to be at least six and eight times higher than that (i.e., 8 kPa) in bare soil, respectively. The additional suction induced in vegetated soils is attributed to plant transpiration through energy interception by plant leaves. For the tree having a LAI of 4.6, the maximum amount of energy interception by tree leaves is up to 50% of the incoming energy supplied in the atmospheric-controlled room. It is important to recognize that the measured vertical distribution of suction in vegetated soil depends on RAI. The magnitude of suction induced is found to be directly proportional to RAI. When RAI is higher, higher surface area of tree roots is available for root-water uptake to take place and higher suction is hence induced.

When comparing soil vegetated with grass and tree, peak suction induced by tree is 34% higher. This is because root diameter and root depth of the tree are larger and deeper than those of grass (which has finer and shorter roots), respectively. As expected, these characteristics of tree roots are more favorable than those of grass for root-water uptake.

7 ACKNOWLEDGEMENTS

The authors would like to acknowledge research grant (2012CB719805) from the National Basic Research Program (973 Program) (No. 2012CB719800) provided by the Ministry of Science and Technology of the People's Republic of China, research grant (HKUSTV/CRF/09) provided by the Research Grants Council (RGC) of the Hong Kong Special Administrative Region, and also financial support from Hong Kong PhD Fellowship Scheme (HKPFS) for the third author of this paper. Experimental work carried out by final-year project students, Messrs Kow Hong Yee, Ho Chu Pan and Wang Zi Jian are acknowledged.

8 REFERENCES


Application of micro-porous membrane technology for measurement of soil-water characteristic curve

Application de la technologie de membrane microporeuse pour la détermination de la courbe de rétention d’eau des sols

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ABSTRACT: This study focuses on the use of micro-porous membranes instead of ceramic disks for improving the time required to reach matric suction equalization. Measurements of the soil-water characteristic curve were conducted using micro-porous membranes in a new pressure plate apparatus. In the testing program, the hydraulic response of the micro-porous membrane under varying water contents corresponding to increasing or decreasing matric suction was measured. Different micro-porous membrane with different air-entry values were investigated in order to compare the time required for matric suction equilibrium with that of the ceramic disk. Soil-water characteristic curves of different types of soil were measured using the axis-translation technique with both the micro-porous membrane and the ceramic disk.

RÉSUMÉ : Cette étude concerne l’utilisation des membranes microporeuses à la place des disques céramiques pour réduire le temps nécessaire pour atteindre l’équilibre en succion matricielle dans les sols. La détermination de la courbe de rétention d’eau a été faite en utilisant des membranes microporeuses dans un nouvel appareil de plaque de pression. Dans le programme d’essais, la réponse hydraulique de la membrane microporeuse sous des teneurs en eau variables correspondant à des augmentations ou diminutions de succion matricielle a été mesurée. Des membranes microporeuses différentes avec des points d’entrée d’air différents ont été étudiées afin de comparer le temps nécessaire pour atteindre l’équilibre en succion matricielle avec le temps dans le cas du disque céramique. Les courbes de rétention d’eau de différents sols ont été déterminées en appliquant la technique de la translation d’axe avec la membrane microporeuse et le disque céramique.

KEYWORDS: unsaturated soils, soil-water characteristic curve, matric suction, pressure membrane technique

1 INTRODUCTION

Unsaturated soil mechanics is becoming more widely accepted in geotechnical engineering and engineering protocols are emerging for a range of geotechnical problems. Matric suction plays an extremely important role in describing unsaturated soil property functions as well as the verification of the unsaturated soils mechanics theories (Gasmo et al. 1999). The axis-translation technique developed by Richards (1941) and the pressure plate technique suggested by Hilf (1956) have contributed significantly towards the measurement and control of matric suction in unsaturated soils laboratory tests. Bishop and Donald (1961) and Bishop and Henkel (1962) developed the triaxial apparatus for unsaturated soils, and used the pressure plate technique in order to separate the pore-air pressure and the pore-water pressure. The pressure plate technique make use of high air entry disks which allow the movement of water but resist the movement of free air. High air entry ceramic disks are generally made of sintered kaolin and have a thickness of 5 mm or 7 mm when used as part of a pressure plate apparatus. The ceramic disk is of extremely low permeability with respect to water flow (i.e., about 1 x 10^-11 m/s; Fredlund and Rahardjo, 1993). Typical high air entry ceramic disk used in unsaturated soil testing equipment, such as the triaxial, direct shear and SWCC apparatuses are rated for air entry value of 1 bar, 3 bar, 5 bar and 15 bar (Padilla et al. 2006). The axis-translation technique is performed by installing (i.e., sealing) a high air entry disk into the base pedestal of a soil testing apparatus. One of the concerns related to the use of high air entry ceramic disks is the time required for equilibrium to be established in a soil specimen. Tinjum et al. (1997) observed that the equilibrium in the pressure plate was established between 5 and 8 days for clayey soils. Consequently, the testing of unsaturated soils was time consuming and therefore, costly. The long time required to reach the equilibrium is particularly of concern for the measurement the soil-water characteristic curves. This study focuses on the use of micro-porous membranes instead of ceramic disks for improving the time required to reach matric suction equalization. Measurements of the soil-water characteristic curve were conducted using micro-porous membranes in a new pressure plate apparatus. In the testing program, the hydraulic response of the micro-porous membrane under varying water contents corresponding to increasing or decreasing matric suction was measured. Different micro-porous membrane with different air entry values were investigated in order to compare the time required for matric suction equilibrium with that of the ceramic disk. Soil-water characteristic curves of different types of soil were measured using the axis-translation technique with both the micro-porous membrane and the ceramic disk.

2 TEST PROCEDURE

2.1 Soil material & micro-porous membrane

Five soil types were used in the study. The grain size distribution curves of the soils are shown in Fig. 1. The micro-porous membranes used in this study are a product
manufactured by Pall Corporation (www.pall.com/lab). Two different types of membranes (i.e., polyether sulfone and acrylic copolymer) were used in this testing program as summarized in Table 1. The air entry values of the membranes range from 40 kPa to 250 kPa depending on the pore size and manufacturing process.

<table>
<thead>
<tr>
<th>No</th>
<th>Thickness (μm)</th>
<th>Air entry value (kPa)</th>
<th>Pore diameter (μm)</th>
<th>Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>140</td>
<td>250</td>
<td>0.45</td>
<td>Polyethersulfone</td>
</tr>
<tr>
<td>2</td>
<td>140</td>
<td>100</td>
<td>0.8</td>
<td>Polyether sulfone</td>
</tr>
<tr>
<td>3</td>
<td>94</td>
<td>60</td>
<td>0.8</td>
<td>Acrylic copolymer</td>
</tr>
<tr>
<td>4</td>
<td>94</td>
<td>40</td>
<td>1.2</td>
<td>Acrylic copolymer</td>
</tr>
</tbody>
</table>

A pressure plate apparatus with a ceramic disk was used to compare the results with those obtained from using the new micro-porous membrane apparatus. The high air entry ceramic disk was installed into the pedestal in place of the micro-porous membrane. The ceramic disk had a thickness of 7 mm and an air entry value of 200 kPa or 500 kPa.

2.3 Soil-water characteristic curve, SWCC, tests

Soil-water characteristic curve tests were performed in the low matric suction range with a maximum matric suction of 20 kPa. Drying and wetting paths were established by progressively increasing and decreasing matric suction. The soils were prepared in a slurry condition at a high gravimetric water content. Soil water moved in response to the externally applied air pressure and accumulated in the burette with elapsed time. The gravimetric water content of the soil specimen was calculated from the changes in the amount of water in the soil. When the water level in the burette attained a steady state condition, it was assumed that equilibrium conditions have been attained with regard to the applied matric suction. As a result, the air pressure supplied to the triaxial chamber was equal to the matric suction in the soil specimen. The matric suction was progressively increased up to about 20 kPa. After the application of the applied maximum matric suction, the air pressure in the chamber was decreased following the path of decreasing matric suction.
3 TEST RESULTS

3.1 Comparison of water flow characteristics

Three different micro-porous membranes (Micro-porous membrane No. 2, 3 and 4) in Table 1 were used to investigate the influence of air entry value on equilibrium time. These micro-porous membranes had different air entry values of 100 kPa, 60 kPa and 40 kPa. The silt was prepared in the steel mold, with an initial degree of saturation close to 100%. An air pressure of 25 kPa was applied to the upper surface of the soil specimen. The soil water passed through the micro-porous membrane and drained into the burette. Figure 4 shows changes of gravimetric water content with time. Once water flow commenced, the gravimetric water content decreased rapidly. The water content reached equilibrium in about 2 minutes.

Similar tests were conducted using the saturated high air entry ceramic disks. Figure 5 shows the changes in water content with time for the ceramic disks with AEV of 200 kPa. The rate of water content decrease was slower as compared to that of the micro-porous membrane. The equilibrium time appears to occur around 3000 minutes. The test results show that considerably more time is required to achieve equilibrium when using the high air entry ceramic disks.

There are differences in the soil-water characteristic curves obtained from the micro-porous membrane and those from the ceramic disk for all soil specimens. On the drying paths, the soil-water characteristic curve obtained from the micro-porous membrane was lower than that obtained from the ceramic disk. The water content using the micro-porous membrane is less than that measured using the ceramic disk. The water content as measured using the ceramic disk was considerably larger as compared to that obtained from the micro-porous membrane. It can be observed that absorption on the wetting path would result in a significantly less water content when the ceramic disk was used. Water appeared to be able to pass through the micro-porous membrane well, increasing the water content of the soil

specimen during the decrease in matric suction. The wetting soil-water characteristic curve is located close to the drying soil-water characteristic curve. The soil-water characteristic curves obtained from the micro-porous membrane showed less hysteresis as compared to those measured using the ceramic disk in the traditional testing method.

3.2 Soil-water characteristic curves for different soil materials

Soil-water characteristic curves for different soil materials as measured using both the micro-porous membrane and the ceramic disk with AEV of 500 kPa are shown in Figs. 6 to 10.
4 CONCLUSIONS

This study focused on the use of micro-porous membrane instead of ceramic disk for measuring soil-water characteristic curve and the results can be summarized as follows:

(1) Soil-water characteristic curve tests were performed in the low matric suction range. The air entry value of the micro-porous membrane was similar to that of the ceramic disk. The equilibrium time required for the SWCC measurements using the micro-porous membrane was much shorter than the equilibrium time required for the measurement using the high air entry ceramic disk. The micro-porous membrane provides more reasonable testing times for determination of SWCC for geotechnical practice.

(2) The hysteresis between the drying and wetting soil-water characteristic curves appeared to be negligible when measurements were made using the micro-porous membrane. The soil-water characteristic curve measured using the ceramic disk demonstrated larger hysteresis than the curve obtained using the micro-porous membrane.

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Determination of soil-water retention curve for a young residual soil using a small centrifuge

Détermination de la courbe de rétention d’eau pour un sol résiduel jeune à l’aide d’une petite centrifuge

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ABSTRACT: The soil-water retention curve (SWRC) determination usually involves time-consuming conventional methodologies such as those based on soil-water plate extractors, tempe cells, suction plates and filter-paper method. In order to overcome the considerable large time lag necessary for SWRC evaluation, an alternative methodology for direct determination of the SWRC of the unsaturated soil was developed using a commercially available small centrifuge with a swinging type rotor assembly without in-flight instrumentation. The testing consists of spinning up four initially saturated soil specimens until constant water content is achieved for a given angular speed. The soil-suction relationship is determined by relating the respective water content to the suction magnitude induced by the ceramic plate at the specimen’s base. This methodology was applied for evaluating the SWRC of a young residual soil from gneiss using both, undisturbed and remolded soil specimens until 900 kPa matric suction magnitude. The testing results show good agreement to the similar SWRC obtained by conventional methods and depicted using the Van Genuchten (1980) mathematical model. Overall, it can be concluded that the methodology proposed ensures good agreement in determining the SWRC of studied soil.

KEYWORDS: unsaturated soil, soil-water retention curve, centrifuge technique, soil suction.

1 INTRODUCTION

The application of a centrifuge for inducing an unsaturated state within a soil sample is not new. Soil drainage tests using centrifugal flow have long been recognized as a valid and efficient way of determining the SWRC. Briggs and McLane (1907) and Russell and Richards (1938) have proposed methodologies for estimating the SWRC using a centrifuge. The proposed methodologies essentially consist in draining an initially saturated soil specimen under a certain induced gravity. Different lower values of water contents can be rapidly achieved by increasing the applied induced gravity. The decrease in soil moisture is then related to the corresponding increase in the soil suction magnitude. Gardner (1937) introduced a “suction” to the induced gravity over the matric potential of the soil, carried out several sets of filter paper tests within a centrifuge apparatus, with soil specimens placed over a free water surface. The matric potential of the soil was evaluated after achieving hydraulic equilibrium and related to the induced gravity applied. The work of Gardner (1937) showed clearly that a relationship exists between the soil matric potential and the induced gravity applied to the soil sample. Other researchers (e.g. Hassler and Brunner, 1945; Croney et al., 1952) have attempted inducing suction magnitudes at the soil specimen’s boundaries through use of ceramic disks or cylinders. Corey (1977), working with soil samples allowed to free drainage due to centrifugal flow, observed that the negative pressure gradient induced within the soil sample acts inward, once the centrifugal force induces water flow on the outward direction. Corey (1977) noted that the capillary pressure induced within the soil sample varies along the length of the sample, from zero magnitude at the outflow boundary (which is open to the atmosphere), to a maximum value at the top boundary of the sample. The author concluded that at each angular velocity, the sample drains until the capillary force equals the centrifugal force induced over the water molecules. Recently, Khanzode et al. (2002) attempted to determine the SWRC at a single induced gravity by simultaneously testing several soil specimens with different ceramic disks attached. The main idea was inducing different magnitudes of suction at each specimen simultaneously. The results, however, exhibit a pore matching when compared to the results obtained by using a pressure plate apparatus. Although the authors suggest the need of complementary studies for understanding the influence of the induced gravity over the soil’s suction, a careful analysis of the methodology applied by Khanzode et al. (2002) indicates that it was not ensured that the specimens tested in the centrifuge had identical initial densities and water contents as those tested in
the pressure plate apparatus. These differences can lead to significantly different SWRC results as compared to the ones obtained with other methodologies.

Seeking the development of an accurate low cost alternative for direct evaluation of the SWRC and also in order to overcome the considerable large time lag necessary for SWRC evaluation by conventional methodologies, an alternative methodology is proposed in this manuscript for SWRC evaluation that uses a commercially available small centrifuge, without the need of in-flight instrumentation. Since there is no external invasive instrumentation (such as TDR probes, tensiometers, etc), the methodology allows evaluating the SWRC of undisturbed soils samples.

The methodology proposed was applied in determining the SWRC of a young residual soil using both, undisturbed and remolded soil specimens. The SWRC testing results show good agreement to the similar data obtained using filter-paper method, porous plate funnel and suction plate extractor.

2 TESTING SETUP AND THEORETICAL BACKGROUND

A schematic drawing of the testing setup developed is depicted in Figure 1.

![Figure 1](https://example.com/figure1.png)

**Figure 1. A schematic drawing of the centrifuge basic principle**

Basically, the setup is composed by a water reservoir located underneath a drainage plate and a high flow ceramic disk fitted above this drainage plate. A 20 mm thick soil specimen, fitted into a stiff stainless steel cylinder, used to avoid any horizontal strains during testing, is placed above the ceramic disk. A saturated filter paper is placed between the soil specimen and the ceramic disk in order to prevent soil particles from migrating into the ceramic disk during testing. The entire setup is assembled into small centrifuge equipment specially modified for receiving four testing setups simultaneously. The drainage plate induces a free drainage surface at the bottom boundary of the ceramic disk in order that all water flow coming from the soil specimen is fully transmitted to the drainage plate induces a free drainage surface at the bottom boundary of the ceramic disk in order that all water flow coming from the soil specimen is fully transmitted to the drainage plate.

The suction test consists in assembling two soil specimens over two 63 mm thick ceramic disks and other two specimens over two 12 mm thick ceramic disks. Once the centrifuge equipment was modified to fit four soil specimens simultaneously, the identical ceramic disks thicknesses setups are displaced on swinging buckets located on opposite sides of the centrifuge center of rotation. This procedure allows submitting two sets of two soil specimens to different values of suction magnitude simultaneously at a given angular velocity.

Table 1. Suction magnitudes attained to different ceramic disks and angular velocity, \( \omega \).

<table>
<thead>
<tr>
<th>( \omega ) (rpm)</th>
<th>Ceramic disks 12 mm</th>
<th>Ceramic disks 63 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>300</td>
<td>2.8</td>
<td>0.3</td>
</tr>
<tr>
<td>500</td>
<td>7.9</td>
<td>25.9</td>
</tr>
<tr>
<td>1000</td>
<td>31.3</td>
<td>105.7</td>
</tr>
<tr>
<td>1500</td>
<td>70.4</td>
<td>232.3</td>
</tr>
<tr>
<td>2000</td>
<td>125.2</td>
<td>414.7</td>
</tr>
<tr>
<td>2500</td>
<td>193.7</td>
<td>647.9</td>
</tr>
<tr>
<td>3000</td>
<td>261.8</td>
<td>852.7</td>
</tr>
</tbody>
</table>

3 EXPERIMENTAL COMPONENT

The testing program was carried at the Civil Engineering Laboratory (LECIV) of the State University of Norte Fluminense Darcy Ribeiro (UENF). The centrifuge equipment used was a Cientec CT 6000 small-scale centrifuge specially adapted with four swinging buckets. The testing program consisted in evaluating the SWRC of a young residual soil from gneiss using both, undisturbed and remolded soil specimens. The soil is classified as clayey silt sand. The undisturbed soil specimens sets, identified herein as undisturbed young horizon (UY), were sampled with a 50mm diameter 20 mm height rings. The remolded samples sets, identified as remolded young horizon (RY), were obtained by handling undisturbed soil samples and re-compacting them statically in order to achieve same dry unit weight in all specimens of each set. This
procedure was adopted in order to minimize potential heterogeneities usually present in undisturbed specimens allowing a better comparison among conventional methods and the alternative methodology using centrifuge. Table 2 presents some typical characteristics and index properties of the soils samples tested. As shown in Table 2, the void ratio of remolded specimens is greater than undisturbed specimens. This is justified in seeking to verify possible deformations in softer soils induced by high acceleration levels. Therefore, the mass of soil, at a specific remolding water content, for a known volume for each soil specimen was calculated. The mass of soil was placed in layers and a tiny compactor was used just to assent them until the complete volume was achieved.

