



Proceedings of the 18th International
Conference on Soil Mechanics and
Geotechnical Engineering

CHALLENGES AND INNOVATIONS IN GEOTECHNICS

*Actes du 18^e Congrès International
de Mécanique des sols et de
Géotechnique*

**DÉFIS ET INNOVATIONS
EN GÉOTECHNIQUE**

Paris 2013

Edited by / Sous la direction de

Pierre Delage, Jacques Desrues, Roger Frank, Alain Puech, François Schlosser

VOLUME 1



Presses des Ponts

Technical Committee 101

**Laboratory Stress Strain Strength Testing
of Geomaterials**

Session I - Time effects and other peculiar observations

Comité technique 101

***Caractérisation en laboratoire du comportement
des géomatériaux***

Session I - Effets du temps et autres aspects

Laboratory testing of geomaterials: time effects and other peculiar observations - General Report of TC 101 (session 1)

Essais de laboratoire sur les géomatériaux: Effets du temps et autres observations spécifiques - Rapport général du TC 101 (session 1)

Ibraim E.

University of Bristol, Bristol, UK

ABSTRACT: This paper presents a General Report of 29 written contributions submitted for one of the parallel sessions entitled "Time effects and other peculiar observation" of the Technical Committee 101: Laboratory Stress Strength Testing of Geomaterials. The origin of the authors shows a wide geographical distribution. The General Report reviews these contributions and presents the current research directions mainly in relation to the experimental behaviour as well as the key outcome results. The topics covered by the written contributions have been grouped within the following thematic strands: time effects, expansive soils, consolidation/compressibility and crushable granular soils.

RÉSUMÉ : Ce document présente un rapport général des 29 contributions écrites pour l'une des sessions parallèles « Effets du temps et autres observations spécifique », soumis au Comité Technique 101: Essais de laboratoire des géomatériaux. L'origine des auteurs montre une large répartition géographique. Le rapport général examine ces contributions et présente les axes de recherche actuels principalement en ce qui concerne le comportement expérimental ainsi que les principaux résultats. Les sujets couverts par les contributions écrites ont été regroupées dans les volets thématiques suivants: les effets du temps, sols gonflants, consolidation/compressibilité et sols granulaires avec rupture des grains.

KEYWORDS: clay soils, time effects, creep, strain rate, consolidation, expansive soil, shrinkage, swelling, breakage, sand.

1 INTRODUCTION

This paper presents a summary in the form of a General Report of the topics, current research directions and key outcomes of the written contributions submitted to the Technical Committee 101: Laboratory Stress Strength Testing of Geomaterials Session. TC101 promotes and actively encourages co-operation and exchange of information concerning research and developments in advanced laboratory geotechnical testing, including apparatus, techniques and interpretation, and their use in practical geotechnical engineering, site characterisation studies and ground modelling approaches.

TC101 session contains a total of 58 written contributions. Their presentation is organised in two parallel sessions of 29 papers each on *Time effects and other peculiar observations* and *Strength properties and treated soils*. This General Report refers only to the papers selected for the former session.

The origin of the authors clearly shows a wide geographical distribution, 12 contributions from Europe, 12 from Asia, 3 from North and South America, and 2 from Australia and New Zealand. Japan, China and France are leading in terms of number of contributions with 5, 4 and 3 papers, respectively.

Although the papers in this session explore mainly the experimental aspects of the behaviour of fine and granular soils, the analysis and interpretation are enhanced due to complementary use of various analytical and constitutive modelling approaches. In general, there is a great concern for conducting multi-scale investigations, including soil structure analyses and correlation with macro soil response. Reference to practical applications is also considered.

Despite some inherent heterogeneity, the topic covered by the written contributions can be grouped within the following thematic strands:

- Time effects: creep, strain rate effects in fine soils;

- Expansive soils: swelling/shrinkage characterisation;
- Consolidation/compressibility (loading and unloading);
- Crushable granular soils.

The following sections develop the issues with reference to the written contributions.

2 TIME EFFECTS

Of all aspects of settlement analysis, the issue of creep and secondary consolidation is one of the area in which least progress has been made in terms of fundamental understanding and in the incorporation of research into practice. While the existence of creep complicates the predictions of the magnitude of settlement of structures founded on clay soils, further difficulties arise from the fact that the concerned elapsed time under the constant load following the end of the construction in a full-scale soft clay deposit is usually very long. As a consequence, the settlement rates are very low, significantly lower than those that can reasonably be measured in conventional laboratory testing (Leroueil 2006). In order to overcome this exertion and evaluate the creep process at very low strain rates as observed in the field (order of 10^{-10} %/s), a new laboratory one-dimensional compression test method is proposed by Kawabe and Tatsuoka (2013). The test procedure consists of application of multiple unloading/reloading (UL/RL) cycles with several sustained - creep - loading (SL) stages. Figure 1 summarises the relationships between the creep strain (for three hours) and the stress ratio between the stress at the start of (UL) or (RL), σ_{UL} or σ_{RL} , and the stress at each (SL) stage, σ_{SL} obtained during multiple UL/RL cycles and several numbers of SL stages on two samples of reconstituted soft clays. The creep strain rates recorded during UL and RL cycles are smaller than those during primary loading and they can reach a very low level. Based on relations of this type, the stress ratio corresponding to $\Delta \epsilon_{a(creep)} = 0$ can be deduced and based

on the Isotach viscous properties of clays, by starting the SL after unloading (point B, Figure 2), the zero-strain-rate relation is reached faster. Therefore, the creep behaviour at very low strain rates is reached in a relatively short period of time, much shorter than if SL would start from point A.

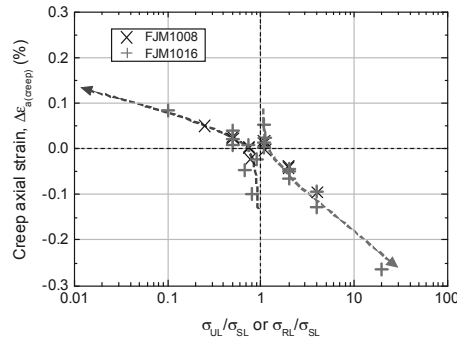


Figure 1. $\Delta\epsilon_{a(creep)} - \sigma_{UL}/\sigma_{SL}$ or σ_{RL}/σ_{SL} relations, for two tests on reconstituted Fujinomori soft clay (Kawabe and Tatsuoka 2013)

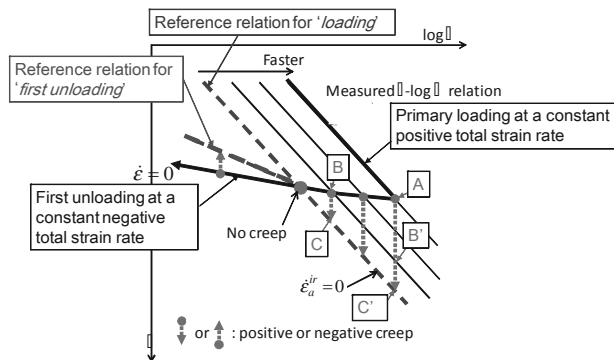


Figure 2 Illustration of $\epsilon_a - \log \sigma_a$ and creep in Isotach theory (Kawabe and Tatsuoka 2013)

In a similar desire to reduce the time for creep investigation of fine soils, Yangsheng et al. (2013) propose a new testing procedure based on the application of dynamic triaxial cycles of frequencies about 5 kHz. The similarity between the measured creep strain evolution and the permanent strains-number of cycles relation in dynamic cyclic tests (Figure 3) is used by the authors for the prediction of the creep strains. The analysis of the microstructure of the clay samples issued from both creep and cyclic loading by the scanning electron microscope shows close resemblances. In a different approach, but somewhat related study by Noda and Xu (2013), a numerical finite element analysis (GEOASIA) of the behaviour of remoulded clay under sustained small cyclic amplitudes followed by creep under undrained conditions reveals that the mechanism of swelling is localised and taking place on the shear bands due to the pore water migration effects.

Re-evaluation of surcharging technique to reduce the secondary compression effects on soft organic clay soils is explored by Feng (2013) by conducting one-dimensional laboratory compression tests on samples with different effective surcharge ratios, R'_s , where

$$R'_s = (\sigma'_s - \sigma'_f) / \sigma'_f \quad (1)$$

and σ'_s is the effective stress achieved at the end of surcharging and σ'_f is the permanent final effective stress. The results show that for small effective surcharge ratios, the magnitude of secondary compression still remains excessive while for the technique to be effective, higher values of R'_s are recommended.

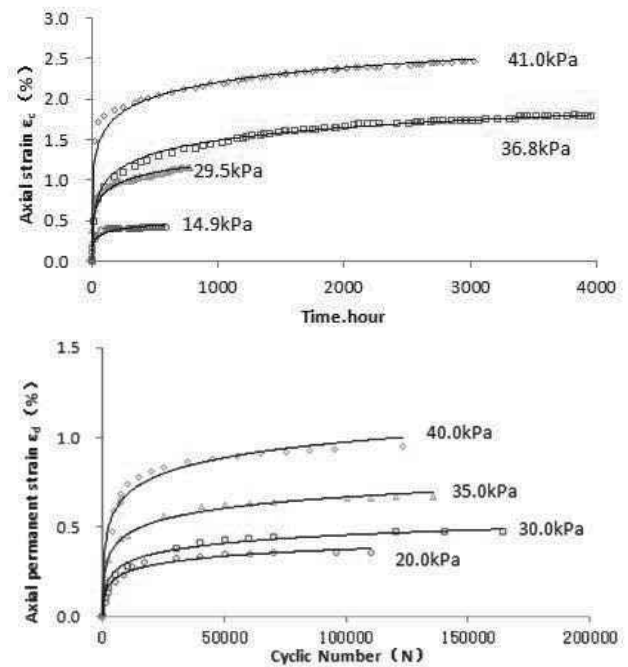


Figure 3 Top: drained creep strains; bottom: dynamic triaxial strain response on remoulded clay, after Yangsheng et al. (2013).

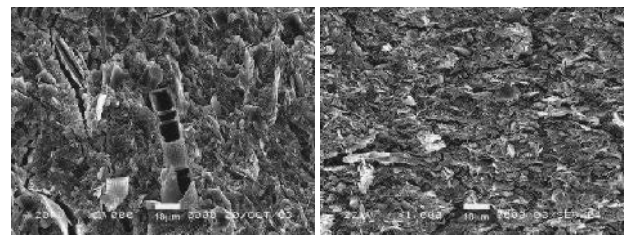


Figure 4. Microstructure of clays observed by SEM (Watabe et al. 2013).

Long-term consolidation laboratory tests on two undisturbed clay soils Osaka Bay (flaky particles, typically smectite, with large number of microfossils, Figure 4a) and Pisa (platy particles, typically illite, with a small number of microfossils, Figure 4b), were conducted and the results reported by Watabe et al. (2013). The interpretation of the tests was also based on the isotach concept (Šuklje 1957) and the relations proposed by Leroueil et al. (1985). While Osaka Bay clay shows high strain rate dependency well approximated by a curve model in line with the response of a wide range of natural clay soils, the Pisa clay revealed particularly smaller strain rate dependency. The origin of this different response appears to be linked to the presence of illite particles in Pisa clay.

The effect of a wide range of strain rates from 0.333 to 60,000 %/hr on the strength and stiffness of reconstituted kaolin clay is investigated in triaxial testing conditions by Robinson and Brown (2013). The strain level and drained conditions are also considered. The influence of the rate effects on the peak shear strength is presented in Figure 5 where the reference data refers to a shear strain rate of 100 %/hr and V is the normalised dimensionless velocity as previously defined by Randolph and Hope (2004). While the experimental results fit well the rate effects equation (Randolph and Hope 2004), the rate effect per log cycle (tenfold) of the shear strength increase is about 22%, higher than detected in various previous studies (about 10%). The applied strain rate was also found to have a significant impact on the small strain response, with the elastic shear strain threshold, γ_{EL} , increasing with strain rate and shear stiffness, G_0 , appearing to reduce with strain rate (Figure 5).

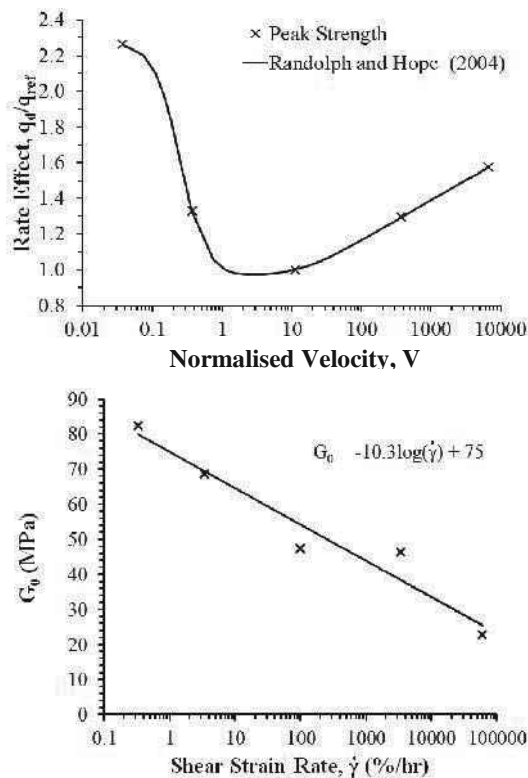


Figure 5. Top: Rate effect against normalised velocity at peak strength experimental data and model by Randolph and Hope (2004) using a shear strain rate of 100%/hr as the reference rate; bottom: variation of G_0 against shear strain rate (Robinson and Brown 2013).

Characterisation in one-dimensional loading and undrained triaxial compression of a bentonite mixed with different amounts of silicon sand performed by Yin and Tong (2013) showed creep, swelling and strain rates effects dependent on the amount of sand. Zhakulin et al. (2013) present results of long-term observations (over 50 years) of settlements of some industrial structures in Kazakhstan founded in clayey soils. Laboratory investigation of the volumetric creep behaviour of the clayey soils is also studied in laboratory on undisturbed samples. Interestingly, it is also shown that the settlements calculated by two different design methods are much higher than the settlements experienced by the real structures.

The settlement behaviour of clay soils, including the effects of time and timescale effects between a thin laboratory specimen and a thick in situ soil layer is not a new research topic, yet still very open as the interpretation of different results remains controversial and does not appear to converge towards a clear and unique framework. The work of Degago et al. (2013) goes at the heart of these controversies by analyzing the validity of creep hypothesis A in open conflict with creep hypothesis B (as previously defined by Ladd et al. 1977). Based on a critical review of relevant experimental investigations from the literature, the main misconceptions around the hypothesis A are discussed and detailed argumentation is advanced. It is also emphasized that the creep hypothesis B agrees well with the measured behaviour of cohesive soils, the experimental results can be consistently explained using a model based on the isotache concept.

The performance of one-dimensional (1D) formulations of two elasto-viscoplastic models for normally consolidated clays, one based on non-stationary flow surface type (Sekiguchi 1977) and the other one over-stress type (Perzyna 1963) is assessed Nakai et al. (2013) against a new model that describes the time-

dependency based on the normal consolidation line (NCL) on $e-\ln \sigma$ plane strain rate dependent shift and using the subloading surface concept by Hashiguchi (1980). The new model is extended for over-consolidated clay cases, by introducing a new state parameter - the difference between the current void ratio and the void ratio on the NCL. The introduction in the model of another state variable, linked to an imaginary increase of density due to bonding effect, successfully captures the response of structured clays as well.

3 EXPANSIVE SOILS

Expansive soils are soils that undergo significant volume change associated with variations in water content. These volume changes can either be in the form of swell or in the form shrinkage and they are derived from clay minerals that undergo hydration due to rainfall and dehydration due to evaporation. However, water access to individual clay particles depends on, and the shrink-swell potential of expansive soils is determined by the soil state (void ratio, the initial water content), internal structure (micropores within soil peds and macropores between the soil peds), vertical stresses, as well as the type and amount of clay minerals like smectite, montmorillonite, nontronite, vermiculite, illite, and chlorite in the soil (Jones and Jefferson 2012).

Maison et al. (2013) present a new experimental device incorporated into an Environmental Scanning Electron Microscope (ESEM). This new apparatus allows the simultaneous measurements of both surface area of clay minerals and water content evolution. The latter is deduced from the sample weighting, while the former is based on the image analysis technique. A complete wetting-dry cycle can be performed in less than 24 hours, faster than the classic tests on representative soil element samples. Figure 6 shows the evolution of the surface strain, ϵ_s , with the moisture content for several homogenous and heterogeneous natural clays. ϵ_s represents the relative change of the surface area S_i of the clay minerals at time t_i in respect with the initial area S_0 :

$$\epsilon_s = \frac{S_i - S_0}{S_0} \quad (2)$$

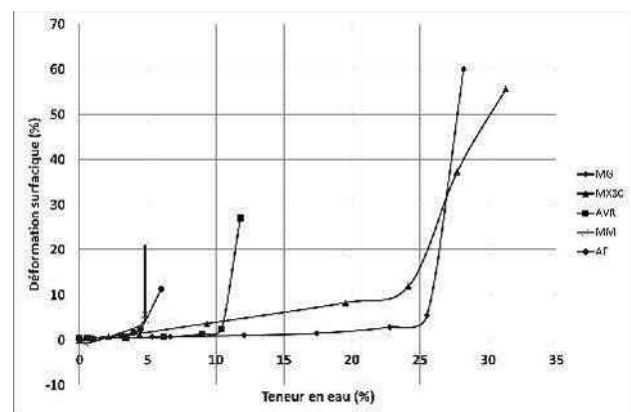


Figure 6. Variation of the surface strain with moisture content measured with the new experimental device developed by Maison et al. (2013).

Internal structure evolution of undisturbed and remoulded expansive soils during triaxial loading conditions and with the application of several wet-dry cycles is studied by Chen (2013) on a CT scanning device. Both loading conditions accelerate the development of the initial structural cracks and fissures (Figure 7). The process of shrinkage causes structural cracks, which on re-wetting, do not close-up and facilitates the access of the water.

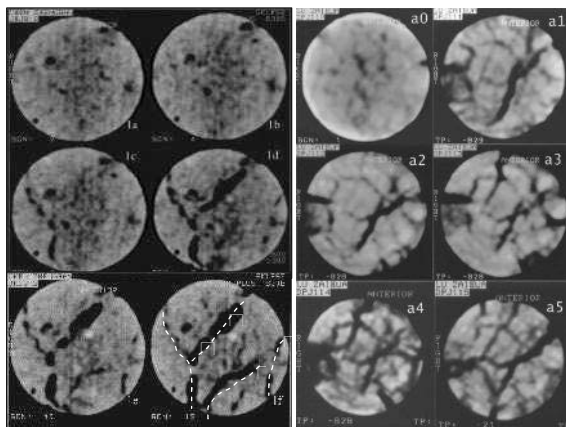


Figure 7. Internal structure evolution by CT scanning (section located at 1/3 of height) during: left - triaxial compression and right - several wet-dry cycles (Chen 2013).

A comprehensive characterisation of the engineering properties of the expansive glacio-lacustrine Regina clay (Canada) is conducted by Azam et al. (2013). The clay contains minerals like smectite, hydrous mica and chlorite, whereas the clay size fraction (material finer than 0.002mm) is around 65%. The soil water characteristic curve (SWCC) determined to investigate the water retention capacity of the soil showed the bimodal shape with two air entry values: an initial low value corresponding to macroporous drainage (through fissures) followed by a high value related to microporous flow (Figure 8, top). The shrinkage curve was found to be S-shaped and included a low structural shrinkage followed by a sharp decline during normal shrinkage and then by a low decrease during residual shrinkage (Figure 8, bottom).

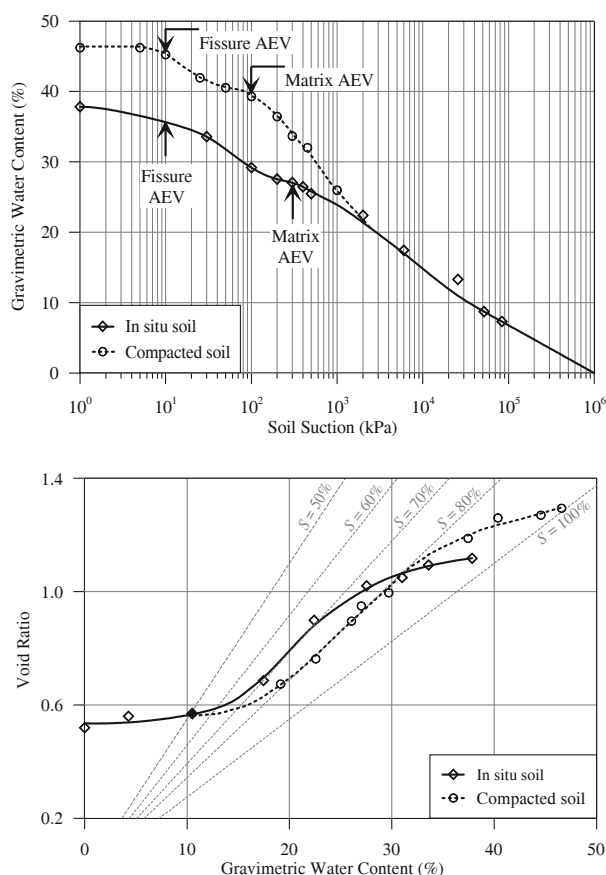


Figure 8. Top: SWCC with gravimetric water content; bottom: shrinkage curves (Azam et al. 2013).

The influence of swelling on the mechanical properties of Clay Geosynthetic Barrier that contains bentonite (predominantly montmorillonite) is studied by Domitrović and Zelić (2013). Initial swelling behaviour is explored through long-term swelling tests using an oedometer. The intensity of swelling decreases as normal stress level increases, however, the time for completion of the primary swelling is independent on the stress level, approximately 31 days. The shear behaviour of the bentonite is studied in direct shear box on samples under three normal stresses and with different hydration time (7, 14 and 21 days). Table 1 shows the value of the peak and residual shear strength parameters; the angle of friction increases while the cohesion decreases with the hydration time.

Table 1. Shear strength parameters (Domitrović and Zelić 2013).

Hydration time	Peak parameters		Residual parameters	
	c (kPa)	ϕ (°)	c (kPa)	ϕ (°)
7 days	11.99	11.23	11.05	7.80
14 days	8.04	12.47	4.79	9.38
21 days	6.32	12.27	3.63	9.31

Residual strength of three clay soils (clay fraction between 70% and 80%) from North-Eastern Greece with high and very high swelling potential is evaluated by performing ring shear tests on remoulded samples at their optimum moisture content by Markou (2013). While the residual failure envelope appears curved, for normal stresses up to 200 kPa it can be considered as linear. In all cases, the residual friction angle does not exceed 14°.

The analysis of expansive properties of natural Neogen clays (smectite, illite minerals) that forms the main foundation stratum of Northern Poland is performed by Kumor (2013). Expansive clays of Northern Poland can be classified as very expansive with a contractibility range ($LL - SL$) of 82.1% > 50%, where LL is the liquid limit and SL is the contractility limit. Swelling pressures that cause lifting of structures and shrinkage that cause differential settlements are the main causes of structural failures in Poland (Figure 9). The examination of the volumetric changes in clays during laboratory shrinkage and swelling phases allowed the refinement of the in-situ shrink/swell predictions.

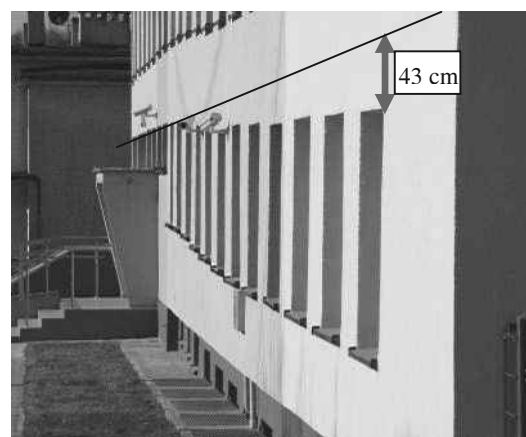


Figure 9. An example of a construction failure resulting from the application of natural swell-shrink cycles in Poland (Kumor 2013).

4 CONSOLIDATION/COMPRESSIBILITY

It is known that three-dimensional effects may significantly accelerate the rate of settlement of foundations on clay, primarily because of the ability of excess pore water pressure to dissipate horizontally as well as vertically. These effects are further accelerated by the use of radial drainage. However, most

of the analytical solutions for consolidation under radial drainage are based on the assumption that only vertical compression occurs. A new physical consolidation model test under plane strain condition, which enables the observation of the deformation of a specimen through digital image analysis, and the measurement of the total vertical stress as well as the pore water pressures, is developed by Kim et al. (2013). A schematic view of the apparatus is shown in Figure 10. Consolidation tests of reconstituted kaolin samples (150 mm height, 140 mm width, and 40 mm thickness) were performed using the new consolidation apparatus under radial drainage, along with test under vertical drainage. Radial deformations as well as vertical deformations during consolidation were monitored and compared for the two different drainage conditions. Significant horizontal displacements are occurring during the early stages of the consolidation process, whereas during the intermediate stage, only minor horizontal displacements are identified. The horizontal displacements are higher near the boundaries and this induces uneven void ratio distribution at the end of consolidation. Meanwhile, for the vertical drainage, only minor variations of the horizontal displacements inferior of 0.006 mm were observed.

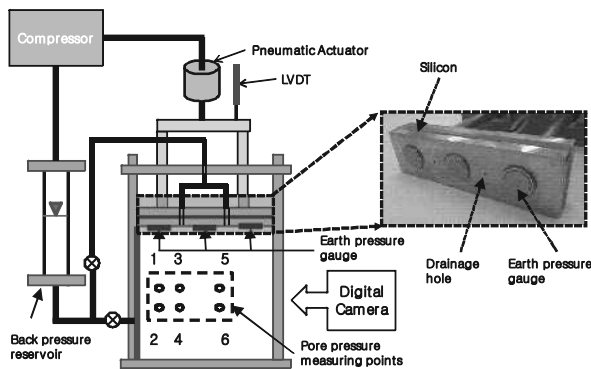


Figure 10. Schematic view of the consolidation test apparatus (Kim et al. 2013)

A laboratory apparatus for axi-symmetric electro-osmotic consolidation of cylindrical samples of 37.6cm diameter and 20cm height is developed by Hu et al. (2013) with capabilities of measuring the electrical voltage, soil mass displacement, water discharge, and electrical current parameters (Figure 11). While tests on kaolin clay showed non-linear variation of soil parameters and complex coupling effects between water flow, soil deformation and electrical properties, a theoretical model that integrates Biot's consolidation equation with the electro-osmotic flow and the equation for an electric field is proposed. Figure 12 shows the comparison of the surface settlement between the numerical results and the experiment data. The simulations of the model with variable electrical conductivity agree better with the experimental data than those of the model with constant electrical conductivity.

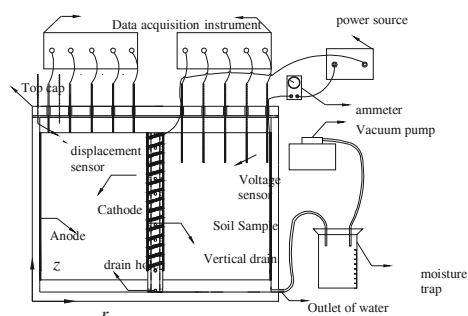


Figure 11. Electro-osmotic apparatus (Hu et al. 2013)

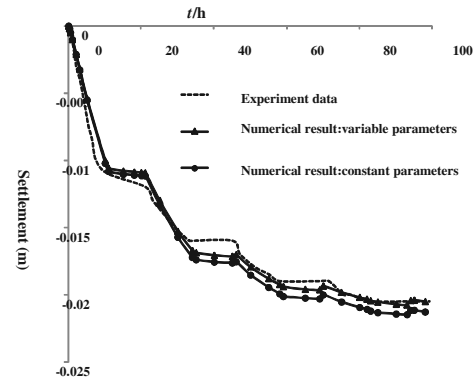


Figure 12. Comparison of the settlement between the numerical results (two models) and recorded data (Hu et al. 2013).