Table 2 – Characteristics and index properties

<table>
<thead>
<tr>
<th>Sand</th>
<th>Silt</th>
<th>Clay</th>
<th>LL</th>
<th>LP</th>
<th>IP</th>
<th>W</th>
<th>γs</th>
<th>Sr</th>
<th>ASTM</th>
</tr>
</thead>
<tbody>
<tr>
<td>RY</td>
<td>55.0</td>
<td>38.0</td>
<td>7.0</td>
<td>50.2</td>
<td>24.6</td>
<td>25.6</td>
<td>19.3</td>
<td>16.6</td>
<td>26.7</td>
</tr>
<tr>
<td>UY</td>
<td>55.0</td>
<td>38.0</td>
<td>7.0</td>
<td>50.2</td>
<td>24.6</td>
<td>25.6</td>
<td>12.0</td>
<td>17.1</td>
<td>26.7</td>
</tr>
</tbody>
</table>

Previously to the centrifuge testing, all soil specimens and all ceramic disks were soaked with distilled - deaired water. The soil specimens wetting procedure adopted in gradually spraying the soil specimens with water until reaching a soaked state, characterized by a thin water layer formed above the top boundary of the specimen. The water content reached at the end of the soaking procedure was assumed to be correspondent to the saturation condition of each specimen. The ceramic disks saturation procedure consisted in submerging them during 48 hours into a recipient filled with distilled water and by spinning them up to 500 rpm in the centrifuge in order to flush air bubbles within them. Thereafter, two setups with 12 mm thick ceramic disks and two setups with 63mm ceramic disks were assembled as shown in Figure 2. Subsequently, the four testing setups were placed into the centrifuge buckets in a symmetric testing configuration with respect to the centrifuge axis of rotation in order to avoid in-flight unbalancing.

Figure 2. Setups Details (a) “Bucket X” (12 mm thick ceramic disks); (b) “Bucket Y” (63mm thick ceramic disks).

Figure 3 presents the general view of the centrifuge arrangement before starting the test. The soil specimens’ top boundary is protected with a PVC cap in order to prevent evaporation during testing. This cap is fitted over a stiff aluminum cap fixed in the centrifuge bucket (Figure 2). Subsequently, the soil specimens are subjected to angular velocities up to 3000 rpm. The tests were performed with initially soaked soil specimens displaced over an initially soaked high flow ceramic disk and submitted to successive increased gravities inducing successive unsaturated states without any external interference (even stopping the centrifuge).

Figure 3. Views of Cientec CT 6000 small-scale centrifuge: (a) external view; (b), (c) internal view.

The methodology allows determining simultaneously two pairs of soil moisture – suction relationships at each induced gravity applied. Applying angular velocities of 300, 500, 1000, 1500, 2000, 2500 and 3000 rpm allows determining experimentally 14 soil moisture – suction relationships. The proposed setup configuration allows evaluating the data repeatability once the specimens with same ceramic disk thickness are subject to similar suction magnitudes and, therefore, they should have similar moisture changes in a specific testing step (denoted by similar changes in specimens’ weight). Due to the absence of in-flight instrumentation, the no-flow steady-state moisture profile condition was checked by stopping the centrifuge equipment and checking any change in each specimen’s weight. The equilibrium condition is yielded when a constant specimens’ weight is achieved. After reaching the no-flow steady-state moisture profile condition at the 3000 rpm run, all soil specimens were oven dried for final water content determination. The water content magnitudes of each intermediary testing step were then back calculated and the respective SWRC plotted.

In order to evaluate any potential effect of consolidation during centrifugal flow, it was evaluated the soil specimen’s height at each centrifuge monitoring stops. It was not observed any volume change for angular velocities lower than 1500 rpm. For angular velocities higher than 1500 rpm, it was observed changes in the specimen’s height of 0.8 mm, 0.4 mm for RY, UY, respectively. These height changes correspond to 4 %, 2 % of volume changes respectively. Once the volume changes observed were small, the corresponding volumetric water contents were evaluated considering the initial soil unit weight even for angular velocities higher than 1500 rpm. The corresponding suction magnitudes of the tests that underwent volume changes were evaluated considering the actual radius magnitude calculated at each testing step.

4 RESULTS AND DISCUSSION

Figures 4 and 5 present the comparison among the SWRC obtained by conventional methods and depicted using the van Genuchten (1980) mathematical model, and the experimental data obtained following the methodology proposed herein. Figure 4 shows the RX testing results while Figure 5 shows the UY testing results.

Analyzing the results in Figures 4 and 5 it can be observed that the experimental data obtained through the proposed methodology agrees with the experimental data obtained by conventional methods such as filter paper method, plate extractor and suction funnel. Concerning the SWRC mathematical description, it can be noted in Figures 4 and 5 that the van Genuchten (1980) model describes accurately the soil’s suction – moisture relationship observed experimentally.
5 CONCLUSIONS

From the research program involving evaluation of the SWRC of undisturbed and remolded specimens of a young residual soil from gneiss conducted it can be concluded that:

- The SWRC determined using the proposed methodology agreed well with the experimental data obtained using conventional methods such as paper filter method, suction plate extractor and porous plate funnel indicating the excellent potential use of the methodology for SWRC determination for studied soils in a reduced time-frame.

- The centrifuge methodology was able of determining the SWRC with good accuracy up to suction magnitudes of the order of 900 kPa.

6 ACKNOWLEDGMENTS

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7 REFERENCES


Interpretation of the Effect of Compaction on the Mechanical Behavior of Embankment Materials Based on the Soil Skeleton Structure Concept

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ABSTRACT: In this research, triaxial compression tests were carried out on 5 materials having different grain size distributions and compaction properties to determine the mechanical properties of compacted soil. In addition, the triaxial test results were reproduced using the SYS Cam-clay model to interpret the mechanical behavior of the compacted soil and the effect of compaction on soil based on the soil skeleton structure concept. Also, seismic response analysis of embankments constructed using these soils was carried out using GEOASIA to determine the relationship between the compaction properties and the seismic stability of embankments. The following were the main conclusions. 1) The maximum deviator stress increased with the increase in degree of compaction Dc, but the trends in the increase differed depending on the material. 2) The mechanical behavior of the compacted soil could be reproduced using the SYS Cam-clay model using one set of material constants for each material. 3) It was found that when the Dc is increased, the seismic stability is increased and that as the maximum dry density increased, the seismic stability increased.

RéSUMÉ: Dans cette étude, nous étudions les caractéristiques mécaniques des sols compactés en mettant en œuvre des essais de compression triaxiale sur 5 échantillons ayant des caractéristiques de granulométrie et de compactage différentes. En outre, nous reproduisons les résultats des essais triaxiaux grâce au modèle SYS Cam-clay d’équation constitutive d’élasto-plasticité. Nous avons également effectué l’analyse de la réponse sismique de la déformation des remblais ainsi créés en utilisant le programme d’analyse dynamique de déformation finie eau-sol GEOASIA pour comprendre la relation entre les caractéristiques du compactage et les performances sismiques du remblai. Les principales conclusions sont indiquées ci-dessous. 1) Avec l’augmentation du degré de compactage, la la contrainte axiale maximale est augmentée, mais cette augmentation dépend de la nature de l’échantillon. 2) Il a été possible de reproduire le comportement mécanique des sols compactés à l’aide du modèle SYS Cam-clay. 3) Quand le degré de compactage augmente, il a été constaté que plus la densité sèche maximale de l’échantillon est grande plus importante est sa résistance aux tremblements de terre.

KEYWORDS: compaction, embankment, triaxial compression test.

1 INTRODUCTION

The stability of the embankment is progressing since fundamental principle of soil compaction proposed by Proctor (Proctor, 1933), and many researcher and engineer have studied and improved quality control standard by compaction. After Han-Shin Awaji Earthquake disaster, road earthwork guideline for construction of embankment was revised to evaluate the seismic resistance of embankments, and design principles are changing from being specification based to being performance based. However, locally occurring materials are still used as embankment materials, and quality control of embankments is carried out using the Dc, so doubts remain regarding whether good quality embankments are being produced. In addition, because prediction of the deformation behavior of embankments due to earthquakes is extremely important, a theory for reproducing the mechanical behavior of compacted materials to enable prediction of the deformation behavior of embankments in an earthquake is necessary.

In this study, 5 types of soil material with different grain size distributions and compaction properties were selected, and the compacted soil specimens were subjected to CU triaxial tests, with the objective of obtaining basic data to reproduce the mechanical behavior of various materials after compaction. The shear behavior after compaction was compared and examined in accordance with the differences in the materials. In addition, the effect of compaction on the mechanical behavior of embankment materials was interpreted based on the soil skeleton structure concept by reproducing the mechanical behavior using the elasto-plastic super/subloading yield surfaces Cam-clay model (hereafter referred to as the SYS Cam-clay model) (Asaoka et al. 2002). Also, seismic stability analysis of embankments were carried out using the soil-water coupled finite deformation analysis program GEOASIA (Noda et al. 2008), which incorporates the SYS Cam-clay model, to determine the relationship between the compaction properties and the seismic stability of embankments.

2 PHYSICAL AND COMPACTION PROPERTIES OF EMBANKMENT MATERIALS

Five types of materials were examined in this study and are referred to as materials A, B, C, D, and E (Nakano et al. 2010, Yokohama et al. 2010). Fig. 1 shows the grain size distributions of the 5 materials. Fig. 2 shows the results of compaction tests on the 5 materials. The compaction tests were carried out by the A-b method for materials B and D, by the A-a method for materials A and C, and by the A-c method for material E. It can be seen that as the amount of coarse-grained fraction of the material increased, the maximum dry density increased and the optimum water content decreased.
3 EFFECT OF COMPACTION ON THE MECHANICAL PROPERTIES OF EMBANKMENT MATERIALS

For 5 types of materials, CU triaxial compression tests were carried out under Dc and confining pressure shown in Table 1. The materials D and E are omitted. When preparing the test specimens, the Dc was adjusted by changing the compaction energy. After setting the specimen in the triaxial compression apparatus, the specimen was saturated with de-aired water using the double-suction method or the back pressure method. Then isotropic consolidation process was carried out, and when it was confirmed that the consolidation completed, the undrained shearing was carried out under constant axial strain rate.

Table 1 Test conditions

<table>
<thead>
<tr>
<th>Material</th>
<th>Confining pressure (kPa)</th>
<th>Dc (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>100, 150, 300</td>
<td>85.90, 95, 100</td>
</tr>
<tr>
<td>B</td>
<td>50, 100, 150, 300</td>
<td>85.90, 95, 100</td>
</tr>
<tr>
<td>C</td>
<td>100, 300</td>
<td>90, 95, 100</td>
</tr>
</tbody>
</table>

Fig. 3 shows that the deviator stress q - shear strain $\varepsilon_s$ relationship and $q$ - mean effective stress $p'$ relationship of the CU triaxial tests when the confining isotropic consolidation pressure was 100 kPa for materials A through C. The materials D and E are omitted. In this section, the increase in $q$ during shearing and maximum of $q$ as a result of compaction is referred to as ‘compaction effect’. For material A at the 85 and 90% of Dc, an increase in $q$ associated with a reduction in $p'$ was seen at the initial shear (shear strain $\varepsilon_s = 0-2\%$). Thereafter, it exhibited the mechanical behavior of normally consolidated soil with both $p'$ and $q$ in a critical state. At the 95% of Dc, there was almost no reduction in $p'$ observed, and thereafter, both $p'$ and $q$ exhibited a critical state. At the 100% of Dc, an increase in $q$ associated with the increase in $p'$ was seen, and the maximum of $q$ increased greatly. It can be seen that if the Dc of material A is not large, a compaction effect is not seen. With material C at the 90 and 95% of Dc, softening behavior was seen associated with plastic compression as a reduction in $q$ associated with a reduction in $p'$. This is behavior seen in soft natural deposited clay. On the other hand, at the 100% of Dc, the shear behavior changed, with $q$ increasing in association with an increase in $p'$, and the compaction effect was exhibited. However, the compaction effect was small compared with materials A and B.

4 INTERPRETATION OF COMPACTION EFFECT BASED ON SOIL SKELETON STRUCTURE CONCEPT

The SYS Cam-clay model is an elasto-plastic constitutive model that expresses soil skeleton structure as 3 properties, structure, overconsolidation, and anisotropy, and describes the evolution of the soil skeleton structure associated with development of plastic deformation. The major characteristic of the SYS Cam-clay model is that it can explain the mechanical behavior of typical clays and sands, as well as intermediate soils, based on the rate of change of the evolution of the soil skeleton structure. In this study, the undrained shear behavior after compaction for 5 materials is simulated by the SYS Cam-clay model, and each compaction effect of each specimens can be interpreted based on soil skeleton structure.

Figs. 4 through 6 show the results of reproducing the mechanical behaviors of materials A through E using the SYS Cam-clay model. The top 2 graphs are the stress-strain relationship and the effective stress path, as in Fig. 3. The bottom left graph shows the decay of structure associated with shear deformation, and the bottom right graph shows how loss of overconsolidation associated with shear deformation occurred; $R^*$ indicates the degree of structure, and the closer $R^*$ is to 1, the lower the structure is, while $R$ indicates reciprocal of OCR.

The material constants and the initial conditions of the materials are shown in Tables 3 and 4, respectively. The calculation results were able to reproduce the test results. In the case of materials for which a large maximum dry density was
obtained in the compaction test, such as material A, the decay of structure and loss of overconsolidation are fast. On the other hand, in the case of materials for which a small maximum dry density was obtained in the compaction test, such as material C, there was little decay of structure, and loss of overconsolidation was moderate. Finally, for material B, the results indicated that there was little decay of structure, and loss of overconsolidation is moderate. At this time, the decay of structure was fast and that the loss of overconsolidation was slow. As the maximum dry density increased, decay of structure occurred faster. However, there was no correlation between the rate of loss of overconsolidation and the maximum dry density. Also, focusing on the initial values, for all materials, it can be seen that as the Dc increased, the structure decayed, and overconsolidation accumulated. Also, it can be seen that for material such as C with small maximum dry densities, structure and overconsolidation tend to remain even though they are compacted. It can be inferred that this is because there is little decay of structure due to shearing. Also, focusing on materials A and B at Dc of 95–100%, overconsolidation increases suddenly as a result of compaction. At this time, the q also increases greatly. It can be seen that the increase in q of compacted soil can be determined by the ease of accumulation of overconsolidation.

Table 2 Material constants

<table>
<thead>
<tr>
<th>Material</th>
<th>A</th>
<th>B</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression index</td>
<td>0.07</td>
<td>0.11</td>
<td>0.13</td>
</tr>
<tr>
<td>Swelling index</td>
<td>0.01</td>
<td>0.02</td>
<td>0.01</td>
</tr>
<tr>
<td>Limit state index</td>
<td>1.48</td>
<td>1.35</td>
<td>1.45</td>
</tr>
<tr>
<td>NCL intercept (98.1 kPa)</td>
<td>1.50</td>
<td>1.71</td>
<td>2.07</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
</tr>
<tr>
<td>Normal consolidation index</td>
<td>5.00</td>
<td>0.50</td>
<td>1.30</td>
</tr>
</tbody>
</table>

Table 3 Initial values

<table>
<thead>
<tr>
<th>Material</th>
<th>Dc</th>
<th>Specific volume</th>
<th>Extent of structure</th>
<th>Overconsolidation</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>90</td>
<td>1.55</td>
<td>1.50</td>
<td>3.77</td>
</tr>
<tr>
<td></td>
<td>95</td>
<td>1.47</td>
<td>1.30</td>
<td>13.2</td>
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<tr>
<td></td>
<td>100</td>
<td>1.40</td>
<td>1.10</td>
<td>32.0</td>
</tr>
<tr>
<td>B</td>
<td>90</td>
<td>1.72</td>
<td>1.30</td>
<td>8.10</td>
</tr>
<tr>
<td></td>
<td>95</td>
<td>1.64</td>
<td>1.20</td>
<td>19.1</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>1.56</td>
<td>1.10</td>
<td>42.5</td>
</tr>
<tr>
<td>C</td>
<td>90</td>
<td>2.17</td>
<td>2.20</td>
<td>5.1</td>
</tr>
<tr>
<td></td>
<td>95</td>
<td>2.08</td>
<td>1.90</td>
<td>9.8</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>1.98</td>
<td>1.40</td>
<td>16.2</td>
</tr>
</tbody>
</table>

5 SEISMIC RESPONSE ANALYSIS OF EMBANKMENTS

Fig. 7 shows a complete cross-section of the embankment and ground used in the analysis. The ground was assumed to be hard ground with poor permeability. Also, it was made sufficiently wide to take into consideration the effects of the side surface boundaries. The height of the embankment was 6 m, with slopes of 1:1.8. Also, the width of the crown was 14 m, assuming an expressway with one lane on each side. The hydraulic boundary conditions were as shown in Fig. 7; the edges on the left, right and bottom were impermeable boundaries, and the top edge was a permeable boundary (atmosphere). Also, the water level was always constant at the ground surface. In other words, the ground and embankment were always saturated. The movement boundary conditions before the earthquake were as follows: all the nodes on the left and right edges were fixed horizontally, and all the nodes on the bottom surface were fixed horizontally and vertically. During and after the earthquake, periodic boundaries were assumed, and both edges were provided with constant displacement boundaries. In addition, in order to prevent all reflections of the seismic waves, a viscous boundary (Joyner et al. 1975) was provided in the horizontal direction on the bottom edge during the earthquake. The seismic motion is measured ground surface wave at Kobe Marine Observatory in the Southern Hyogo prefecture earthquake in 1995. The input seismic motion was assumed to be a level 2 inland earthquake. In this section, the materials analyzed were materials A and C.

Figs. 8 and 9 show the shear strain distribution immediately before and after the earthquake for materials A and C, respectively. Also, the values shown in the figures indicate the amount of settlement in the center of the crown after the earthquake. For all the materials, as the Dc is increased, the strain due to the earthquake becomes smaller, and the amount of settlement is reduced to about one-third. It can be seen that increasing the Dc is extremely effective for improving the seismic stability of embankments. For material A at all Dc, the strain due to the earthquake did not extend, and stable behavior
was exhibited during and after the earthquake. For material C at the 90% of Dc, a slip plane occurred from the top of the slope of the embankment due to the earthquake, resulting in collapse. However, it can be seen that as the Dc increased, the shear strain due to the earthquake reduced, and the seismic stability increased. From the above, it can be seen that greater seismic stability can be obtained from embankments constructed with materials with fast decay of structure and loss of overconsolidation (material A) than from embankments constructed from materials with little decay of structure, and loss of overconsolidation (material C).