Compression properties of Swedish fine-grained sulphide clay soils are explored by Westerberg and Andersson (2013) through the monitoring and analysis of the long-term response of two instrumented real scale test embankments. Actually, the predicted settlements of a construction founded on sulphide soils deviates significantly from those measured in situ, and normally the predicted settlements are too small. In sulphide soils, the structure is often relatively porous and the voids between the mineral grains and clay particles are filled with pore water, organic material and iron sulphide. Field investigations of the properties of the sulphide soil were performed by cone penetration tests, field vane tests and Swedish piston sampling and an extensive program of laboratory investigation, in oedometer, both by incremental loading and constant rate of strain, creep tests, permeability as well as undrained direct simple shear tests, were conducted for the determination of compression and strength properties. Very good agreement between data given by different field instrumentations is reported.

The heave rebound strain prediction of overconsolidated soils represents an important engineering design issue since the development of major road infrastructures that involve deep excavations and trenches. Based on the use of simple oedometer tests combined with an analysis of stress paths approached by Ylight model (Leroueil, Magnan & Tavenas, 1985), Petit et al. (2013) present a quantification method of those heave rebound strains. The results of the calculations of the heave rebound over a 120m deep clay deposit for three excavation depths of 10, 20 and 40m are shown in the Figure 13. The estimations show that the elastic rebound are relatively small, between 0.01 and 0.02m.

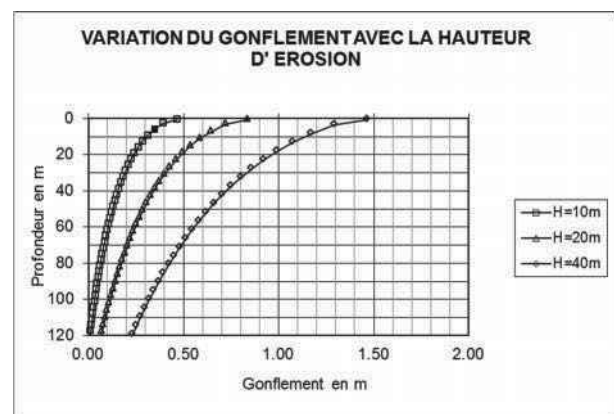


Figure 13. The profile of the heave rebound for three different excavation depths (Petit et al. 2013).

The problem of estimation of the at rest coefficient K_0 for overconsolidated soils is treated by Bohac et al. (2013). At a site containing highly overconsolidated clays, K_0 was measured by using Marchetti (1980) flat dilatometer. The K_0 values are higher than those estimated from oedometer yield point and empirical correlations by Main and Kulhawy (1982). Numerical modelling of the flat dilatometer penetration into the soil based on a hypoplastic model (Mašin, 2005) combined with the intergranular strain concept proposed by Niemunis and Herle (1997) were also performed. However the K_0 values are slightly underpredicted.

5 CRUSHABLE GRANULAR SOILS

The significance of particle crushing to the mechanical behaviour of granular materials has been well identified and in the particular field of geomechanics, it is recognized as having a major effect in a wide range of practical problems involving shallow foundations, bearing capacity of piles, stability of earth structures, seepage properties, hydraulic conductivity, and pore-pressures distribution in earth dam water retaining structures, pavement and railway substructures. However, the laboratory investigation and interpretation of crushable materials show some inherent difficulties and there is a need for consideration of appropriate approaches that may differs from non-crushable, cohesionless sands. In this respect, Wils et al. (2013) identify and analyse several issues like: determination of the minimum and maximum density, sieving and the need for advanced grain morphology characterisation by means of microscopy, the abrasion of particles and the apparent cohesion due to interlocking of the angular particles creating sand clusters with the appearance of larger particles, as well as the existence of larger shells sheltering smaller grains.

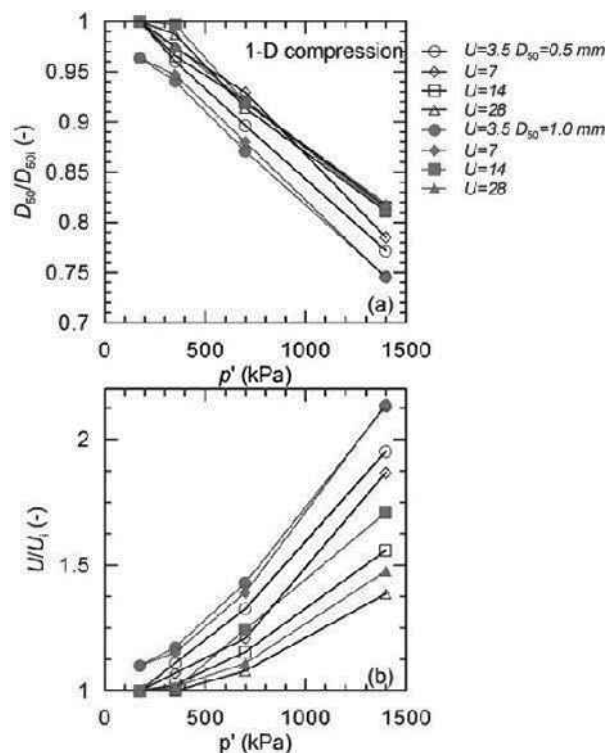


Figure 14. Evolution of ratios: (a) d_{50}/d_{50i} and (b) U/U_i with mean effective stress applied in 1D-compression (Casini et al. 2013).

An experimental investigation into the mechanical response of an artificial granular material, consisting of crushed expanded clay pellets, LECA (Light Expanded Clay Aggregate)

under various stress path tests is conducted by Casini et al. (2013). The material was reconstituted, with a maximum particle size of 2 mm, into grading curves with the same mean diameter, d_{50} , and different coefficients of uniformity, U ($= 3.5, 7, 14, 28$) or the same U and different d_{50} ($= 0.5, 1 \text{ mm}$). Changes in of the grading at the end of the tests (one dimensional, isotropic and triaxial compression) were described using two parameters defined as the mean diameter and coefficient of uniformity of the final distribution over the values of the initial distributions, d_{50}/d_{50i} and U/U_i , respectively. Poorly graded samples show more pronounced decrease in mean diameter and increase of uniformity with higher applied stresses (Figure 14).

Experimental evidence of size effects in rockfill shear strength and the link with the crushing strength of rock aggregates is presented by Frossard et al. (2013). Compression test results on individual chalk (CP) and quartz rock (STV) particles have been statistically analysed and it was shown that the probability of survival for each grain size fraction follows Weibull distribution given by:

$$P_s(d) = \exp \left(- \left(\frac{d}{d_o} \right)^{n_d} \cdot \left(\frac{\sigma}{\sigma_o} \right)^m \right) \quad (3)$$

where σ_o is the value of the strength of d_o size particles such that 37% of the total number of tested particles survive. The exponent m is called the Weibull modulus, which increases with decreasing variability in strength of the material. For each material, drained triaxial tests on dry samples (maximum particle diameter sizes of 40mm (CP1 and STV1) and 160mm (CP2 and STV2)) under different confining pressures were performed. Considering that individual particle breakages affect the shear strength of the entire granular medium, a method to predict the effects of scale on shear strength envelopes of rockfills is proposed. Figure 15 shows the comparison between measurements and predictions for shear strength envelopes for both CP2 and STV2 materials, based on the best fit of CP1 and STV1 data with the failure envelope given by De Mello (1977) ($\tau = A \cdot \sigma_n^b$).

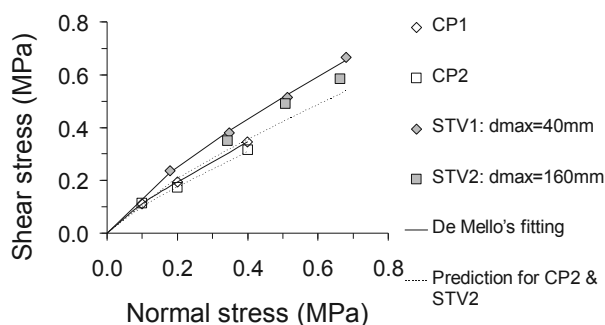


Figure 16. Scale effect on shear strength (Frossard et al. 2013).

Investigations performed by Shahnazari et al. (2013) on the mechanical behaviour in triaxial apparatus of two calcareous natural soils obtained from the Persian Gulf, Hormuz Island sand (HI) and Bushehr Port sand (BP) (Figure 16), and comparison with the triaxial response of a silica sand, Firoozkuh (F) showed behavioural contrast between the sand materials, and through global measurements, clear evidence of particle breakage of the former soils.

Results from a series of basic characterisation tests conducted on reconstituted samples of carbonate sand to understand its behaviour are reported by Safinus et al. (2013). Compared to silica sand, carbonate sand has considerably higher angularity, lower grain hardness and higher intra-particle

porosity, which result in high friction angles and compressibility. The corresponding dilatancy is affected strongly by the confining stress (Figure 17): even for low relative densities, dilation occurs at low confining stresses, reflecting the greater particle interlocking compared to silica sand, while with the increase of confining stress, the dilatancy is suppressed quickly, and finally diminishes completely at a relatively low stress level, due to particle degradation. The influence of the observed characteristics of carbonate and silica sands on practical applications was examined through model tests in drum centrifuge of spudcan foundations penetrating through four-layer soils, with a carbonate or silica sand layer interbedded in soft clay. All measures of spudcan punch-through severity were significantly lower for interbedded carbonate sand despite its higher friction angle ($\phi_{crit} = 40^\circ$) compared to silica sand ($\phi_{crit} = 34^\circ$), Figure 18.

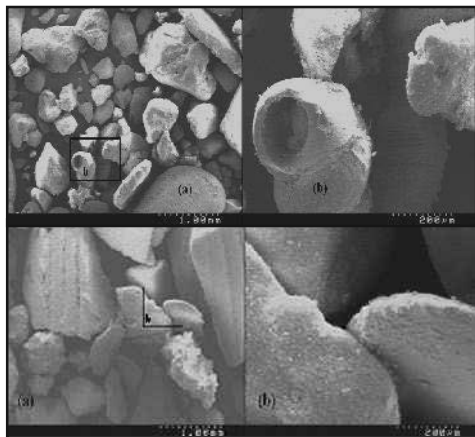


Figure 16. Microscopic images of Hormuz Island sand (top) and Bushehr Port sand (bottom) from Shahnazari et al. (2013).

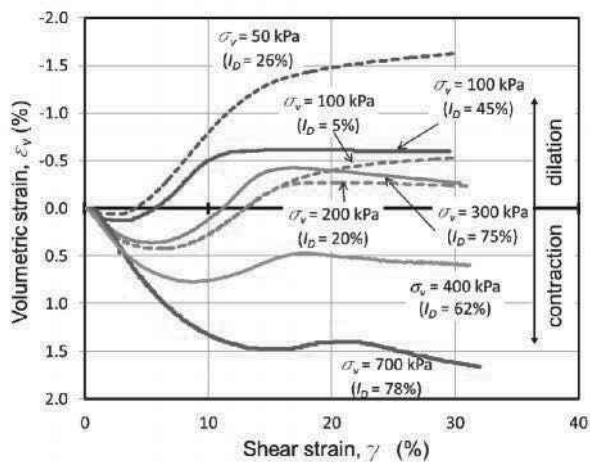


Figure 17. Volumetric change of carbonate sand in drained simple shear test with lateral stress ratio $K = 0.4$ (Safinus et al. (2013)).

Observations of the experimental behaviour of two pumice sands from Japan and New Zealand are made by Kikkawa et al. (2013). While the composition of Pumice sands is dominated by silica and aluminium oxide, they are normally characterised by the presence of particles that are easily crushed against a hard surface under fingernail pressure. Considerable particle breakage is occurring in triaxial compression loading, while mobilization of the drained shear strength is increasing steadily with the shear strain. However, owing to a different stress path, less crushing is occurring in undrained testing. A particularly interesting feature of the drained shear behaviour of these two materials is that with a sufficiently large confining pressure they

tend to deform in one-dimensional compression way, with practically no change in the diameter of the sample, also a consequence of the large amount of particle crushing. An example of observed incremental strain ratio evolution for one of the materials during triaxial testing is shown in Figure 19.

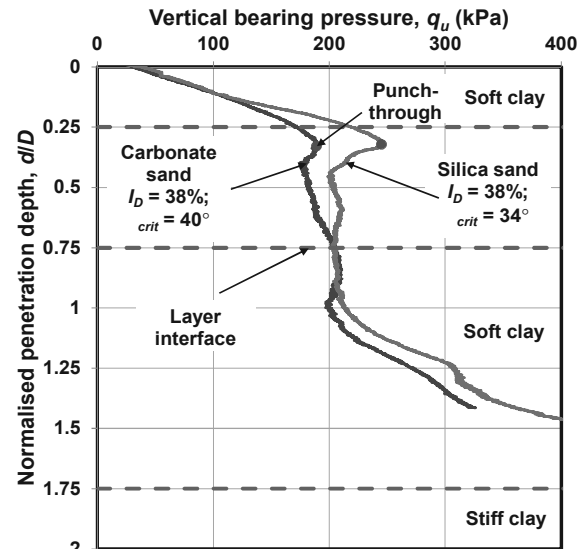


Figure 18. Effect of interbedded sand mineralogy on load penetration response: severity of punch-through (Safinus et al. (2013)).

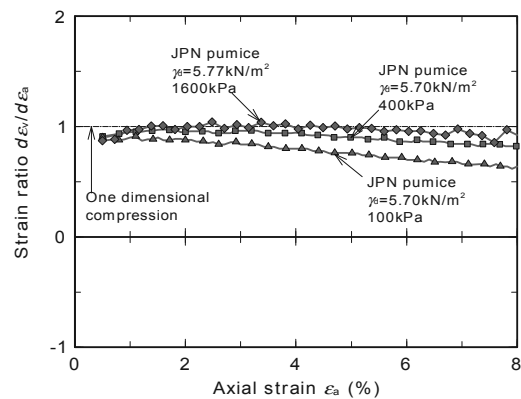


Figure 19. Incremental strain ratio against axial strain for drained triaxial tests on pumice sand from Japan (Kikkawa et al. 2013).

6 CONCLUSION

This paper presents a General Report of 29 written contributions from authors representing 22 countries submitted for one of the parallel sessions, *Time effects and other peculiar observation*, of the Technical Committee 101: Laboratory Stress Strength Testing of Geomaterials. The General Report reviews these contributions and presents the current research directions mainly in relation to the experimental behaviour as well as the key outcome results. The topics covered by the written contributions have been grouped within the following thematic strands:

- Time effects: creep, strain rate effects in fine soils;
- Consolidation/compressibility (loading and unloading);
- Expansive soils: swelling/shrinkage characterisation;
- Crushable granular soils.

Various analytical and constitutive modelling approaches successfully assisted the interpretation of the presented experimental data and in general, the research methodology

adopted considered multi-scale investigations, including soil structure analyses and correlation with macro soil response. Reference to practical applications was also considered.

REFERENCES

- Azam, S, Ito, M and Chowdhury, R. 2013. Engineering properties of an expansive soil. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Boháč, J, Mašin, D, Malát, R., Novák, V and Rott, J. 2013. Methods of determination of K_0 in overconsolidated clay. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Casini, F, Leu, J, Low, B, Wanninger, F, Zimmermann, A, Zwicker, P, Springman, SM, Viggiani, GMB. 2013. Stress-path effects on the grading of an artificial material with crushable grains. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Chen, Z. 2013. Characteristics of structure evolution of expansive soil and loess during loading and wetting. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Degago SA, Grimstad, G, Jostad, H P, Nordal, S. 2013. Misconceptions about experimental substantiations of creep hypotheses A. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- De Mello, VFB. 1977. Reflections on design decisions of practical significance to embankment dams. *Géotechnique*, Vol. 27, No.3, pp. 281-355
- Domitrović, D, and Zelić, BK. 2013 The relationship between swelling and shear strength properties of bentonites. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Feng, TW. 2013. Reappraisal of Surcharging to Reduce Secondary Compression. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris.
- Frossard, E, Ovalle1, C, Dano, C, Hicher P-Y., Maiolino, S. Hu, W. 2013. Effets d'échelle dus à la rupture des grains sur la résistance au cisaillement d'énrochements. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Hu, L, Wu, H. and Wen, O. 2013. Electro-osmotic consolidation: laboratory tests and numerical simulation. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Jones, LD, Jefferson, I. 2012 Expansive soils. In: Burland, J., (ed.) ICE manual of geotechnical engineering. Volume 1, geotechnical engineering principles, problematic soils and site investigation. London, UK, ICE Publishing, 413-441
- Kawabe, S and Tatsuoka, F. 2013. Creep characteristics of clay in one-dimensional compression with unloading/reloading cycles. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris.
- Kikkawa, N, Pender MJ, and Orense RP. 2013. Comparison of the geotechnical properties of pumice sand from Japan and New Zealand. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Kim, JY, Chung, CK, Cho, NG, Yune, CY. 2013. Evaluation of Consolidation Behavior of Soils under Radial Drainage Condition Using Digital Image Analysis. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Kumor, MK. 2013. The expansive properties of Poland's clay subsoil. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Ladd CC, Foott R, Ishihara K, Schlosser, F and Poulos, HG 1977. Stress-deformation and strength characteristics. State-of-the-art report. *Proc. 9th ICSMFE*, Tokyo 2, 421-494.
- Leroueil S., Kabbaj M., Tavenas F. and Bouchard, R. 1985. Stress-strain-strain rate relation for the compressibility of sensitive natural clays, *Géotechnique*, 35(2), 159-180.
- Leroueil, S., Magnan, JP and Tavenas, F. 1985 F. Remblais sur argile molle. Lavoisier Eds.
- Leroueil, S. 2006. Suklje's Memorial Lecture – The isotache approach: Where are we fifty years after its development by Professor Suklje? *Proc. European-Danube Conference on Geotechnical Engineering*, Ljubljana, 1, 55-88.
- Mayne, P.W. and Kulhawy, F.H. (1982) K_0 -OCR relationships in soil. *J. Geotech. Eng. Div. ASCE*, GT6, 851-872.
- Maison, T., Laouafa, F., and Delalain, P. 2013. Mise au point d'un dispositif expérimental pour l'analyse du retrait-gonflement des sols argileux. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Marchetti, S. (1980) In situ tests by flat dilatometer. *J. Geot. Eng. Div. ASCE*, Vol. 106, NoGT3, 299-321.
- Markou, IN 2013. Residual shear strength behavior of swelling soils. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Mašin, D. (2005). A hypoplastic constitutive model for clays, *International Journal for Numerical and Analytical Methods in Geomechanics*, 29, No. 4, 311-336
- Nakai, T, Shahin, HM and Kyokawa, H. 2013. Rational expression of time-dependent behavior from normally consolidated soil to naturally deposited soil. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Niemunis, A, and Herle, I. (1997). Hypoplastic model for cohesionless soils with elastic strain range. *Mech. Cohesive-Frict. Mater.*, 2, 279-299
- Noda, T and Xu, B. 2013. Oscillation of Acceleration Accompanying Shear Band and Subsequent Time-Dependent Behavior in Overconsolidated Clay under Undrained Plane-Strain Conditions. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Randolph, M.F. and Hope, S. 2004. Effect of cone velocity on cone resistance and excess pore pressures. *Proc. Int. Symp. Eng. Practice and Performance of Soft Deposits*, 147-152.
- Robinson, S. and Brown, M.J. 2013. Rate effects at varying strain levels in fine grained soils. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Petit, G., Virollet, M., and Bernard, Ph. 2013. Quantification des gonflements des sols surconsolidés en fond de déblai. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Safinus, S, Hossain MS and Randolph M.F. 2013. Comparison of Stress-Strain Behaviour of Carbonate and Silicate Sediments. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Shahnazari, H, Salehzadeh, H, Rezvani R, and Tutunchian M.A. 2013. Experimental study on drained shearing behavior of calcareous sands. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Šuklje L. 1957. The analysis of the consolidation process by the isotache method, *Proc. 4th Int. Conf. on Soil Mech. Found. Engng.*, London, Vol.1, 200-206.
- Zhakulin, A.S., Zhakulina, A.A., Orazaly, E.E., Orazalin, Z.Y. 2013. Special Features of Creep of Clayey Soils. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Watabe, Y., Sassa, S. and Udaka, K. 2013 Comparative study of long-term consolidation for subsoils under Kansai Airport and Pisa Tower. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Westerberg, B, and Andersson, M. 2013A new test field in sulphide clay with test embankments for study of compression properties. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Wils, L., Van Impe, WF, and Van Impe, P.O. 2013. Laboratory testing issues related to crushable sands. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Yangsheng, Y, Qianli, Z, Degou.CF, Chen, JY and Lijun, W. 2013. Study on New Method of Accelerated Clay Creep Characteristics Test. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris
- Yin, JH and Tong, F. 2013. Test study and constitutive modelling of the time-dependent stress-strain behavior of soils. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris

Engineering properties of an expansive soil

Propriétés mécaniques d'un sol gonflant

Azam S., Ito M., Chowdhury R.

Environmental Systems Engineering, University of Regina, Regina, SK, Canada

ABSTRACT: The engineering properties of an expansive soil were investigated using in situ and compacted samples. The soil water characteristic curve was found to have a bimodal shape composing of two air entry values: an initial low value corresponding to macroporous drainage followed by a high value related to microporous flow. Likewise, the shrinkage curve was found to be S-shaped and included a low structural shrinkage followed by a sharp decline during normal shrinkage and then by a low decrease during residual shrinkage.

RÉSUMÉ: Les propriétés mécaniques d'un sol gonflant ont été étudiées à l'aide d'échantillons compactés obtenues sur le terrain. La courbe caractéristique en eau du sol s'est révélée avoir une forme bimodale avec deux valeurs d'entrée d'air: une valeur initiale correspondant à un faible drainage macroporeux suivi d'une valeur élevée par rapport à l'écoulement microporeux. De même, la courbe de rétrécissement est en forme de S et inclus un faible rétrécissement structurelle suivie d'une forte baisse pendant le rétrécissement normal, suivie d'une diminution faible durant le rétrécissement résiduel.

KEYWORDS: expansive soil, soil water characteristic curve, shrinkage curve.

1 INTRODUCTION

The capital of Saskatchewan, Canada, is founded on a glacio-lacustrine clay deposit that exhibits significant volume changes due to seasonal weather variations. Alternate swelling and shrinkage in the expansive Regina clay has impaired civil infrastructure such as transportation networks (Kelly et al. 1995), residential, industrial, and commercial facilities (Ito and Azam 2010), and water supply and sewage collection systems (Hu and Hubble 2005). Damages to engineered facilities are clearly manifested in the form of differential heave in roadways and sidewalks, inclined cracking in slab-on-grade basements and masonry walls, and fatigue and breakage in underground storage tanks and buried pipelines. The associated repair cost is usually quite enormous. For example, the breakage rate in the 850 km long water supply network in the city has now reached a 30-year maximum of 0.27 breaks/km/year, costing more than \$2 million in annual maintenance. Furthermore, the city is currently going through a period of infrastructure development including mega-projects such as the Global Transportation Hub and the Downtown Covered Stadium with a 55000-seat capacity. Clearly, there is a need to study site-specific soil properties for the continuous maintenance and improved design of civil infrastructure systems in Regina.

Generally, volume changes in expansive soils are derived from clay minerals that undergo hydration due to rainfall and dehydration due to evaporation. This process is governed by the attraction of bipolar water molecules to the negatively charged clay particles possessing high specific surface areas (Mitchell and Soga 2005). However, water access to individual clay particles primarily depends on the following two factors: (i) soil structure (micropores within soil peds and macropores between the soil peds) and (ii) soil state (void ratio and degree of saturation). These parameters are respectively governed by parent geology and construction practices prevalent in an area. Recent research on local expansive clays has focused on the determination of unsaturated soil properties using “undisturbed samples” from the geological deposit (Azam and Ito 2011) and

on the correlation of these properties with morphological observations using “cryogenic specimens” in a scanning electron microscope (Ito and Azam 2013). These studies concluded that the geologically-induced soil structure governs the water migration and the swell-shrink patterns through the expansive clay. The present study extends the current body of knowledge to compacted soils thereby capturing the effect of soil state on the properties of local clays. Overall, a generalized theoretical framework is developed to understand the behavior of expansive soils.

The main objective of this paper is to understand the engineering properties of Regina clay using in situ and compacted specimens. Geotechnical index properties were determined for preliminary soil assessment. The soil water characteristic curve (SWCC) was determined to investigate the water retention capacity of the soil. Likewise, the shrinkage curve was determined to correlate volume changes with soil saturation and desaturation.

2 RESEARCH METHODOLOGY

The expansive clay was retrieved from a local soil deposit that was found to be desiccated in early Fall and exhibited extensive fissuring oriented in all directions. High quality undisturbed samples were obtained using the ASTM Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes (D1587-08) from a depth of 0.6 m to 1.2 m. Likewise, disturbed samples were obtained from bore cuttings according to the ASTM Standard Practice for Soil Investigation and Sampling by Auger Borings (D1452-09). All of the specimens were plastic-wrapped and wax-coated and the entire collection was transported and stored at the University of Regina as per the ASTM Standard Practice for Preserving and Transporting Rock Core Samples (D5079-08). The latter samples were compacted in accordance with the ASTM Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (D1557-12).

The geotechnical index properties were determined according to the ASTM test methods as follows: (i) water content (w) by the Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass (D2216-05); (ii) dry unit weight (γ_d) by the Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method (D2937-10) for the in situ sample and by the above-mentioned method for the compacted sample; (iii) specific gravity (G_s) by the Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer (D854-10); (iv) liquid limit (w_l), plastic limit (w_p) and plasticity index (I_p) by the Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils (D4318-10); and (v) grain size distribution (GSD) by the Standard Test Method for Particle-Size Analysis of Soils (D422-63(2007)). The entire GSD data is not given in this paper.

The SWCC was determined according to the ASTM Standard Test Methods for Determination of the Soil Water Characteristic Curve for Desorption Using a Hanging Column, Pressure Extractor, Chilled Mirror Hygrometer, and/or Centrifuge (D6836-02(2008)e2) on 10 mm thick samples obtained from both the undisturbed core and the compacted sample. Predetermined values of matric suction were applied using pressure plate and pressure membrane extractors manufactured by Soil Moisture Equipment Inc. These equipment included the following: (i) a 5 bar pressure plate extractor (Model 1600) for up to 200 kPa suction; (ii) a 15 bar pressure plate extractor (Model 1500F1) for suction values ranging from 300 kPa to 500 kPa; and (iii) a 100 bar pressure membrane extractor (Model 1020) for suction values between 2000 kPa and 7000 kPa. The porous plates and the cellulose membranes were submerged in distilled and de-aired water for 24 hours to expel air bubbles. Thereafter, the specimens along with the retaining ring were placed on their respective porous plate or cellulose membrane and allowed to saturate. Next, the excess water was removed and each plate or membrane was placed in the designated extractor. For each suction value, the expelled water from the samples was monitored in a graduated burette. When two consecutive readings nearly matched over a 24 hour period, the test was terminated and the sample water content was determined.

The dew point potentiometer (WP4-T) was used for suction measurement at low water content corresponding to total suction values greater than 7000 kPa. The sampling cup was half filled with soil to ensure accurate suction measurement (Leong et al. 2003) by using about 5 mg of material with a known water quantity. The unsaturated sample was forwarded to the head space of the sealed measurement chamber, set at 25°C temperature, through a sample drawer and was allowed to equilibrate with the surrounding air. Equilibration was usually achieved within 10 min to 20 min, as detected by condensation on a mirror and measured by a photoelectric cell. From knowledge of the universal gas constant, R (8.3145 J/mol°K), sample temperature, T (°K), water molecular mass, X (18.01 kg/kmol), and the chamber relative humidity, p/p_o , soil suction was calculated ($\psi = RT/X \ln(p/p_o)$) and displayed on the potentiometer screen. The water content of the soil was measured as described earlier.