2) The mechanical behavior of each material was reproduced with the SYS Cam-clay model using one set of material constants for each material and representing the differences in Dc by different initial conditions of structure and overconsolidation. It was possible to interpret the increase in Dc as decay of structure and accumulation of overconsolidation. When the q increased beyond CSL, it was found that the overconsolidation tended to increase. Also, it was found that a large maximum dry density, such as material A, exhibits that the decay of structure and loss of overconsolidation are fast, while a small maximum dry density, such as material C, there was little decay of structure, and loss of overconsolidation was moderate.

3) From seismic response analysis using GEOASIA, it was found that the seismic stability of embankments was increased by increasing the Dc. Materials with fast decay of structure and loss of overconsolidation, such as material A, produce embankments with high seismic stability, so they are good embankment materials.

7 ACKNOWLEDGEMENTS

Data for this report were provided by Atsuko Sato of the Civil Engineering Research Institute for Cold Region and Professor Seiichi Miura of Hokkaido University with the assistance of the 2009 Ministry of Land, Infrastructure, Transport and Tourism Construction Technology Research and Development Subsidy Program, for which we wish to express our thanks.

8 REFERENCES


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6 CONCLUSIONS

In this research, various laboratory tests were carried out on 5 types of embankment material, and their mechanical behaviors were reproduced using the SYS Cam-clay model. Also, seismic response analysis was carried out for embankments constructed with 2 types of material with 3 Dc. The following are the conclusions obtained from this research.

1) For all materials, an increase in q associated with an increase in p’ during shearing was seen as a result of compaction, and the maximum of q increased. However, the trend in the increase was different for each material; for some materials, the maximum of q did not increase with compaction, and for some materials, the maximum of q suddenly increased from a certain Dc.
Mechanisms of Strength Loss during Wetting and Drying of Pierre Shale

Mécanismes de la perte de force pendant humidification et séchage de Pierre Shale

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ABSTRACT: The physio-chemical and morphological role on the residual strength of Pierre Shale during wetting and drying cycles was investigated. Ring-shear tests were conducted on intact and cycled material to assess the residual strength. The mineralogy and chemistry were determined from x-ray diffraction and x-ray fluorescence results. Minor mineralogical changes were observed during the cycling process. The material degraded from a firm, dense shale to a massive, clayey material after three to four cycles. Gypsum concentrations decreased during the wet-dry cycles. The low residual friction angles of 6.1 to 6.8 degrees decreased an additional 0.8 to 1.4 degrees during the wet-dry cycles. A significant fabric contrast was apparent after three cycles as the material’s structure became more massive. The material with higher amounts of montmorillonite in the mixed-layer clay mineral showed little change in the liquid limits with cycling, in contrast to the illite materials. A decrease in residual strength was observed for the first two wet-dry cycles, but little change for successive wet-dry cycles. The results suggest the disintegration of particles during slaking is the main determinant of strength loss. The initial mineralogy was also observed to be a factor on the slaking rate and the residual strength behavior.

RÉSUMÉ : Le rôle physio-chimique et morphologique sur la résistance résiduelle de Pierre Shale au cours des cycles de mouillage et de séchage a été étudié. Les essais de cisaillement annulaires ont été effectués sur ce matériau intact et recyclé pour évaluer sa résistance résiduelle. La minéralogie et la chimie ont été étudiées à partir de diffraction des rayons X et de rayons X de fluorescence. Des modifications minéralogiques mineures ont été observées au cours des cycles hydriques. Le schiste dense a subi une dégradation après 3 à 4 cycles et a été transformé en une argile. La concentration en gypse a également été diminuée. Les faibles angles de frottement interne résiduels de 6,1 à 6,8 degrés ont diminué d’environ 0,8 à 1,4 degrés supplémentaires au cours des cycles de mouillage-séchage. Une modification importante de la structure interne et une densification ont été notées après trois cycles. Les limites de liquidité du matériau avec un pourcentage élevé de montmorillonite n’ont pas été modifiées contrairement au matériau contenant de l’illite. Une diminution de la résistance résiduelle a été observée pour les deux premiers cycles hydriques, mais peu de changement a été observé pour des cycles ultérieurs de mouillage-séchage. Les résultats suggèrent que la désintégration des particules pendant l’humidification est la principale cause de perte de résistance. La minéralogie initiale a également été considérée comme un facteur important influençant cette désintégration et la résistance résiduelle.

KEYWORDS: Shale, weathering, strength, clay mineralogy, residual friction angle

1 INTRODUCTION

The tendency for clay shales to weather, soften, and slake upon drying and rewetting has been well documented. The degradation causes the material to soften and lose strength, possibly leading to slope failures. Skempton (1964) noted strength losses of up to 80% in some deposits after softening. The slaking rate has been observed to be dependent on the mineralogy and physico-chemical behavior, especially in materials with high activity clay minerals (Perry and Andrews 1984). While the mineralogy mechanisms are well known, very few studies on clay shales have been conducted to analyze the role of physico-chemical occurrences on the strength loss.

Weathering in overconsolidated clays and clay shales has been observed to be a significant process due to the mode of deposition and the bonding from diageneesis, especially in outcroppings. One of the largest and most problematic clay shales in the United States is the Pierre Shale Formation (Fleming et al. 1970). The drop in strength from the peak to residual strength is a source of the stability problems associated with these materials. Initial fissuring results from rebound in clay shales and leads to clay swelling, strain softening and weathering (Brooker and Peck 1993). Much of the past work on residual strength has focused on the mechanical aspects with less emphasis on the role of the more dynamic, physico-chemical effects on residual strength.

This paper reports the results of tests conducted to relate the strength of a series of laboratory wetting and drying cycles on unweathered Pierre Shale to its chemistry, mineralogy, and micromorphology. The study analyzes the role of the initial mineralogy, breakdown of particle size, and the overall changes in the mineralogy.

1 MATERIALS AND METHODS

1.1 Materials

As described by Bjerrum (1967), the behavior of overconsolidated materials is strongly correlated to their geologic history. Pierre Shale is a heavily overconsolidated clay shale formed from a marine/non-marine environment sedimentation during the Cretaceous Period approximately 60 to 80 million years ago (Fleming et al. 1970). The formation extends throughout Canada and as far south as the Gulf of Mexico. Significant slope failures have been observed throughout the formation, but are mainly focused in the upper Missouri and South Saskatchewan River basins. The
mineralogy of Pierre Shale is primary clay minerals, specifically smectite and mixed-layer clays. Unweathered samples were obtained from a boring at the Oahe Dam site, South Dakota, at depths below 60 meters. Firm shale, without any weathering, has an average dry density of 1.71 g/cm³, a moisture content of 25%, and specific density of 2.7 at the site. Unweathered Pierre Shale at the site exhibits unconfined strengths of 0.5 to 17.4 MPa and a Young’s modulus between 137 and 965 MPa (Johns et al. 1963). The wide variation in strength shows the engineering complications in classifying Pierre Shale as either a rock or soil. Extensive problems were encountered during the construction of the Oahe Dam due to the rapid strength loss in the material (Knight 1963). After many failures, slopes had to be redesigned with φ=8.5 degrees and 14.4 kPa for cohesion (Johns et al. 1963). Weathering was observed to cause the durable, brittle, rock-like shale to turn into a weathered, soil-like material at the site. The peak strength of the weathered material was found to be φ=11.9 degrees and 24.9 kPa for cohesion, but due to the possibility of slip surfaces in the shale, the residual strength controlled the design parameters. In a laboratory slaking test, Botts (1986) observed nearly a 75% drop in the shear strength of Pierre Shale samples, or a 6-degree drop in the internal friction angles and a reduction of the cohesion from 848 kPa to 0 kPa, after one wetting and drying cycle.

1.2 Sample Preparation

1.2.1 Residual Shear Strength
The testing procedure for the Bromhead Ring Shear device used a modified version of ASTM D6467. The material was crushed and passed through a US standard #50 sieve, remolded with distilled water to their plastic limit and allowed to hydrate for 48 to 72 hours. Higher water contents are suggested by ASTM (2003) but Bromhead (1979) advises drier samples in the 48 to 72 hours. Higher water contents are suggested by ASTM (2003) but Bromhead (1979) advises drier samples in the 48 to 72 hours. Tension was obtained for 100, 200, and 400 kPa normal stress reached, typically an additional 10 to 15 mm. The residual strength was obtained for 100, 200, and 400 kPa normal stress increments.

1.2.2 Micromorphology
Central to the quality of any fabric analysis in soils is a preparation method preventing disturbance. Dry samples are required for analysis in the vacuum environment of the scanning electron microscope (SEM). The freeze-drying technique was used to obtain relatively undisturbed samples. A sample from each weathering cycle was placed in an intermediate freezer for twelve hours at a temperature of -75 degrees Celsius and then placed in a VirTis Ultra.35 8-shelf model freeze dryer at a temperature of -25 degrees Celsius until a vacuum of <100 millitorr was achieved. The temperature was raised to 26 degrees Celsius and held constant for 48 to 72 hours. Tension fractured and placement on carbon stubs preceded the analysis in the SEM.

1.2.3 Mineralogical and Chemical Analysis
X-ray fluorescence (XRF) and x-ray diffraction (XRD) analyses used both bulk samples and size fractions to determine the composition of the material. Bulk, silt, and sand mineralogy was obtained from random-oriented mount samples mounted in the XRD unit and analyzed at a speed of 2°/two theta per minute with copper K-alpha radiation. Minerals were identified by a computerized catalog of the Joint Committee on Powder Diffraction Standards (JCPDS) Powder Diffraction File system (The International Centre for Diffraction Data® 2004). Particle-size separation was performed on the material on the basis of Stoke’s law, where particles <2 µm equivalent spherical diameter were obtained. This fraction was further separated by the same method into the coarse clay fraction, particles less than 2 µm, and the fine clay fraction, particles less than 0.5 µm. The oriented-oriented samples intensified the (001) reflections and reduce (hk0) reflections by removing non-platy minerals, dispersing clay minerals into individual colloidal particles, and laying the clay particles flat (Moore and Reynolds 1997). The filter transfer method, as recommended by Moore and Reynolds (1997), was used to transfer the material to a filter. The type of clay mineral is identified by the characteristic expansion, contraction, or collapse of the clay mineral’s d-spacing through five subsequent treatments: air drying, glycolation with ethylene glycol, heating to 400°C, and heating to 550°C. The methods described in Shulz (1978) were used to estimate the concentrations of the clay minerals in the mixed-layer.

2 TESTING AND ANALYTICAL METHODS.

2.1 Weathering Cycle
ASTM standard C593 (ASTM 2003) was used as an outline for the saturation of the Pierre Shale. The vacuum-saturation strength testing procedure described in the standard was modified to allow for saturation of the soil samples. The test consists of obtaining multiple two cubic-inch samples of unweathered Pierre Shale and placing them on a filter in a vacuum chamber. Samples were vacuumed at 24 inch Hg (11.8 psi) for one hour in the chamber; the chamber was then flooded; and the samples were soaked for one hour in the distilled water bath. The samples were removed from the bath and allowed to air-dry for 48 to 72 hours, completing one weathering cycle.

2.2 Classification and Ring Shear
Classification testing followed applicable American Society for Testing and Materials (ASTM 2003) standards. Atterberg limit and hydrometer analyses were performed on the samples. The Bromhead Ring Shear Device was used for measurement of the residual strength. The device was first proposed by Bromhead (1979) and has provided results that are in good agreement with back-calculations for slope stability analysis (Bromhead and Dixon 1986, Skempton 1985). The mold dimensions are 100-mm outer diameter, 70-mm inner diameter, and a thickness of 5-mm. Replicate testing was not conducted for the cycling studies due to sample limitations.

The prepared sample was molded into the ring at a water content near its plastic limit and placed in a distilled water bath. Multistage testing was used to obtain the residual strength. This technique uses progressive loading after the formation of a shear surface. The samples were initially consolidated at the normal stress of 100 kPa and then sheared at a rate of 0.16 degrees per minute (0.119 mm per minute) for a displacement of one revolution to form the shear surface. The rate was slowed down to 0.048 degrees per minute (0.036 mm per minute) and allowed to shear until the residual strength was reached, typically an additional 10 to 15 mm. The residual strength was obtained for 100, 200, and 400 kPa normal stress increments.

2.3 Scanning Electron Microscopy
Freeze-dried samples were analyzed with a Hitachi S-2460N VP scanning electron microscope (SEM) with energy dispersive spectrogrographic (EDS) and digital imaging capability. Two sources were used to obtain surface topography: backscattered electron (BSE) and secondary electron (SE). BSE mode was used for lower-resolution imaging and elemental analyses while SE mode was used for higher magnifications. Samples were coated with gold to prevent charging for samples analyzed in the SE mode. Digital elemental maps were produced using the EDS function on the SEM. Mineral contents were inferred by coinciding elemental maps.
3 RESULTS

3.1 Engineering Properties

Samples from three different depths below 60 meters were cycled through wetting and drying. Table 1 shows the index properties for the cycles. The material at the 63.0 and 70.3-meter depths consisted of approximately 2% sand, 50% silt, and 48% clay sized particles. The 63-meter samples exhibited a substantial increase in the liquid limit, as the material was wetted and dried. Minor increases in the liquid limit were observed in the 63.6-meter sample. The 70.3-meter sample showed insignificant changes in the Atterberg limits. The plastic limit increased by ten and five percent for the 63.0 and 66.6 meter cycles. The plastic limit remained almost unchanged for the 70.3 meter depth.

Table 1. Weathering cycle Atterberg limits.

<table>
<thead>
<tr>
<th>Depth (M)</th>
<th>Depth (Ft)</th>
<th>Weathering Cycle</th>
<th>Liquid Limit (%)</th>
<th>Plastic Limit (%)</th>
<th>Plasticity Limit (%)</th>
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<tbody>
<tr>
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<td>208</td>
<td>0</td>
<td>79</td>
<td>33</td>
<td>46</td>
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<tr>
<td>63.0</td>
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<tr>
<td>63.0</td>
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<td>5</td>
<td>71</td>
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The residual friction angle for each cycle for the three depths was determined from ring-shear tests. The formulated residual strength-normal stress plot for each sample was constructed using a spreadsheet program and analyzed for both the trend line through the origin and with a cohesion intercept. The residual friction angle (ϕ′) and residual cohesion are summarized in Table 2. The residual friction angle dropped almost 1.5 degrees for the 63.0 meter depth, nearly one degree for the 63.6 meter sample, and 1.3 degrees for the 70.3 meter depth during the wet/dry cycles. An unexpected increase in strength was observed in the later stages of the 70.3 meter cycles. Similarities in the 63.6 and 70.3 meter residual friction angle plots are apparent for the first two cycles.

3.2 SEM Images and Visual Observations

The unweathered material below 60 meters showed dense, high-laminae fabric. The three depths analyzed had similar fabrics. Various other fabrics and particle shapes were observed over the depths including burrows of pyrite frambooids and cemented calcite accretions. Weathering created a more homogenous fabric. Weathering led to a more open fabric. Visual observations of yellow, sulfuric, sand particles scattered over the sample between the laminae were made. Fabrics with large amounts of yellow particles showed a significant amount of degradation after one to two wet/dry cycles as well. After the third cycle, the material turned into a soil-like material. As the material dried, large cracks were observed, an indicator of the shrink/swell potential of the clay minerals in the material.

Following each weathering cycle, the sample was fractionated in a sedimentation cylinder to obtain the clay fraction for XRD analyses. Observations of the time rate of settlement of the particles during this process provide insight into the soils' behavior. Settlement analyses showed a contrast in the suspension time for wet/dry samples. The fine fraction of the wet/dry cycled samples stayed in suspension for many weeks longer than the un-cycled samples. A sample after five wet/dry cycles continued to be in suspension after three months.

The elemental composition of the materials was inferred from elemental maps of the EDS analyses. Elemental maps of the unweathered Pierre Shale showed K- and Na-ions are dispersed over the material, a possible indicator of the adsorbed cation on the montmorillonite mineral. The material has high aluminum, silicon, and oxygen suggesting clay minerals. The particles with high reflection were determined to be pyrite due to the occurrence of iron and sulfur in the bright particles.

3.3 Mineralogy and Chemistry

Bulk mineralogy was determined from powder samples. Sand, silt, clay, and very fine clay fractions were divided to determine the mineralogy of the fractions. Sand and silt mineralogy was determined by random-mount samples and clay mineralogy was found from oriented-mount samples. Bulk mineralogy indicated quartz (SiO2), gypsum (CaSO4.2H2O), and pyrite (FeS2). The mineralogy was similar over the depths except for higher concentrations of gypsum at the 70.3 m depth.

The sand fraction was observed to be highly heterogeneous, with a significant amount of quartz. Large calcite accretions were observed visually in the bulk sample and were found in the sand fraction mineralogy. These observations were verified by EDS analyses on SEM specimens. Pyrite, gypsum, and bassanite (CaSO4.1/2H2O) were also observed in the sand fraction. Gypsum is a hydrated form of bassanite. Quartz and pyrite were found in the silt fraction with minor amounts of feldspars including orthoclase (KAlSi3O8) and albite (NaAlSi3O8).

The clay minerals were primarily mixed-layer clays as well as minor amounts of illite clay mineral. Shultz (1978) provides an outline for the determination of mixed-layer clay minerals in Pierre Shale. The 63.0 meter depth showed a mixed-layer clay mineral with montmorillonite, illite, and beidellite concentrations of 20, 45, and 35 percent, respectively. The 70.3 meter depth had a mixed-layer clay mineral closer to bentonite with montmorillonite, illite, and beidellite concentrations of 60,
15, and 20 percent, respectively. Data and diffractograms for the mixed-layer clay determinations are shown in Birchnier (2005). These values fall within the data compiled by Schultz (1978) for mixed-layer clay minerals in Pierre Shale. Overlays of diffractograms of bulk samples for the wet/dry cycles of the 70.3 meter depth showed little change over the weathering cycles except for the reduction of gypsum particle size during the initial cycles. The combination of the decrease in the gypsum peak intensity at 7.63 Å, and the constant 4.28 Å and 3.8 Å peak intensities during cycling suggests the breakdown of the gypsum particles.