The shrinkage curve was determined in accordance with the ASTM Standard Test Method for Shrinkage Factors of Soils by the Wax Method (D4943-08). To obtain the void ratio, the volume of soil specimens was determined using the water displacement method. Each specimen was coated with molten microcrystalline wax ($G_s = 0.9$) and allowed to cool down at room temperature. After wax solidification, the sample was submerged in a 250 mL graduated cylinder that was filled with distilled water. The water height in the cylinder was carefully recorded using a Vernier caliper before and after sample submersion in the cylinder. A graduated syringe was used to remove the increased amount of water displaced by the sample

thereby bringing the water height back to the initial reading. The displaced water mass was determined by weighing the graduated syringe before and after water filling and recording the difference. This quantity was readily converted to water volume representing the volume of the wax-coated soil. The volume of soil was obtained from the difference of volume of the wax coated sample and the volume of wax (mass/0.9). A 7.4% correction was applied to account for the underestimation due to air entrapment at the soil-wax interface, as suggested by Prakash et al. (2008). The sample mass was also determined to estimate the bulk unit weight of the soil that, in turn, was converted to the void ratio using basic phase relationships.

3 RESULTS AND DISCUSSION

Table 1 summarizes the geotechnical index properties of the investigated soil. The water content and the dry unit weight of the in situ sample were found to be 31% and 1.34 g/cm³, respectively. In early Fall when the sample was collected, the soil generally experiences a net water deficit given the semi-arid climate prevalent in the region. This was evident from the unsaturated ($S = 82\%$) state of the sample in the field: the field void ratio was calculated to be 1.05. Similar initial conditions ($w = 38\%$ and $\gamma_d = 1.29$ g/cm³) were chosen for the compacted sample to obtain comparable data: the corresponding saturation and void ratio were found to be 86% and 1.18, respectively. The high liquid limit and plastic limit indicate the high water adsorption capability of the clay. These values are attributed to the presence of expansive clay minerals such as smectite, hydrous mica, and chlorite (Ito and Azam 2009). Likewise, the clay size fraction (material finer than 0.002 mm) was found to be around 65%. The fine grained nature of the soil suggests a high water retention capacity. The calculated soil activity ($A = I_p / C$) of about 0.8 is associated with moderate swelling. Overall, the soil was classified as CH (clay with high plasticity) according to the Unified Soil Classification System (USCS).

Table 1. Summary of geotechnical index properties

Property	In situ	Compacted
Water Content, w (%)	31	38
Dry Unit Weight, γ_d (g/cm ³)	1.34	1.29
Specific Gravity, G_s	2.75	2.74
Void Ratio, e^*	1.05	1.18
Degree of Saturation, S (%)†	81	86
Liquid Limit, w_l (%)	83	77
Plastic Limit, w_p (%)	30	27
Plasticity Index, I_p (%)	53	50
Clay Size Fraction, C (%)	66	64
USCS Symbol	CH	CH

$$* e = (G_s \gamma_w / \gamma_d) - 1$$

$$† S = w \gamma_s / e$$

Figures 1 shows SWCC with gravimetric water content. The samples were put in a water tub for one week and the water content measured 38% for the in situ sample and 46% for the compacted sample. Irrespective of the initial water content, the SWCC data fitted well to bimodal distributions with two air entry values: a lower value (10 kPa) corresponding to drainage through fissures followed by a higher value (300 kPa and 100 kPa for the two samples, respectively) associated with seepage through the soil matrix. When the samples were gradually desaturated, air first entered into the fissures at low suction.

Although the fissures are sealed due to hydration of clay minerals, these discontinuities have much lower tensile strengths than the soil aggregates (Azam and Wilson 2006). This led to a quick drainage through these paths of least resistance. Subsequent application of suction affected the soil aggregates and eventually forced air to enter into the pore system of the aggregate. The upward SWCC shift of the compacted sample with respect to the in situ sample is attributed to the high initial water content of the former sample. Upon water inundation, this sample favored particle hydration because of a comparatively looser state ($e = 1.18$). Likewise, the relatively homogeneous structure of the compacted sample correlated well with its smaller difference between the two air entry values. The effect of soil structure was eliminated when the two curves merged at higher suction. Desaturation occurred at an increased rate up to residual suction of 200000 kPa ($w = 5\%$) and the curves finally joined the abscissa at 10^6 kPa.

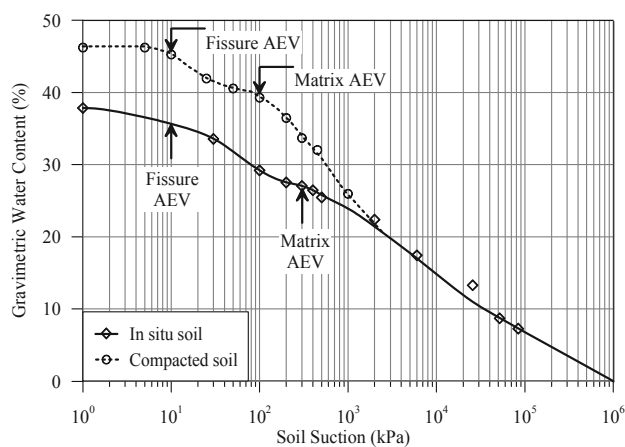


Figure 1. SWCC with gravimetric water content

Figure 2 presents the SWCC in the form of void ratio as a function of soil suction. The measured data closely followed unimodal distributions showing a single air entry value (300 kPa for the in situ sample and 100 kPa for the compacted sample). The latter sample plotted at a higher void ratio at low suction and merged with the in situ sample at 500 kPa. Thereafter, the singular curve exhibited a sharp decrease in void ratio up to the residual condition, became asymptotic to the abscissa after the residual suction and never reporting to 10^6 kPa on complete drying. This is because void ratio of a soil pertains to an average value for all voids and does not differentiate between inter-aggregate fissures and intra-aggregate pores. Clearly, SWCC representation in the form of void ratio is not suitable for expansive soils.

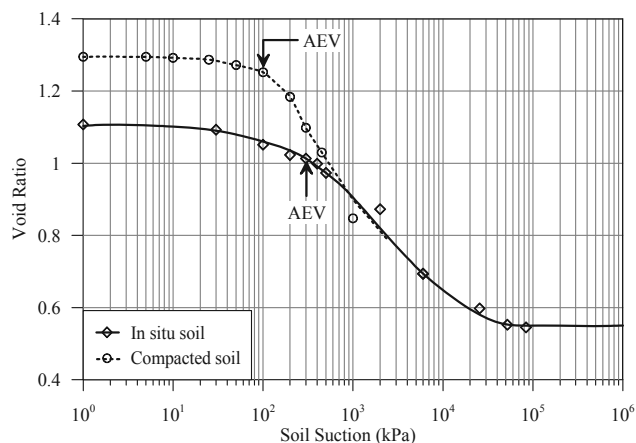


Figure 2. SWCC with void ratio

Figure 3 gives the SWCC in the form of degree of saturation versus suction. Similar to Figure 1, the laboratory measured

data depicted a bimodal function with a fissure air entry value of 10 kPa. For the in situ sample, the average degree of saturation decreased due to drainage through fissures and reached about 70% when most of the fissures were filled with air. The corresponding saturation for the compacted sample was found to be around 85% indicating the presence of relatively smaller and less frequent discontinuities. Once the fissures were desaturated, water flow had to occur through voids in the soil aggregates. Microporous drainage required a high suction (6000 kPa) for air to enter into the soil matrix. In contrast to Figure 1, the significantly higher matrix air entry value in this figure is attributed to the gradual decrease in soil volume with increasing suction. As explained later, this volume reduction is primarily due to reduced fissure sizes and and, as such, was not captured in Figure 2 that is based on an average void ratio.

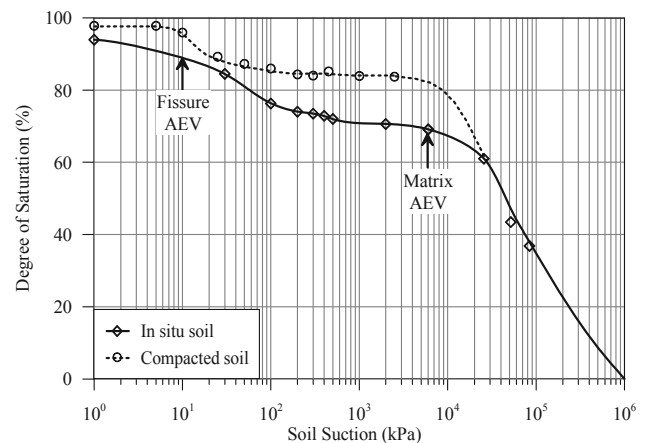


Figure 3. SWCC with degree of saturation

The SWCC given in the form of water content versus matric suction is the most accurate representation for expansive soils. This is because gravimetric water content (measured for each suction value) captures the water drainage through fissures and is independent of volume changes due to water adsorption by clay particles. Likewise, the SWCC represented in the form of degree of saturation versus soil suction is most suitable for understanding volume decrease in fissures due to suction application. Marinho (2005) reported that for plastic soils, drainage through water filled pores is associated with pore compressibility due to capillarity and this phenomenon renders such soils to remain saturated over higher suction values. This representation implies that the expansive soil aggregates remain saturated over a wider range of suction generally prevalent in the field, as postulated by Fityus and Buzzi (2008).

Figure 4 shows the shrinkage curve for the investigated expansive clay. Theoretical lines representing various average saturation degrees were obtained from basic phase relationships and using $G_s = 2.75$. The initially unsaturated samples were first wetted to achieve close to saturation conditions and subsequently desaturated by applying different suction values. The void ratio and water content of each sample were determined as described earlier in this paper. The data depicted in Figure 4 indicate S-shaped shrinkage curves for both sample types and represent the progressive drying of the investigated expansive soil. The curves are composed of an initial low structural shrinkage followed by a sharp decline during normal shrinkage and then by a low decrease during residual shrinkage (Haines, 1923). During structural shrinkage, water within the fissures and some of the larger and relatively stable voids is removed such that the decrease in soil volume is less than the volume of water lost. Volume decrease in soil is equal to the volume of water lost during normal shrinkage thereby leading to a 45° straight line, which is almost parallel to the 100% saturation line. This suggests that drainage primarily takes place through the soil matrix in the normal shrinkage zone. During

residual shrinkage, air enters the pores close to the shrinkage limit and pulls the particles together due to suction. This leads to a further decrease in soil volume albeit lower than the volume of water lost. Furthermore, the high volume change and the closeness to the saturation line during normal shrinkage corresponded well with the comparatively looser state and a relatively homogeneous structure of the compacted sample. Finally, the observed shrinkage curve is reversible because the in situ soil has undergone numerous swell-shrink cycles since deposition. Likewise, Tripathy et al. (2002) reported that equilibrium conditions are usually attained after about four cycles in compacted soils.

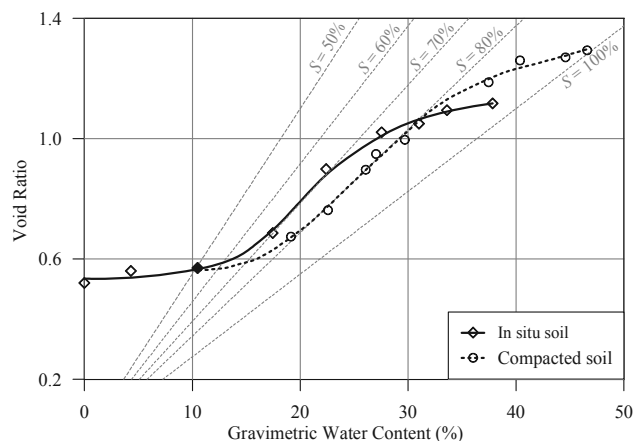


Figure 4. Shrinkage curve

Theoretically, the shrinkage curve comprises of two straight lines: a sloped line closely following the $S = 100\%$ line that joins a horizontal line at a void ratio associated with the shrinkage limit of the soil. This means that soils essentially remain saturated up to the shrinkage limit following a J-shaped curve. Due to the presence of fissures, the investigated soil (in both in situ and compacted conditions) exhibited deviations from this theoretical behavior.

The definition the degree of saturation for expansive soils is not straight forward. Since such soils consist of discontinuities and soil aggregates, the calculated degree of saturation pertains to an average value for the entire soil mass. This definition was used in Figure 3 because it permits easy calculations. A more accurate approach is to consider only the soil aggregates as saturated up to the matrix air entry value of Figure 3 and the fissures as air filled cracks. This is close to an equilibrium soil microstructure that allows alternate swelling and shrinkage. In this approach, the change of water volume in the soil mass equates to the volume change of the soil aggregates and that of the cracks. Gens and Alonso (1992) explained the two levels of soil structure in their framework as follows: the micro-level is governed by physicochemical interactions between the expansive clay minerals thereby forming aggregates whereas the macro-level includes both the aggregates and the fissures.

Overall, water flow through expansive soils is governed by soil structure as indicated by bimodal SWCCs for both undisturbed and compacted samples: sample type becomes irrelevant when the flow starts to occur through the soil matrix. Likewise, the S-shaped swell-shrink curve confirms the structure dominant volume change behavior of such soils for both samples. In the absence of an adequate definition for the degree of saturation, soil state pertains to the initial void ratio. This parameter governs the extent of volume change that, in turn, must be calculated from the reversible swell-shrink curve for expansive soils.

4 CONCLUSIONS

The engineering properties of a typical expansive soil (from Regina, Saskatchewan, Canada) were investigated under in situ and compacted conditions. For both sample types, the clay behavior was characterized by its internal structure comprising of fissures and aggregates. The SWCC using water content on the ordinate showed a bimodal distribution with two air entry values: a lower value (10 kPa) corresponding to drainage through fissures followed by a higher value (300 kPa for the in situ sample and 100 kPa for the compacted sample) associated with seepage through the soil matrix. Sample type became irrelevant when the flow started to occur through the soil matrix. The matrix air entry value was found to be about 6000 kPa when the SWCC was plotted in the form of the degree of saturation versus soil suction. Likewise, the shrinkage curve was found to be S-shaped and included a low structural shrinkage followed by a sharp decline during normal shrinkage and then by a low decrease during residual shrinkage. The extent of volume change that depends on the initial void ratio must be calculated from the reversible swell-shrink curve.

5 ACKNOWLEDGEMENTS

The authors acknowledge the material and financial support provided by the Saskatchewan Ministry of Highways and Infrastructure and the University of Regina for providing laboratory space.

6 REFERENCES

- Azam S. and Ito, M. 2011. Unsaturated soil properties of a fissured expansive clay. *Proceedings, 64th Canadian Geotechnical Conference*, Toronto, Canada. 313:1-5.
- Azam, S. and Wilson, G.W. 2006. Volume change behavior of a fissured expansive clay containing anhydrous calcium sulfate. *Proceedings, 4th International Conference on Unsaturated Soils*, Carefree, Arizona, USA. 1, 906-915.
- Fityus, S. and Buzzi, O. 2008. The place of expansive soils in the frameworks of unsaturated soil mechanics. *Applied Clay Science* 43, 150-155.
- Gens, A. and Alonso, E.E. 1992. A framework for the behaviour of unsaturated expansive clays. *Canadian Geotechnical Journal* 29, 1013-1032.
- Haines, W.B. 1923. The volume change associated with variations of water content in soil. *Journal of Agricultural Science* 13, 296-310.
- Hu, Y. and Hubble, D.W. 2005. Failure conditions of asbestos cement water mains. *Canadian Geotechnical Journal* 34, 608-621.
- Ito, M. and Azam, S. 2013. Engineering properties of a vertisolic expansive soil deposit. *Engineering Geology*. 152(1):10-16
- Ito, M. and Azam, S. 2010. Determination of swelling and shrinkage properties of undisturbed expansive soils. *Geotechnical and Geological Engineering* 28, 413-422.
- Ito, M. and Azam, S. 2009. Engineering characteristics of a glacio-lacustrine clay deposit in a semi-arid climate. *Bulletin of Engineering Geology and the Environment* 68, 551-557.
- Kelly, A.J., Sauer, E.K., Barbour, S.L., Christiansen, E.A., and Widger, R.A. 1995. Deformation of the Deer Creek bridge by an active landslide in clay shale. *Canadian Geotechnical Journal* 32, 701-724.
- Leong, E.C., Tripathy, S. and Rahardjo, H. 2003. Total suction measurement of unsaturated soils with a device using the chilled-mirror dew-point technique. *Geotechnique* 53, 173-182.
- Marinho, F.M.A. 2005. Nature of soil-water characteristic curve for plastic soils. *Journal of Geotechnical and Geoenvironmental Engineering* 131, 654-661.
- Mitchell, J.K. and Soga, K. (2005). *Fundamentals of Soil Behaviour*. 3rd ed., John Wiley and Sons, Inc. New York, NY, USA.
- Prakash, K., Shidharan, A. Baba, J.A., and Thejas, H.K. 2008. Determination of shrinkage limit of fine-grained soils by wax method. *Geotechnical Testing Journal* 32, 86-89.
- Tripathy, S., Subba Rao, K.S., and Fredlund, D.G. 2002. Water content-void ratio swell-shrink paths of compacted expansive soils. *Canadian Geotechnical Journal* 39, 938-959.

Methods of determination of K_0 in overconsolidated clay

Méthodes de détermination de K_0 dans une argile surconsolidée

Boháč J., Mašín D., Malát R., Novák V., Rott J.

Charles University in Prague, Faculty of Science, Albertov 6, 12843 Praha 2, Czech Republic

ABSTRACT: In situ effective stresses, expressed usually by the at rest coefficient K_0 , influence the mechanical behaviour and thus any geotechnical analysis. In normally consolidated soils K_0 can be computed according to the Jaky formula. For overconsolidated clays however neither a general formula nor a general experimental procedure are available. The paper summarizes briefly the available methods and then presents some methods in more detail. First the use of flat dilatometer is discussed. Further, K_0 is determined by back analysing the convergence of a circular test gallery. Finite element analysis using the hypoplastic constitutive model to represent the clay behaviour is adopted in the analyses.

RÉSUMÉ : Les contraintes effectives in situ, exprimées habituellement à travers le coefficient des terres au repos K_0 , ont une influence sur le comportement mécanique des sols, et donc, sur toute analyse géotechnique. Dans les sols normalement consolidés, K_0 peut être calculé à partir de la formule de Jaky. Pour les argiles surconsolidées cependant, il n'existe ni formule générale, ni procédure expérimentale reconnue pour évaluer ce paramètre. La communication présentée résume brièvement les méthodes actuellement disponibles, puis présente quelques méthodes plus en détail. D'abord, l'utilisation du dilatomètre plat et d'une cellule de pression en forme de pelle est discutée. En outre, K_0 est déterminé à partir de l'analyse en retour de la convergence d'une galerie d'essai circulaire. Une approche en éléments finis, basée sur l'utilisation d'une loi de comportement hypoplastique pour modéliser le comportement des argiles, est adoptée dans l'analyse.

KEYWORDS: clay, earth pressure at rest, horizontal stress, anisotropy, hypoplasticity, tunnelling.

MOTS-CLÉS : argile, pression des terres au repos, contrainte horizontale, anisotropie, hypoplasticité, tunnel

1 INTRODUCTION

The in situ effective stresses represent an important initial condition for geotechnical analyses. Typically, the horizontal stress is computed from the vertical stress using the coefficient of earth pressure at rest $K_0 = \sigma_h' / \sigma_v'$, where σ_h' and σ_v' are effective horizontal and vertical stresses, respectively. In the case of deep foundations (friction piles), retaining structures and tunnels, K_0 influences the mechanical behaviour dramatically. Franzius et al. (2005) made a direct investigation into the influence of K_0 conditions in 3D finite element analysis of a tunneling problem using $K_0 = 1.5$ and $K_0 = 0.5$. The unrealistically low K_0 value for London Clay led to better predictions: the normalised settlement trough was narrower and deeper. In absolute values, however, low K_0 caused overprediction of surface settlements by a factor of 4. With $K_0 = 1.5$ the predicted trough was too wide and vertical displacements were underpredicted by the factor of 4.

For normally consolidated soils the estimation of horizontal stresses is not a major problem. Jaky's equation in its usual simplified form of $K_{0nc} = 1 - \sin \phi_c'$ may be used in determining the K_{0nc} for normally consolidated soils (Jaky, 1948; ϕ_c' is the critical state friction angle). There is a lot of experimental evidence throughout the literature that the Jaky formula represents the at rest coefficient of normally consolidated soils well provided the critical state effective friction angle ϕ_c' is used (Mesri and Hayat, 1993; Mayne and Kulhawy, 1982).

For overconsolidated clays however neither a general formula nor a generally applicable experimental procedure for determining the initial stress are available to date. In summarising the knowledge about the mechanical behaviour and characterisation of a typical example of overconsolidated clays – the Tertiary London Clay, which has been a subject of

very intensive research for many decades, Hight et al. (2003) noted: „Still the most difficult parameter to determine for the London Clay is K_0 “.

1.1 Direct methods of K_0 determination

Horizontal stress in clay is most often determined by selfboring pressuremeter (e.g., 'Camkometer' - Wroth and Hughes, 1973), by the flat dilatometer (Marchetti, 1980), or different types of pushed-in spade-shaped pressure cells (e.g., Tedd and Charles, 1981). The use of push-in instruments in stiff clays is often questioned due to possible problems with the installation and due to the soil disturbance. The latter reason together with the possibility of imperfect fit in the borehole seems to have disqualified the Menard-type pressuremeter in stiff clays. A good agreement of K_0 values obtained by push-in spade-shaped pressure cells and Camkometer for London Clay was reported by Tedd and Charles (1981), the 'spade' producing a smaller scatter and better reproducibility. Hamouche et al (1995) reported results by Marchetti dilatometer consistent with those obtained with the self boring pressuremeter in overconsolidated sensitive Canadian clays.

A hydraulic fracturing technique for clays for measuring the horizontal total stress was developed by Bjerrum and Andersen (1972). The method is based on measuring the stress at closing of a vertical crack that had previously been formed by pressurised water. The method can hardly be used under the condition of $K_0 > 1$ as a horizontal crack would be formed instead of the vertical one, and „...the method will just measure the weight of the overburden...“ (Bjerrum and Andersen, 1972). A recent 2D numerical study by Wang et al (2009) also considers horizontal cracks being formed in the case of $K_0 > 1$, i.e. in overconsolidated clays. However, Lefebvre et al. (1991)

using methylene blue tracer in studying the shapes of clay fracturing reported vertical cracks formed in overconsolidated clays of $K_0 > 1$. The measured K_0 values were higher than when approximated by the established K_0 -OCR correlations (by Mayne and Kulhawy, 1982). A similar conclusion was made by Hamouche et al. (1995), who also found that the horizontal pressure determined by fracturing corresponded well to the self boring pressuremeter and Marchetti dilatometer results.

1.2 Indirect methods of K_0 determination

Skempton (1961) made use of four ways of determining the capillary pressure of the undisturbed samples in the laboratory: direct and indirect measurement of the load preventing swelling, analysis of the undrained strength measured in the triaxial device, and measurement of pore water suction in the triaxial specimen. The averaged capillary pressure from the four methods was used to compute the effective horizontal stress, and the pore pressure coefficient was determined in the triaxial apparatus.

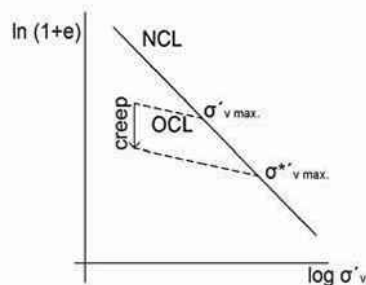


Figure 1. The influence of creep on $\sigma'_{v \max}$ position at oedometer test.

Burland and Maswoswe (1982) used the method in supporting the use of direct measurements of horizontal stresses in London clay: Their suction based results agreed well with the self boring pressuremeter and the push-in 'spade' by Tedd and Charles (1981).

The current version of the Skempton's procedure makes use of the „suction probe“ capable of direct measurement of capillary suctions within undisturbed samples taken by a thin walled samplers (e.g., Hight et al., 2003). Doran et al. (2000) studied the changes of pore pressures and effective stresses in the laboratory specimens on sampling and preparation. They concluded that using isotropic elasticity in the 'suction method' results in underestimating the K_0 . The only up-to-date alternative in London clay projects seems to be to estimate K_0 on the basis of lift-off pressures measured in self-boring pressuremeter tests, although interpretation remains controversial (Hight et al., 2003).

The correlation methods for determination of K_0 are represented by the Jaky formula for normally consolidated soils and by its extensions to cope with the overconsolidated soils in the form of $K_{0oc} = (1 + \sin \phi_c) \times (\text{OCR})^\alpha$. The most common alternative for the exponent is $\alpha = \sin \phi$ (Mayne and Kulhawy, 1982), or $\alpha = 0.5$ (Meyerhof, 1976). Some studies indicated $\alpha \approx 1.0$ (Lefebvre et al., 1991; Hamouche et al., 1995). Using such correlations however neglects other effects than the stress history (unloading), for example creep and cementation that might have developed in the soil in situ, and may lead to erroneous estimation of the values of K_0 . Creep moves the position of the real maximal vertical stress to the position of an apparent maximal vertical stress (Fig. 1). The oedometer test is a common technique for evaluating $\sigma'_{v \max}$. Due to creep however the test produces a pseudo-overconsolidation value of $\sigma_{v \max}^*$ instead of the present overconsolidation pressure $\sigma_{v \max}$. Hence, both the OCR and $K_{0,OC}$ values determined by the correlations and not considering creep (ageing) are overestimated.

An experimental determination using the advanced triaxial instrumentation (stress path testing, local LVDT gauges mounted on the specimens etc.) was suggested by Garga and Khan (1991) and Sivakumar et al. (2009). The latter proposed and experimentally confirmed a new expression $K_{0oc} = 1/\eta(1 - (1 - \eta K_{0oc})\text{OCR}^{(1-\chi)})$, which takes account of OCR (parameter χ is determined by 1-D and isotropic compression tests on undisturbed specimens) and of anisotropy (parameter η is determined from a CIUP test). K_{0oc} can be determined, for example, by Jaky's formula.

Doležalová et al. (1975 in Feda, 1978) made use of the displacements measured after unloading the massif by means of a gallery. The deformation parameters of the rock were determined by independent in situ testing and then the FEM was used to simulate elimination of the monitored displacements of the gallery. The stresses necessary for the simulation were considered the in situ stresses in the massif. A similar approach using an advanced hypoplastic model is presented further.

The review shows that in determining initial stresses in overconsolidated clays a single method can hardly be sufficient. The best way seems taking good quality samples (thin wall sampler) and measuring suctions, and comparing the result with a direct measurements, for which Marchetti dilatometer (DMT), push-in spade-shaped pressure cells or self boring pressuremeter seem most promising. If available, convergence measurements of a underground cavity (gallery) evaluated using a numerical model with an advanced anisotropic constitutive model is believed the best method.

2 GEOLOGY AND CHARACTERISTICS OF CLAY INVESTIGATED

Different methods were used to evaluate K_0 of clay from Brno, Czech Republic. The investigated calcitic silty Brno Clay („Tegel“) of Miocene (lower Badenian) age belongs to the Neogene of Carpathian foredeep, and reaches the depth of several hundred metres. Sound Tegel has a greenish-grey colour, which changes to yellow-brown to reddish-brown colour at the zone of weathering closer to surface. According to X-Ray analysis there is a substantial percentage of CaO (ca 20%) and the main minerals are kaolinite (ca 23%) and illite (22%), calcite (20%), quartz (17%), chlorite (up to 10%) and feldspar (Boháč et al., 1995). Tegel exhibits stiff to very stiff consistency. The clay is overconsolidated but the height of eroded overburden is not known. Above the Miocene clay there are Quaternary gravels overlain by loess loam. The clay is tectonically faulted. The groundwater is mostly bound to Quaternary fluvial sediments, and the collectors are typically not continuous. However the clay is fully water saturated.

In Tegel there is about 50% of clay fraction, w_L is about 75%, I_p about 43%, the soil plots just above the A-line at the plasticity chart and its index of colloid activity is about 0.9.

3 MARCHETTI DILATOMETER MEASUREMENTS

At the site the current phreatic water table was 4.7 metres under the surface and top layer of about 5.5 metres was excavated some 30 years ago. This generated negative pore water pressures, which have not fully dissipated yet. At the current depth of 11.7 metres the pore pressure of -32 kPa was measured (after dissipation of excess pore pressures caused by the sounding) by a push-in spade pressure cell. The present vertical effective stress in the depth of 11.7 metres calculated from the soil unit weight and pore water pressure was $\sigma'_v = 185$ kPa.