4 DISCUSSION

Residual friction angles for the wet/dry cycles decreased from 6.8° to 5.4°, 6.1° to 5.2°, and 6.7° to 5.4° for the depths 63.0, 63.6, and 70.3 m, respectively. A one degree drop in a material with an initial residual friction angle of 6.5° is significant. For a given normal stress and negligible residual cohesion, the factor of safety would be reduced by a ratio of 1.2. Exposure and removal of confining stresses during construction activities in Pierre Shale could cause wetting and drying to occur and lead to slope failures.

The residual friction angle for each cycle for the three depths was determined from ring-shear tests. The residual friction angle dropped almost 1.5 degrees for the 63.0 m depth, nearly one degree for the 63.6 m sample, and 1.3 degrees for the 70.3 m depth during the wet/dry cycles. An unexpected increase in strength was observed in the later stages of the 70.3 meter cycles. Similarities in the 63.6 and 70.3 m residual friction angle plots are apparent for the first two cycles. Residual friction angles for the wet/dry cycles decreased from 6.8° to 5.4°, 6.1° to 5.2°, and 6.7° to 5.4° for the depths tested. A degree drop in a material with an initial residual friction angle of 6.5° is significant. For a given normal stress and negligible residual cohesion, the factor of safety would be reduced by a ratio of 1.2. Exposure and removal of confining stresses during construction activities in Pierre Shale could cause wetting and drying to occur and lead to slope failures.

5 CONCLUSIONS

Minor mineralogical changes were observed in the wet/dry cycles similar to the weathering occurrences in Pierre Shale. Gypsum concentrations decreased initially in the wet/dry cycles. The low residual friction angles of 6.1° to 6.8° decreased an additional 0.8° to 1.4° during the wet/dry cycles. A significant fabric contrast was apparent after three cycles as the material's structure became more massive. The most noticeable difference in the cycles was the particle settlement rates. Excessive cycling caused particles to stay in suspension for weeks to months longer than the un-cycled material. This observation indicates clay aggregates are becoming smaller and going towards their unit-cell size. The reduction in size increased the clay fraction, contributing to the residual strength decrease.

The mechanical behavior varied for the samples analyzed. The material with larger amounts of illite in the mixed-layer clay mineral showed a decrease in residual strength following Stark and Eid's (1994) curves. The material with higher amounts of montmorillonite in the mixed-layer clay mineral showed little change in the liquid limits, a contrast to the other sample. A decrease in residual strength was observed for the first two cycles but increased thereafter. The contrasting behavior shows the heterogeneity of the material and the difficulties in determining design parameters. The mineralogical changes and the disintegration of aggregates during wetting and drying are concluded to be more influential than physico-chemical effects.

6 ACKNOWLEDGEMENTS

The authors thank the engineers of the U.S. Army Corps of Engineers Project Office in Pierre, SD for their help in obtaining the samples of Pierre Shale. Ashley Schwaller, undergraduate student at Iowa State University, is thanked for conducting the XRD tests. This material is based on work supported by the National Science Foundation under Grant Nos. CMS-0201482 and CMS-0227874. This support is gratefully acknowledged. Any opinions, findings and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect the views of the National Science Foundation.

7 REFERENCES.

Effect of confining stress on the transient hydration of unsaturated GCLs

ABSTRACT: Geosynthetic clay liners (GCLs) are often used within composite landfill liner systems in combination with a geomembrane to provide an effective barrier. In order for the GCL to function as a barrier, however, it must hydrate from its initially low moisture content. With the geomembrane above the GCL limiting moisture uptake from the surface, the GCL must hydrate by taking moisture from the foundation soil below. Numerical simulations of hydration showed that in an isothermal closed system, the foundation soil sets the suction value towards which the GCL migrates. In this paper the impact of normal stress on the rate of hydration and equilibrium moisture content is investigated numerically. The laboratory tests were completed with 2 kPa normal stress, however, in a landfill, GCL hydration could occur at significantly higher normal stresses. Therefore the question arises whether additional normal stress will aid or hinder GCL hydration. The results showed the impact of normal stress is to constrain swelling of the GCL which will reduce the equilibrium moisture content as well as reduce the time to achieve equilibrium. This provides additional motivation to cover the composite barrier system in a timely manner within the construction process.

KEYWORDS: Geosynthetic clay liner, hydration, numerical simulations, parametric study, landfill, barrier systems

1 INTRODUCTION

Geosynthetic clay liners (GCLs) are often used in combination with a geomembrane (GMB) as a composite landfill liner system. For a GCL to function as a hydraulic barrier, however, it must hydrate from its initially low moisture content. The source of this moisture cannot be from the atmosphere as the GMB barrier is installed on top of the GCL. Instead, a GCL must hydrate with moisture from the foundation soil below the GCL. This is not to say that the GMB does not play a significant role in defining the hydration behaviour of a GCL. If the surface of the GMB is left exposed to solar radiation during construction of the landfill liner, peak daily temperatures have been observed in the range of 60-75 °C for black GMBs (Pelte et al. 1994; Chappel et al., 2012; Rowe et al., 2012). These daily thermal cycles have been shown in the laboratory to suppress the ability of GCLs to significantly hydrate (Rowe et al. 1994; Chappel et al., 2012; Rowe et al., 2012). These daily thermal cycles have been shown in the laboratory to suppress the ability of GCLs to significantly hydrate (Rowe et al. 2011).

If best practice is followed and the GMB is covered in a timely fashion with a granular protection layer or leachate collection system, the combination of a lack of direct contact with solar radiation and the thermal insulation provided by these layers of granular material will result in a much less severe thermal regime allowing GCL hydration to occur. Because these layers are designed to be as thin as possible (i.e. maximising landfill volume), GCL hydration in this scenario could be viewed as occurring under low confining stresses and relatively isothermal conditions. Research quantifying the rate of GCL hydration under these conditions was performed by Rayhani et al. (2011) which showed that GCLs readily hydrate from the subsoil under these conditions with the rate of hydration and final equilibrium water content of the GCL varying on the initial moisture content of the foundation soil.

The key to quantifying the moisture uptake and retention behaviour of GCLs is the material’s unsaturated water retention curve (WRC). The water retention behaviour of GCLs at low confining stresses has recently been quantified using measurements of GCL suction using high capacity tensiometers and relative humidity sensors (Beddoe et al., 2010) for four geotextile-encased GCLs containing granular bentonite (Beddoe et al., 2011). The results of this study have indicated that the method of GCL manufacture, in particular the presence or absence of a scrim reinforced nonwoven carrier geotextile and thermal treatment, can have a significant impact on the water retention behaviour of GCLs. Using the water retention curves (WRCs) defined by Beddoe et al. (2011), Siemens et al. (2012) performed an unsaturated numerical modelling parametric study of the experimental isothermal hydration experiments of Rayhani et al. (2011). The results of this analysis are shown schematically in Figure 1. The underlying foundation soil was observed to act as a suction boundary condition. As a result, the soil followed a drying path, whereas the GCL followed its wetting path until an equilibrium suction value was reached. Due to the relatively small mass of water required to hydrate the GCL compared to the mass of moisture held in the soil column, the equilibrium suction value was observed to be that represented by the foundation soil’s initial moisture content.
the increased confining stress will reduce the height of the GCL and its hydration behaviour. In particular, confining stress will have a similarly significant impact on the WRC of the GCL after it is fully swollen to its saturated moisture content when subjected to the higher confining stress associated with waste placement. Based on the effect of higher normal stress on other geomaterials (e.g., Fredlund and Pham 2006) it is hypothesised that confining stress will have a similarly significant impact on the WRC of the GCL and its hydration behaviour. In particular, the increased confining stress will reduce the height of the GCL after it is fully swollen to its saturated moisture content when compared to a similar GCL at a nominal 2 kPa confining stress (Figure 1). A GCL hydrated at high confining stress will therefore have a lower void ratio than a GCL hydrated at low confining stress. This change in the pore structure of the encased bentonite of the more confined GCL will result in a reduction in the GCLs saturated conductivity and an increase in the air entry value of the GCL from the values measured at low confining stress.

Although there being clear indications that confining stress will significantly impact the WRCs of GCLs, there is currently a paucity of WRC data for GCLs at higher confining stresses (the one exception being the two GCL specimens tested by Southen and Rowe, 2007). As a result, the practical impact (if any) of confining stress on the magnitude and rate of hydration is not immediately clear. However, data exists that describes the water retention behaviour of GCLs at low confining stress, and the consolidation behaviour of saturated GCLs to confining stress. In this paper we report a numerical sensitivity analysis to quantify the potential impact higher normal stress could have on the rate of hydration of GCLs to assess whether it is likely to have any practical impact on the hydration behaviour of GCLs assumed in landfill design.

2 MODEL DESCRIPTION

The effect of confining stress on the moisture uptake of GCLs was investigated for a typical geotextile-encased GCL consisting of a woven cover and a non-woven carrier geotextile with the material properties listed in Table 1. The water retention behaviour of this GCL has been quantified at a low (2 kPa) confining stress by Beddoe et al. (2011). The WRC relationship for this GCL and a typical silty-sand foundation soil are listed in Table 2 and plotted in terms of volumetric water content against suction in Figure 2.

The material properties for the GCL at high confining stress (Figure 2 and Table 2) were estimated from experimental results. Southen and Rowe (2007) reported drying WRCs for the GCL at 100 kPa vertical stress, which allowed comparison with the low stress data. Comparing the fitted curves indicated an increase in the ‘a’ parameter of 100 kPa was necessary to match the results (Table 2). Beddoe et al. (2011) compared WRCs of various GCLs and reported that at suction values greater than approximately 3000 kPa, the WRCs were similar. Therefore the ‘a’ parameter for the wetting curve was increased by two orders of magnitude as listed in Table 2. The ‘m’ and ‘n’ parameters were adapted to ensure the WRC agreed with other GCLs for higher suction values. Lake and Rowe (2000) reported on the compressibility and swell behaviour of GCLs. The results were used to estimate the height and saturated porosity of the hydrated GCL at 100 kPa vertical stress. Finally, conductivity data obtained at various confining stresses (Rowe and Hosney, 2013) was used to estimate the saturated conductivity at 100 kPa.

The finite element model (Figure 3) procedure was described in detail earlier (Siemens et al. 2012) and a brief summary will be given here. The initial moisture conditions are set in the GCL and foundation soil and then the model steps forward in time under closed conditions. As-manufactured the GCL is at nominal moisture content and approximately 105 kPa suction. Under closed isothermal hydration the foundation soil undergoes minor drying while the GCL approaches the suction boundary condition provided by the soil. At equilibrium the system achieves a hydrostatic state.

Table 1. Properties of GCL1

<table>
<thead>
<tr>
<th>Property</th>
<th>GCL</th>
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<tr>
<td>Average GCL mass per unit area (g/m²)</td>
<td>Measured 4679, Minimum Acceptable Roll Value 3965</td>
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<td>Mass per unit area (g/m²)</td>
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<tr>
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<tr>
<td>Mass per unit area (g/m²)</td>
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<tr>
<td>Bentonite As-delivered form</td>
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<tr>
<td>Montmorillonite content (%)</td>
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</tr>
<tr>
<td>Structural</td>
<td>Needle-punched</td>
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Table 2. GCL and foundation soil Fredlund and Xing fitting parameters for water retention curves and saturated hydraulic conductivity values.

<table>
<thead>
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<th>Parameter</th>
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<th>GCL 100 kPa</th>
<th>Soil</th>
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<tr>
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<td>1310</td>
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<tr>
<td>n</td>
<td>0.84</td>
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<tr>
<td>m</td>
<td>0.51</td>
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<td>0.78</td>
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<tr>
<td>ψ (kPa)</td>
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<td>5856</td>
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<tr>
<td>Saturated VWC (%)</td>
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<td>0.39</td>
</tr>
<tr>
<td>Ksat (m/s)</td>
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<td>3.5x10⁻¹³</td>
<td>9.5x10⁻⁴</td>
</tr>
</tbody>
</table>
3 RESULTS

3.1 Typical Model Results – GCL on Soil at GWC=16%

A typical model result is illustrated in Figure 4. The low confining stress model (Siemens et al. 2012) is also included for comparison. The foundation soil has an initial suction of 27.6 kPa while the GCL initial suction is set to 362,000 kPa. These conditions are representative of a foundation soil compacted to a constant moisture content with the GCL immediately installed following compaction. In Figure 4a both GCLs start at their initial low VWC and increase to similar equilibrium moisture content with the higher confining stress model achieving equilibrium in a shorter duration. From Figure 4b, both GCLs plot along their wetting curves and the reason for the similar equilibrium VWC is apparent. The soil provides the suction level to which the GCLs migrate. For the suction level applied in this model, the GCL WRCs of the 2 kPa and 100kPa cross, which results a similar equilibrium VWC.

Figure 3. Finite element model and boundary conditions.

Figure 4. Typical results for foundation soil at 16% gravimetric moisture content: a) GCL volumetric water content versus time and b) unsaturated paths plotted on WRCs.

3.2 Model Results

The calculated uptake of moisture during GCL hydration is shown in Figure 5a in terms of degree of saturation versus time. Relationships are shown for GCL hydration occurring at either 2kPa or 100 kPa normal stress for a range of four foundation soil moisture contents. As observed by Siemens et al. (2012), the equilibrium degree of saturation of a GCL increases with foundation moisture content. The effect of normal stress can be observed by comparing the hydration behaviour at 2 kPa (closed symbols) and 100 kPa (open symbols). For each of the four foundation moisture contents examined in this study, the GCL hydrating at a higher confining stress was observed to achieve a higher degree of saturation than the unconfined GCL. The magnitude of this difference is shown in Figure 5b expressed as the ratio of $S_{r100kPa}/S_{r2kPa}$ at their equilibrium state of hydration. This data indicates that although GCLs at both confining stresses will achieve their near-fully saturated state on the wettest of foundation soils, at progressively lower foundation moisture
contents, the GCL hydrated under 100 kPa confining stress will be at an increasingly higher degree of saturation.

For a given foundation soil moisture content, a GCL hydrating at a higher confining stress will achieve a higher degree of saturation than an unconfined GCL. The magnitude of this difference is a function of the foundation moisture content. This is due to the change in shape of the WRC resulting from the decrease in saturated VWC and increase in air entry value.

The objective of the parametric study was to assess the likely impact of confining stress on the hydration of GCLs in landfill liner applications. The results of these analyses indicate that confinement will play a significant role on both the rate and magnitude of hydration in GCLs, warranting further laboratory studies to experimentally quantify the WRCs for typical GCLs at a higher confining stress. In the absence of this specific information, the analyses indicate that the use of WRCs defined for GCL hydration at low confining stress will be generally conservative (i.e. underpredict) the rate and magnitude of hydration for GCLs at higher confining stresses.

5 REFERENCES


4 CONCLUSIONS

The lack of unsaturated water retention curve data for GCLs at high confining stresses has made the assessment of the effect of confining stress on the magnitude and rate of hydration currently unclear. In this paper, existing data describing the water retention behaviour of GCLs at low confining stress has been combined with data describing the consolidation behaviour of saturated GCLs to perform a numerical sensitivity analysis to investigate whether higher confining stresses are likely to have a practical impact on the hydration behaviour of GCLs in landfill design.

The numerical analyses indicate:

1. GCLs hydrating at high confining stresses will hydrate faster than a GCL at lower confining stresses for lower suction ranges. GCLs at higher confining stresses have the benefit of having a higher suction where the saturated hydraulic conductivity is achieved and their thinner thickness results in a shorter flow path distance.

2. For a given foundation soil moisture content, a GCL hydrating at a higher confining stress will achieve a higher degree of saturation than an unconfined GCL.

The numerical analyses indicate:

- The lack of unsaturated water retention curve data for GCLs at high confining stresses has made the assessment of the effect of confining stress on the magnitude and rate of hydration currently unclear. In this paper, existing data describing the water retention behaviour of GCLs at low confining stress has been combined with data describing the consolidation behaviour of saturated GCLs to perform a numerical sensitivity analysis to investigate whether higher confining stresses are likely to have a practical impact on the hydration behaviour of GCLs in landfill design.

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5 REFERENCES


Soil chart, new evaluation method of the swelling-shrinkage potential, applied to the Bahlui’s clay stabilized with cement.

L’empreinte du sol, une nouvelle méthode d’évaluation du potentiel de gonflement, appliquée à l’argile de Bahlui stabilisée avec du ciment.

Stanciu A., Aniculaesii M., Lungu I.
The “Gheorghe Asachi” Technical University from Iaşi, România

ABSTRACT: Clays, in both their remoulded and natural state, can present a certain swelling-shrinkage potential. A specific behaviour characterized by extreme swelling and shrinkage values is exhibited by the colloidal clays such as London’s, Swellhaven’s, Cambridge’s clay. Bahlui’s clay from Romania fits the same domain, based on the authors’ proper tests. The indirect evaluation of the Bahlui’s clay swelling potential, besides using the classical methods has been performed also based its own “chart”. By consequence, the variation of the swelling potential was analyzed in this paper, based on the indirect methods (Seed 1962, Van der Merwe 1964) and as novelty, also based on the activity coefficient (CA) (Stanciu et al. 2011) for the Bahlui’s clay both in its natural state and mixed with Portland cement and respectively with ecologic cement. The laboratory test results indicated the decrease of the swell-shrinkage potential presenting reduced values of $CA = 0.58 \div 0.18$.

RÉSUMÉ: Les argiles, à la fois naturelles et restructurées, disposent d’un certain potentiel de retrait-gonflement. Un comportement spécifique des argiles coloidales telles que celles de Londres, de Swellhaven, de Cambridge, etc. est caractérisé par des valeurs extrêmes du gonflement et du retrait. Dans la même catégorie, basée sur des mesures propres, on peut aussi encadrer l’argile de Bahlui, de Roumanie. L’évaluation indirecte du potentiel de gonflement de l’argile de Bahlui, avec des méthodes traditionnelles, a été effectuée sur la base de son “empreinte”. Dans cet article, basé sur des méthodes indirectes (Seed 1962, Van der Merwe 1964) et comme nouveauté, basé sur l’empreinte des sols (Stanciu et al. 2011), nous avonsanalysé la variation du potentiel de gonflement de l’argile de Bahlui, à la fois naturelle et mélangée avec de ciment de Portland et avec du ciment écologique. Les résultats ont indiqué une diminution du potentiel de retrait-gonflement mis en évidence par une diminution des valeurs $CA = 0.58 \div 0.18$.