The K_0 was measured using Marchetti (1980) flat dilatometer. The measured K_D according to Marchetti (1980) was 8.0 and K_0 derived using the empirical equation $K_0 = (K_D/1.5)^{0.47} - 0.6$ was $K_0 = 1.6$. This is substantially

higher than K_0 determined from oedometric yield point and the empirical correlation of Mayne and Kulhawy (1982) $K_0 = 1.2$.

4 NUMERICAL ANALYSIS OF MARCHETTI DILATOMETER

An attempt was made to explain this discrepancy by numerical modelling of the flat dilatometer penetration into the soil. For the numerical analysis the hypoplastic model (Mašin, 2005) was used in combination with the intergranular strain concept (Niemunis and Herle, 1997). The model predicts nonlinear stiffness depending on the strain level. The input value of K_0 of 1.2 was considered. Both the calibration and the parameters for the hypoplastic model were taken from Svoboda et al. (2010) and Mašin (2012). The parameters are summarised in Table 1.

Table 1. Parameters of the hypoplastic model

φ_c	λ^*	κ^*	N	r
22°	0.128	0.015	1.51	0.45
m_R	m_T	R	β_r	χ
16.75	8.375	1.e-4	0.2	0.8

The numerical analysis was carried out using Plaxis 2D finite element code. The modelling sequence involved three phases:

1. Generation of the initial stress condition with $K_0 = 1.2$,
2. Excavation of the 5.5 metres thick layer in order to reach the measured pore water pressure of -32 kPa at the depth of 11.7 metres. Consolidation time was varied using the consolidation analysis until the measured excess pore water pressure was obtained.

3. The installation of the dilatometer was simulated in a simplified manner using two approaches. In the first one, displacement was prescribed at the left boundary of the model, as depicted in Fig. 2. The second analysis involved prescribed load. The dilatometer was 200 millimetres high and 14 millimetres wide (7 mm horizontal displacement was considered in the model thanks to its symmetry) and it was installed in the depth of 11.6 – 11.8 metres. In the analyses, load/displacement was evaluated in the centre of the dilatometer. These phases employed a plastic undrained analysis.

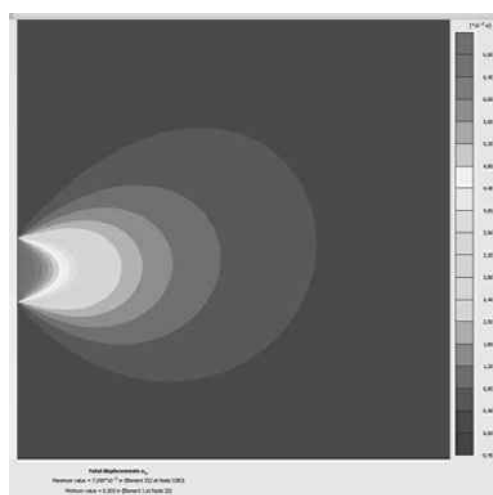


Figure 2. Distribution of horizontal displacements calculated by the hypoplastic simulation of Marchetti (1980) dilatometer.

The calculated coefficient K_D was 4.51 for the load controlled analysis and 4.06 for the displacement controlled analysis, which leads to K_0 equal 1.07 and 1.00 respectively. This preliminary analysis thus indicated slight underprediction of K_0 using Marchetti (1980) empirical equation. Limitations of

the model need, however, be considered. In particular the simplified geometry and limitations of the adopted constitutive model, which does not allow for an explicit consideration of inherent stiffness anisotropy. To overcome this limitation, a new anisotropic version of the hypoplastic model is currently being developed.

5 BACKANALYSIS OF CIRCULAR ADIT

In the second numerical study presented, the K_0 coefficient is evaluated by means of backanalysis of convergence measurements within a circular exploratory adit. The adit was excavated as part of a geotechnical site investigation preceeding the excavation of Královo Pole Tunnels in Brno (see Svoboda et al., 2010).

The adit was located 26 m below the ground level, and its diameter was 1,9 m. Its geometry is shown in Fig. 3. The adit was protected by a steel net and rolled steel arches. These were installed for safety reasons only, and the support was never in full contact with the cavity wall. The monitored convergence of the cavity is thus assumed to be representative of the displacement of an unsupported massif. Its convergence was monitored by means of push-rod dilatometer in four different directions (vertical, horizontal and two sections inclined at 45 degrees).



Figure 3. Circular adit used in backanalyses of the earth pressure coefficient at rest K_0 (Pavlik et al., 2004).

The adit has been simulated in 2D and 3D using finite element method. The model properties were taken over from Svoboda et al. (2010). Hypoplastic model parameters are in Tab. 1. In the analyses, it was assumed that the massif properties were known. The initial value of K_0 was varied by a trial-and-error procedure until the model correctly reproduced the measured ratio of horizontal and vertical convergence of the adit. The analyses were performed under undrained conditions.

The analyses were performed using the softwares PLAXIS 2D and PLAXIS 3D. The 2D analyses adopted the load reduction method (see Svoboda and Mašin, 2011). In these analyses, the load reduction factor was varied to achieve the monitored displacement magnitude, and the coefficient K_0 was adjusted to reproduce the ratio of displacements in horizontal and vertical directions.

Geometry assumed in the 3D analyses is in Fig. 4. No effort was made to vary model properties to reach the exact monitored displacement magnitude. As in 2D analyses, K_0 was

backanalysed to fit the displacement ratio. To represent the real excavation and monitoring procedure, displacements were reset in simulations once the adit face passed the monitored section. They are thus not biased by the pre-convergence displacements, which are not registered by the rod dilatometers. Evaluation of horizontal displacements in the monitored section is demonstrated in Fig. 4.

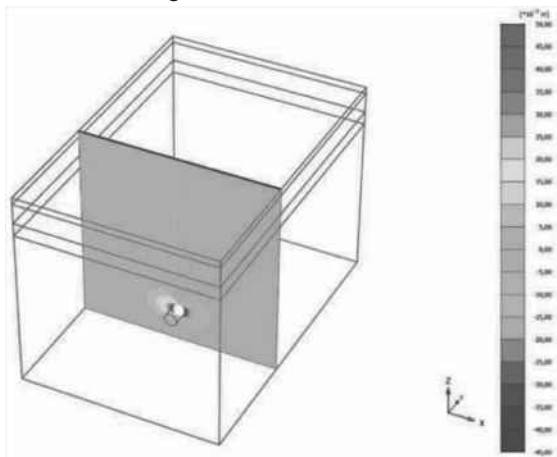


Figure 4. 3D model geometry and predictions of horizontal displacements.

Results of backanalyses are summarized in Table 2. It is clear that the 2D and 3D analyses were consistent in the estimation of K_0 (1.37 and 1.45 respectively). The 3D analyses overpredicted the displacement magnitude.

Table 2. Results of numerical backanalysis of circular exploratory adit.

	monitoring	2D model	3D model
horiz. conv. (u_h) [mm]	19.8	19.8	33.4
vert. conv. (u_v) [mm]	15.9	15.4	26.1
Ratio u_h/u_v	1.25	1.25	1.28
K_0	-	1.37	1.45

Similarly to Sec. 4, it is expected that the results obtained are negatively influenced by inaccurate representation of soil anisotropy using the hypoplastic model. The development of the new model is ongoing.

6 CONCLUSIONS

In the paper, we summarized different methods for evaluation of the earth pressure coefficient at rest K_0 . Due to limitations of different methods, it is always advisable to combine different approaches based on laboratory investigation, field measurements and numerical analysis. Several results of the K_0 evaluation of the Brno Clay were presented, in all cases leading to K_0 higher than unity. These analyses are preliminary and they will be adjusted in the forthcoming work.

7 ACKNOWLEDGEMENTS

The financial support by the grants P105/11/1884 and P105/12/1705 of the Czech Science Foundation, and by the grant MSM0021620855 is gratefully acknowledged.

8 REFERENCES

Bjerrum, L. and Andersen, K. (1972) In-situ measurement of lateral pressures in clay. 5th ECSMFE, Madrid, Vol 1, 12-20.

- Boháč, J., Feda, J., Herle, I. and Klablena, P. (1995), Properties of fissured Brno Clay. Proc. 11th Eur. Conf. Soil Mech. Fdn. Eng., Copenhagen, 28 May - 1 June, Danish Geotechnical Society, Bulletin No. 11, Vol. 3, 19-24.
- Burland, J.B. and Maswoswe, J. (1982) Discussion on „In situ measurements of horizontal stress in overconsolidated clay using push-in spade-shaped pressure cells“. *Géotechnique* 32, No. 2, 285-286.
- Doležalová, M., Truncová, D. and Vorlíčková, Z. (1975) In situ determination of initial state of stress in a rock mass and the finite element method (in Czech). MS, Report of Hydroprojekt, Praha, 21pp.
- Doran, I.G., Sivakumar, V., Graham, J. and Johnson, A. (2000) Estimation of in situ stresses using anisotropic elasticity and suction measurements. *Géotechnique* 50, No. 2, 189-196.
- Feda, J. (1978) Stress in subsoil and methods of final settlement calculation. Academia Prague.
- Franzius, J. N., Potts, D. M., and Burland, J. B. (2005). The influence of soil anisotropy and K_0 on ground surface movements resulting from tunnel excavation. *Geotechnique*, 55(3), 189-199.
- Garga, V.K. and Khan, M.A. (1991) Laboratory evaluation of K_0 for overconsolidated clays. *Can. Geotech. J.* 28, 650-659.
- Hamouche, K., Leroueil, S., Roy, M. and Lutenecker, A.J. (1995) In situ evaluation of K_0 in eastern Canada Clays. *Canadian Geotech. J.* 32 (4), 677-688.
- Hight, D.W., McMillan, F., Powell, J.J.M., Jardine, R.J. and Allenou, C.P. (2003) Some characteristics of London Clay. Proc. Characterisation and Engineering Properties of Natural Soils – Tan et al. (eds). Swets & Zeitlinger, 851-907.
- Jáky, J. (1948) Pressure in silos. Proc. 2nd ICSMFE, Rotterdam, Vol. 1, 103-107.
- Lefebvre, G., Bozozuk, M., Philibert, A. and Hornych, P. (1991) Evaluating K_0 in Champlain clays with hydraulic fracture tests, *Can. Geotech. J.* 28 (3), 365-377.
- Marchetti, S. (1980) In situ tests by flat dilatometer. *J. Geot. Eng. Div. ASCE*, Vol. 106, NoGT3, 299-321.
- Mašin, D. (2005). A hypoplastic constitutive model for clays, *International Journal for Numerical and Analytical Methods in Geomechanics*, 29, No. 4, 311-336.
- Mašin, D. (2012). Clay hypoplasticity with explicitly defined asymptotic states. *Acta Geotechnica* (in print; doi: 10.1007/s11440-012-0199-y).
- Mayne, P.W. and Kulhawy, F.H. (1982) K_0 -OCR relationships in soil. *J. Geotech. Eng. Div. ASCE*, GT6, 851-872.
- Mesri, G. and Hayat, T.M. (1993) The coefficient of earth pressure at rest. *Canadian Geotech. J.* 30(4), 647-666.
- Meyerhof, G.G. (1976) Bearing capacity and settlement of pile foundations. *J. Geotech. Engng. Div. ASCE* 102, No. GT3, 197-228.
- Niemunis, A., and Herle, I. (1997). Hypoplastic model for cohesionless soils with elastic strain range. *Mech. Cohesive-Frict. Mater.*, 2, 279-299.
- Pavlik, J., Klímek, L., Rupp, O. (2004). Geotechnical exploration for the Dobrovského tunnel, the most significant structure on the large city ring road in Brno. *Tunel* 13(2), 2-12.
- Sivakumar, V., Navaneethan, T., Hughes, D. and Gallagher, G. (2009) An assessment of the earth pressure coefficient in overconsolidated clays. *Géotechnique* 59, No. 10, 825-828.
- Skempton, A.W. (1961) Horizontal stresses in an over-consolidated Eocene clay. Proc. 5th ICSMFE, Vol. 1 351-357.
- Svoboda, T., Mašin, D. and Boháč, J. (2010) Class A predictions of a NATM tunnel in stiff clay. *Computers and Geotechnics* 37(6), 817-825.
- Svoboda, T. and Mašin, D. (2011). Comparison of displacement fields predicted by 2D and 3D finite element modelling of shallow NATM tunnels in clays. *Geotechnik* 34(2), 115-126.
- Tedd, P. and Charles, J.A. (1981) In situ measurements of horizontal stress in overconsolidated clay using push-in spade-shaped pressure cells. *Géotechnique* 31, No. 4, 554-558.
- Wang, S.Y., Sun, L., Au A.S.K., Yang, T.H. and Tang, C.A. (2009) 2D numerical analysis of hydraulic fracturing in heterogeneous geo-materials. *Construction and Building Materials* 23, 2196-2206.
- Wroth, C.P. and Hughes, J.M.O. (1973) An instrument for the in-situ measurements of the properties of soft clays. Proc 8th ICSMFE, Vol 1.2, 487-494.

Stress-path effects on the grading of an artificial material with crushable grains

Stress-trajectoire effets sur le granulométrie d'un matériau artificiel avec des grains déformables

Casini F.

Universitat Politècnica de Catalunya, Barcelona, Spain

Leu J., Low B., Wanninger F., Zimmermann A., Zwicker P., Springman S.M.

Institute for Geotechnical Engineering, ETHZ, Zurich, Switzerland

Viggiani G.M.B.

Università degli Studi di Roma Tor Vergata, Roma, Italy

ABSTRACT: Granular materials forming natural slopes, embankments, foundations, pavement structures, and rail track structures are subjected to static and dynamic loads, which may cause particle breakage to occur. Results are presented of an experimental investigation into the mechanical response of an artificial granular material, consisting of crushed expanded clay pellets, commercially known under the brand name LECA (Light Expanded Clay Aggregate) to various stress path tests. The material was reconstituted, with a maximum particle size of 2 mm, to obtain grading curves with the same mean diameter d_{50} , and different coefficients of uniformity, U ($= 3.5, 7, 14, 28$) or the same U and a different d_{50} ($= 0.5, 1$ mm). The constant volume friction angle and the minimum and maximum densities corresponding to each grading were determined before stress path testing in one dimensional and triaxial compression at different stress levels. Changes in the LECA grading after the stress path tests were described using two parameters defined respectively as the mean diameter and coefficient of uniformity of the final distribution over the value of the initial distributions, both of which were assumed to be consistent with self-similar grading with varying fractal dimension.

RÉSUMÉ: Les matériaux granulaires formant des pentes naturelles, des talus, des fondations, des structures de chaussées et des structures de voie ferrée se sont soumis à des charges statiques et dynamiques, qui peuvent causer la rupture des particules de se produire. Les résultats sont présentés sur une recherche expérimentale sur le comportement mécanique d'un matériau granulaire artificielle, composée de boulettes d'argile expansée concassées, commercialement connu sous le nom de marque LECA (Light agrégat d'argile expansée) à différents tests chemin de stress. Le matériau a été reconstituée, avec une granulométrie maximale de 2 mm, pour obtenir des courbes de gradation avec le même diamètre moyen d_{50} , et différents coefficients d'uniformité, U ($= 3,5, 7, 14, 28$) ou même U et une autre d_{50} ($= 0,5, 1$ mm). L'angle de volume constant de friction et les densités minimale et maximale correspondant à chaque classement ont été déterminés avant le test dans un chemin de contrainte de compression triaxiale dimensions et à différents niveaux de contrainte. Les changements dans le classement LECA après les essais de chemin de stress ont été décrits en utilisant deux paramètres définis respectivement comme le diamètre moyen et le coefficient d'uniformité de la distribution finale au-dessus de la valeur des distributions initiales, qui étaient tous deux censés être compatibles avec auto-similaire classement avec plus ou moins la dimension fractale.

KEYWORDS: artificial material, grain crushing, grain size distribution, breakage, stress path testing.

INTRODUCTION

Particle breakage describes the response to loading in which soil particles become smaller, while other mechanisms of deformation, such as slippage, dilation and creep occur. Degradation processes associated with loading-induced grain crushing affect the macroscopic mechanical behaviour of granular materials. For a given material, breakage is affected by both stress level and stress path direction and can cause volume loss leading to settlements and a reduction in the hydraulic conductivity, as finer particle fractions fill voids. Moreover, elastic and frictional properties of the soil are modified due to changes in grain size distribution. Understanding the mechanisms of grain crushing is therefore crucial, as this affects the stress-strain response of the soil under loading.

Different measures have been suggested to quantify the amount of particle breakage in a sample of granular material. Hardin (1985) introduced the relative breakage, B_r , based on the relative position of the current cumulative particle size distribution from the initial cumulative distribution and a cut-off value of 'silt' particle size (of 0.074 mm). The use of the latter implied that all particles would eventually become finer than the (arbitrary) cut-off value in the fragmentation process. This denies the growing understanding that the grain size distribution of an aggregate of any initial grading, under large confining pressure and extensive shear strains, tends to become self-similar (fractal) (Turcotte, 1986; McDowell and Bolton, 1998).

Several studies (e.g.: Sammis *et al.*, 1987; Tsoungui *et al.*; 1999) have shown that the main effect of particle crushing is to increase the proportion of fine material without significantly changing the size of the largest particles. Larger particles are cushioned by surrounding smaller particles (Imre *et al.*, 2010; 2011), making them more resistant to crushing and giving them a higher coordination number, which is defined as the number of the particle's nearest neighbours. Smaller particles, with smaller coordination numbers, are more likely to be crushed in the fragmentation process. In other words, the cushioning effect due to the large coordination number for larger particles outweighs the effect of reducing strength with increasing particle size (Casini and Viggiani, 2011; Casini *et al.*, 2013).

Methods and the results are presented for an experimental investigation on an artificial granular material under different loading conditions. In particular, the paper explores the evolution of the main physical properties, such as the angle of friction and the range of voids ratio together with the grading, sphericity and angularity of the particles under different loading conditions.

For practical reasons, the experimental programme was carried out on an artificial granular material, which was reconstituted at different initial grading with grains that crush at relatively low stress. Samples of the artificial, 'crushable', granular soil, have been subjected to different effective stress paths in one dimensional and triaxial compression, in order to understand the mechanisms of grain crushing better.

1. MATERIAL TESTED

A systematic experimental investigation of grain crushing for natural materials is often difficult due to the relatively high stress required to crush the grains and the variability and heterogeneity of natural deposits, which makes it difficult to obtain repeatable results. For these reasons the experimental work was carried out on an artificial granular material.

The material used is commercially available under the acronym LECA (Light Expanded Clay Aggregate) and is obtained through an industrial process. The expanded clay pellets are screened into their various fractions and made commercially available both as intact (so-called "granular") with a characteristic a round shape and a hard outer shell (see Fig. 1a), or crushed, at different grain sizes (see Figs 1b-c).

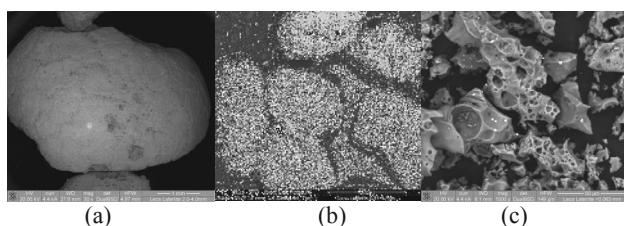


Figure 1. LECA pellets whole/broken with particle diameters: (a) $d=2-4$ mm; (b) $0.71-1$ mm; (c) $d<0.063$ mm (Wanninger and Zwicker, 2010).

The material has a very low apparent unit weight; this is because the particles are characterised by the existence of a double order of porosity: "inter-granular", i.e. voids existing between particles, and "intra-granular", i.e. closed voids existing within individual particles (see Fig. 2). Because of the voids existing within the particles, their apparent unit weight, γ_s , depends on their diameter d as $\gamma_s(d) = a \cdot (d_0/d)^b$, with $a = 12.64$ kN/m³, $b = 0.268$, and $d_0 = 1$ mm (Casini *et al.*, 2013).

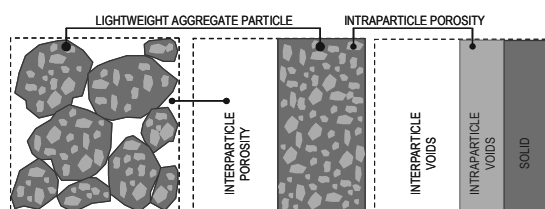


Figure 2. Inter-granular and intra-granular porosity (after Casini and Viggiani, 2011).

1.1 Initial grading

The maximum particle size of the tested material was 4 mm; the material was reconstituted to obtain grading curves with the same mean diameter d_{50} , and different coefficients of uniformity, U ($= 3.5, 7, 14, 28$) or the same coefficient of uniformity, U and different d_{50} ($= 0.5, 1$ mm) (Figure 3).

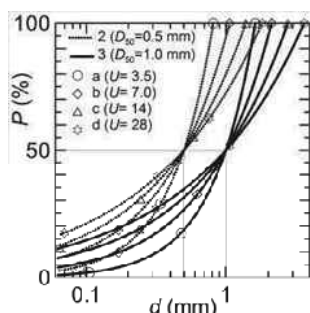


Figure 3. Grain size distributions of reconstituted LECA samples.

1.2 Basic properties

The constant volume friction angle ϕ'_{cv} , and the minimum and maximum densities corresponding to each grading were determined before testing. ϕ'_{cv} was obtained by pouring the material on a rough, flat surface from a given height, and measuring the slopes of the resulting granular cone. The experimental values of ϕ'_{cv} (Fig. 4) are in the range of $30-33^\circ$ and increase slightly with the coefficient of uniformity. There is no obvious dependence on the mean grain size.

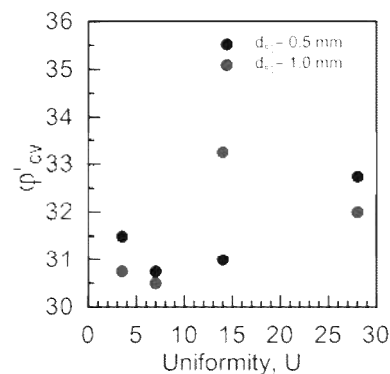


Figure 4. Constant volume angle of friction as function of uniformity for crushed LECA

The maximum (e_{max}) and minimum (e_{min}) voids ratio were determined using non-standard procedures so that particle crushing would not falsify the results. In particular, e_{min} was obtained vibrating the samples with very low input energy.

The experimental values of ($e_{max}-e_{min}$) obtained for the granular material (Fig. 5) at different values of U and d_{50} , of the order of $0.9 \div 1.0$, are much larger than those obtained for other granular materials with grains that do not exhibit significant intra-porosity, such as natural river sands, lightweight aggregates and glass ballottini (*e.g.* Miura *et al.*, 1997). LECA can probably be reconstituted at very high values of voids ratio due to the rough surface of the particles of crushed material, as shown in Figure 1, which causes a very "open" structure with loads carried by arching between particles, some of which are effectively unloaded or redundant, and large inter-porosity.

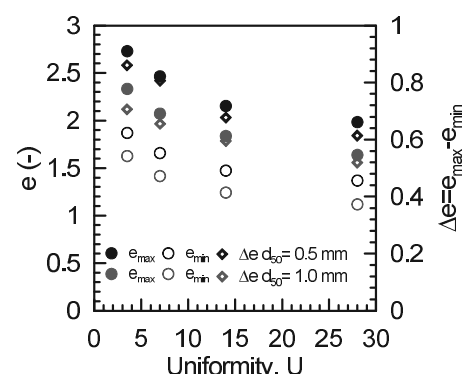


Figure 5. Minimum and maximum void ratios as a function of uniformity for crushed LECA

1.3 Microstructural features

Examples of Scanning Electron Microscopy (SEM) micrographs of crushed LECA particles of different dimensions are reported in Figure 6a-b. SEM micrographs of portions of grains belonging to different fractions were manipulated using the image editing program GIMP (Peck, 2008); the exposed intra-granular pores were coloured progressively in black and the contrast in the image was raised until all the pixels were either black (pores) or white (matrix), (compare Figures 6a and

6b). The processed images were then imported in Matlab and the number of white (N_W) and black (N_B) pixels counted with a simple algorithm; it was then possible to attribute the exposed intra-granular porosity of grains, $n_{ei} = N_B/(N_B + N_W)$ to different fractions. Figure 7 shows n_{ei} as a function of particle size, together with the bulk intra-granular porosity, $n_{bi} (= 1 - \gamma_{as}/\gamma_s)$, obtained from the measurement of the apparent unit weight of particles of different sizes, γ_{as} . The exposed intra-granular porosity is always smaller than the bulk intra-granular porosity as the first is related to the ratio of the average void size to the particle size squared, while the second is related to the same ratio raised to a power 3. Both n_{ei} and n_{bi} increase with increasing grain size, tending to constant values at particle sizes larger than about 3.5 mm, where the apparent unit weight of the particles, γ_{as} , becomes constant, with a final ratio $n_{ei}/n_{bi} \cong 1.3$.

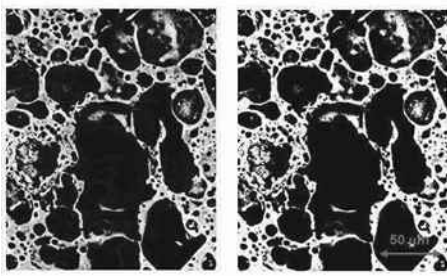


Figure 6. Intra-porosity detected through SEM processing

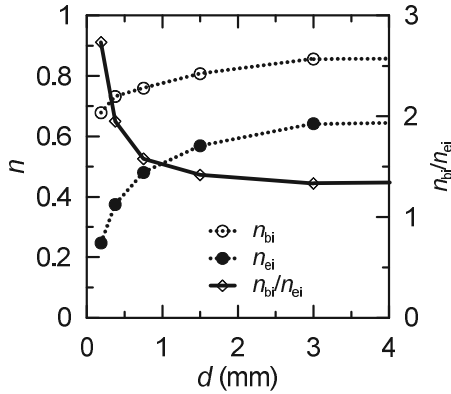


Figure 7. Exposed and bulk intra-granular porosity of crushed LECA particles as a function of grain size.

2 EXPERIMENTAL PROGRAMME The samples were subjected to isotropic, one dimensional and triaxial compression (Wanninger and Zwicker, 2010; Leu *et al.*, 2011) at increasing confining pressures (see Fig. 7).

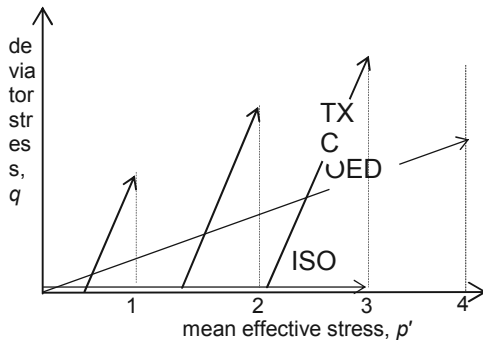


Figure 7. Stress-paths followed in the laboratory tests.

2.1 Particle sphericity and angularity

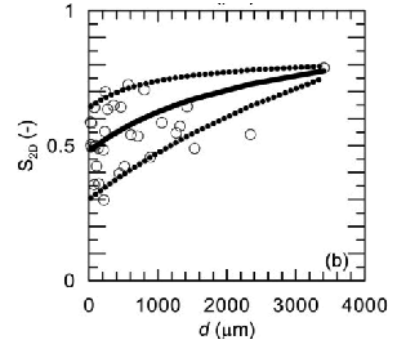


Figure 8. 2D sphericity of crushed LECA as a function of grain size.

SEM micrographs were also used to determine 2D sphericity and angularity of the different fractions systematically. Following the suggestion of Cho *et al.* (2006), 2D sphericity, S_{2D} , was evaluated as the ratio between the diameter of the smallest circle inscribed in the 2D projection of the particle shape and the diameter of a larger circle that contains the whole particle. It is logical that S_{2D} will increase with particle diameter due to surface texture (Fig. 8).