KEYWORDS: expansive clay, swell potential, soil chart, activity coefficient, stabilization with ecologic cement

1 INTRODUCTION

The swell potential evaluation for a certain soil has a very significant importance in the mitigation and limitation of potential structural damages of future constructions founded on active soils.

Among soils with special behaviour, the expansive ones take an important place due to their volume variations determined by moisture variations. These volume variations are reflected in soil differential swells/shrinkages due to soil uneven drying or wetting processes developed both beneath foundations and near them. Various solutions such as chemical stabilization techniques have been developed with more or less satisfactory results to limit the potential degradations of constructions on these soils (Gueddouda et al. 2011). The objective of the expansive soils stabilization is to reduce the swell potential within acceptable limits. The expansive soils stabilization with Portland cement reduces the swell potential as well as increases their mechanical strength. The substitution of the Portland cement in soil stabilization with 50% eco-cement has lead to promising results. The substitution solution of the Portland cement with eco-cement has the purpose of reducing the negative environmental impact, especially due to the manufacturing process of the Portland cement. The direct determination of the physical parameters that characterize the expansive soil behaviour such as the swell pressure, free swelling, volumetric shrinkage and of other properties takes a long time and is costly, due to the rather complex laboratory works. By consequence, it is necessary to develop and use simple and fast estimation methods for the swell potential. There is an important number of empirical methods to estimate the swell potential available in publications. It is generally considered that the swell potential is directly correlated with: the Atterberg limits ($w_p$, $w_L$), he colloidal clay fraction ($A_{2\mu}$), the shrinkage limit ($w_s$), the dry density ($\rho_d$), etc. These correlations are currently semi-empirical and based on statistics (Yilmaz 2006).

2 EVALUATION METHODS OF THE SWELL POTENTIAL

There are many methods to determine the parameters that characterize the swell-shrinkage of clays. The most utilized parameter to estimate the swell-shrinkage potential is the plasticity index ($I_p$). It is well known that the swell-shrinkage potential is dependent on the granulometric composition of the investigated clay ($A_{2\mu}$) as well as on the specific available surface for the interface phenomena display (Stanciu 2006).

Numerous attempts have been made to find an acceptable system to evaluate the swell potential. The most utilized systems for the indirect evaluation and classification of the swell potential are based on the graphical correlation between two or more geotechnical indices (Van Der Merwe – Figure 1 and respectively Casagrande – Figure 2).

The principal physical properties of the representative clays investigated in this research are presented in Table 1.

The swell potential has been estimated based on these properties reflected by the indices ($A_{2\mu}/w_s/I_p$) and their representation on the diagrams in Figures 1 and 2. The swell potential of the investigated clays, established using the Van Der Merwe’s and respectively Casagrande’s diagrams is presented in Table 2. The domains of the swell potential, from very high to low, within these two diagrams, have generally a conventional representation on a (0-1) or (0-100) scale.
reflects the attempt to correlate the degradation magnitude of constructions founded on such soils, with the intensity of the swell potential (Figure 1 and Figure 2) (Das 1995).

Table 1. The physical properties of the representative investigated clays.

<table>
<thead>
<tr>
<th>Soil</th>
<th>Liquid limit (%)</th>
<th>Plasticity index (%)</th>
<th>Colloidal fraction (%)</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hubballi clay- India</td>
<td>51–71</td>
<td>32–53</td>
<td>51–56</td>
<td>Hakari, 2010</td>
</tr>
<tr>
<td>Ankara clay- Turkey</td>
<td>64–75</td>
<td>29–34</td>
<td>39–55</td>
<td>Hakari, 2010</td>
</tr>
</tbody>
</table>

It was noticed though, both from the published data and from the results in Table 2 that these two approaches (Van Der Merwe and Casagrande) using only two indices ($I_P$, $w_L$), indicate different magnitudes of the swell potential for the same soil.

By consequence, the Romanian Norms (STAS 1913/12-88 and Code N.E. 0001-96) propose the use of the “soil chart” to identify and characterize the expansive soils. This representation, considered unique (Andrei 1980) and specific for each soil, is obtained by joining the points I; II; III; IV and V on a composite diagram by assembling on the same graph, the Casagrande-Chleborad diagram, the Skempton-Van Der Merwe diagram and the granulometric curve (Figure 3). The size of the figure area (I, II, III, IV and V), ($A_N$) constitutes a first criterion to characterize the soil swell potential (Andrei 1997). The reference circle that intersects the 50% $w_L$, $I_P$, $X_{15}$ points and the 1mm diameter as the point of interest on the diameter axis is introduced to scale the graph on the four axes. The normalized surface is calculated ($A_N$) using the reference circle area ($A_{circle}$) with the formula:

$$A_N = A / A_{circle}$$

It is defined an activity coefficient ($C_A$), constructed similarly as the consistency index ($I_C$), to evaluate the magnitude of the swell potential (Stanciu et al. 2011), fixing as the variation limits, the chart minimum and maximum areas for two soils with extreme behaviour: with maximum content of kaolinit, as with the lowest potential, and of sodium montmorillonite as for soils with maximum volume variations.

By consequence, the activity coefficient $C_A$ for the normalized area of the soil chart $A_N$ is given by the relation:

$$C_A = (A_N^o - A_N) / (A_N^o - A_M)$$

where: $C_A$ – the activity coefficient; $A_N^o$ - the normalized area of the chart referring to the investigated soil; $A_N^o$ - the normalized area of the kaolin chart; $A_M$ - the normalized area of the sodium montmorillonite chart.

The clay swell potential can be classified based on the value of the activity coefficient as presented in Table 3.

Table 3. The classification of the soil swell potential based on the activity coefficient, after Stanciu 2011

<table>
<thead>
<tr>
<th>Activity coefficient $C_A$</th>
<th>Swell potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 0.24</td>
<td>low</td>
</tr>
<tr>
<td>0.25 + 0.49</td>
<td>medium</td>
</tr>
<tr>
<td>0.50 + 0.74</td>
<td>high</td>
</tr>
<tr>
<td>0.75 + 1.00</td>
<td>very high</td>
</tr>
</tbody>
</table>

The example of this new evaluation procedure for the clay swell potential is presented in Figure 3, where the average charts have been plot for four representative clays: from London, Hubballi, Ankara and Bahlui-Romania, together with the corresponding Romanian kaolinitic and montmorillonitic clays. The values of the normalized area $A_N$ and respectively the values of the activity coefficient $C_A$ have been calculated for each soil, based on the plotted charts from Figure 3 (Table 4).

The resulted swell potential from the classification based on the activity coefficient $C_A$, referring to the diagrams of

Figure 1. The classification of the swell potential, after Van Der Merwe, 1964.

Figure 2. The classification of the swell potential based on the Casagrande’s plasticity chart.

Table 2. The classification of the investigated clays.

<table>
<thead>
<tr>
<th>Soil</th>
<th>Van Der Merwe</th>
<th>Casagrande</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bahlui clay- Romania</td>
<td>Very high</td>
<td>Very high</td>
</tr>
<tr>
<td>London clay-U.K.</td>
<td>Very high</td>
<td>Very high</td>
</tr>
<tr>
<td>Hubballi clay- India</td>
<td>Very high</td>
<td>Very high</td>
</tr>
<tr>
<td>Ankara clay- Turkey</td>
<td>High</td>
<td>High</td>
</tr>
</tbody>
</table>
Skempton-Van Der Merwe, Casagrande-Chleborad and the granulometric curve has been reduced from high to medium for the London, Hubbali and Ankara clays, and from very high to high for the Bahlui clay (Tables 2 and 4).

Methods to reduce the swell potential is the chemical engineering practice on such sites. One of the most used expansive soils to overcome the difficulties in foundation stabilization using Portland cement.

Many methods exist to reduce the swell-shrinkage potential of expansive soils to overcome the difficulties in foundation engineering practice on such sites. One of the most used methods to reduce the soil swell potential is the chemical stabilization using Portland cement. Ecological issues resulting from the end-product pollutants related to the manufacturing of Portland cement lead to the development of new chemical binders named eco-cements.

This paper intends, apart from the introduction of the index (\(C_A\)) characterizing the swell potential, to reduce this potential by introducing new chemical binders that would provide less environmental pollution during manufacturing. Thus, a comparative analysis has been performed regarding the Bahlui clay stabilization with Portland cement and respectively with Portland and eco-cement mixture.

During the first testing series, the Portland cement participation was 2.5 - 10% from the soil dry mass. For the second testing series, the previous Portland cement amount was 50% substituted with eco-cement.

The results quantified by the main parameters (\(w_L\), \(w_P\), \(I_p\), \(A_{30}\)) are presented in Tables 5a and 5b.

Table 5a. The physical properties of the Bahlui clay stabilized with Portland cement.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Bahlui clay stabilized with Portland cement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2.5%</td>
</tr>
<tr>
<td>(w_L) (%)</td>
<td>88.3</td>
</tr>
<tr>
<td>(w_P) (%)</td>
<td>35.5</td>
</tr>
<tr>
<td>(I_p) (%)</td>
<td>52.8</td>
</tr>
<tr>
<td>(A_{30}) (%)</td>
<td>78</td>
</tr>
</tbody>
</table>

The charts of the stabilized Bahlui clay with Portland cement (Figure 4) and with the mix (50% Portland cement and 50% eco-cement) (Figure 5) were plotted based on the obtained results.

The new material structure will display a reduced swell potential determined based on the activity coefficient for a 10% Portland cement mix (Table 6a).

![Figure 3. The average soil charts of the investigated clay.](image)

![Figure 4. The chart of the Bahlui clay stabilized in various percentages with Portland cement.](image)
analyses and plasticity limits determination (Figure 5).

Table 6a. The swell potential of the Bahlui clay stabilized with Portland cement.

<table>
<thead>
<tr>
<th>Bahlui clay with cement</th>
<th>Normalized area (A^*)</th>
<th>Activity coefficient (C_s)</th>
<th>Swell potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>0% CP*</td>
<td>5.198</td>
<td>0.578</td>
<td>high</td>
</tr>
<tr>
<td>2.5% CP*</td>
<td>5.162</td>
<td>0.571</td>
<td>medium</td>
</tr>
<tr>
<td>5% CP*</td>
<td>4.321</td>
<td>0.395</td>
<td>medium</td>
</tr>
<tr>
<td>7.5% CP*</td>
<td>3.756</td>
<td>0.277</td>
<td>medium</td>
</tr>
<tr>
<td>10% CP*</td>
<td>3.185</td>
<td>0.157</td>
<td>low</td>
</tr>
</tbody>
</table>

(*) Portland cement

Table 6b. The swell potential of the Bahlui clay stabilized with Portland and eco-cement.

<table>
<thead>
<tr>
<th>Bahlui clay with cement</th>
<th>Normalized area (A^*)</th>
<th>Activity coefficient (C_s)</th>
<th>Swell potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>0% cement*</td>
<td>5.198</td>
<td>0.578</td>
<td>high</td>
</tr>
<tr>
<td>2.5% cement*</td>
<td>4.729</td>
<td>0.48</td>
<td>medium</td>
</tr>
<tr>
<td>5% cement*</td>
<td>4.109</td>
<td>0.35</td>
<td>medium</td>
</tr>
<tr>
<td>7.5% cement*</td>
<td>3.983</td>
<td>0.324</td>
<td>medium</td>
</tr>
<tr>
<td>10% cement*</td>
<td>3.272</td>
<td>0.175</td>
<td>low</td>
</tr>
</tbody>
</table>

(*) Portland and eco-cement with 50% participation each

The chart of the stabilized Bahlui clay has been plotted for each cement percentage used in the mix (Portland and eco-cement) using the results obtained from the granulometric analyses and plasticity limits determination (Figure 5).

- The ecologic cement presents a smaller influence than the Portland cement.

4 CONCLUSIONS.

The comparative analysis on the evaluation of the swell potential of the London, Hubbali, Ankara and Bahlui-Romania clays confirmed, as presented in previous publications, that the use of only two indices \(I_p\), \(w_l\) or \(A_2\mu\), may lead to different classifications of the swell potential of active clays.

The assemblage in one representation of the Skempton-Van Der Mewe diagram, the Casagrande-Chileborad diagram and the granulometric curve has made possible the plotting of a specific soil “print” for each soil. On its basis, a “unifying” coefficient has been defined \(C_s\), namely the soil’s activity coefficient. Thus, active soils can display a low, medium, high and very high swell potential.

This coefficient was also utilized to study the influence of the Portland and ecologic cement stabilization on the swell potential of the Bahlui clay from Romania. Stabilization with the Portland or the ecologic cement presents a decrease of this potential from high to low, along with the increase of the cement participation from 2.5% to 10%.

The eco-cement is not providing a significant decrease of the swell potential by comparison with the Portland cement, but it reduces the environment pollution during the manufacturing process.

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Andrei S. and Manea S. 1997. La systematisation, le stokage,et la reutilisation de information geotechniques, Revue francaise de Geotechnique 78, 51-61


It can be noticed, based on the analysis of the data from Tables 6a and 6b:
- the reduction of the swell potential along with the Portland cement increase;

Figure 5. The chart of the Bahlui clay stabilized with various percentage of cement mix (50% Portland cement and 50% eco-cement).
Measurement of Unsaturated Ground Hydraulic Properties using a Dynamic State Soil Moisture Distribution Model

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ABSTRACT: This paper presents a new in-situ test for producing a moisture characteristic curve and estimating unsaturated hydraulic conductivity using a reduced number of measurement parameters. Measurement parameters are restricted to determining the amount of infiltration and the pressure head, and can be used to determine the hydraulic properties of unsaturated soils. The model expresses the dynamic state of moisture distribution using a sigmoid function, and does not require measurement of the amount of soils moisture. Experimental results showed that suction of the moisture characteristic curve was small and estimated based on the influence of pore air pressure, and the values obtained for saturated and unsaturated hydraulic conductivity were in agreement with the results of laboratory tests.

RÉSUMÉ : Nous présentons ici la mise en œuvre d'une expérience in situ, destinée à définir une courbe caractéristique de l'humidité et la conductivité hydraulique d'un sol non saturé. Cette méthode, basée sur un modèle de distribution proposé par les auteurs a l'avantage d'éliminer un paramètre de mesure. Les paramètres de mesure nécessaires pour déterminer la conductivité hydraulique du sol non saturé sont seulement le degré d'infiltration et la pression hydraulique du haut. Le modèle développé par les auteurs utilise une courbe sigmoïde de mesure de distribution de l'humidité dynamique, ceci permet de supprimer la mesure du taux d'humidité. Nous avons comparé la succion obtenue pour la courbe caractéristique d'humidité et observé qu'elle est un peu plus faible sous l'influence de la pression d'air des pores, mais nous pouvons conclure que nos résultats de conductivité hydraulique, en sol saturé et non saturé, sont comparables aux résultats expérimentaux en laboratoire.

KEYWORDS: Unsaturated soils, Hydraulic conductivity, In-situ test, soil water characteristic curve

1 INTRODUCTION

Hydraulic conductivity of the ground depends on the degree of saturation. Since boundary conditions vary, hydraulic conductivities in in situ tests are classified into four categories; unsaturated and saturated hydraulic conductivities of soils below the groundwater level, which are usually saturated, and unsaturated and saturated hydraulic conductivities of soils that are above the groundwater level, which are usually unsaturated. In embankments or slopes where groundwater levels are deep, soils are typically in an unsaturated state. However, intense rainfall and seepage water change the soil condition from being unsaturated to saturated. Consequently, the development of an in-situ test for evaluating the hydraulic properties of saturated and unsaturated soils is required. Current in situ permeability tests of unsaturated sediments require special equipment, and the method used to saturate the foundation in such tests is often difficult. These problems of soils have made it difficult to develop effective techniques for assessing the hydraulic characteristics of soils in practice.

This study therefore sought to develop a simple method for evaluating the hydraulic properties of foundations, which are typically unsaturated. This method only measures osmotic flow and pressure head, but it is capable of estimating hydraulic conductivity and producing a moisture characteristic curve of the unsaturated foundation by using a dynamic state moisture distribution model developed by the authors.

2 WETTING AND SATURATION FRONTS

In order to clarify the hydraulic properties of foundations in-situ, a simple examination method is desirable. A simple permeation examination was therefore used for increasing the degree of saturation of the foundation. Wetting fronts advance within foundations and increase the degree of saturation in permeation tests. Yong (1975) showed that the distance over which a wetting front advances in a one-dimensional lateral flow is proportional to the square root of time. They also observed that the relation was established under conditions of one-dimensional vertical flow. In this study, we referred to the depth of boundary that reaches saturation from being unsaturated in the one-dimensional vertical flow as the “saturation front” (Fig.1). We then used a numerical simulation to determine whether the square root of time could be applied to estimate the depth of the saturation front. The results of the numerical simulations are shown in Fig. 2.

Figure 1. Permeation test in one-dimensional vertical flow and variables used in permeation test.
The numerical simulations showed that the depth of a saturation front was proportional to the square root of time when a constant water level of 10 cm was applied. It is considered that nonlinearity arises in the second half of the experiment due to the presence of silt. Fig. 3 shows the experimental results obtained when water moving through sand was observed using four-point moisture sensors. Since the time of onset of watering is not clear, there is a gap in the range and time, but the results obtained by numerical simulation and in the experiments produced the same results.

These data show that the depth of a saturation front using one proportionality factor and the following equation:

\[ H_s = s \sqrt{t} \]  

where, \( H_s \) is the depth of saturation front, \( t \) is elapsed time and \( s \) is the regression parameter.

3 OUTLINE OF PERMEATION TEST

3.1 Experimental apparatus and measured parameters

A soil chamber was used to imitate the experimental apparatus in-situ (Fig. 5). As shown in Fig. 4, the main part of the experimental apparatus has a Marriott tank and a circular water supply part. The circular water part is inserted into the foundation and a perpendicular flow is maintained. The tensiometer which measures pore pressure was installed at a depth of 50 mm in the centre of the water supply part. In order to obtain a constant pressure with the water supply part, the difference in the hydraulic head was set to 100 mm using the Marriott tank.

4 ESTIMATED MOISTURE DISTRIBUTION

4.1 Linear approximation of the soil water distribution

In order to calculate the hydraulic conductivity of unsaturated ground, information on the degree of saturation is required. In particular, in conditions of unsteady flow, since the amount of moisture changes with depth, it is necessary to accurately determine the distribution of moisture over time. The authors therefore assumed a moisture distribution changes over time using the relation between Fig. 1 and Eq. 1. It is assumed that any infiltration assumes a trapezoidal distribution consisting of two components; the depth of the saturation front
from ground surface and the depth of the wetting front. The saturation front in t hours can be calculated from Eq. 1. The wetting front can be determined by assuming by that it is equal to the trapezoidal area and the accumulation amount of infiltration. It is considered that the saturation front was 50 mm because the pressure head will be 0 mm at the tensiometer installed to a depth of 50 mm. The regression parameter of Eq. 1 can be estimated as follows:

\[
H_f = \frac{8Q}{(\theta_f - \theta_i)RD^2} - H_s ,
\]

where \(\theta\) is the volumetric water content of saturated soil and \(Q\) is the amount of infiltration, and \(D\) is the radius of cylinder.

4.2 Presumption of the moisture distribution using dynamic state soil moisture distribution model

We performed a separate numerical simulation to verify whether the moisture distribution had a trapezoidal distribution. The result is shown in Fig. 7. In comparison with numerical experimental results the inclines of the soil moisture distribution that was approximated with a trapezoid increase and become estranged. Moreover, the depth of the wetting front differed from the result obtained in the numerical analysis.

On the other hand, the trapezoid distribution accords with the numerical experimental result at the depth of the mean soil moisture. The authors therefore selected the dynamic state moisture distribution model to model moisture distribution (Eq.3 and Fig. 8). In this model, parameter \(a\), which indicates the depth of the average moisture, and parameter \(b\), which moisture \(\theta_m\), is \((\theta_f - \theta_i)/2 + \theta_i\), the conditions of \(\exp(0) = 1\) are acquired. Therefore, if the depth \(zm\) is used as the average moisture, then Eq.4 can be derived as follows:

\[
\theta = \frac{\theta_f - \theta_i}{1 + \exp(a + bz_m)} + \theta_i
\]

\[
a + bz_m = 0
\]

\[
\frac{\partial \theta}{\partial z} = -\frac{(\theta_f - \theta_i)\exp(a + bz)}{(1 + \exp(a + bz))^2}
\]

\[
\left.\frac{\partial \theta}{\partial z}\right|_{z_m} = -\frac{(\theta_f - \theta_i)\theta_f}{4}
\]

where, \(a\) and \(b\) are parameters of the dynamical moisture distribution model. Furthermore, the slope of the moisture distribution differentiates an Eq.3 with respect to \(z\), which can be used to obtain Eq.5. Since Eq.5 is equivalent to the slope (Eq.6) of the moisture distribution expressed using the depth of a saturation front and \(a\) wetting front, parameter \(b\) can be obtained.

Parameter \(a\) can be obtained using Eq.4 and the parameters of an Eq.3 are identified. The result of the experimental and numerical moisture distributions is shown in Fig. 9. The dashed line shows the moisture distribution of the numerical experimental result, the solid line the estimated moisture distribution from Eq.3. Fig. 9 shows that both are in agreement. The experimental result in the permeation test (Fig.4) obtained
using the dynamic state moisture distribution model is shown in Fig. 10.

5 MOISTURE DISTRIBUTION IN RESPONSE TO HYDRAULIC PROPERTIES

5.1 Moisture characteristic curve

Fig. 11 shows the relationship between the water volume presumed from the pressure head and moisture distribution model for a depth of 50 mm, which is installing the tensiometer. Moreover, what showed the relation of the moisture characteristic from these is shown in Fig. 11. The moisture characteristic curve obtained in the laboratory experiments for reference is also shown in Fig. 12. Compared to the laboratory experiments, the absolute value of the pressure head was low. In order to carry out load of the water pressure of 100 mm to a permeation surface with the start of test, the air below a

5.2 Assumption of unsaturated hydraulic conductivity

The parameters of the van Genuchten model can be estimated by our moisture characteristic curve of Fig. 12. The unsaturated hydraulic conductivity obtained using the Mualem model and the estimated parameters is shown in Fig. 13. The result of the proposed method was in agreement with other laboratory experimental results. By using the van Genuchten and Mualem models, since parameter n of the moisture characteristic curve is used, as shown in an Eq.7, it is considered that the slope of the moisture characteristic curve was evaluated correctly.

\[ k_{\text{unsat}} = k_{\text{sat}} \cdot \left( \frac{\theta}{\theta_s} \right)^{0.5} \left( 1 - \left( \frac{\theta}{\theta_s} \right)^n \right)^{1-n/\alpha} \]  

where, \( S_r = (\theta - \theta_d)/(\theta_s - \theta_d) \), \( k_{\text{unsat}} \) is the hydraulic conductivity of unsaturated soil, \( k_{\text{sat}} \) is the hydraulic conductivity of saturated soil, and \( n \) is the parameter of the Genuchten and Mualem model.

6 CONCLUSIONS

The experimental results of the permeation tests and dynamic state soil moisture distribution model, the following conclusions could be drawn:

1) When there is no volume change, not only the wetting front but the depth of the saturation front is proportional to the square root of time.

2) The measurement values obtained for the amount of infiltration by permeation and the pore pressure of one point, can be used to calculate the parameters of a dynamic state moisture distribution model.

3) Hydraulic properties of the unsaturated foundation could be evaluated using the observed value of the presumed moisture distribution and pore pressure.

4) The pressure head of the moisture characteristic curve was affected by gap air and shifted to positive pressure; this needs to be improved in the future.

7 ACKNOWLEDGEMENTS

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New devices for water content measurement

Les appareils nouveaux pour la mesure de la teneur en eau

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ABSTRACT: Two new devices for water content measurement are described: (i) an automated multi-electrode resistivity system and resistivity probe and (ii) a coiled TDR probe that can be used in conjunction with a high suction tensiometer. A flexible resistivity data acquisition system has been developed to acquire resistivity data using different arrays and which automatically switches electrodes interchangeably. A wide range of high precision reference resistors and soils have been used to test the system and the output data have been compared with a commercial resistivity system. The system developed has been used to investigate wetting and drying of clay using a new resistivity probe with a square electrode configuration that can be used for localised water content determination. The novel coiled TDR device uses a two-pronged TDR wrapped around the body of the Durham University high capacity tensiometer. The calibration of the device takes account of the contact with the tensiometer body. The device can be used with a steel bodied tensiometer and provides accuracy in volumetric water content of ±0.075 over a range of volumetric water contents of 0 to 0.9. A ceramic bodied device has also been investigated that does provide improved accuracy of ±0.047.

RÉSUMÉ : Deux appareils nouveaux pour la mesure de la teneur en eau sont décrites: (i) un système automatique multi-électrode de résistivité et une sonde de résistivité et (ii) une sonde TDR qui peut être utilisée en parallèle avec un tensiomètre de forte capacité de succion. Un système d'acquisition de données de résistivité a été développé pour acquérir des données de résistivité en utilisant des tableaux différents et qui commute automatiquement par électrodes interchangeables. Une large gamme de résistances de référence de haute précision et les sols ont été utilisés pour tester le système et les données de sortie ont été comparées à partir d'un système commercial de mesure de résistivité. Le système mis au point a été utilisé pour étudier l'humidification et le séchage de l'argile à l'aide d'un nouvel appareil de résistivité avec une configuration d'électrodes carrée qui peut être utilisée pour déterminer la teneur en eau localisée. Le dispositif de TDR utilise un TDR à deux volets enveloppé autour du corps d'untensiomètre de grande capacité de l'Université de Durham. L'étalonnage du dispositif tient compte du contact avec le corps de tensiomètre. Le dispositif peut être utilisé avec un tensiomètre en acier et offre une précision de la teneur en eau volumétrique de ± 0,075 sur une plage de teneurs en eau volumétriques de 0 à 0,9. Un dispositif en céramique a également été étudié qui donne une meilleure précision de ± 0,047.

KEYWORDS: Resistivity, TDR, water content

1 INTRODUCTION

An accurate knowledge of soil water content is crucial to understanding the impact of climate change on engineered earth structures. However, quantifying water content in unsaturated soils is difficult due to the complexity of unsaturated soil systems and the difficulties associated with gathering representative measurements. A large spectrum of techniques has been developed to measure soil water content. These include; neutron scattering, dielectric methods such as Time Domain Reflectometry (TDR) and Frequency Domain Reflectometry (FDR), capacitance probes and remote sensing techniques that provide measurements at regional scale. Robinson et al. (2008) and Vereecken et al. (2008) have presented detailed reviews of these techniques.

In geotechnical testing there is an increasing demand to develop efficient techniques to measure soil water content. Among the options available, TDR is becoming more widely used in geotechnical testing and electrical resistivity has also emerged as a cost effective and non-invasive tool to map the spatiotemporal variability of water content that cannot be provided by more traditional techniques (Zhou et al., 2001). In this paper, two new systems are described: (i) an automated multi-electrode resistivity system and resistivity probe (ii) a coiled TDR probe that can be used in conjunction with a high suction tensiometer to provide measurements of water content and suction at the same position. The devices have been developed to carry out experimental studies to monitor water content changes in unsaturated soil specimens submitted to drying and wetting cycles.

2. ELECTRICAL RESISTIVITY

2.1 Theoretical Background

An unsaturated soil is a multi-phase system consisting of air, water and soil grains. Electrical resistivity (the reciprocal of electrical conductivity) is an intrinsic physical property of a material that describes its ability to resist the ionic mobility in pore water. Since electrical conduction is mainly electrolytic and takes place through the pore water (Bryson, 2005), electrical properties of soils are mainly controlled by water content. A traditional four-electrode resistivity system therefore is based on the principle that the potential drop across a pair of electrodes due to a direct (DC) or low frequency current injected via another pair of electrodes is proportional to the electrical resistivity, that is:

$$\rho = K \times \frac{\Delta V}{I}$$  \hspace{1cm} (1)

Where, $\rho$ is resistivity (Ohm.m), the ratio of $\Delta V$, the potential drop (Volts), and $I$, the current (Amps), is the material resistance (Ohm). $K$ is a geometric factor (m) representing the electrode arrangement. For 2D and 3D resistivity studies, traditional four-electrode systems are time consuming and impractical. Therefore, the development of automated multi-
A multi-electrode resistivity system is based on the traditional four-electrode principle combined with automatic multiplexing for a larger number of electrodes (Damasceno et al., 2009). The system described here consists of: a constant current power source, a switching system and acquisition software. A 30V/2A programmable DC power supply type EL302P and MSL Datascan logger type 7220, both connected to a PC via RS 232 interface, were used to measure the voltage and log the current by measuring the voltage drop across a 1 Ω high precision shunt resistor. A similar approach has been adopted in commercial equipment e.g. MPT/ERT 2004 system from Multi-Phase Technologies, LLC (MPT) (LaBrecque and Daily, 2008).

Windows based data acquisition and control software named Resist has been developed to integrate the hardware and to control the data collection process. The user can set the current injected into the soil specimen and read the current, the voltage drop, and hence the resistance in a fully automatic procedure. To prevent electrode polarization (LaBrecque and Daily, 2008) short current pulses are used and an average reading (i.e. stacking) of a number of normal and reverse polarity readings are automatically acquired.

The aim of the laboratory testing described here was to check the data quality of the developed system. A wide range of high precision reference resistors (ASTM G57, 2006) was used to calibrate the system, and the measurements were compared with those acquired with a Terrameter SAS 300C (ABEM) system. The results are reported in Table 1. It can be seen that Resist gives better results than the commercial Terrameter with a maximum error of 0.8%.

### Table 1. A comparison between Terrameter SAS 300C system and Resist reading for a range of reference resistors

<table>
<thead>
<tr>
<th>Reference Resistor (Ohm)</th>
<th>Average reading (Ohm)</th>
<th>Percentage Error (%)</th>
<th>Average reading (Ohm)</th>
<th>Percentage Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>9.9</td>
<td>1.00</td>
<td>10.0</td>
<td>0.00</td>
</tr>
<tr>
<td>56</td>
<td>56.1</td>
<td>0.18</td>
<td>56.3</td>
<td>0.54</td>
</tr>
<tr>
<td>100</td>
<td>98.0</td>
<td>2.00</td>
<td>99.2</td>
<td>0.80</td>
</tr>
<tr>
<td>120</td>
<td>119.3</td>
<td>0.58</td>
<td>120.2</td>
<td>0.17</td>
</tr>
<tr>
<td>150</td>
<td>149.0</td>
<td>0.67</td>
<td>150.1</td>
<td>0.07</td>
</tr>
<tr>
<td>220</td>
<td>217.0</td>
<td>1.36</td>
<td>218.9</td>
<td>0.50</td>
</tr>
<tr>
<td>270</td>
<td>268.0</td>
<td>0.74</td>
<td>270.5</td>
<td>0.19</td>
</tr>
<tr>
<td>370</td>
<td>368.0</td>
<td>0.34</td>
<td>368.9</td>
<td>0.30</td>
</tr>
<tr>
<td>490</td>
<td>486.0</td>
<td>0.82</td>
<td>489.0</td>
<td>0.20</td>
</tr>
<tr>
<td>590</td>
<td>585.0</td>
<td>0.85</td>
<td>589.1</td>
<td>0.15</td>
</tr>
<tr>
<td>1000</td>
<td>998.0</td>
<td>0.40</td>
<td>998.4</td>
<td>0.16</td>
</tr>
<tr>
<td>1120</td>
<td>1118.0</td>
<td>0.18</td>
<td>1118.4</td>
<td>0.14</td>
</tr>
<tr>
<td>1220</td>
<td>1217.0</td>
<td>0.24</td>
<td>1215.9</td>
<td>0.33</td>
</tr>
</tbody>
</table>

The experimental data followed the power law function reported in the literature (Calamita et al., 2012), within the typical range of clay resistivity (1-100 Ohm.m) (e.g. Loke 2011). As resistivity is mainly controlled by water content, in both drying or wetting the resistivity is relatively low at high water content (the range of adsorbed, capillary and gravitational water ranges) and high at low water content (the range of desorbed, film, and film-capillary water) (Pozdnyakov et al., 2006). However, the rate of the resistivity changes is higher at low water content due to air replacement of water in the pores. The well defined resistivity-water content relationship obtained in this study with high correlation coefficient 0.945 and 0.966 for drying and wetting respectively, suggest that it can be used to calibrate resistivity against water content (Muñoz-Castelblanc et al., 2011) and to estimate in situ water content changes (Calamita et al., 2012).
Although resistivity provides an excellent technique for non-intrusive measurement of the spatiotemporal variation in water content on a large scale in the field, it can also be used to provide localised measurements in the laboratory (e.g. Muñoz-Castelblanc et al., 2011). The system described here has also been adopted for use in large-scale laboratory lysimeters (Asquith et al., 2012).

3 TIME DOMAIN REFLECTOMETRY

The TDR technique (Topp et al., 1980) is a method to measure soil water in hydrological and geotechnical testing, by measuring the soil bulk permittivity or dielectric constant that determines the velocity of an electromagnetic wave transmitted through the soil via a TDR probe (Tarantino et al., 2008). Since the dielectric constant of water ($K=80$) is larger than air ($K=1$) and soil constituents ($K=2-5$), the bulk permittivity is mainly governed by soil water content. To estimate water content from the dielectric constant, $K$, the empirical equation of Topp et al. (1980) is commonly used.

3.1 Coiled TDR

In geotechnical testing it would be hugely beneficial to have a device that is capable of simultaneous measurements of soil water content and pore water pressure at the same position. To achieve this, a coiled TDR device was developed that could be wound around a high suction tensiometer. The tensiometer was developed at Durham University (Lourenço et al., 2006) and is capable of measuring negative pore water pressures down to -2 MPa.

A double pronged TDR device was constructed by coiling copper wire around the insulated stainless steel housing of the tensiometer (Figure 3). A second device was also constructed using an impermeable ceramic tensiometer housing (Figure 4). The ceramic chosen was an impermeable Macor machinable glass ceramic, with a Young’s Modulus of 66.9 GPa and a compressive strength of 345 MPa.

Each housing had two helices (0.8 mm wide, 0.4 mm deep) cut into them at a pitch of 6 mm. This was so that the TDR prongs sat 3 mm apart as shown in Figure 3. This ensured that the probe diameter to spacing ratio was within the recommended region given by Noborio (2001) and Knight (1992), thus promoting an even distribution of electric field between the TDR prongs. The stainless steel body was insulated using five coatings of an insulating varnish.

The devices were tested alongside a conventional three-pronged TDR probe in three different soils (Leighton Buzzard sand, Birtley Clay and a very loose organic soil) over a range of known water contents.

The device could be simply calibrated based on the measured dielectric constant $K_1$ for known soil water contents. However, to better understand the effect of coiling the probe around a steel or ceramic body and to take account of the fact that the coiled TDR is measuring the effect of the steel or ceramic housing that it is wound around, as well as the properties of the soil surrounding it, a mixing model approach (Roth et al., 1990) was investigated for interpreting the data. The aim was to split the apparent dielectric constant $K_a$ into two parts, the dielectric constant of the tensiometer housing $K_{house}$ and the dielectric constant of the soil $K_{soil}$.

Ferré et al. (1998) showed that for the special case where the rod surface was divided equally between two materials, the apparent dielectric constant could be described as:

$$K_a = 0.5K_1 + 0.5K_2$$  \(3\)

where $K_1$ and $K_2$ are the dielectric constants of the two surrounding materials.

The helix which seats the TDR probe was designed so that half of each prong was exposed to the soil. Therefore $K_1$ can be replaced by $K_{house}$ and $K_2$ by $K_{soil}$. By measuring $K_{house}$, $K_{soil}$ was then interpreted by rearranging eq. (3) and finding a suitable value of $K_{house}$.

The manufacturer’s specifications give the dielectric constant of the ceramic to be 6.03 at 1 kHz and 5.67 at 8.5 GHz. Since the TDR bandwidth extends to around 1.5 GHz, a value of 6.0 was taken as the first approximation of $K_{house}$. This value, however, still caused large underestimations of volumetric water content, $\theta$. By using trial and error and measuring the standard deviation of the difference between the actual dielectric constant calculated from $\theta$ and $K_{soil}$ obtained from the mixing model, the best value of $K_{house}$ was found to be 3.5.