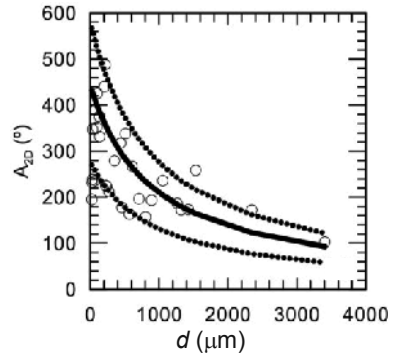


Figure 9. 2D angularity of crushed LECA as a function of grain size.

2D angularity (A_{2D}) of the particles was determined using the definition proposed by Miura *et al.*, 1997. The values of A_{2D} are reported in Figure 9 as a function of particle diameter, where A_{2D} decreases as the particle diameter increases. It should be mentioned that the angularity as evaluated above is a macro angularity of the particles, while further investigations are needed to evaluate the micro-angularity or smoothness of the particles.

2.2 Evolution of grading

Figure 8 shows the cumulative grain size distribution by weight obtained for the grading with $U = 3.5$ and $d_{50} = 0.5$ mm at increasing mean effective stress. The final grain size distribution (GSD) is rotated upwards and translated leftwards, with an increase of the fine fraction at an almost constant value of the maximum particle size, d_M . The maximum particle size d_M is likely to be different from Δn (maximum dimension of the sieve series) and is unknown, even though d_M must always be less than the sieve dimension Δn . Small changes of d_M with load and stress path are difficult to detect in the laboratory because the spacing of two successive sieves around d_M is finite and not fine enough. The experimental results have been fitted using the equation $P(\%) = (d/d_M)^\beta$, represented by the dotted line in Figure 10, which fits the experimental results quite well.

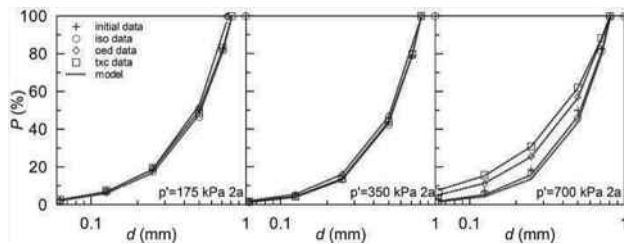


Figure 10. Grain size distribution evolution with $U = 3.5$ and $d_{50} = 0.50$ mm.

Grain crushing has been quantified, as a first approximation, using the ratios between d_{50}/d_{50i} and U/U_i , where d_{50i} and U_i represent respectively the initial mean diameter and coefficient of uniformity. Figure 11 (a) and (b) show the evolution for the ratios d_{50}/d_{50i} and U/U_i after 1-D compression for all of the initial GSDs tested, as a function of maximum mean effective stress p' .

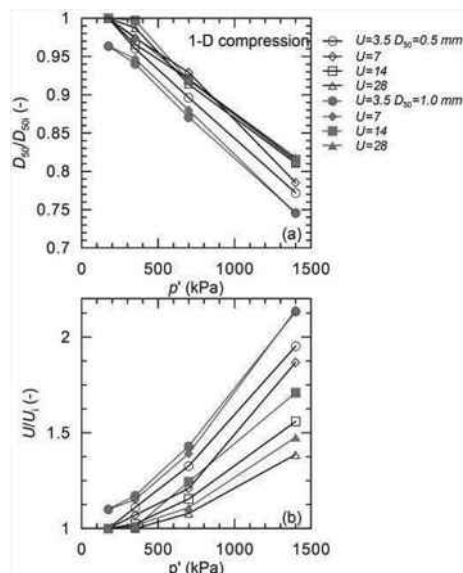


Figure 11. Evolution of ratios: (a) d_{50}/d_{50i} and (b) U/U_i with mean effective stress applied in 1D-compression.

The reduction of d_{50}/d_{50i} with increasing p' is more pronounced for samples with lower initial coefficient of uniformity ($U_i = 3.5$) and higher mean diameter ($d_{50i} = 1$ mm). Likewise, as the initial U decreases, the ratio U/U_i increases by a factor greater than 2 for the poorly graded samples with the greater d_{50i} . The ratio U/U_i is consistently higher for higher initial mean diameter ($d_{50i} = 1$ mm) for all the GSDs tested, which is probably due to the lower coordination number of the particles. The stress acting on the neighbour is higher for the same magnitude and direction of stress applied, as the coordination number decreases, as larger particles tend to be cushioned by surrounding smaller particles. This leads to higher coordination numbers and makes the larger particles more resistant to crushing. Smaller particles, with smaller coordination numbers, are more likely to be crushed in the fragmentation process so that the cushioning effect for larger particles is more relevant than the lower particle strength with increasing particle size.

3 CONCLUSIONS

An extensive laboratory investigation has been conducted on an artificial granular, expanded clay pellets LECA, composed of grains that break at relatively low stresses. These lightweight expanded clay aggregates are used in road construction, tunnelling, structural backfill against foundations, retaining

walls and bridge abutments, because of their low unit weight and good drainage properties. In many practical cases, the stress levels to which the material is subjected are comparable to those explored in the present experimental investigation.

The final grain size distribution measured after loading is rotated upwards and translated leftwards (on a standard particle size distribution plot) for all tested samples. The percentage of finer particles increases with increasing mean effective stress and stress-path obliquity. The grain size distributions can be described satisfactorily using simple equations derived from a fractal evolution of grading. Breakage has been quantified as a first approximation through the evolution of non-dimensional ratios of mean diameter and coefficient of uniformity. Poorly graded samples show more pronounced decrease in mean diameter and increase of uniformity with higher stress applied.

Further investigations will be undertaken to reproduce the observed behaviour through a constitutive model to account for breakage and its effects on mechanical behaviour.

ACKNOWLEDGEMENTS

The authors are grateful to Ralf Herzog (Institute for Geotechnical Engineering) and Gabriele Peschke (Institute for Building Materials), ETH Zurich, for their technical support.

REFERENCES

- Casini, F. and Viggiani, G.M.B. (2011). Experimental investigation of the evolution of grading of an artificial material with crushable grains under different loading conditions. *Proc. Int. Symp. on: Deformation Characteristics of Geomaterials*, IS-Seoul 2011, 957-964.
- Casini F., Viggiani G.M.B. and Springman S.M. (2013). Breakage of an artificial crushable material under loading. *Granular Matter* (submitted).
- Cho G., Dodds J. and Santamarina J. (2006). Particle Shape Effects on Packing Density, Stiffness, and Strength: Natural and Crushed Sands. *J. Geotech. Geoenviron. Eng.*, 132(5), 591-602.
- Hardin, B.O. 1985. Crushing of soil particles. *Journal of Geotechnical Engineering*, ASCE 111(10), 1177-1192.
- Imre B., Laue, J. and Springman, S.M. (2010). Fractal fragmentation of rocks within sturzstroms: insight derived from physical experiments within the ETH Geotechnical Drum Centrifuge. *Granular Matter* 12(3): 267-285. DOI: 10.1007/s10035-009-0163-1.
- Imre B., Wildhaber, B. and Springman, S.M. (2011). A Physical Analogue Material to Simulate Sturzstroms. *IJPMG*. 11(2): 69-86. doi:10.1680/ijpmg.2011.11.2.69
- Leu J., Low B. and Zimmermann (2011). Effects of grain size distribution and confining stress on the mechanical behaviour of an artificial crushable material *Project thesis*, ETH Zurich, Switzerland.
- McDowell, G.R. & Bolton, M.D. (1998). On the micromechanics of crushable aggregates. *Géotechnique* 40(5): 667-679.
- Miura, K., Maeda, K., Furukawa, M. and Toki, S. (1997). Physical characteristics of sands with different primary properties. *Soils and Foundations* 37(3): 53-64.
- Peck, A. (2008). *Beginning GIMP: From Novice to Professional*, Second Edition. Apress, 548 pp.
- Wanninger F. and Zwicker P. (2010). Effects of grain crushing on compressibility of an artificial material. *Semester Thesis*, ETH Zurich, Switzerland.
- Sammis, C.G., King, G. and Biegel, R. (1987). The kinematics of gouge deformations. *Pure Appl. Geophys.* 125, 777-812.
- Tsounghi, O., Vallet, D. and Charnet, J.C. (1999). Numerical model of crushing of grains inside two-dimensional granular materials. *Powder Technology*, 105(1-3), 190-198.
- Turcotte, D.L. 1986. Fractals and fragmentation. *J. of Geophysical Research* 91(B2): 1921-1926.

Characteristics of structure evolution of expansive soil and loess during loading and wetting

Caractéristiques de l'évolution structurale du sol expansif et du loess lors du chargement et du mouillage

Chen Z.

Logistical Engineering University, Chongqing 400041, China

ABSTRACT: A CT-triaxial apparatus was used to study systematically the meso-structure evolution of expansive soil and loess. All soil samples were unsaturated. A group of CT-triaxial drained tests with net cell pressure and suction controlled and a group of wet-dry circles for expansive soil samples were conducted. A series of CT-triaxial inundation-collapse tests were conducted under certain suction and net mean stress as well as deviator stress for undisturbed Q_3 loess. A number of CT images and CT data of meso-structure evolution of the samples were obtained. Some regularity of holes and cracks developing in samples were discovered. The structure damage evolution equations for expansive soil and loess were proposed, respectively. A method to determine the yield stress in triaxial test was suggested.

RÉSUMÉ : Un appareil CT-triaxial a été utilisé pour étudier systématiquement l'évolution méso-structurale du sol expansif et du loess. Tous les échantillons de sol étaient insaturés. Un groupe d'essais CT-triaxial de drainage ont été faits sous le contrôle de la pression et de la succion de la salle blanche, ainsi qu'un groupe d'essais d'alternance humide-sèche pour les échantillons de sol expansif. Une série d'essais CT-triaxial de mouillage ont été effectués sur le loess Q_3 intact sous une valeur définie de succion, de contrainte moyenne nette et de contrainte déviatorique. Un certain nombre d'images et de données CT de l'évolution méso-structurale des échantillons ont été prélevées. Quelques régularités du développement des trous et des fissures dans les échantillons ont été découvertes. Les équations d'évolution des dégâts structuraux du sol expansif et du loess ont été proposées respectivement. Une méthode de déterminer la limite d'élasticité en essai triaxial a été suggérée.

KEYWORDS: CT-triaxial apparatus; suction; loading; wet-dry circle; collapse; structure evolution.

1 INTRODUCTION

Each type of soils, such as collapsible loess and expansive soil, has its own special structures. The mechanical properties of soils are related to their interior factors (i.e., mineralogical composition and structure) and the outer factors (i.e., loading and environmental changes). For example, the strength of collapsible loess and expansive soil are very high during the dry seasons. However, the strength will decrease drastically when they are inundation.

Since 2000, a series of researches have been conducted on the meso-structure analysis of loess and expansive soil using the CT-triaxial apparatus developed by the author. The meso-structure and evolution laws of expansive soil and loess under the loading and moisture changes were mainly investigated.

2 TEST EQUIPMENT

In recent years, great effort has been paid to study meso-structures and constitutive relations of special soils such as loess and expansive soil, et al, by the authors. A multifunctional triaxial apparatus of unsaturated soil combining with CT machine developed successfully (Chen, et al, 2001; Chen, et al, 2007), and over 30 of stress path tests on saturated soil, unsaturated soil and special soils can be conducted with the equipment (Figure 1). The CT machine made by GE is a spiral scanner type of Prospeed AL. Its spatial resolution is 0.38 mm and its density resolution is 0.3 % (3 Hu).

3 STRUCTURE EVOLUTION OF EXPANSIVE SOIL

Crack is one of important structural feature, which influences the mechanical properties of expansive soil. In order to discover the characteristics of the crack evolution of expansive soil, 9 groups of tests including 55 samples were conducted. These tests were CT-triaxial tests in loading and wet-dry circles. A total of 666 CT images and a number of CT data of the test samples were obtained (Lu, 2000; Wei, 2007; Yao, 2009; Chen,

2011; Yao, 2011; Wang, 2012). However, only a part of test results are presented in the paper because space is limited.



Figure1. CT-triaxial apparatus

3.1 The characteristics of structure damage evolution of intact expansive soil during loading

Fifteen CT-triaxial drained tests of intact expansive soils with net cell pressure and suction controlled were conducted. The size of samples was 39.1mm in diameter and 80mm in height. The initial conditions and index of samples are shown in table 1 (only the data of sample 2[#] are listed).

The scanning cross-sections which were marked 1 and 2 located at 1/3H and 2/3H from the top of specimen. Each sample was scanned 6 times and the scanning pictures were marked as a, b, c, d, e and f successively. 1a, 1b, 1c, 1d, 1e and 1f were corresponding to the cross-section of 1/3H (Fig. 2).

The CT images of sample 2[#] clearly illustrate that the structures of the samples change obviously from initial deformation to failure and several cracks gradually developed. The test results of the samples indicate similar crack evolution rules. 1) The initial damage of structure of intact expansive soil is great and heterogeneous in space. 2) Shear makes an obvious

Table 1 Initial conditions and test parameters of sample 2#

Initial conditions		Test parameters		
		Scanning sequence	Deviator stress / kPa	Deviator strain / %
		a	0	0.00
Dry density $\rho_d / (g/cm^3)$	1.68	b	153	0.95
Water content $w / \%$	22.1	c	302	3.30
Void ratio e	0.63	d	283	5.38
Matric suction s / kPa	100	e	238	8.19
Net cell pressure p / kPa	50	f	211	11.24

development of structure damage (which is from fissure development to several crack connection). 3) Both suction and cell pressure affect damage evolution. The higher the cell pressure or suction is, the lower the damage degree of sample will be. 4) The formation of fracture plane of undisturbed soil has several characteristics as follows: a) Inevitability: preexisting fissures, or weakness planes or large cavity, which are the weak areas of samples, easily extend to fracture plane (dash line ① in figure 2). b) Randomness: there are no signs before fracture plane forming, however, fissure occurs with shear and develops to fracture plane (dash line ② in figure 2). c) Correlation: new fracture plane grows due to other fracture planes (dash lines ③ and ④ in figure 2)

Taking the mean value of CT data of undisturbed samples as the base value, a damage variable D_1 can be defined as the relative difference of base value and the mean value of CT data in the process of tests. Damage evolution equation can be obtained through analysis on test date:

$$D_1 = D_0 + \exp\left(\frac{p_0}{p} \varepsilon_s \frac{p}{p_{atm}}\right) - 1 \quad (1)$$

where D_0 is the initial value of damage variable, ε_s is deviator strain, s is suction, p_{atm} is atmospheric pressure, p and p_0 are spherical stress and pre-consolidation pressure, respectively.

Equation (1) can reflect the influence of deviator stress, pre-consolidation pressure, suction and initial damage degree on expansive soil in the process of shear. It is convenient to use equation (1) in constitutive relationship because of its simple style and a few of parameters involved.

3.2 Characteristics of structure damage evolution of remolded expansive soil in the process of wet-dry circles

Twelve wet-dry cycle tests of remolded expansive soils were carried out. Each sample went through 5 wet-dry circles. The sizes and the scanning cross-sections of the samples were as same as in section 3.1. However, the scanning pictures were marked as a0~a5 and b0~b5, successively, were corresponding to the cross section of 0.3H and 0.7H from the top of the samples, respectively. The picture marked as a0 and b0 associated with the initial state of the specimen. A total of 66 CT images were obtained. Let H and SD denote the mean value and the standard deviation of CT data of a scanning section, respectively, the results of the tests show that H decrease and SD augment as the times of wet-dry circles increase. The images show that the cracks initiate and extend continuously. At last, the crannies becomes connectivity each other as shown in figure 3.

Taking the CT data corresponding to the initial state of specimen as the base value, a damage variable D_2 can be defined as the relative difference of base value and the value of CT data in the process of wet-dry circles tests.

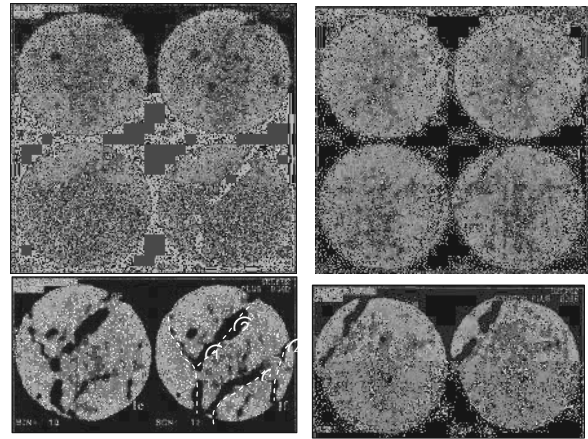


Figure 2. Scanning images during triaxial shear test of sample 2# of intact expansive soil.

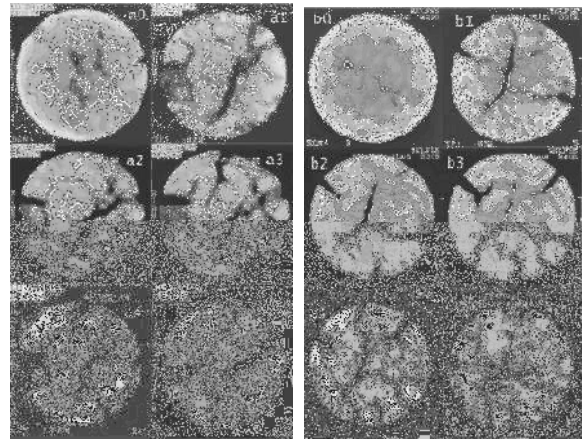


Figure 3. Scanning images through 5 times of wet-dry circles of sample 5# of remolded expansive soil

The damage values related to various volumetric strains in wet-dry circle tests are show in figure 4. Through curve-fitting, the damage evolution equation upon the wet-dry circle test is:

$$D_2 = \exp(A\varepsilon_v) \quad (2)$$

Where A is a soil parameter, and ε_v is volumetric strain. It is obviously that the damage evolution equation is in good agreement with the experiments.

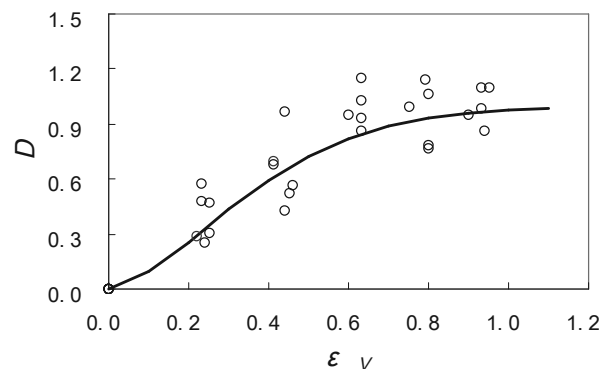


Figure 4. The damage values vs. volumetric strains of samples in the process of wet-dry circles test

Tab.2 Initial physical parameters and stress states of samples before soaking

No.of sample	Initial dry density g / cm ³	Initial water content / %	Void ratio	Net cell pressure ($\sigma_3 - u_a$) / kPa	Matric suction s / kPa	Deviator stress q / kPa	Water pressure during soak / kPa
1	1.31	11.0	1.08	100	150	0	4
2	1.31	11.0	1.08	200	150	0	12
3	1.32	11.0	1.04	100	150	200	14
4	1.32	11.0	1.05	100	250	0	12
5	1.31	11.0	1.06	100	150	100	14
6	1.30	11.0	1.08	100	150	250	14

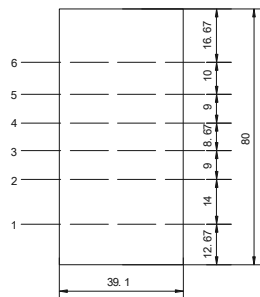
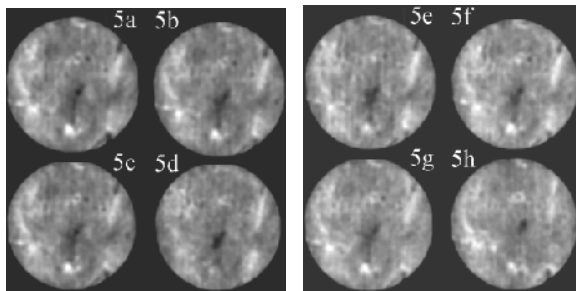


Figure 5. Scanning position (length unit: mm)


 Figure 6 Scanning images of 5th section of sample 3[#] of Q₃ loess during loading

Tab.3 CT data and stress state of sample 3# during loading

Stress state	Scan sequence	5 th Section	
		H	SD
Initial state	a	821.1	49.4
End of consolidation	b	854.6	44.7
q = 25 kPa	c	856.5	44.7
q = 50 kPa	d	852.7	41.4
q = 75 kPa	e	854.1	42.6
q = 100 kPa	f	854.0	41.7
q = 150 kPa	g	887.0	45.9
q = 200 kPa	h	927.2	43.4

4 STRUCTURE EVOLUTION OF LOESS Q₃

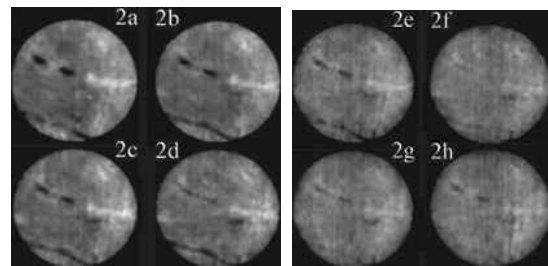
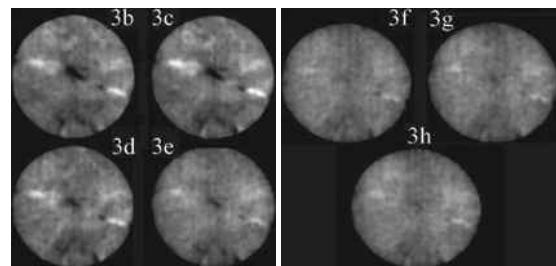
Collapseability is the most important feature of loess. However, collapseability depends on the meso-structure of loess. In order to discover the characteristics of the meso-structure evolution of loess, 10 groups of CT-triaxial tests including 49 samples of Q₃ loess were conducted. A total of 847 CT images and a number of CT data were obtained (Zhu, 2007; Fang, 2008; Li, 2010).

4.1 The structure evolution of Q₃ loess during loading and collapsing

Six tests of intact Q₃ loess were conducted. The size of samples was 39.1mm in diameter and 80mm in height. The initial conditions and test parameters of the samples are shown in table 2. Each sample was scanned 6 sections (Figure 5), and each section was scanned from 6 to 8 times. The scanning pictures were marked as a, b, c, d, e, f, g and h corresponding to a section in successive scanning (Zhu, 2008a). A total of 273 CT images were obtained.

Figure 6 shows the Scanning images of 5th section of sample 3[#] of Q₃ loess during loading. The scanning pictures were marked as a, b, c, d, e, f, g and h corresponding to the initial state, the end of consolidation, and deviator stress (q) equal to 25, 50, 75, 100, 150 and 200 kPa of the 5th section (Table 3). H and SD in table 3 are the mean value and the standard deviation of CT data of the 5th section.

It is clear from Figure 6 and Table 3 that consolidation has significant effect on the meso-structure of sample, and there is no evident change in the meso-structure of sample 3[#] before deviator stress less than 150 kPa. In addition, the defect region (black part in Figure 6) not vanishes completely at the end of loading.


 Figure 7 Scanning images during triaxial collapse of 2nd section of sample 4# of intact Q₃ loess

 Fig.8 Scanning images during triaxial collapse of 3rd section of sample 4# of intact Q₃ loess

The CT-triaxial collapse tests of Q_3 loess have been done under various mass of inundation. The scan images of CT-triaxial inundation tests for sample 4[#] are show in figures 7 and 8. The following understanding can be summarized from CT images:

1) For certain stress state, the original structure of the loess is damaged during collapse and a new stable homogeneous structure is formed simultaneously. In the CT images, the fissures and the cavity in the undisturbed loess samples shrink gradually, even disappear at last.

2) Either spherical stress or shear stress can lead to collapse including volume strain and shear strain. The 1[#], 2[#] and 4[#] samples (Table 2) are in hydrostatic state of stress, and their collapsible volume strains are 4.3 %, 1.27 % and 4.79 %, respectively. It can be seen that the structure of 4[#] sample occurs a substantial change from a non-homogeneous (even with cracks) to a quite homogeneous state. For example, from Fig.7-2b to Fig.7-2c, the mean value of the CT data increases from 829.06 to 983.26 with an increment of 154.2. However, from Fig.7-2f to Fig.7-2g and from Fig.8-3f to Fig.8-3g, the mean values of the CT data change only in the fractional part.

The above two opinions proposed by the first author of the paper in 1986 firstly (Chen, 1986a, 1986b) are proved by the CT images, which give the visual evidence to the structure changes of the loess in the process of collapse. These two characteristics of collapse also show that collapse is different from shear failure, in other words, shear deformation during collapse process of loess is generally limited and taking the M-C criteria as initial collapse condition is unreasonable, which was pointed out by the first author of the paper in 1986 (Chen, 1986b).

4.2 Determining yield stress of intact loess with the help of CT scanning data

Figure 9 shows the relations of CT number vs. stress of two scanning sections of 3[#] simple Q_3 loess during shear (Table 2). It can be seen that there is a characteristic point at each curve. The CT numbers of pre- and post- the point change significantly, which means that the point is yield point. Thus, a method to determine yield point is proposed with the help of CT scanning data.

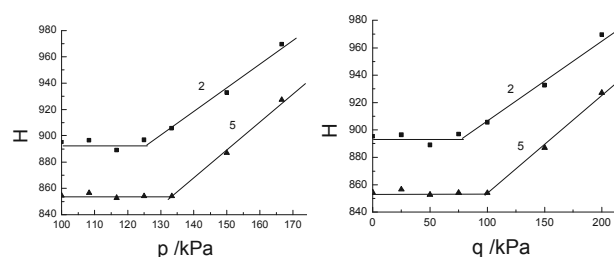


Fig.9 CT number vs. stress of two scanning sections of 3[#] simple Q_3 loess during shear

5 CONCLUSIONS

(1) The macro-scale behaviors of the soil samples are closely related with their mesostructural evolutions.

(2) Fissures sprout and grow during loading or during wet-dry cycles for undisturbed expansive soil for remolded expansive soil.

(3) The original structure of the loess is damaged and a new homogeneous structure is formed during wetting and for certain stress state. The holes and fissures of intact loess may become gradually small even disappear during loading or inundation depending on stress state of soil.

(4) Based on CT data, the definitions of the structure damage variable of the loess and expansive soil are given. The structural

damage evolution equations of two kinds of soils are obtained in various test conditions.

CT technology give the visual evidence to the structure changes of soils, and makes a solid test foundation to establish the damage evolution equation and structure constitutive model of soils.

6 ACKNOWLEDGEMENT

This work has been supported by the National Natural Science Foundation of China (Grant No. 11072265 & 11272353).

7 REFERENCES

- Chen Z.H. & Liu Z.D. 1986a. Mechanism on collapse deformation of loess. *Chinese Journal of Geotechnical Engineering* 1986(2): 1-12.
- Chen Z.H. & Liu Z.D. 1986b. Some problems on collapsibility of loess. *Chinese Journal of Civil Engineering* 1986(3): 62-69.
- Chen Z.H. 1994. New development in research of unsaturated soils. *Sino-Canadian Symposium on Unsaturated / Expansive soils*, June 1994: 145-152.
- Chen Z.H. 1999. Deformation, strength, yield and moisture change of a remolded unsaturated loess. *Chinese Journal of Geotechnical Engineering* 21(1): 82-90.
- Chen Z.H. , Lu Z.H. & Pu Y.B. 2001. The matching of computerized tomography with triaxial test apparatus for unsaturated soils. *Chinese Journal of Geotechnical Engineering* 23(4): 387-392.
- Chen Z.H., Sun S.G., Fang X.W., Zhu Y.Q. & Xie Y. 2007. Development and application of multifunction triaxial apparatus. *Journal of Logistical Engineering University* 23(4): 1-5.
- Chen Z. H, Yao Z. H. & Miao Q. Q. 2011. Influence of structure damage to the yielding of expansive soils[c]. Proc. AP-UNSAT, 14-16 November 2011, Thailand, Vol.II : 679-684.
- Li J.G., Chen Z.H., Huang X.F. , et al. 2010. CT-triaxial shear tests on the meso-structure evolution and strength of unsaturated loess Q_3 during unloading confining pressure. *Chinese Journal of Rock and Soil Mechanics*, 31(4): 1084-1091.
- Lu Z.H. 2001. Elasto-plasticity-damage constitutive model of unsaturated expansive soil and applications on multi-field coupling numerical analysis for Soil Slope. Chongqing: Ph. D. dissertation, Logistical Engineering University.
- Wang S.J., Chen Z. H. & Li X. The verification of damage variable of expansive soil based on CT numbers. *Chinese Journal of Rock and Soil Mechanics*, 2012, Underreview.
- Wei X.W. 2007. Research on wet-swelling deformation and characteristics of structure evolution of expansive soil. Chongqing: Master Degree Dissertation, Logistical Engineering University.
- Yao Z.H. 2009. Evolution characteristics of meso-structure of expansive soils in triaxial soaking test. Chongqing: Master Degree Dissertation, Logistical Engineering University.
- Yao Z. H , Zang J. H & Chen Z. H. 2011. Study on meso-structure changes of remolded expansive soils during wetting-drying cycles and triaxial soaking test[c]. Proc. AP-UNSAT 2011, 14-16 November, Thailand, Vol.II : 685-692
- Zhu Y.Q. & Chen Z.H. 2008b. New methods for research of collapsibility of loess. *Chinese Journal of Geotechnical Engineering* 30(4): 524-528
- Zhu Y.Q. 2008a. Research on constitutive model of unsaturated undisturbed collapse loess based on evolution of meso-structure. Chongqing: Ph. D. dissertation, Logistical Engineering University.

Misconceptions about experimental substantiation of creep hypothesis A

Les idées fausses justifiant l'hypothèse A de fluage au laboratoire

Degago S.A.

Norwegian Public Roads Administration, Trondheim, Norway

Grimstad G.

Oslo and Akershus University College of Applied Sciences, Oslo, Norway

Jostad H.P.

Norwegian Geotechnical Institute, Oslo and Norwegian University of Science and Technology, Trondheim, Norway

Nordal S.

Norwegian University of Science and Technology, Trondheim, Norway

ABSTRACT: Ample laboratory experiments as well as field observations show existence of rate effects or creep during the primary consolidation phase of clayey soils. However, the role of creep during the primary consolidation phase has been a subject of active debate among researchers. As a result, two totally different hypotheses referred to as creep hypothesis A and B have been defined as a basis for the discussion. Despite being opposite extreme to each other, both creep hypotheses seem to be experimentally supported leading to confusion as to which of them is correct. This paper aims to consistently clarify the apparent misconceptions involved in the experimental substantiation of hypothesis A as well as discuss some of its unphysical implications. This means to provide simple and convincing arguments as to why creep hypothesis A is not experimentally substantiated. It is shown that cohesive soils behave in conformity with hypothesis B. It is also illustrated that a constitutive model based on hypothesis B can give excellent prediction of long-term field measurements of settlements and excess pore pressure responses.

RÉSUMÉ : De nombreuses observations sur le terrain et au laboratoire ont montré que le fluage existe au cours de la phase de consolidation primaire des argiles. Le rôle du fluage pendant la phase de consolidation primaire fait toutefois l'objet d'une vive controverse dans la littérature. Au cours des ans deux hypothèses complètement différentes ont été définies pour discuter du fluage dans les argiles ; l'hypothèse A et B. Les deux hypothèses semblent être soutenues par des résultats en laboratoire et cela mène à savoir laquelle des deux est correcte. Le but de cette étude est de clarifier certaines idées fausses concernant l'hypothèse A basé sur des résultats expérimentaux. Ceci vise à renforcer les arguments qui montre que l'hypothèse A n'est pas valide. L'étude montre également que les sols cohérents se comportent pratiquement de la façon d'écrite par l'hypothèse B. Finalement, un modèle de comportement basé sur l'hypothèse B produit des résultats qui concordent très bien avec des mesures de tassements en chantier ainsi qu'avec la réponse des pressions interstitielles due à la charge.

KEYWORDS: Creep, primary consolidation, settlement, clays

1 INTRODUCTION

Settlement of saturated soils under increased loading consists of two successive phases, commonly referred to as the primary and secondary consolidation phases. The primary consolidation phase is dominated by pore pressure dissipation and effective stress increase; whereas, the secondary consolidation phase is dominated by creep at almost constant effective stress.

The existence of creep during primary consolidation is evident, but there exist opposing opinions on the role of creep in the primary consolidation phase. In 1977, Ladd et al. formally proposed two creep hypotheses referred to as creep hypotheses A and B. Creep hypothesis A implies that the end of primary consolidation (EOP) strain and EOP preconsolidation stress (p'_c) are unique, independent of the consolidation duration; while, hypothesis B implies that the EOP strain increases or EOP p'_c decreases with increasing duration of the primary consolidation.

The two creep hypotheses have significance implications when it comes to practical applications such as prediction of field settlements where settlement behaviours of laboratory tests (short primary consolidation duration) have to be extrapolated to describe in-situ performance of clay deposits with very long primary consolidation duration. Mesri (2003) and Leroueil (2006) summarised several experimental and numerical substantiations that have been independently presented to advocate the two hypotheses. However, these voluminous substantiations had little effect in deciding which of the two hypotheses was correct as experimental observations were presented that seem to advocate two opposing soil behaviours.

2 SUBSTANTIATIONS OF THE CREEP HYPOTHESES

The substantiations and arguments for hypothesis B are more general as compared to the extreme hypothesis A which says that the EOP strain and p'_c are identical irrespective of the time it takes to reach an EOP state. Hence, this motivates to take a closer look at the experimental substantiation of hypothesis A. Four groups of substantiations have been put forward by the advocates of hypothesis A (Mesri, 2003). These arguments can briefly be stated as; (1) comparing EOP strain of specimens with different heights (here 127 and 508 mm thick specimens), (2) studying sub-specimen compressibilities in interconnected tests (3) predicting field settlements and excess pore pressures using a numerical model developed based on hypothesis A concept (referred to as ILLICON methodology) and (4) comparing field and laboratory preconsolidation stresses.

Degago et al. (2009, 2010, 2011a and 2011b) and Degago (2011) have thoroughly investigated the experimental and numerical substantiations of hypothesis A and provided explanations using a consistent framework as to why hypothesis A seemed to be wrongly substantiated. Actually, the very same data have been used to substantiate hypothesis B. However, recent work by Mesri and Feng (2009) (published in 2011) indicates a series of misconceptions by the advocates of hypothesis A regarding substantiation of hypothesis A. Therefore this paper attempts to briefly provide clarifications for most of these misconceptions. In addition, it presents some of the recent works performed with regard to the experimental and numerical substantiations of the two creep hypotheses.

3 SOME NUMERICAL ASPECTS

The isotache concept, proposed by Šuklje (1957), can conveniently be used for modeling rate dependency of clays. The isotache concept states that there is a unique relationship between the current strain rate (change in void ratio), effective stress state and strain (void ratio).

Under oedometer testing conditions, a direct implication of the isotache concept is that the experienced p'_c is dependent on the time between the load increments, or the rate of loading. Under EOP incremental loading scheme, the implication of the isotache concept is sketched in Figure 1 for fast and slow consolidation durations and the experienced p'_c is shown to be rate dependent. In opposition to this, hypothesis A implies a unique EOP effective stress-strain relationship irrespective of consolidation duration (Mesri and Choi, 1985b). Hence, the distinction between the two hypotheses basically comes down to whether the resulting p'_c is rate dependent or not.

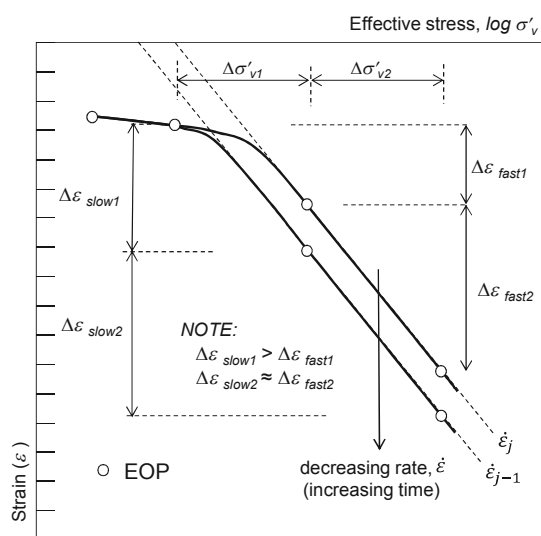


Figure 1. Implication of the isotache concept for EOP states of fast and slow consolidation times under incremental loadings up to EOP states.

In the isotache concept the strain rate is determined by the current void ratio and effective stress. In water-saturated soil, change in void ratio can only take place when water is expelled from the soil. Therefore the strain rate is indirectly controlled by the global consolidation process. However, a soil element inside a soil layer (or sample) has neither any direct information of this global consolidation process nor remaining time until the EOP consolidation state is reached. However, for hypothesis A to hold true, the response in all soil elements must be a function of this remaining time and its advocates argue that “no sub layer, including the drainage face, experiences any secondary compression until the simultaneous completion of primary consolidation of all sub layers” (Mesri and Vardhanabhuti 2006). Such assertion, however, violates some basic axioms of continuum mechanics such as axiom of *material invariance* and axiom of *objectivity* (see e.g. Eringen (1967)).

4 SOME MISCONCEPTIONS AND CLARIFICATIONS ON SUBSTANTIATIONS OF CREEP HYPOTHESIS A

Degago et al. (2009) re-evaluated the EOP experiments conducted on 127 and 508 mm thick specimens by Feng (1991) and showed that the experiments actually substantiate hypothesis B. In addition, Degago et al. (2009) used a numerical tool based on hypothesis B to analyse the raw data of the tests as they were originally conducted and showed that they are explainable using this model. Mesri and Feng (2009) questioned the validity of hypothesis B and attempted to provide an

explanation for their tests. However, a series of misconceptions are visible in Mesri and Feng (2009) that needs clarifications and these are given in the following sections by classifying the apparent misconceptions into laboratory and field studies.

4.1 Laboratory studies

As illustrated in Figure 1, the specific load increment that starts below initial p'_c and exceeds it is critical. This has been treated in greater detail in Degago et al. (2011a). Degago et al. (2009) focused on this increment and showed that the tests conducted by Feng (1991), on the 127 and 508 mm thick specimens of Batiscan and St. Hilaire clays, did not have the same EOP state.

To determine if there is any possibility that hypothesis A has a practical use, evidences for hypothesis A must be found. This requires first of all giving an objective definition of time at the end of “primary consolidation”. The obvious criterion would be the remaining excess pore pressure. However, this requires a detailed knowledge on the excess pore pressure. Mesri and Feng (2009), referring to Mesri et al. (2005), admit that such an exact criterion does not exist.

Mesri and Feng (2009) claim that Degago et al. used “micro-management” to evaluate the EOP criterion adopted in the test by Feng (1991). However, a clear criterion is exactly what one needs for answering this fundamental question, especially to study the validity of hypotheses A where EOP is a key state. With this regard, it is worthwhile to mention that EOP definition is not important for hypothesis B where there is a smooth transition from primary to secondary consolidation phases. Still, it is important to understand the nature of excess pore pressure around EOP state where creep starts to dominate and governs the dissipation of the remaining excess pore pressure. At this stage, the soil can continue to deform without a significant change in excess pore pressure. Consequently, the EOP criteria can easily be misused and there is a potential of exposing specimens being compared to unsystematic creep durations. In such cases, comparisons may end up being not genuine enough to reflect reality. The excess pore pressures for the 127 and 508 mm thick specimen of Batiscan clay were 0.1 and 0.8 kPa and for St. Hilaire clay they were 1.0 and 2.2 kPa, respectively. Under these conditions one cannot claim that the thin and the thick specimen have had the same EOP state.

One fundamental proof that was overlooked in the discussion of Mesri and Feng (2009) is time considerations aspects. From the classical consolidation theory, the ratio of the time needed to achieve the same degree of consolidation between two specimens is equal to the square of the ratio of the heights. However, because of the consolidation time being increased by creep, a thick specimen would need more time than the one calculated based on the classical consolidation theory concept. Accordingly, one can compare the time needed to achieve EOP state for the 127 mm (t_{127}) and 508 mm (t_{508}) thick specimens studied by Feng (1991). In fact the ratio t_{508}/t_{127} in the actual tests of Batiscan and St. Hilaire clay were only 7 and 9 instead of being larger than 16. Therefore the tests do not even qualify as tests conducted in accordance to hypothesis A where the ratio t_{508}/t_{127} is expected to be equal to 16 (Ladd et al., 1977).

Based on the final excess pore pressure of the 508 mm thick specimen, Degago et al. (2009) established the time that corresponds to the same EOP state of the 127 mm thick specimen. This gave a ratio t_{508}/t_{127} of 19 and 20 for both clays (>16), and most importantly an EOP strain that increases with specimen thickness. Figure 2 shows details of the excess pore pressure and volumetric strain development of St. Hilaire clay, for the step of interest, for both sample thicknesses. It is seen that a small variation in excess pore pressure gives significant difference in the “primary consolidation” duration and the corresponding strains. To achieve same EOP criterion with the 508 mm sample, the 127 mm sample should have been loaded for 14 days instead of the actual 33 days adopted in the tests.

Mesri and Feng (2009) present two figures of excess pore pressure versus time for Batiscan clay to show that sufficient time is given for the 508 mm thick specimen to reach EOP. The stress increments they presented were for a step well before the initial p'_c and for a step after exceeding the initial p'_c . However, they did not present the most important step, i.e. the step that starts below and exceeds the initial p'_c . Here lies the main misconception, as Mesri and Feng (2009) tend to continue to underrate the importance of the load increment that exceeds the initial p'_c .

Applying the interpretation shown in Figure 2, the resulting stress-strain relationship of the St. Hilaire clay is shown in Figure 3. The interpreted experimental data clearly imply hypothesis B, unlike the original curves where inconsistent EOP criteria were used to wrongly advocate hypothesis A. Mesri and Feng (2009) wondered about the re-interpretation of the load increment after exceeding p'_c (the broken lines in Fig 3). However, referring to the broken line in Figure 3, Degago et al. (2009) clearly stated “a similar re-interpretation procedure could not be adopted for the following steps since the thin specimen has already exceeded its EOP condition”. To establish the broken lines Degago et al. (2009) simply used the fact that, after exceeding p'_c , the incremental strain for a thin and thick specimen are almost the same, see Figure 1.

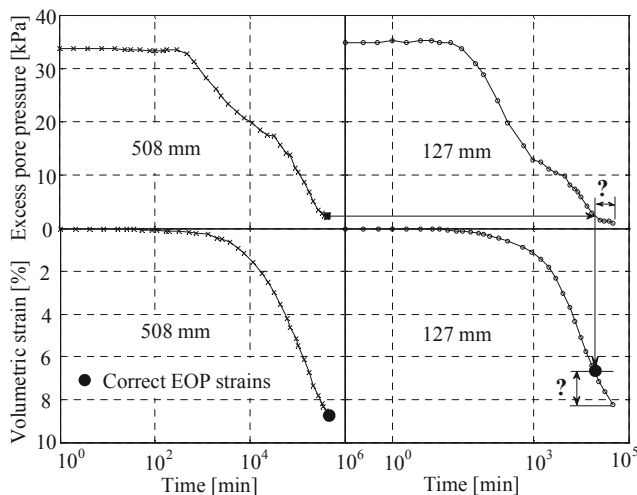


Figure 2. Re-evaluation of experimental results of St. Hilaire clay for the load increment that exceeds p'_c (raw data after Feng, 1991)

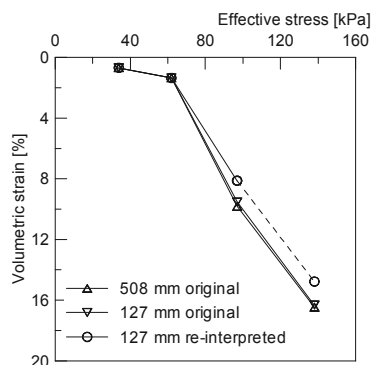


Figure 3. EOP volumetric strain-effective stress relationship of thin and thick specimen of St. Hilaire clay for the load increment that exceeds p'_c (raw data after Feng, 1991)

Regarding soil element compressibility, Degago et al. (2010) conducted a test to show that a soil element close to the drainage boundary does not wait for the global EOP state before starting secondary compression. This was also found when Degago et al. (2011a) revisited the sub-specimen nominal strain versus time relationships of the 508 mm samples of Batiscan and St. Hilaire clay conducted by Feng (1991). For the step

exceeding the initial p'_c , the tests showed largest deformation on the sub-specimen closest towards the drainage boundary. This is in accord with predictions by the isotache concept and contrary to the claims made by Mesri and Vardhanabhati (2006).

4.2 Field studies

Mesri and Feng (2009) argued that “the computer program based on Soft Soil Creep (SSC) model should be applied to field situation with primary consolidation duration of 30 to 50 years before reaching any general conclusion on EOP compression and on the uniqueness of preconsolidation pressure”. In addition they presented excellent settlement predictions of field cases to illustrate the validity of hypothesis A. In connection to this, Degago et al. (2011b) studied 57 years of settlement data from the Väsby test fill and have clarified two misconceptions observed in the arguments by Mesri and Feng (2009) regarding their field predictions. First, it was clearly illustrated that the excellent settlement predictions and somehow lower excess pore pressure predictions by ILLICON were due to use of soil data from highly disturbed samples along with inappropriate analysis assumption. Secondly, it was demonstrated that when one interprets soil data from high quality samples then the isotache model (SSC) gives excellent settlement as well as excess pore pressure predictions, see Figure 4 and Figure 5.

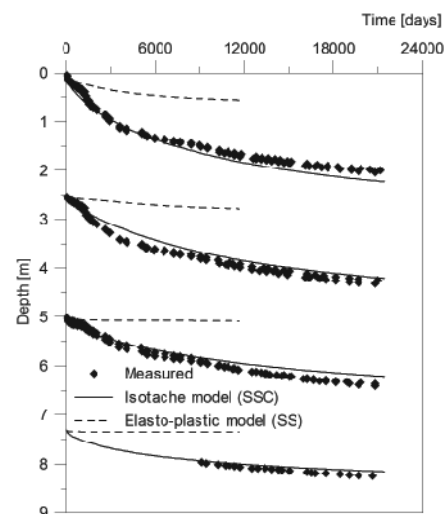


Figure 4. Time-displacement curves for different depths (Degago et al. 2011b)

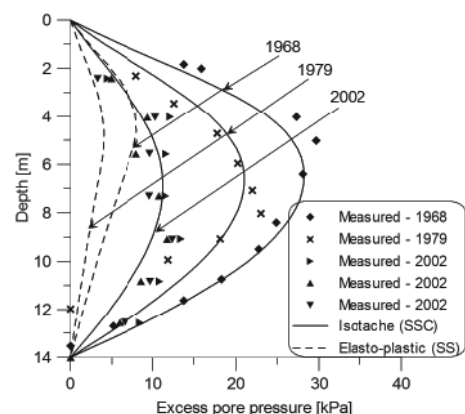


Figure 5. Excess pore pressure profiles for different times plotted against undeformed soil layer depth (Degago et al. 2011b)

A typical oedometer test used in the analysis of Väsby test fill by ILLICON (Mesri and Choi, 1995a) had an EOP overconsolidation ratio (OCR) value of 1.3 as compared to the more correct value of 1.8 interpreted from high quality block samples (Leroueil and Kabbaj 1987). The implication of this is that, had Mesri and Choi (1985a) used high quality data,

ILLICON would have significantly under predicted both the excess pore pressure and the settlement measured at the Väsby test fill. In addition, Mesri and Choi (1985a) also ignored an important role of buoyancy in large deformations. In the time period analyzed by Mesri and Choi (1985a), a 42% load reduction was estimated due to buoyancy effect. This effect is too large to be neglected. Taking into account this key aspect would have even led ILLICON to further underestimate measurements in a similar manner to the results of the elastoplastic model (SS) shown in Figure 4 and Figure 5.

In connection to another field aspect, Mesri and Feng (2009) also present a summary data, detailed in Mesri et al. (1995), claiming that field and laboratory vertical preconsolidation stresses (σ'_p) are the same. The authors of this paper found it difficult to access these data in Mesri et al. (1995), in order to analyze the quality of the test data (e.g. sample disturbance effects) and to assess how and which procedures are used in the determination of σ'_p 's, see further discussion in Degago (2011). An exception to this was the data by Sällfors (1975) which constitutes one third of the data gathered by Mesri et al. (1995). Therefore the authors have thoroughly looked into the laboratory and field σ'_p data as presented in Sällfors (1975).

A good starting point to evaluate the data by Sällfors is to understand the background behind the data. Sällfors studied and determined field σ'_p based on pore pressure response to an applied stress increment. Then he proposed a method to directly predict the field σ'_p based on laboratory CRS tests. Because of the background and aim of the method, the σ'_p interpreted in this way would naturally give a σ'_p lower than those determined for laboratory cases (Olsson, 2010). In simple terms, the good match between laboratory and field σ'_p of Sällfors (1975) merely show that the Sällfors method serves its purpose. However, Mesri et al (1995) took the final data and made a wrong conclusion and this may relate to a lack of understanding of the objective behind the data gathered by Sällfors (1975).

5 SUMMARY OF THE MAIN MISCONCEPTIONS

The most important misconceptions as observed in the works of the advocates of hypothesis A arises from overlooking some important aspects of clay compressibility. These are discussed in detail in this paper and are briefly summarized as follows:

1. The importance of the load (effective stress) increment that starts below and exceeds the initial p'_c has not been considered and understood properly by the advocates of hypothesis A and its role has continually been underrated.
2. Without a unique and consistent EOP criterion, the results from samples with different specimen heights can be interpreted inconsistently resulting in misleading conclusions. The discrepancy is not necessary significant within the duration of laboratory tests, but become important when extrapolating to field condition. EOP is of no interest when using a model based on hypotheses B as it gives a smooth transition between primary and secondary consolidation phases.
3. Effect of sample disturbance needs proper assessment. Successful prediction of long-term field performance demands use of high quality data with creep considerations. However, by using results from tests on highly disturbed samples and disregarding creep, one may obtain reasonably good estimate of settlements.

6 FINAL REMARKS

Degago et al. (2009, 2010, 2011a and 2011b) and Degago (2011) clearly showed that there exist definitive data to demonstrate that the creep hypothesis B agrees very well with the measured behaviour of cohesive soils. It is also illustrated that the isotache approach describes this soil response

properly. In closure, the main points of these studies with regard to this paper are briefly stated as follows;

1. The experienced preconsolidation stress as well as EOP strain are rate dependent even for EOP loading conditions and this fact has been experimentally supported by several EOP tests and field observations. All the experimental evidences that were used to wrongly advocate hypothesis A actually imply hypothesis B.
2. Hypothesis A would require that the soil element close to the drainage boundary would wait for the global EOP state before starting its secondary compression. However, various tests conducted on sub-specimen compressibility clearly showed that this does not hold true and the compressibility of a soil element is controlled by prevailing conditions at that particular element rather than what is happening elsewhere.
3. A model based on the isotache approach gives excellent agreement between field measurement and numerical simulations when soil data are derived from high quality samples along with appropriate analyses assumptions.
4. Future developments related to the compressibility of natural clays, such as modeling anisotropy and destructuration effects, should be based on the isotache framework (hypothesis B) along with use of soil data from high quality samples.

7 REFERENCES

- Degago S. A. 2011. *On creep during primary consolidation of clays*. PhD thesis, Norwegian University of Science and Technology, Trondheim, Norway.
- Degago S. A., Grimstad G., Jostad H. P. and Nordal S. 2009. The non-uniqueness of the end-of-primary (EOP) void ratio-effective stress relationship. *Proc. 17th ICSMGE*, Alexandria 1, 324–327.
- Degago S. A., Jostad H. P., Olsson M., Grimstad G. and Nordal S. 2010. Time- and stress-compressibility of clays during primary consolidation. *Proc. 7th NUMGE*, Trondheim, 125–130.
- Degago S. A., Grimstad G., Jostad H. P., Nordal S. and Olsson M. 2011a. Use and misuse of the isotache concept with respect to creep hypotheses A and B. *Géotechnique* 61(10), 897–908.
- Degago S. A., Nordal S., Grimstad G. and Jostad, H. P. 2011b. Analyses of Väsby test fill according to creep hypothesis A and B. *13th Int. Conf. of IACMAG*, Melbourne 1, 307–312.
- Eringen A. C. 1967. *Mechanics of Continua*. John Wiley & Sons.
- Feng T. W. 1991. *Compressibility and permeability of natural soft clays and surcharging to reduce settlements*. PhD thesis, University of Illinois at Urbana-Champaign, Urbana-Illinois, USA.
- Ladd C. C., Foott R., Ishihara K., Schlosser F. and Poulos H. G. 1977. Stress-deformation and strength characteristics. State-of-the-art report. *Proc. 9th ICSMFE*, Tokyo 2, 421–494.
- Leroueil S. and Kabbaj M. 1987. Discussion of 'Settlement analysis of embankments on soft clays'. *ASCE* 113(9), 1067–1070.
- Leroueil S. 2006. Šuklje Memorial Lecture: The isotache approach. Where are we 50 years after its development by Professor Šuklje? *Proc. 13th Danube Eur. Conf. Geotech. Engng*, Ljubljana 2, 55–88.
- Mesri G. and Choi Y. K. 1985a. Settlement analysis of embankments on soft clays. *ASCE* 111 (4), 441–464.
- Mesri G. and Choi Y. K. 1985b. The uniqueness of the end-of-primary (EOP) void ratio-effective stress relationship. *Proc. 11th ICSMFE*, San Francisco. 2:587–590.
- Mesri G., Feng T. W. and Shahien M. 1995. Compressibility parameters during primary consolidation. *Proc. Int. symposium on compression and consolidation of clayey soils*, Hiroshima, 2, 1021–1037.
- Mesri G. 2003. Primary and secondary compression. In *Soil behavior and soft ground construction* (eds Germaine, Sheahan & Whitman), *ASCE Geotechnical Special Publication* 119, 122–166.
- Mesri G. and Feng T. W. 2009. Discussion of Degago et al. (2009). *Post conference proc. 17th ICSMGE*, Alexandria 5, 3559–3561.
- Olsson M. 2010. *Calculating long-term settlement in soft clays - with special focus on the Gothenburg region*. Licentiate thesis, Chalmers University of Technology, ISSN 1652-9146; nr 2010:3.
- Šuklje L. 1957. The analysis of the consolidation process by the isotaches method. *Proc. 4th ICSMFE*, London 1, 200–206.
- Sällfors G. 1975. *Pressure of soft high plastic clays*. Ph.D Thesis, Chalmers University of Technology, Gothenburg, Sweden.

The relationship between swelling and shear strength properties of bentonites

La relation entre les propriétés de résistance au cisaillement de l'enflure et des bentonitiques

Domitrović D., Kovačević Zelić B.

University of Zagreb, Faculty of Mining, Geology and Petroleum Engineering, Zagreb, Croatia

ABSTRACT: Clay Geosynthetic Barriers are manufactured hydraulic barriers. The mineral component usually consists of bentonite, which belongs to a group of expansive soils. The aim of this study was to establish the influence of swelling on mechanical properties of bentonites. For this purpose, granular bentonite Volclay was chosen for laboratory testing. Its mineralogical composition was predominantly montmorillonite (80% - 85% by mass). The correlation between swelling and water content on one side and changes in shear strength i.e. shear strength parameters on the other were tested using a direct shear device. During several series of testing, different levels of swelling of bentonite were simulated under the conditions of changing normal stress and hydration times. Swelling behaviour of bentonite was determined through the long-term swelling tests using oedometer, under changing normal stress. Shear strength testing show decreasing cohesion with longer hydration of bentonite. The friction angle increases with hydration that lasts from 7 to 14 days. There is no significant change in the friction angle with hydration time longer than 14 days.