This value of $K_{house}$ for the ceramic was significantly lower than the dielectric constant given by the manufacturer. Adopting a $K_{house}$ value of 6.0 would be assuming that there was a perfect contact between the copper wire and the ceramic within the helix. However, as the grooves cut into the ceramic were not perfectly smooth and some tension in the prongs was lost when gluing them in place, this could introduce a small air gap between the copper wire and the ceramic body, changing the effect that the housing would have on the measured result.

For the stainless steel probe, using the same approach gave the optimal value of $K_{house}$ to be 2.65. In the case of the stainless steel body, the dielectric constant of the insulation was unknown so comparisons could not be made.

The results of applying the simplified mixing model to the data (using $K_{house}$ as 2.65) are shown in Figure 5. It can be seen that the results are slightly underestimated for clay and overestimated for sand, compared to the readings obtained from the conventional 3-prong TDR device.

It is likely that the higher values for sand are due to poor contact with the probe. If these higher values for sand were
neglected, a different optimal value of $K_{\text{soil}}$ would be achieved that would provide a closer fit to the observed values.

Ignoring the anomalous results for sand and comparing calculated and measured water contents it was found that the ceramic probe gave an accuracy for water content determination of $\pm0.047$. This resulted in an $R^2$ value of 0.966 for $K_{\text{soil}}$, when compared to the actual $K_{\text{soil}}$ found from known $\theta$. Likewise for the stainless steel probe, accuracy was found to be $\pm0.075$ with an $R^2$ value of 0.937. Improved accuracies can be obtained from direct calibration, rather than applying a mixing model.

It can be seen that Topp’s equation does not provide a good fit to the results (from either device) for the very loose organic soil. It is known that Topp’s equation is not appropriate for high volumetric water contents (>0.5).

4 CONCLUSIONS

The design and laboratory testing of new devices for water content measurement are described. A flexible multi-electrode resistivity system has been developed to acquire resistivity data and to carry out experimental studies to monitor water content changes in unsaturated soil specimens submitted to drying and wetting cycles.

5 REFERENCES


LaBrecque, D., and Daily, W., 2008, Assessment of measurement errors for galvanic-resistivity electrodes of different composition: Geophysics, 73, F55 – F64.


Figure 5. Comparison between the Coiled TDR (with a steel tensiometer housing) and a conventional 3 prong TDR device for Leighton Buzzard sand, Birtley Clay and a very loose organic soil.
ABSTRACT: Alonso et al. (1990) have presented the most comprehensive theory for partly saturated soils. Their constitutive equations present very complex formulations that rely on a large number of parameters, which are difficult to achieve unless advanced laboratory tests are performed. This paper presents a simple model for predicting the oedometric collapse of soils compacted with low density. The model has a minimum complexity, only needs two parameters, and establishes a linear relationship between log suction and volume change for different vertical pressures, until the moment when suction reaches the field capacity; then volume change remains at a constant value. This linear relationship is controlled by the Instability Index, \( I_{pt} \). Suction controlled oedometer test have been carried out, and the results agree with sufficient degree of accuracy with the proposed model.

RÉSUMÉ : Alonso et al. (1990) on présenté la théorie la plus avancée pour des sols partiellement saturés. Les équations constitutives présentent des formulations très complexes qui dépendent d’un grand nombre de paramètres, qui sont difficiles à évaluer sans des essais de laboratoire très avancés. Cet article présente un modèle simple pour prédire l’effondrement oedométrique du sol compacté avec une faible densité. Le modèle est une complexité minimale, nécessite seulement deux paramètres, et établit une relation linéaire, entre le logarithme de la succion et le changement de volume pour différentes pressions verticales, jusqu’au moment où la succion atteint la capacité de champ ; à partir de ce moment le changement de volume reste constante. La relation linéaire est contrôlée par l’Indice d’Instabilité, \( I_{pt} \). Les essais oedométriques avec succion contrôlée ont été réalisés, et les résultats sont en accord avec le modèle proposé.

KEYWORDS: unsaturated soil, model, collapse, suction, oedometer.

INTRODUCTION.

Expansive and collapsing soils have, generally, the common condition of being partly saturated.

In partially saturated soils with an open structure, the increase in the degree of saturation resulting from environmental or manmade changes can produce irrecoverable volume reductions without any change in the external forces. This phenomenon receives the name of collapse.

The first technical description of the collapse phenomenon may be the one made by Terzaghi and Peck (1948) when they describe the loss of strength and increase in compressibility of loess upon saturation. The word collapse to designate this phenomenon was already used by Jennings and Burland (1962). In general terms, a soil will swell or collapse after being flooded, depending upon whether the external pressure is smaller or larger than the swelling pressure. That is the reason why significant collapse may occur in a wide variety of open-structure soils ranging from well-graded sand, gravel and rockfill to plastic clay under high pressure, as long as the degree of saturation is low enough (Justo and Saetersdal 1979). In these soils and under these conditions, collapse is produced as suction decreases. As the external pressure increases, collapse increases up to a maximum; then the particles are so tight that any further decrease in suction produces volume expansion (Booth 1975, Yudhbir 1982, Maswoswe 1985).

During the wetting process in suction-controlled oedometer tests, sometimes swelling has been followed by collapse reached at very low suction values (Escario and Sáez 1973, Cox 1978, Alonso et al. 1987).

Oedometric cells similar to the ones described by Escario (1969), and Escario and Sáez (1973) have been used to investigate swelling, shrinkage and collapse under constant vertical net stress, as well as the loading and unloading behaviour under a constant matrix suction (Balanceda 1991; Yuk Gehling 1994; Vilari 1995 and Romero 1999).

In the deformational behaviour of partly saturated soils, the soil is sometimes considered elastic and isotropic. Fredlund and Morgenstern (1976), and Justo et al. (1984 a and b) use different elastic moduli with respect to the external stresses and suction. Justo and Saettersdal (1979) present an analysis of expansive and collapsing soils and a revision of calculation methods, including the elastic methods.

Alonso et al. (1987) analyse the volumetric deformations of these soils in the space of net stress and suction. Other authors have continued in this line, generating models that agree with a good approximation to the behaviour of partly saturated, non expansive soils (Josa et al. 1992, Cui et al. 1995, Wheeler and Sivakumar 1995, Habibagahi and Mokhberi 1998, Sheng et al. 2004).

The latest tendencies in the study of partly saturated soils are addressed to coupling in the same model the expansive and collapsing behaviour of soils (v. Justo and Saetersdal 1979), generating the so called consistent models (Li and Fang 2011). Along this line, it is proposed in this paper a model for open structure collapsing soils based upon two parameters obtained from the relationship between the volumetric deformation (under oedometric conditions), the suction and the vertical pressure.

From the results obtained, it will be observed that the model describes, with sufficient precision, the behaviour of a collapsing mixture of clay, when subject to oedometric conditions, and to a wetting stress path under constant vertical stress.
2 MATERIAL USED IN THE RESEARCH

2.1 Composition and characterization of material

The material used is a mixture of sand (30%), silt (32%) and clay (38%) (v. Figure 1). The analysis by X-ray diffraction indicates that the soil mixture consists of quartz, calcite, dolomite and vermiculite.

Figure 1. Soil mixture

Table 1 summarizes the results obtained from the analysis of the major chemical compounds present in the samples tested by the method of quantitative Pyrite.

<table>
<thead>
<tr>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>MgO</th>
<th>CaO</th>
<th>K₂O</th>
</tr>
</thead>
<tbody>
<tr>
<td>47.9%</td>
<td>6.0%</td>
<td>2.8%</td>
<td>1.5%</td>
<td>9.8%</td>
<td>1.3%</td>
</tr>
</tbody>
</table>

Calcination Others

loss

19.0% 1.72%

Table 2 summarizes the characterization tests of the soil mixture. The initial suction (Ψ₀), was obtained with a sphygmanometer T5x-UMS.

<table>
<thead>
<tr>
<th>T200 (%)</th>
<th>w₀ (%)</th>
<th>ρₘax (kg/m³)</th>
<th>ρₙ (kg/m³)</th>
<th>e₀ ( )</th>
<th>Sᵣ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>73.49</td>
<td>14.5</td>
<td>1795</td>
<td>2700</td>
<td>0.68</td>
<td>45</td>
</tr>
<tr>
<td>w₀ (%)</td>
<td>I₀ (%)</td>
<td>Ψ₀ (kPa)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>31</td>
<td>13.6</td>
<td>95.4</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

To obtain a more open structure that guarantees a collapsing behavior, the sample was compacted to 90% of the Proctor standard maximum dry density (1615 kg/m³). As the moisture content (8.8%), corresponding to this density in the Proctor curve, prevented obtaining acceptable handling conditions (it crumbled when removed from the compaction ring), it was finally compacted to a moisture content of 11.3% and ρₙ=1615 kg/m³.

2.2 Soil water characteristic curve (SWCC)

The techniques used to implement and control the suction in the different samples may be grouped by ranges as indicated below:

1. Pressure membrane for suctions less than 500 kPa.
2. Vapour equilibrium in vacuum desicator, with saline solutions (NaCl, CaCl₂) for suctions ranging from 500 kPa to 150 MPa.
3. Vapour equilibrium in vacuum desicator, with acid solution (H₂SO₄) for suctions exceeding 150 MPa.

The time required to reach equilibrium under the imposed suction was determined before performing the experimental tests scheduled. In test specimens in the pressure membrane apparatus, a suction of 400 kPa was applied for several days. Every two days the samples were weighed and it was found that the weight stabilized in eight days.

In the experimental phase stabilization times were greater, at least ten days for all samples tested, thus ensuring that suction equilibrium was reached.

Figure 2 shows the SWCC for the drying paths from the initial suction and the subsequent wetting paths.

Figure 2. Soil water characteristic curve

3 EQUIPMENT AND EXPERIMENTAL METHODOLOGY

3.1 Experimental techniques

Oedometer tests have been performed on cells that control the suction of the soil mixture, using the technique of axis translation and keeping constant the vertical pressure (see Figure 3).

All tests have been carried out in a room with controlled temperature (20±1 °C) and relative humidity of 65±2%.
3.2 Experimental results.

The experimental program intends to simulate the behaviour of a collapsing soil when it is progressively wetted until saturation. The wetting process is carried out decreasing progressively the suction until zero.

Oedometer tests have been performed in the prepared specimens maintaining suction values in the range initial suction-zero. Specifically, the suction steps applied were: 95.4 kPa, 50 kPa, 15 kPa, 10 kPa y 0 kPa (suction for saturation). Figure 4 shows the test results.

Figure 5 shows the volume increase in samples tested at different external pressures when suction is decreased from the initial value.

3 SIMPLIFIED COLLAPSE-SUCTION MODEL.

Following the results extracted from oedometer tests, a relationship has been sought between volumetric strain under oedometric conditions, suction and applied external vertical stress.

Tests carried out on expansive soils indicate that for constant external pressure there is a linear relationship between volumetric strain and log of suction (Meintjes 1992, Gordon 1992).

The model proposed in this paper is associated to the behaviour of a collapsing soil instead of an expansive soil, but uses the same linear relationship between volume strain ($\varepsilon_v$) and log relative suction (Eq. 1).

$$\varepsilon_v = I_{pt} \cdot \log \left( \frac{\Psi}{\Psi_0} \right)$$

where $\Psi_0$ is the initial suction and $I_{pt}$ is the “Instability Index”, proposed by Aitchison et al. (1973) for swelling and shrinking test on soils.

The Instability Index is a function of the vertical stress applied.

The proposed relationship is valid until suction arrives to a low value, corresponding to the field capacity, when volumetric strain becomes constant (v. Figure 6). Water is not absorbed anymore by the soil.

In Figure 5, the Instability Index is the slope of the regression line relating volumetric strain and log ($\Psi/\Psi_0$) for every vertical pressure.

Figure 6 indicates a linear relationship between Instability Index and vertical pressure drawn in semilog scale.

This relationship can be expressed by equation Eq. 2.

$$I_{pt} = C \cdot \log(\sigma_v)$$

Substituting Eq. 2 into Eq. 1, a relationship between vertical stress and suction with vertical strain is obtained.

$$\varepsilon_v = C \cdot \log(\sigma_v) \cdot \log \left( \frac{\Psi}{\Psi_0} \right)$$

Figure 7 is a 3D picture of the experimental relationship between vertical strain, relative suction and vertical pressure. The lines corresponding to constant $\sigma_v$ values may be approximated by straight lines as indicated by equations (1) and (3). The picture includes the line when the field capacity is reached and the subsequent constant volumetric strain indicated in Figure 5. The projection of this line to the $\varepsilon_v - \log(\Psi/\Psi_0)$ plane is a potential, corresponding to the equation:

$$\varepsilon_v = 24.5 \cdot (\Psi/\Psi_0)^{0.2}$$
Eq. 3 depends upon two fundamental parameters: the initial suction, $\Psi_0$, and the constant, $C$.

The initial suction may be obtained by any of the techniques for measuring suction in soils compatible with the range of suctions of the sample: tensiometer, filter paper, psychrometer, etc. To obtain the constant $C$, suction controlled oedometer test must be performed subjecting the sample at least to two different vertical pressures.

This way, with the determination of $\Psi_0$ and the $C$ coefficient, it is possible to establish a simple method that will represent with sufficient approximation the collapsing behavior of a collapsing soil from the initial suction until saturation.

5 CONCLUSIONS

Recent models to predict the collapsing behaviour in low density, partially saturated soils, obey to very complex formulations with a high number of parameters. These parameters can only be carried out using advanced laboratory tests, not commonly available even in advanced laboratories.

For many problems of foundations on collapsing soils it may be assumed that displacement are one dimensional and also that wetting of the soil and collapse occurs after the soil is loaded. For this cases, a simple model has been presented in this paper that describes, with sufficient precision, the behaviour of a collapsing soil (a mixture of sand, silt and clay) when subject, under oedometric conditions, to a wetting stress-path under constant vertical stress.

The model proposed is based on a linear relationship between volumetric deformation, the log of relative suction and the log of the vertical pressure. The equation depends upon a coefficient called the Instability Index, which in turn is proportional to the log vertical pressure.

The model is valid until the samples have reached the field capacity; then the volumetric strain becomes constant.

This model needs only two parameters to be defined: the initial suction and the coefficient $C$ that relates the Instability Index with the log of the vertical pressure. This parameter is obtained in suction controlled oedometer tests, for two different constant vertical pressures.

The simplicity of the proposed model makes it interesting for a quick estimate of the collapse vertical strains in partially saturated soils.

6 ACKNOWLEDGEMENTS

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7 REFERENCES


Critical State for Unsaturated Soils and Steady State of Thermodynamic Process

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ABSTRACT: Critical state is an important concept in modern soil mechanics. It was developed by Roscoe, Schofield and Wroth (1958) and Schofield and Wroth (1968) for saturated soils. Currently, the critical state for unsaturated soils has not been clearly defined and the necessary conditions and constraints to attain to critical state have not been definitely developed. More studies on critical state of unsaturated soils are required. Based on theory of thermodynamics, the conditions and constraints to attain to steady state of thermodynamic process for unsaturated soils are proposed in this paper. This paper points out that the steady state of a deformation process of unsaturated soils is the ultimate state of the deformation process and it is, by concept, the critical state of unsaturated soils. The conditions and constraints for critical state of unsaturated soils presented in the paper are more general and complete, and are based on more rigorous thermodynamic theory instead of only being based on the laboratory test results of particular unsaturated soil samples.

RÉSUMÉ :

Le concept d’état critique est un concept important en mécanique des sols. Il a été développé pour les sols saturés par Roscoe et al.; (1958), et Schofield et Worth (1968). En revanche, ce concept n’est pas encore clairement décrit pour les sols non saturés. Il est donc nécessaire de mettre en place des études plus approfondies dans ce sens. Dans cet article, en se basant sur la théorie de la thermodynamique, les conditions et les limites pour atteindre l’état critique dans les sols non saturés sont exposées. Ici on considère que l’état permanent de la déformation dans les sols non saturés est l’état ultime du processus de déformation, il peut donc être considéré comme étant l’état critique. Les conditions et les limites pour l’état critique des sols non saturés présenté dans cet article sont plus générales et complète, car elles sont basées sur la théorie de la thermodynamique au lieu d’être issues des essais de laboratoire sur des échantillons de sols non saturés.

KEYWORDS: unsaturated soils; critical state; steady state of thermodynamic process

1 INTRODUCTION

Critical state is an important concept in modern soil mechanics. It acts as the cornerstone of critical state soil mechanics since it gives the final point of a soil deformation process, which is necessary for establishing soil constitutive models. Without the concept of critical state, it would not be able to know which direction a soil deformation process evolves, and where and when the deformation process ends. If the end point of a deformation process is not defined, significant deviations might occur in constructing soil constitutive models. Once the initial state and the end point (the critical state) of a deformation process are defined, relatively more accurate models can be established by taking advantage of interpolations between the initial state and the critical state, rather than only using one-sided extrapolation from the initial state (without critical state). Therefore critical state is a fundamental concept that most constitutive models for soils are based on.

The critical-state concept was proposed by Roscoe, Schofield and Wroth (1958), and Schofield and Wroth (1968), and is defined as the end or ultimate state of a deformation process for saturated soils, in which soil keeps shear deforming with constant stress and volume in large strains. In three-dimensional axisymmetric space (i.e. in triaxial stress state), when a soil deformation process evolves to a critical state, the stress and strain should satisfy the following necessary conditions and constraints:

\[ p' = 0; \quad q = 0; \quad \nu = 0; \quad \dot{\varepsilon}_s \neq 0 \]  

The critical state can be expressed in \( q - p' \) space and \( e - \ln p' \) space by the following equations, respectively:

\[ (q / p') = M; \quad \nu = \nu_s = \Gamma + \lambda \ln(p') \]  

where \( p' \) is the mean effective stress of saturated soils, \( q \) is the deviator stress, \( \nu \) is the specific volume, \( \varepsilon_s \) is the deviator strain, \( M \) is the slope of the critical state line in \( q - p' \) space, \( \Gamma \) and \( \lambda \) are the intercept (at \( p' = 1 \text{kPa} \) ) and the slope of the critical state line, respectively, in \( e - \ln p' \) space.

Previous and current studies of critical state have been conducted mainly on saturated soils. Although the constitutive models for unsaturated soils within the framework of critical state soil mechanics have been developed by Alonso, Gens and Josa (1990), Gens and Alonso (1992), Wheeler and Sivakumar (1995), Wheeler, Sharma and Buisson (2003), Sheng, Sloan and Gens (2004), Li (2007), Sun, Sheng, Cui and Sloan (2007) to mention only a few from the existing literature, the definition and the necessary conditions and constraints for critical state of unsaturated soils have not been provided explicitly and clearly. So it is affected to determine the value of critical state for unsaturated soils correctly and accurately.