RÉSUMÉ : Les géosynthétiques bentonitiques sont des produits manufacturés pour les barrages hydrauliques. La composante minérale est le plus souvent l'argile bentonitique qui appartient à la catégorie des sols gonflants. L'objectif de cette recherche est de déterminer l'influence du gonflement sur la propriété mécanique de l'argile bentonitique. Pour la recherche est choisie l'argile bentonitique en granulés Volclay, de composition minérale où domine la montmorillonite 80-85 %. La corrélation du gonflement et des changements d'humidité avec les variations de la résistance au cisaillement et les paramètres de résistance au cisaillement a été étudiée dans le dispositif de cisaillement direct. Voici la série de tests à différentes contraintes normales, à des temps variés d'hydratation, simulant divers degrés de gonflement de l'argile bentonitique. La courbe de gonflement d'argile bentonitique est définie par les expériences de gonflement prolongées à différentes contraintes normales dans l'œdomètre. Les résultats montrent une réduction significative de cohésion avec la prolongation de l'hydratation de l'argile bentonitique. L'angle de frottement augmente avec l'hydratation (de 7 à 14 jours). Il n'y a pas de changement significatif dans la valeur de l'angle de frottement lors d'une prolongation de l'hydratation supérieure à 14 jours.

KEYWORDS: Bentonite, Swelling, Direct shear test, Shear strength.

1 INTRODUCTION

Clay Geosynthetic Barriers (GBR-C) - a member of the family of synthetic materials - are extensively used nowadays as sealing barriers or their components in a wide range of engineering applications (Guyonnet et al. 2009, Kang and Shackelford 2010, Shackelford et al. 2010). In the clay geosynthetic barrier, bentonite makes the mineral component that ensures low hydraulic conductivity (Koerner 1996, Shackelford et al. 2000, Bouazza 2002, Katsumi et al. 2008), while layers of geotextile make the supporting component. Layers of geotextile are usually needle-punched or stitch-bonded in so called reinforced GBR-Cs. There are also unreinforced GBR-Cs, but their use are limited to relatively flat slope applications that do not present significant shear stress.

In deciding about the most suitable design for the sealing system, stability is one of the important factors to consider. When using clay geosynthetic barriers, it is important to pay attention to interface and internal shear strength. The interface strength is developed at the contact between the clay geosynthetic barrier and adjacent material, be it soil or other geosynthetic material. The internal shear strength is different with reinforced and unreinforced clay geosynthetic barriers. The internal shear strength of reinforced clay geosynthetic barriers depends on the strength of their components, more precisely the ultimate tensile strength of reinforcement yarns and the shear strength of bentonite. Hydrated bentonite has very low shear strength, which is why its impact on the internal shear strength will be reduced. The internal shear strength of unreinforced clay geosynthetic barriers is identical to the shear strength of bentonite clay (Gilbert et al. 1996, Kovačević Zelić 2000, Zornberg and McCartney 2009, Fox 2010).

Bentonite clays fall into the group of expansive soils. It was the bentonite clay swelling property the key mechanism that ensures very low levels of hydraulic and gas conductivity, and therefore its role of the clay geosynthetic barrier's sealing component. The cause for swelling is found in the fact that smectite clay minerals are the main component of bentonite, with montmorillonite as the predominant mineral. Electrical charge and the colloidal particle size are the reasons why this group of minerals is hydrophilic. Their ability to attract molecules of water allows them to increase volume several times, and it plays an important part in the mechanical properties of bentonite, including strength, deformability and hydraulic conductivity. What contributes further to the low hydraulic conductivity of bentonite is an increase in the content of montmorillonite; specific surface area (decrease in the size of particles); electrical charge deficit and sodium content (Na^+) in the system of exchange (Mitchell and Soga 2005, Guyonnet et al. 2009). The influence of these factors on the quality of bentonite can mostly be seen macroscopically through increases in ion exchange capacity, plasticity and swelling capacity (in the presence of fluid), against decreases in hydraulic conductivity and strength. It should be stressed here that sodium bentonite clays demonstrate a very low level of hydraulic conductivity and a high swelling capacity.

For the purpose of quantifying the role of bentonite within the clay geosynthetic barrier, laboratory tests were conducted with samples of Volclay granular bentonite, using direct shear and oedometer tests. As a part of this study, a detailed characterization of bentonite was carried out. Mineralogical composition of bentonite was: montmorillonite 80-85%, cristobalite around 5%, quartz around 5%, plagioclase 5%. Index properties of bentonite were as follows: liquid limit

437.0%, plastic limit 52.86%, specific surface area 700 m²/g and ion exchange capacity 88.63+/-6.51 meq/100g. The influence of swelling and water content of bentonite samples on the measured values of total shear strength and shear strength parameters was observed using the direct shear device. In a series of tests, under conditions of varying effective stress, hydration times were changed depending on the results of oedometric tests in order to simulate different levels of bentonite swelling. Swelling behavior of bentonite was defined through long-term oedometer tests with varying effective stress. In order to prevent the change in chemical and mineralogical composition of bentonite, demineralised water was used as the test fluid. Shear displacement rate was 1 mm/min. This displacement rate enables relatively short shear stage in relation to previously finished hydration stage, that is, the reduced impact of additional hydration and creep during shear stage on the test results.

2 OEDOMETER SWELL TEST

Bentonite swelling tests were performed using standard oedometer cells of 74 mm in diameter. The placement procedure was relatively simple i.e. pouring of granular bentonite into the oedometric cell was performed without the application of external loading. The initial water content (as-received) of granules was approximately 12%. Identical amounts of bentonite were used for all specimens, making sure that the bentonite is not compacted, but only slightly flattened. After installation, the specimens were loaded to normal stress levels of 50, 100 and 200 kPa. The next step was to add demineralised water into the cell, leaving the specimens to swell under applied normal stress levels for the next 276 days. At the end of experiments, specimens were taken out of the device, and final moisture content was determined.

Test results showed the highest level of swelling and relative vertical deformation for those specimens that were under the lowest levels of normal stress. Therefore, the intensity of swelling decreased as normal stress level increased. It is evident on the basis of the swelling curves that relative vertical deformation (swelling) of specimens after the period of 276 days was 65.80% (6.787 mm) under normal stress of 50 kPa; 38.54% (3.945 mm) under normal stress of 100 kPa and 13.93% (1.339 mm) under normal stress of 200 kPa.

The analysis of time required for the primary swelling stage indicates that these times were identical for all normal stress levels (Figure 1). In this particular case, the time for completion of the primary swelling was approximately 31 days, looking at all three series.

On the basis of analysis of vertical deformation development for these specimens upon completion of the primary swelling stage, it is evident that in the period which remained the stage of secondary compression started. Following conclusions were drawn by observing vertical deformations over the remaining period of 245 days during which there was secondary compression of the specimens:

- with specimen subjected to normal stress intensity of 50 kPa, there was compression by 0.092 mm, resulting in vertical deformation of 0.54%;
- with specimen subjected to normal stress intensity of 100 kPa, there was compression by 0.196 mm, resulting in vertical deformation of 1.36%;
- with specimen subjected to normal stress intensity of 200 kPa, there was compression by 0.203 mm, resulting in vertical deformation of 1.82%.

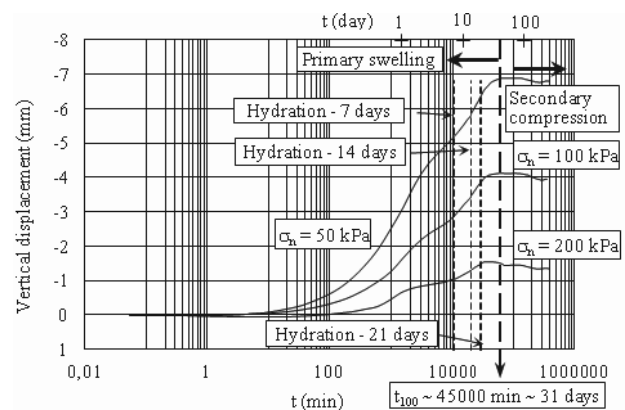


Figure 1. Bentonite swell tests.

Therefore, same as with the primary swelling, the rate of vertical deformation in the stage of secondary compression and how it develops with time also depends on normal stress levels, but in this case the intensity of secondary compression increases with the increase of normal stress level.

The specimens were subjected to secondary compression over a long period of time, so it is assumed that the impact of changing temperature in the laboratory on the vertical deformation curves is possible during the period of secondary compression. A combination of very low vertical deformations and variable temperatures in the laboratory during measurement may lead to a change in the rate of vertical deformation increment during secondary compression. Some changes in temperature in the laboratory were expected, and it is assumed that they affect measuring sensors used in this test.

3 DIRECT SHEAR TEST

3.1 Laboratory testing program

Clay geosynthetic barriers are composite materials. Considering the specific form of clay geosynthetic barriers, their shear strength is mainly tested using modified direct shear devices. This test was aimed at quantifying the performance of bentonite clay component within the clay geosynthetic barrier. It was conducted on a sample of granular bentonite, not including the geosynthetic component. The shear strength tests on some unreinforced and particularly on reinforced clay geosynthetic barriers indicate that special attention is required relating to the size of specimen. However, the specimen size is not crucial when testing the shear strength of bentonites. Therefore, a standard direct shear device with box dimensions of 60×60 mm was used in this study.

Previous studies of the shear strength of bentonite indicate that the key influence on its behavior comes from the property of swelling i.e. moisture content in the specimen. In order to establish the influence of bentonite swelling on its shear strength, specimens were tested in three series under normal stress of 50, 100 and 200 kPa, with varying hydration times (7, 14 and 21 days).

The specimen placement procedure consisted of pouring bentonite into the shear box. The as-received water content level in granules was approximately 12%. The same amount of granulated bentonite was always used, making sure that the bentonite is not compacted, but only slightly flattened. The described procedure ensured approximately identical initial values for thickness, dry mass and dry density of all specimens (their thickness was approximately 8 mm). This kind of procedure was believed to provide a representative simulation for the conditions under which bentonite is used as part of clay geosynthetic barriers. After the placement, normal stress loading was applied on the specimens immediately followed by the initiation of hydration procedure. After finished hydration

procedure lasting 7, 14 and 21 days for three series of testing, specimens were sheared in the shearing stage by the same rate of shearing. Normal stress levels remained unchanged in both stages.

3.2 Test results

Measured vertical displacements in the shear stage indicate consolidation under all normal stress levels and hydration times. The displacement rate in these tests was 1 mm/min and the assumption was that compression during the shear would cause pore pressure inside the specimen to increase.

Table 1 shows the results of testing in the shear stage. Residual values of shear strength were obtained at specimen relative deformation of 15%, and this was also the maximum horizontal shear that could be obtained in the standard shear box. Shear strength values were constantly growing with the increase of normal stress and with decrease of final moisture content. With specimens subjected to stress levels of 50 and 100 kPa, longer hydration times caused higher values of final moisture content and decreasing shear strengths. With specimens subjected to stress level of 200 kPa, final moisture content and shear strength of bentonite did not change irrespectively on hydration times.

Table 1. Summary of direct shear test results.

Hydration time	Normal stress (kPa)	Shear strength		Final moisture content (%)
		Peak (kPa)	Residual (kPa)	
7 days	50	21.4	17.3	113.12
14 days	50	19.4	12.9	124.32
21 days	50	17.2	11.8	140.26
7 days	100	32.2	25.6	85.36
14 days	100	29.2	21.4	97.82
21 days	100	27.5	20.0	104.05
7 days	200	51.1	38.0	71.33
14 days	200	52.0	37.6	69.66
21 days	200	49.4	36.2	71.91

Table 2 shows the value of shear strength parameters obtained by shearing of bentonite specimens. Peak and residual strength envelopes are shown in Figures 2 and 3.

Table 2. Shear strength parameters.

Hydration time	Peak parameters		Residual parameters	
	c (kPa)	ϕ (°)	c (kPa)	ϕ (°)
7 days	11.99	11.23	11.05	7.80
14 days	8.04	12.47	4.79	9.38
21 days	6.32	12.27	3.63	9.31

By observing peak (Figure 2) and residual (Figure 3) strength envelopes, it is evident that the specimens sheared after 14 and 21 days of hydration have almost identical values of peak and residual friction angle, while peak and residual values of cohesion were decreasing as specimen hydration times were increased. In the case of specimen hydrated for seven days, there was obvious change in behavior, in comparison with specimens hydrated for 14 and 21 days. This specific specimen shows significantly higher values for peak and residual cohesion and somewhat lower values for peak and residual friction angle, in comparison to specimens hydrated for 14 and 21 days.

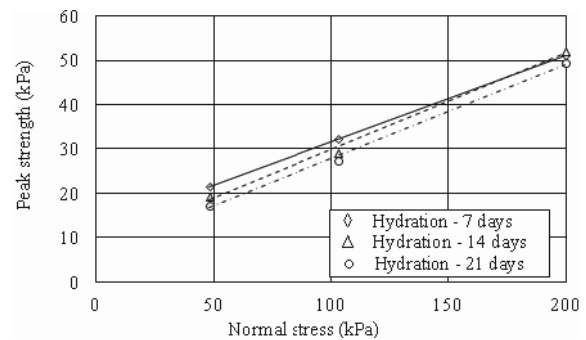


Figure 2. Peak strength envelopes.

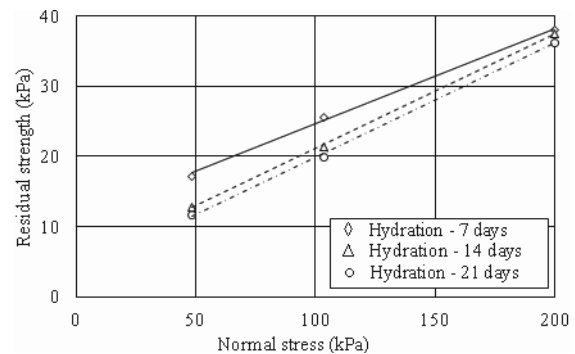


Figure 3. Residual strength envelopes.

Figure 4 presents the influence of final moisture content on the test result values for peak and residual shear strength. It is evident that, irrespectively to the normal stress level, shear strength has tendency to decrease as final moisture content increases.

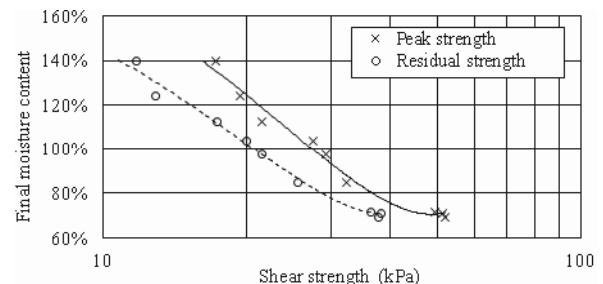


Figure 4. Final moisture content vs. shear strength.

Figure 5 presents the correlation between peak and residual friction angles and the time of hydration. For hydration times in this test, a change in the friction angle was visible up to the 14th day of hydration. After that, the friction angle decreased insignificantly.

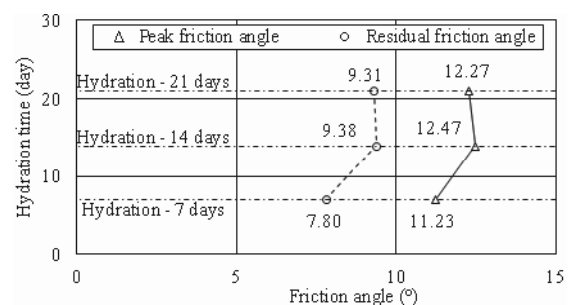


Figure 5. Hydration time vs. friction angle.

Figure 6 presents the correlation between peak and residual cohesion and the time of hydration. Observing the change in cohesion, it is evident that cohesion decreases constantly with longer hydration times.

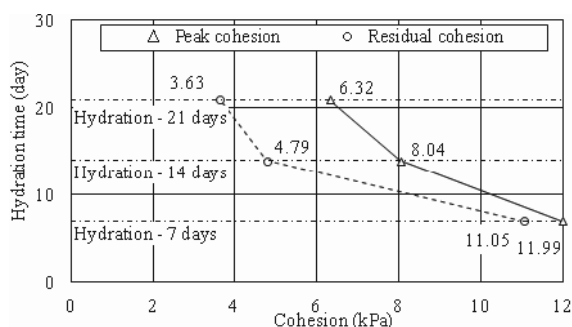


Figure 6. Hydration time vs. cohesion.

Friction angle and cohesion values were normalized and expressed in percentages in relation to the parameter values after seven days of hydration. It is evident that a 14-day hydration causes peak friction angle to increase by around 10%, while extended hydration time results in insignificant decrease of the friction angle (Figure 7). The increase is higher with the residual friction angle – by around 20% after 14 days of hydration, while after that it decreases insignificantly.

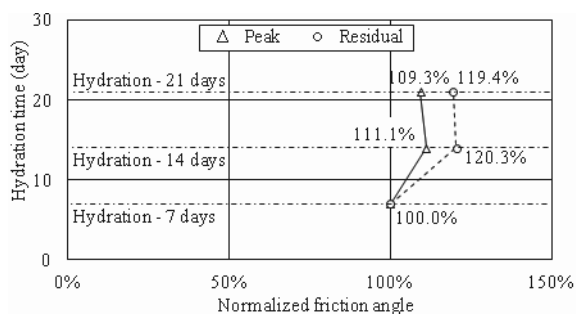


Figure 7. Hydration time vs. normalized friction angle.

Cohesion values continuously decrease with the increase of hydration time, and this is particularly evident with residual cohesion, where this reduction is more obvious (Figure 8). Specifically, the value of peak cohesion equals 67% of normalized value (in comparison to 7-day hydration) after 14 days of hydration and 52.7% after 21 days of hydration; the value of residual cohesion equals 43.3% of normalized value, which is by 13.7% higher reduction in residual cohesion. After 21 days of hydration, the value of residual cohesion equals 32.9% of normalized value, which is by 9.8% less in comparison with peak cohesion.

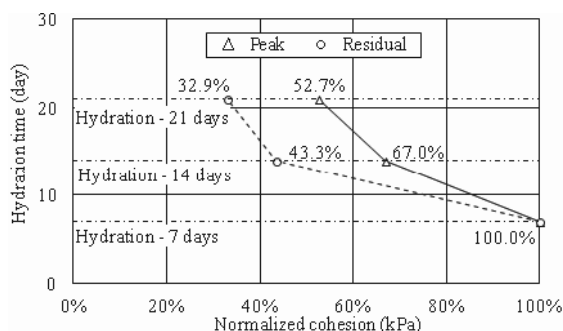


Figure 8. Hydration time vs. normalized cohesion.

4 CONCLUSIONS

Test results show that the swelling of bentonite clay is a long-lasting process. Tests carried out on the Volclay sample have shown that the primary swelling stage is completed after 31 days, regardless of the normal stress intensity. After primary swelling, the stage of secondary compression and creep develops. The extent of swelling and secondary compression depends on the normal stress levels.

The analysis of shear strength parameters of specimens hydrated for 7, 14 and 21 days shows decreasing cohesion with extended times of hydration. When hydration time is longer than 14 days, the intensity of change in cohesion decreases. The intensity of change in residual cohesion in the first 14 days of hydration is higher than for peak values, presumably as a result of increasing pore pressure during the shear stage.

With extended hydration time (from 7 to 14 days), the friction angle initially increases, but further extension of hydration (to more than 14 days) produces almost no change in the friction angle value. Peak and residual friction angles change in line with this pattern, but the extent is somewhat smaller for the residual friction angle.

Knowing the process of swelling of bentonite is important from the aspect of shear strength of bentonite and clay geosynthetic barriers. Also, the displacement rate influences peak and residual values of bentonite shear strength. Lower displacement rate means a longer test duration overall, and consequently more time for hydration/swelling in case of normal stress levels that are smaller than the bentonite swelling pressure. Potential continuation of the hydration process in the shearing stage will depend on the stage of swelling that the specimen is in. If the time required to finalize primary swelling has been reached, it can be assumed that hydration of the specimen would end, and that after this the specimen would enter the stage of secondary compression.

5 ACKNOWLEDGMENTS

This paper was produced with financial support from the Croatian Ministry of Science, Education and Sports, through a project "Long-term performance of mineral barriers"; 195-0831529-1847.

6 REFERENCES

- Mitchell J. K. and Soga K. (2005): Fundamentals of Soil Behavior, John Wiley and Sons, New York
- Shackelford, C. D., Benson, C. H., Katsumi, T., Edil, T. B. and Lin, L. (2000): Evaluating the hydraulic conductivity of GCLs permeated with non-standard liquids, *Geotextiles and Geomembranes*, 18 (2–4), 133–162.
- Shackelford, C. D., Sevik, G. W. and Eykhol, G. R. (2010): Evaluating the hydraulic conductivity of GCLs permeated with non-standard liquids, *Geotextiles and Geomembranes*, 18, 133–162.
- Zornberg, J. G. and McCartney, J. S. (2009): Chapter 8: Internal and interface shear strength of geosynthetic clay liners, In: *GCLs for Waste Containment*, Bouazza, A. & Bowders, J.J. (Ed), CRC Press.
- Kovačević Zelić, B. (2000): Ispitivanje posmične čvrstoće bentonitnih tepiha, *Doktorska disertacija*, RGN-fakultet, Zagreb.
- Guyonnet, D., Touze Foltz, N., Norotte, V., Pothier, C., Didier, G., Gailhanou, H., Blanc, P. and Warmont, F. (2009): Performance-based indicators for controlling geosynthetic clay liners in landfill applications; *Geotextiles and Geomembranes* 27, 321–331.
- Fox, P. J. (2010): Internal and interface shear strengths of geosynthetic clay liners, 3rd Int. Symp. on GCLs, 15-19.9.2010, Germany, Zanzinger, H., Koerner, R. M. & Touze-Foltz, N. (Eds.), 203–218.
- Gilbert, R. B., Fernandez, F. F. and Horsfield, D. (1996): Shear strength of a reinforced clay liner, *Journsl of Geot. and Geoenv. Eng.*, 122, 259–266.
- Bouazza, A. (2002): *Geosynthetic Clay Liners*, Geotextiles and Geomembranes 20, Elsevier Science.

Reappraisal of Surcharging to Reduce Secondary Compression

Remise en cause de l'imposition de frais supplémentaires pour réduire la Compression secondaire

Feng T.W.

Department of Civil Engineering, Chung Yuan Christian University, Taiwan, R. O. C.

ABSTRACT: For structures on soft soils with relatively high secondary compression characteristics, post-construction settlement may be excessive so that application of surcharging may be called for to reduce secondary compression. The principle of surcharging is simple, but the post-surge secondary compression behavior requires further study. This paper looks into effects of surcharging loading history on post-surge secondary compression characteristics of soils. Current knowledge on this subject is primarily based on unloading from surcharge stress to permanent stress. There could be other types of surcharging stress history. For example, all preloads, including permanent load and surcharge load, are removed before the permanent load is reapplied. Laboratory data of secondary compression with different surcharging stress history are presented and discussed. These data provides useful information for understanding the post-surge secondary compression behavior of soils.

RÉSUMÉ : Pour les structures sur des sols mous avec des caractéristiques de compression secondaire relativement élevé, règlement après la construction peut être excessive pour que l'application d'imposer des frais supplémentaires peut-être être appelée pour réduire la compression secondaire. Le principe de l'imposition de frais supplémentaires est simple, mais le comportement de compression secondaire post-surge nécessite une étude plus approfondie. Cet article se penche sur les effets de surcharge chargement histoire sur les caractéristiques de compression secondaire post-surge des sols. Les connaissances actuelles sur ce sujet repose essentiellement sur le déchargement de stress en supplément à un stress permanent. Il pourrait y avoir d'autres genres d'imposer des frais supplémentaires histoire des contraintes. Par exemple, toutes les précharges, y compris la charge permanente et la charge de la surtaxe, sont supprimés avant que la charge permanente est réappliquée. Données de laboratoire de compression secondaire avec différents histoire surcharge de stress sont présentées et discutées. Ces données fournit des informations utiles pour comprendre le comportement de compression secondaire post-surge des sols.

KEYWORDS: Surcharging, precompression, soft soils, secondary compression.

1 INTRODUCTION

Surcharging is a special condition of precompression or preloading technique (e.g. Jamiolkowski et al. 1983, Johnson 1970) for soft ground improvement. The philosophy of surcharging is quite simple that a state of overconsolidation in soil is generated after surcharging executions. It is expected that post-construction settlements of structures are thus reduced. Although, there are some unsuccessful surcharging applications (e.g. Chang 1981, Sower 1964) around the world in which surcharge loads are removed too early during primary consolidation so that post-construction settlements resulting from both primary consolidation and secondary compression are still excessive. It is apparent that the philosophy of surcharging is not followed in these cases. To bring the soil to a state of overconsolidation, primary consolidation should have been eliminated completely at the time of surcharge removal. The other situation is that the part of settlement resulting from secondary compression of soil is not usually considered in practice. It should be emphasized that the magnitude of secondary compression settlement may still be significant if conditions allows it to occur. Equation (1) can be used for calculation of secondary compression settlement:

$$s_a = [(C_a/C_c) \times C_c / (1 + e_0)] H_0 \log(t/t_p) \quad (1)$$

Where C_a is the secondary compression index, C_c is the compression index, e_0 is the initial void ratio, H_0 is the initial

thickness of soil layer, t_p is the time of end of primary consolidation, and t is any time during secondary compression. A sample calculation can be made with Eq. (1) for a normally consolidated soil layer of 10 m thick doubly drained from both top and bottom, with $C_a/C_c = 0.05$, $C_c = 1.0$, $e_0 = 1.2$, coefficient of consolidation $c_v = 0.001 \text{ cm}^2/\text{sec}$. The time of end of primary consolidation t_p is then estimated as 9 years and the secondary compression settlement at consolidation time of 20 years is calculated using Eq. (1) as 0.08 m. If vertical drains were used to speed up primary consolidation, the time of end of primary consolidation would be greatly shortened. It is assumed for the sample calculation that t_p is reduced to 6 months. Then the secondary compression settlement at consolidation time also of 20 years is calculated again using Eq. (1) as 0.36 m. The above sample calculation demonstrates that shortening the time of end of primary consolidation has an undesirable effect on the magnitude of secondary compression settlement. Secondary compression of natural soils has been observed both in the field and in the laboratory. As can be seen from Eq. (1), the magnitude of secondary compression settlement is in direct proportion to secondary compression index, thickness of soil layer, and consolidation time considered for calculation. In general, structures on thick soft soils with high natural water contents are highly susceptible to excessive secondary compression settlement. It is easier to estimate secondary compression settlement during design than to face unacceptable post-construction long term settlement problems.

When it is determined during the design stage that secondary compression settlement is excessive or detrimental, surcharging technique can be considered for ground improvement to eliminate part of the undesirable secondary compression settlement. Surcharging application involves using a temporary surcharge load in excess of the permanent load. A question can then be raised as to how much surcharge load is sufficient. Knowledge about relationship between the magnitude of surcharge load and the post-surge secondary compression characteristics are required to answer this question. As discussed earlier, the surcharge load should be removed at the end of primary consolidation or without leaving excess pore water in the consolidating soil layer. This renders the soil an overconsolidation state after rebounding with time. The more the surcharge load is removed, the higher the overconsolidation is induced in soil and the longer the time is spent to reach the end of rebound. Furthermore, it has been clearly observed in the laboratory that secondary compression may reappear after the end of rebound. The magnitude of surcharge removal affects both the time of end of rebound and magnitude of post-surge secondary compression (Mesri and Feng 1991). Extensive laboratory data of post-surge secondary compression of natural soils, including soft clays, silts, and peats, show that the magnitude of post-surge secondary compression decreases with increasing the effective surcharge ratio (Mesri and Feng 1991, Mesri et al. 1997, Mesri et al. 2001). When the surcharge load is removed at the end of primary consolidation, the effective surcharge ratio is defined by Eq. (2):

$$R'_s = (\sigma'_s - \sigma'_f) / \sigma'_f \quad (2)$$

where σ'_s is the effective stress achieved at the end of surcharging and σ'_f is the final effective stress. These data also show that post-surge secondary compression always appears when the effective surcharge ratio is less than 1.0, is noticeable for effective surcharge ratios ranging from 0.5 to 1.0, and becomes important for effective surcharge ratios between 0.1 and 0.5. It is the last case in which post-surge secondary compression settlement may be excessive. Figure 1 shows consolidation curves obtained from laboratory oedometric surcharging tests, on a thin-wall tube sample from a lacustrine clay deposit in Taipei, Taiwan, with a final effective stress of 356 kPa and an effective surcharge ratio of 0.23. It is clearly seen from Fig. 1 that secondary compression reappears after surcharge removal and the slope of the post-surge secondary compression curve increases gradually up to a maximum value, which is about the same as that of the later portion of the $R'_s = 0$ curve.