The engineering properties and behaviors of unsaturated soils are much more complicated than those of saturated soils because of the existence of air in unsaturated soils. Comparing
with saturated soils, the stress states and the phase volume changes of unsaturated soils are more complicated. The effective stress for unsaturated soils is not only related to the pore water pressure, but also related to the pore air pressure and even the degree of saturation (Zhao, Liu and Gao 2010). Therefore the critical state of unsaturated soils could not be accurately defined by Eq. (1). For this reason it is important to determine which conditions and constraints should be satisfied when the unsaturated soils attain the critical state, and which additional conditions and constraints should be added to Eq. (1) to completely and accurately describe critical state for unsaturated soils. It is impossible for the theory of critical state unsaturated soil mechanics and the corresponding constitutive models to be further developed if these problems are not fully solved.

Until now most researches on critical state of unsaturated soils have been based on laboratory triaxial tests, such as Toll (1990), Wheeler and Sivakumar (1995), Maatouk et al. (1995), Adams and Wulfssohn (1997), Rampino et al. (1998), Wang et al. (2002), Kayadelen et al. (2007), etc. By means of laboratory triaxial tests, these researches have been focused on whether the critical state for unsaturated soils exists, and if the critical state for unsaturated soils exists, then what necessary conditions and constraints should be satisfied when unsaturated soils attain to critical state. Many researchers, such as Wheeler and Sivakumar (1995), Maatouk et al. (1995), Adams and Wulfssohn (1997), etc., selected the mean net stress, the deviator stress, the suction and the specific volume as the state variables to describe critical state of unsaturated soils. In addition to the above-mentioned state variables, other researchers such as Wang et al. (2002), Kayadelen et al. (2007), etc., suggested an additional variable, specific water volume \( \nu_s \), or degree of saturation \( S_o \), to define critical state of unsaturated soils. Some laboratory test results, such as from Wheeler and Sivakumar (1995), showed that \( \nu_s \) did not affect the steady state, ultimately, possibly due to the limitations of equipment and experimental conditions. Whereas Rampino et al. (1998) reported that \( \nu_s \) could keep constant at the end of deformation test for unsaturated soils. Wang et al. (2002) pointed out that it may not be reliable to use \( \nu_s \), as an indicator of critical state for unsaturated soils, and more data from tests and more researches are required before a conclusion can be made on this matter.

Thermodynamics is a universally applicable theory. The deformation process for unsaturated soils must follow thermodynamic laws. By using thermodynamic theory, the deformation process and behavior of unsaturated soils can be investigated with more common situations and on more general perspectives in order to reveal the complex soil behaviors and properties. The objective of this paper is to establish the necessary conditions and constraints for unsaturated soils to reach critical state based on thermodynamic theory.

With more general perspectives based on the theory of thermodynamics, critical state for unsaturated soil is studied in this paper. Steady state in thermodynamic process is more common and general than critical state in soils. Steady state describes the final state of a thermodynamic process, and critical state of soils is a special case of it. This paper demonstrates that steady state in a deformation process of unsaturated soils is the final state of the process just as the critical state in soil mechanics, and it includes the more state variables and restrictions than those in critical state of saturated soils.

2 THERMODYNAMIC CONDITIONS FOR STEADY STATE

It is assumed that unsaturated soils satisfy the assumption of local equilibrium thermodynamics, and the theory of local equilibrium thermodynamics can be used to approximately describe the irreversible deformation process of unsaturated soils. The theory of local equilibrium thermodynamics (Kuiken 1994, Wark and Richards 1999) demonstrates that under the environmental force disturbance, a closed system that keeps constant temperature but cannot exchange both mass and heat energy with its surroundings can evolve from a non-equilibrium state to a steady equilibrium state with system entropy reaching the maximum. In accordance with the assumptions widely used in unsaturated soil mechanics, it is assumed that a representative volume element (RVE) of unsaturated soils is a closed system that cannot exchange mass with its surroundings (actually the real system can exchange mass with its surroundings. If the system were not supposed to be a closed system, Gibbs’s thermodynamic theory would not be applied to it. As usual applied thermodynamics, here a representative volume element (RVE) of unsaturated soils is supposed to be a closed system). Therefore, a RVE evolves from a non-equilibrium state to a steady equilibrium state with the maximum entropy. Under the isothermal and isometric conditions, the Helmholtz free energy \( \Psi \) of the RVE reaches the minimum value and remains constant at steady state according to the theory of local equilibrium thermodynamics (Kuiken 1994, Wark and Richards 1999).

The first law and the second law of thermodynamics (Wark and Richards 1999) are given as following:

\[
\begin{align*}
W + Q &= U \\
S &= \frac{\dot{S}}{\dot{T}} \geq 0
\end{align*}
\]

where \( W \), \( Q \), \( U \), \( T \), \( S \), and \( S \) are work, heat supplied, internal energy, temperature, internal entropy and total entropy, respectively. From Eq. (3), the following equation can be developed:

\[
W - T S_s = \dot{U} - T \dot{S}_s = \Psi
\]

In light of Gibbs’s theory of local equilibrium thermodynamics (Kuiken 1994, Wark and Richards 1999) when the RVE attains to steady equilibrium, Helmholtz free energy, \( \Psi \), reaches the minimum value, i.e. \( \Psi = 0 \), and all state variables must keep constant, which can be expressed as:

\[
W - T S_s = \dot{U} - T \dot{S}_s = \Psi = 0
\]

where the subscript \( s \) stands for steady state.

3 CONDITIONS AND CONSTRAINTS TO ATTAIN TO CRITICAL STATE FOR UNSATURATED SOILS

To analyze the problems of soil mechanics, it is important that some independent state variables should be selected at first. In continuum mechanics, total stress \( \sigma \) and corresponding strain \( \varepsilon \) are used to describe material mechanical properties. As there are solid, liquid and gas phases in soils, and each phase has its dual variables, stress and the volume fraction, the properties of soils cannot be analyzed strictly by continuum mechanics. However, theory of porous media can be used to describe behaviors and properties of soils. According to theory of porous media, such as de Boer (2000), the volume fraction and the stress of each phase are selected as the dual state variables. In theory of porous media, the velocity is defined as a mass weighted average quantity, but in soil mechanics or hydrology, the velocity of soil skeleton \( \nu_s \) is generally used as a reference configuration to construct seepage or other equations. The control equations based on the theory of porous media may have different forms from the ones developed in some engineering fields, such as soil mechanics and hydrology, due to the selection of different basic kinematical variables. In this
paper, the variables and parameters widely used in soil mechanics are selected as the state variables and parameters, the same as those used in Zhao, Liu and Gao (2010), e.g., the total stress $\sigma$ and the dual variables in Eq. (6), effective stress $\tilde{\sigma}$ and strain $\varepsilon'$ of soil skeleton that determines soil deformation, suction $s$ and degree of saturation $S_s$, and air pressure $P_a$ and air volume strain $\varepsilon_a$.

Based on porous media theory, it is assumed that the solids and the pore water are incompressible, neither heat nor mass is transferred among the three phases, and the velocities of seepage and airflow are sufficiently small such that the diffusion effects on internal energy, stress, heat and entropy are all negligible, then, as in Zhao, Liu and Gao (2010), the work for unsaturated soils can be expressed as:

$$W = \left[\text{tr}(\tilde{\sigma} \cdot \varepsilon') + snS_s + P_a n' \varepsilon_a'ight]$$ (6)

Houlsby (1997) gave a similar form as Eq. (6). Substituting Eq. (6) into Eq. (4) results in the following:

$$[\text{tr}(\tilde{\sigma} \cdot \varepsilon') + snS_s + P_a n' \varepsilon_a'] - TS = U - TS = \Psi$$ (7)

where $\varepsilon'$ is soil skeleton strain, $s = P_a - P_i$ is suction, $P_i$ is the pore air pressure, $P_a$ is the pore water pressure, $n$ is the porosity of soil, $S_s$ is degree of saturation, $n' = n(1 - S_s)$ is volume fraction of air phase, $\varepsilon_a'$ is volume strain of air phase and $\tilde{\sigma}$ is the effective stress of unsaturated soils, as given by Zhao, Liu and Gao (2010) as:

$$\tilde{\sigma} = \sigma - (S_a P_a + (1 - S_a) P_i) \delta$$ (8)

where $\delta$ is unit tensor.

In soil mechanics, some fundamental concepts and constitutive models are usually developed based on the results of triaxial tests, and the conditions and constraints for critical state of saturated soils given by Eq. (1) and Eq. (2) are in three-dimensional axisymmetric space. Following this convention, Eq. (8) is rewritten in the three-dimensional axisymmetric stress and strain space as:

$$\{\tilde{\sigma} \varepsilon' + q \varepsilon_a' + snS_s + P_a (1 - S_a) \varepsilon_a'\} - TS = U - TS = \Psi$$ (9)

where $\varepsilon'$ and $\varepsilon_a'$ are volume and deviator strains, respectively, of solid phase in triaxial stress space. $\tilde{\sigma} = 1/3(\tilde{\sigma}_{rr} + \tilde{\sigma}_{rr} + \tilde{\sigma}_{rr})$ is the mean effective pressure dual to $\varepsilon'$, and $p = 1/3(\sigma_{rr} + \sigma_{rr} + \sigma_{rr})$ is the mean total pressure.

The objective of this paper is to discuss the stress and strain conditions and constraints of each phase to attain to the ultimate state or steady state of deformation process for unsaturated soils from thermodynamic perspective. According to the theory of thermodynamics as mentioned in the above section, when the unsaturated soil continually being sheared to reach the ultimate point of the deformation process, a local steady equilibrium state should be attained, and then the local state variables should not change with time, and Helmholtz free energy, $\Psi$, reaches the lowest value, i.e. $\Psi = 0$. In other words, all state variables except shear strain should be constant. When soil deformation attains to steady state, according to Eq. (5), Eq. (9) equals to zero. Following the principle proposed by Roscoe, Schofield and Wroth (1958), and Schofield and Wroth (1968), the shear strain energy is entirely dissipated at steady state, and then Eq. (9) can be decomposed into:

$$\left\{\tilde{\sigma} \varepsilon' + q \varepsilon_a' + snS_s + P_a (1 - S_a) \varepsilon_a'\right\} - TS = U - TS = \Psi; U = 0; S_s = 0; \Psi = 0$$ (10)

The first equation in Eq. (10) demonstrates that when the deformation process of unsaturated soils attains to steady state, its state variables should not include shear strain. The shear strain energy is supposed to be the only dissipated energy, and it should be dissipated (assume that the elastic shear strain is sufficiently small and its corresponding energy is ignored) to make the system entropy increase. The system entropy reaches the maximum value, or the Helmholtz free energy reaches the lowest value at steady state. From the second equation in Eq. (10), it can be learnt that because the pressure of each phase is not null when the deformation process attains to steady state, the volume increment of each phase in the left side of the equation must be zero. This result is very important since currently there is not a clear definition about whether the volume or volume fraction of each phase keeps constant at the steady state or critical state. Some researchers might believe intuitively the above point, but there has not been any rigorous theoretical proof. Based on the theory of thermodynamic, it has been proven theoretically that the volume or volume fraction of each phase must keep constant at steady state. So according to the second equation in Eq. (10), the necessary conditions and constraints for steady state of unsaturated soils can be expressed as:

$$\varepsilon_a' = 0; \ S_s = 0; \ \varepsilon_a' = 0; \ \varepsilon_a' \neq 0$$ (11)

where $n$ is the porosity of the soil. It should be constant in order to keep the volume of each phase constant, otherwise the change of $n$ would make $\varepsilon_a'$, $S_s$ and $\varepsilon_a'$ change, which is contradictory to Eq. (11).

On the other hand, when the deformation process of unsaturated soil attains to steady state, in addition to $\varepsilon_a'$, $S_s$ and $\varepsilon_a'$ keeping constant, the stress variables $\tilde{\sigma}$, $q$, $s$ and $P_a$ is in the second equation of Eq. (10) should also keep constant, otherwise the deformation process would not attain to steady state in light of thermodynamics. If $s$ and $P_a$ is constant at steady state, it is obvious that $P_a$, $p$ and net pressure $\tilde{\sigma} = p - P_a$ must be constant.

From above discussion, a thermodynamic process evolves continuously and finally attains to steady state. For the deformation process of unsaturated soils, steady state or steady balance means that deformation process of unsaturated soils attains to the ultimate state, namely the critical state in soil mechanics or the steady state in thermodynamics, and at this state all the state variables do not change. The necessary conditions and constraints for critical state of unsaturated soils based on thermodynamics are being stated as: 1) the volume change of each phase should satisfy the requirement of Eq. (11) and porosity $n$ should keep constant; 2) the stress variables $\tilde{\sigma}$, $q$, $s$ and $P_a$ (also including $P_i$, $p$ and $\tilde{\sigma}$) should also be constant. Comparing with the conditions and constraints for critical state of saturated soils, i.e. Eq. (1), more conditions and constraints are needed for critical state of unsaturated soils.

Toll (1990), Wheeler and Sivakumar(1995), Maatouk et al. (1995), Adams and Wolffsohn (1997), Rampino et al. (1998), Wang et al. (2002), and Kayadelen et al. (2007) have conducted some pioneering work on critical state of unsaturated soils by laboratory triaxial tests, and suggested some conditions required for critical state of unsaturated soils. Comparing the conditions from these researchers, the conditions proposed in this paper are more complete and generalized with rigorous theoretical basis. In some laboratory tests on samples of unsaturated soils, at the end of the deformation processes, the conditions and constraints given in this paper are not all satisfied. This does not mean that the conditions and constraints provided in this paper are incorrect, since they are established based on the universally applicable laws of thermodynamics. The explanation may be that these tests might be limited by laboratory equipment and experimental conditions, and the deformation of unsaturated soil samples might not be able to attain to critical state, just as those with saturated soils.

Based on above the necessary conditions and constraints for critical state of unsaturated soils, two special cases need further discuss: 1) When air pressure, $P_a$, is not considered as an
independent state variable, identical to ignoring the item of \( P_a \) in Eq. (10), the conditions and constraints to attain to critical state can be rewritten as: the volume increment of each phase should satisfy \( \varepsilon_i' = 0 \) and \( S_i = 0 \), the porosity \( n \) keeps constant, and the stress variables \( p, \sigma, p', q, s \) and \( P_a \) remain unchanged. Alonso, Gens and Josa (1990) developed the Barcelona model, in which they selected mean net stress \( \bar{p} = p - P_a \), matric suction \( s \), deviator stress \( q \), specific volume \( \nu \), and shear strain \( \varepsilon' \) as state variables, the conditions to attain to critical state given by them belong to the second special case described above. In the models of coupling of hydraulic hysteresis and stress-strain behaviour developed by Wheeler, Sharma and Buisson (2003), Sheng, Sloan and Gens (2004), Li (2007), Sun, Sheng, Cui and Sloan (2007), etc., the conditions to attain to critical state belong to the first special case described above. Besides when the degree of saturation, \( S_i \), equals to 1, the soil is saturated, and matrix suction and pore air presser are zero, then the necessary conditions and constraints for critical state of unsaturated soils are degraded into those for saturated soils, e.g. equal to what are given in Eq. (1). Of course, more study is needed for some special circumstances whether the conditions and constraints for critical state of unsaturated soils are all necessary.

It should be noted that when the deformation process of soils attains to critical state, the soil structure might be anisotropic. Li and Dafalias (2010) discussed the uniqueness of critical state line with anisotropic structure and the enhanced critical state conditions for saturated soils. But more research on critical state with anisotropic structure and its uniqueness for unsaturated soils is needed.

4 CONCLUSIONS

This paper develops the necessary conditions for the deformation of unsaturated soils to reach critical state based on the theory of local equilibrium thermodynamics. The necessary conditions are developed using state variables and parameters widely used in soil mechanics and the expression of work \( W \) for unsaturated soils proposed by Zhao, Liu and Gao (2010). Comparing with the conditions given by other pioneering researchers based on the laboratory triaxial tests, the conditions given in this paper are more complete and generalized with rigorous theoretical basis, and these conditions are not based on the laboratory test results on some special samples. In addition, it is shown that when some variables are not treated as independent variables, the conditions given by other researchers are a special case of the conditions presented in this paper.

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CHALLENGES AND INNOVATIONS IN GEOTECHNICS

Actes du 18e Congrès International de Mécanique des sols et de Géotechnique

DÉFIS ET INNOVATIONS EN GÉOTECHNIQUE

The 18th International Conference on Soil Mechanics and Geotechnical Engineering (Paris 2013) was devoted to “Challenges and Innovations in Geotechnics”. The call for abstracts was based on a large series of themes covering most aspects of Geotechnical Engineering and around 800 abstracts were received from the Member Societies. According to the ISSMGE new vision for strengthening the role of the Technical Committees, the papers were distributed to the corresponding Technical Committees, which then selected the General Reporters, the Invited Speakers and the Poster presentations. The involvement of TCs (which could also propose Workshop Sessions on an independent basis) was enthusiastic and successful, which probably explains the success of the Conference with more than 1500 delegates.

The four volumes of the Proceedings contain the Terzaghi Oration, the Honour lectures, the Special lectures followed by the papers presented according to the relevant TC and introduced by the TC General Report. All volumes, together with late papers, will also be made available online free of charge. These volumes will provide a state of the art and serve as an essential reference for practitioners, academics and researchers involved in Soil Mechanics and Geotechnical Engineering.

Le 18e Congrès International de Mécanique des sols et de Géotechnique (Paris 2013) a été dédié aux « Défis et Innovations en Géotechnique ». L’appel à résumés était ouvert sur un large éventail de thèmes couvrant la plupart des aspects de la Géotechnique ; les Sociétés Membres ont sélectionné environ 800 résumés. En cohérence avec le souhait de la Société Internationale (SIMSG) de renforcer le rôle des Comités Techniques (CTs), les contributions ont été transmises aux CTs correspondants, en charge de choisir les Rapporteurs Généraux, les Orateurs Invités et les Présentations sur Posters. L’implication des CTs, qui pouvaient en outre proposer des sessions d’Atelier à leur convenance, a été enthousiaste et fructueuse ; ceci explique probablement le succès du Congrès avec plus de 1500 délégués.


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