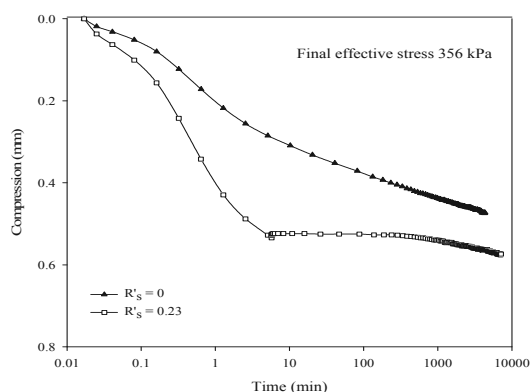


Figure 1. Consolidation curves of Taipei lacustrine clay sample with or without surcharging

In fact, the execution of surcharging from overburden pressure to final effective stress could have a number of different stress histories, as shown in Fig. 2(a)~2(d). Figure 2(a) shows that the consolidation pressure is increased from the overburden pressure to the surcharging stress and then unloaded to the final effective stress. This type of stress history was commonly adopted in most laboratory surcharging tests. Figure 2(b) shows that the consolidation pressure is increased from the overburden pressure to the surcharging stress and then unloaded to the overburden pressure before reloading to the final effective stress. This type of stress history depicts a practical situation in which all preloads, including surcharge load, are removed for the construction of permanent structures. Figure 2(c) shows that the consolidation pressure is increased from the overburden pressure to the final effective stress without surcharging and then unloaded to the overburden pressure before reloading to the final effective stress. This type of stress history depicts a practical situation in which all preloads, without surcharge load, are removed for the construction of permanent structures. Figure 2(d) shows that the consolidation pressure is increased from the overburden pressure to the final effective stress without unloading. This type of stress history represents a situation that the permanent structure is constructed without employing precompression technique to reduce secondary compression. It is noted again that these four types of stress histories are different but all ended at the same final effective stress. A series of laboratory oedometric surcharging tests with these stress histories are conducted to study the effects, if any, of consolidation stress history on secondary compression. Test results are presented and discussed in this paper.

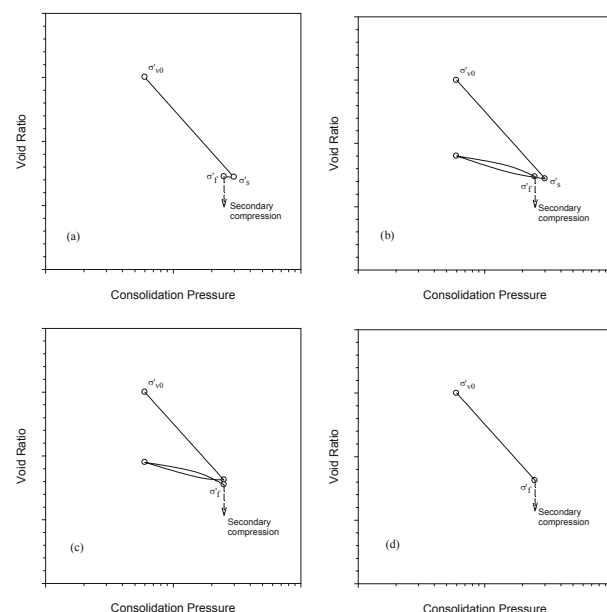


Figure 2. Illustrated surcharging stress histories adopted in this study: (a) unloading from surcharge stress directly to final effective stress, (b) unloading from surcharge stress to overburden pressure before reloading to final effective stress, (c) unloading from final effective stress to overburden pressure before reloading to final effective stress, and (d) without preloading.

2 LABORATORY SURCHARGING TESTS

2.1 Test program

The surcharging tests are conducted using conventional oedometer set up with a data acquisition system for taking LVDT (Linear Variable Differential Transformer) readings of specimen compression. The soil tested is a native organic clay

sample from Nantou in central Taiwan. The liquid limit and plastic limit of the soil sample are 106% and 44%, respectively. The soil sample is reconstituted in the laboratory with a water content of 155% which is about 1.5 times of the liquid limit. A 200-mm diameter consolidometer is used to prepare identical soil specimens for the surcharging tests. Five oedometer rings are placed with proper spacing in between in the consolidometer and then the consolidometer is filled up with the 155% water content soil sample. The soil in the consolidometer is incrementally loaded to 50 kPa and then unloaded. By this way, five identical oedometer specimens are obtained with a preconsolidation pressure of 50 kPa. These specimens have initial water contents around $88\% \pm 1\%$. Four specimens are used to conduct oedometer tests with different consolidation stress histories illustrated in Figs. 2–5. Specimen #5 is tested according to the stress history described in Fig. 2(a) but is unloaded prior to the end of primary consolidation. Secondary compression characteristics of natural soils can be expressed in terms of the compressibility ratio C_α/C_c (Terzaghi et al. 1996), where C_α is the secondary compression index and C_c is the compression index. The compressibility ratio and compression index of the organic clay tested are 0.063 and 0.7, respectively. The secondary compression of the laboratory specimens tested with or without surcharging is observed at two different final effective stresses, i.e. 109.6 kPa and 438.4 kPa, for a period that is about two times of the time of end of primary consolidation. Effective surcharge ratios of 0.25 and of 0.125 are used for final effective stresses of 109.6 kPa and of 438.4 kPa, respectively. Significant surcharging test results are presented and discussed herewith.

2.2 Test results and discussions

Effect of surcharging on secondary compression, if any, can be directly observed from the consolidation curves with or without surcharging as shown in Fig. 3 and Fig. 4. In these two figures, curve A is the consolidation curve without surcharging, curve B is the consolidation curve with surcharging by the type described in Fig. 2(a), curve C is the consolidation curve with surcharging by the type described in Fig. 2(b), curve D is the consolidation curve with preloading by the type described in Fig. 2(c), and curve E is the consolidation curve with surcharging by the type described in Fig. 2(a) but the specimen is unloaded prior to the end of primary consolidation.

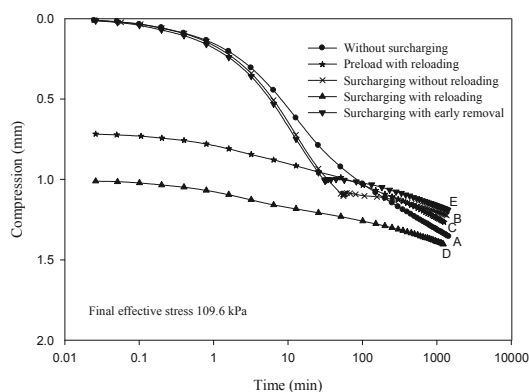


Figure 3. Consolidation curves of different surcharging stress histories with effective surcharge ratio equal to 0.25

It can be seen from Fig. 3 that the slopes of the secondary compression portion of curves B–D are smaller than that of curve A. These data indicate that surcharging or preloading of the types used has reduced the magnitude of secondary compression to somewhat different degrees. Curve E shows

about the same magnitude of secondary compression as that of curve A, which can be expected since the specimen is unloaded too early to receive a beneficial effect from surcharging. Similar test results are shown in Fig. 4 for higher final effective stress and smaller effective surcharge ratio than those shown in Fig. 3. However, it appears from Fig. 4 that the slopes of the later secondary compression portion of all five curves are very close to each other. This is probably partly because the effective surcharge ratio of 0.125 used is rather small so that the reduction in secondary compression is highly limited. These findings suggest that the magnitude of post-surge secondary compression is a function of the consolidation stress history imposed on the soft soil. For small effective surcharge ratios achieved in engineering practice, the magnitude of secondary compression may not be reduced to an acceptable level so that long term secondary compression settlement may still be excessive.

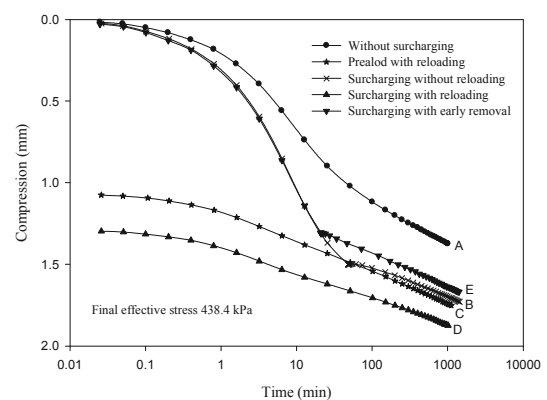


Figure 4. Consolidation curves of different surcharging stress histories with effective surcharge ratio equal to 0.125

The post-surge secondary compression characteristics of the Taipei clay are quite different from those of the Nantou clay. Figure 5 shows consolidation curves of these two clays with or without surcharging, all in the normally consolidated range. The load increment ratio used for the increments without surcharging is 1. The loading increment ratio used for the increments with surcharging is higher than 1 since the surcharge load is added with the final load at the same time. The surcharge load is removed at a time near the end of primary consolidation. During the primary consolidation stage, the Nantou clay specimen consolidates much slower and compresses more than those of the Taipei clay specimen. The compressibility ratio and compression index of the Taipei clay are 0.039 and 0.3, respectively, which are much smaller than those of the Nantou clay. It can be seen from Fig. 5 that, after surcharge removal, the Taipei clay specimen rebounds slightly and continuously for a relatively long period of time before secondary compression appears. As a comparison, the Nantou clay specimen rebounds very slightly and shortly before secondary compression appears. The effective surcharge ratios used in these tests are about the same and do reduce the secondary compression to a certain degree. But it appears that characteristics of post-surge secondary compression vary from one soil to another soil.

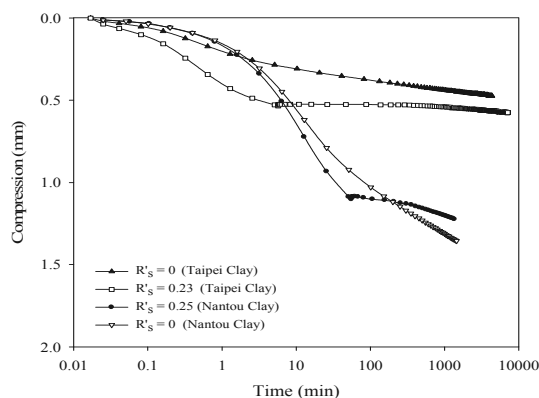


Figure 5. Consolidation curves of different soils with or without surcharging

3 CONCLUSIONS

The following conclusions are based on data and discussions presented in previous paragraphs. Surcharging with higher effective surcharge ratio reduces secondary compression to a lower magnitude. For the same effective surcharge ratio, surcharging without reloading reduces more secondary compression than surcharging with reloading. The post-surge secondary compression is also a function of the surcharging stress history achieved in the consolidating soil. For small effective surcharge ratios realized in practice, the secondary compression of soil may not be reduced sufficiently and excessive long term settlement resulting from secondary compression may still be expected. Structures on natural thick soil deposit with high compressibility ratio, e.g. greater than 0.04, and improved by prefabricated vertical drains are highly susceptible to excessive long term secondary settlement. In this case, surcharging technique is capable of reducing the secondary settlement, as long as a proper value of effective surcharge ratio is used in design and achieved in soil.

4 ACKNOWLEDGEMENTS

This study is partly supported by the National Science Council of Taiwan, under Grant No. NSC 98-2221-E-033-052-MY2, and by Chung Yuan Christian University. Great thanks are given to these organizations for their continuous support.

5 REFERENCES (TNR 8)

- Chang Y.C.E. 1981. Long term consolidation beneath the test fills at Vasby, Sweden. *Swedish Geotechnical Institute*, Report 13, Linköping, Sweden.
- Jamiolkowski M., Lancellotta R. and Wolski W. 1983. Precompression and speeding up consolidation. *Proc. 8th ECSMFE*, 1201-1206.
- Johnson S.J. 1970. Precompression for improving foundation soils. *Journal of the Soil Mechanics and Foundations Division*, ASCE, 1, 111-114.
- Mesri G., Ajlouni M.A., Feng T.W. and Lo, D.O.K. 2001. Surcharging of soft ground to reduce secondary settlement. *Proc. of the 3rd International Conference on Soft Soil Engineering*, Swets & Zeitlinger, Hong Kong, 55-65.
- Mesri G. and Feng T.W. 1991. Surcharging to reduce secondary Settlement. *Proceedings of the international conference for coastal development -Theory to Practice*, 1, Yokohama, Japan, 359-363.
- Mesri G., Stark T.D., Ajlouni, M.A. and Chen C.S. 1997. Secondary compression of peats with or without surcharging. *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 123 (5), 411-421.
- Sowers G.F. 1964. Fill settlement despite vertical sand drains. *Journal of the Soil Mechanics and Foundation Division*, ASCE, 90 (SM5), 289-302.
- Terzaghi K., Peck R.B. and Mesri G. 1996. *Soil Mechanics in Engineering Practice*, 3rd ed., John Wiley & Sons, Inc., 549.

Effets d'échelle dus à la rupture des grains sur la résistance au cisaillement d'enrochements

Size effects due to grain crushing in rockfill shear strength

Frossard E.

Tractebel Engineering - Coyne et Bellier, France

Ovalle C.^{1,2}, Dano C.¹, Hicher P-Y.¹

¹*GeM, Ecole Centrale Nantes, CNRS UM 6183, Nantes, France*

²*Department of Structural and Geotechnical Engineering, Pontificia Universidad Católica de Chile, Chile*

Maiolino S.

CETE de Lyon, France

Hu W.

State Key Laboratory of Geohazard Prevention and Geoenvironment Protection, Chengdu U. of Technology, China

RÉSUMÉ: Des preuves expérimentales de l'effet d'échelle sur la résistance au cisaillement de petits enrochements et son lien avec la résistance à l'écrasement des granulats de roches sont présentées et analysées. Les relations d'effet d'échelle qui décrivent ce lien, portent une application pratique directe : une nouvelle méthode pour l'évaluation de la résistance au cisaillement d'enrochements.

ABSTRACT: Experimental evidence of size effects in rockfill shear strength and the link with the crushing strength of rock aggregates is presented and analyzed. The scale effect relations describing this link bear a direct practical application: a new method for the evaluation of rockfill shear strength.

MOTS-CLÉS: Effet d'échelle, Ruptures des grains, Enrochement, Grande cellule triaxiale, Weibull.

KEYWORDS: Size effect, Grain crushing, Rockfill, Shear strength, Large triaxial, Weibull.

1 INTRODUCTION

L'étude de la résistance au cisaillement des matériaux granulaires grossiers, largement utilisés dans le génie civil, présente des défis expérimentaux et théoriques considérables. Une méthode originale pour résoudre ce problème réside dans l'utilisation des effets d'échelle issus de la mécanique de la rupture des grains. Ainsi, pour tout matériau fragile existe un effet d'échelle sur la résistance à l'écrasement des granulats eux-mêmes (McDowell & Amon, 2000). Cela signifie que la contrainte à la rupture est fonction décroissante d'une dimension caractéristique de l'échantillon. L'explication physique en a été donnée par Griffith (1921), qui a montré que la probabilité de trouver des défauts internes d'une taille donnée augmente avec la taille de l'échantillon. Ce phénomène est à l'origine des effets d'échelle sur la résistance au cisaillement des assemblages granulaires. Ainsi, lorsque la taille des particules augmente, il y a (1) une augmentation du taux de rupture des grains, (2) une augmentation de la contractance volumique et (3) une diminution de l'angle de frottement au pic (Marachi et al., 1969).

L'objectif principal du présent article est de présenter des preuves expérimentales de la relation entre l'effet d'échelle sur la résistance à la rupture de particules individuelles, et l'effet d'échelle sur la résistance au cisaillement de l'assemblage granulaire, ainsi que de la formulation théorique exprimant cette relation.

2 EFFETS D'ECHELLE DANS LA RESISTANCE A LA RUPTURE DE GRANULATS DE ROCHE

La théorie statistique de la résistance des matériaux de Weibull (1939) est basée sur le concept du maillon faible (WLC). Elle décrit la probabilité de survie d'un matériau de taille d soumis à une contrainte de traction σ par:

$$P_s(d) = \exp \left(- \left(\frac{d}{d_o} \right)^{n_d} \cdot \left(\frac{\sigma}{\sigma_o} \right)^m \right) \quad (1)$$

où σ_o est la résistance caractéristique ($P_s = 37\%$ pour un échantillon de taille d_o) et m donne la dispersion des données. Le concept de maillon faible est basé sur l'idée que, pour qu'une chaîne de longueur d survive sous une contrainte uniaxiale, chaque élément de longueur d_o doit aussi survivre. Le terme $(d/d_o)^{n_d}$ est le rapport d'échelle indiquant combien de liens forment la chaîne. D'après Bazant et Planas (1998), n_d est la dimension de similitude géométrique du problème mécanique: $n_d = 1, 2$ ou 3 , pour une similarité linéaire, surfacique ou volumique, respectivement. Pour une probabilité de survie donnée et des paramètres empiriques connus, une relation d'effet d'échelle est obtenue entre la contrainte de traction induite à la rupture (σ_f) et la taille caractéristique de l'échantillon (d): $\sigma_f \propto d^{-n_d/m}$. Pour les granulats, la taille caractéristique est un diamètre des particules d . Pour les applications pratiques, n_d est simplement calé sur des données expérimentales, afin de pouvoir utiliser l'Eq. 1 comme un outil statistique phénoménologique.

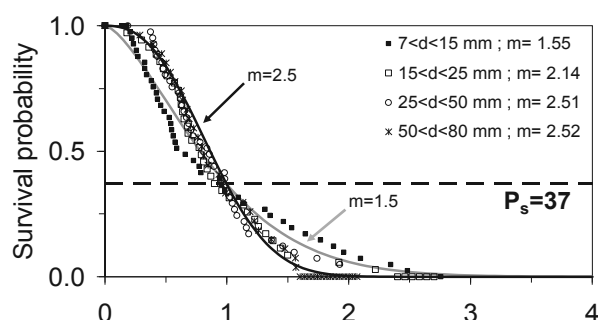


Figure 1. Distributions de Weibull pour les granulats CP.

144 essais d'écrasement des particules, de diamètres compris entre 7 à 80 mm, ont été réalisés sur un enrochement calcaire (CP) provenant d'une carrière située à Préfontaines, France (Hu et al., 2011). Les essais ont été effectués entre deux plaques rigides parallèles à l'aide d'une presse de 10 kN et une vitesse de chargement de 0,1 mm/min. Les particules sont posées sur le plateau inférieur fixe, selon leur plus grande dimension afin d'assurer leur stabilité. Les résultats ont été analysés statistiquement en distinguant chaque fraction de taille. La résistance à la traction induite est considérée comme

$\sigma_f \propto f_f / d^2$ (où f_f est la force d'écrasement). La figure 1 montre (i) les valeurs de m pour chaque fraction granulaire, avec une valeur moyenne $m_{avg}=2,18$, et (ii) les probabilités de survie avec un très bon accord avec la distribution de Weibull.

La même analyse statistique a été faite à partir de 100 essais d'écrasement sur un enrochement de schiste quartzitique (STV) de la carrière des Trois Vallées, France. Le module moyen est $m_{avg}=1,57$ et on a eu également un bon ajustement de la distribution de Weibull. La figure 2 montre la résistance à l'écrasement pour l'ensemble des essais sur CP et STV, la résistance caractéristique σ_o et la prédiction obtenue à partir de l'Eq. 1 ($\sigma(d)=\sigma_o(d/d_o)^{nd/m}$; avec $n_d=1$).

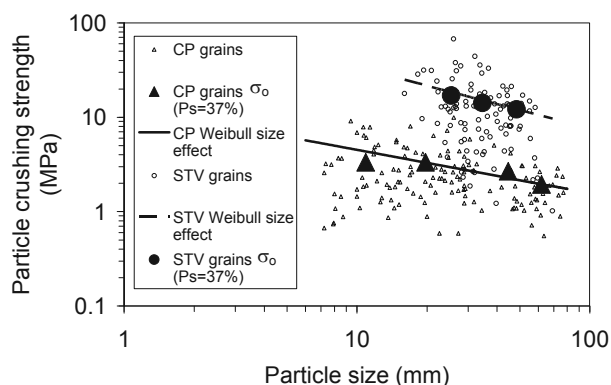
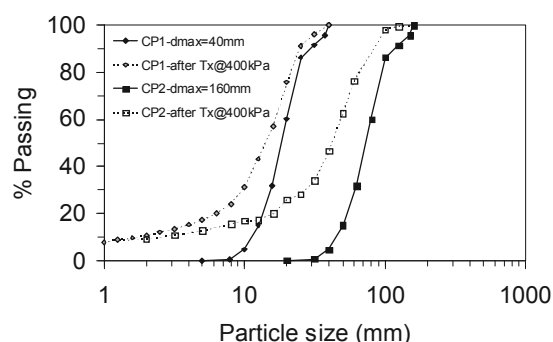
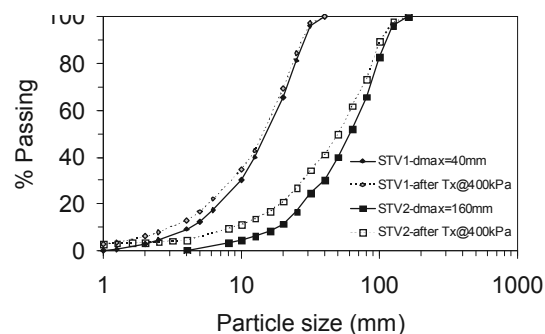


Figure 2. Résistance à l'écrasement des grains et prédictions de Weibull.



(a)

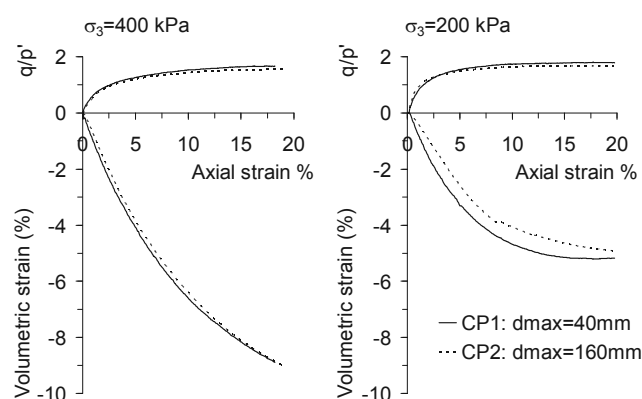


(b)

Figure 3. Granulométries avant et après essais triaxiaux.

3 ESSAIS TRIAXIAUX

Pour chaque enrochement (CP et STV), on a effectué une série d'essais triaxiaux drainés sur du matériau sec en utilisant deux granulométries homothétiques avec un diamètre maximal d_{max} de 40 et 160 mm respectivement (Figure 3), compactés à la même densité. Les matériaux de $d_{max}=40$ mm sont appelés CP1 et STV1 et ceux de $d_{max}=160$ mm CP2 et STV2. On suppose que la forme des particules et la minéralogie sont préservées entre les deux matériaux homothétiques, parce que chacun a été obtenu par broyage de la même roche-mère en carrière. Les essais sont conduits sur deux grandes cellules triaxiales. Pour $d_{max}=40$ mm, les échantillons ont un diamètre de 250 mm, une hauteur de 375 mm et sont protégés par deux membranes en néoprène de 1 mm d'épaisseur. Pour $d_{max}=160$ mm, les échantillons font 1 000 mm de diamètre, 1 500 mm de hauteur et sont protégés par deux membranes en PVC de 1,5 mm d'épaisseur et une couche de géotextile. La plus petite des deux cellules (échantillon de 250 mm) a été conçue pour être placée sous un vérin de 500 kN. Pour les essais sur du matériau sec, les variations de volume sont mesurées à partir du volume de fluide de confinement échangé entre la cellule et le contrôleur pression-volume.



(a)

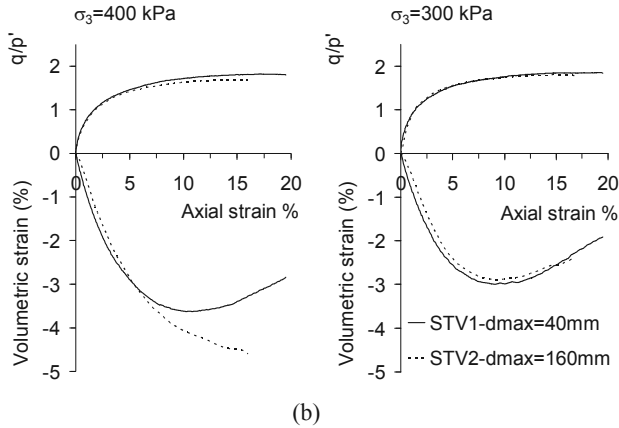


Figure 4. Courbes de contrainte-déformation pour (a) CP and (b) STV

La grande cellule triaxiale (1 000mm) a été conçue pour supporter des charges verticales jusqu'à 2 000 kN. Le système est composé d'une base en acier rigide fixée sur une dalle en béton précontrainte. La cellule peut supporter une pression de confinement de 1,5 MPa. L'enveloppe cylindrique de la cellule est constituée d'une double paroi. Un système de contrôle-commande en continu maintient la même pression dans les deux chambres, en permettant les échanges à l'aide d'une servovalve automatique. Par conséquent, le cylindre intérieur reste indéformable pendant l'essai et la variation de volume d'eau dans le compartiment intérieur est uniquement due à la déformation volumique de l'échantillon et à la pénétration du piston. Une description exhaustive des appareils triaxiaux se trouve dans Hu et al. (2011).

Les échantillons sont préparés par couches successives, compactées par vibrations à l'aide d'une aiguille vibrante. Les indices des vides initiaux sont de $0,854 \pm 0,03$ pour CP et de $0,493 \pm 0,03$ pour STV. La Figure 4 montre la réponse mécanique lors des essais de cisaillement drainés à une contrainte effective de confinement σ'_3 entre 200 et 400 kPa. La Figure 3 montre les granulométries associées avant et après les essais à $\sigma'_3=400$ kPa. Sur les Figures 3 et 5, on voit qu'il existe un effet d'échelle sur la résistance mécanique des deux enrochements, avec un taux de rupture des grains plus élevé (Figure 3) et un angle de frottement au pic inférieur (Figure 5) pour les matériaux plus grossiers. Ceci est cohérent avec l'effet d'échelle sur la résistance à l'écrasement des particules (voir Figure 2). Néanmoins, nous n'avons pas observé un effet d'échelle significatif dans la réponse volumique (Figure 4). Marachi et al. (1969) montrent des résultats similaires lorsqu'ils comparent des enrochements de granulométries homothétiques. Une explication possible de cette évolution particulière de la déformation volumique pourrait être le rapport relativement élevé entre d_{max} et le diamètre de l'échantillon (environ 1/6). De même, les essais sont conduits jusqu'à une déformation axiale de 20% : les particules grossières pourraient ne pas se déplacer suffisamment pour se démêler après la rupture et ainsi permettre plus de contractance. En fait, comme le montre la Figure 6, nous avons observé, après essai, plusieurs grosses particules cassées qui semblent être intactes sans déplacement des fragments brisés.

4 METHODE PREDICTIVE

En considérant que les ruptures individuelles de granulats affectent la résistance au cisaillement de l'ensemble du milieu granulaire, Frossard (2009) (voir aussi Frossard et al., 2012) a proposé une méthode rationnelle afin de prédire les effets d'échelle sur les enveloppes de résistance au cisaillement des enrochements. Il a considéré deux granulométries homothétiques (G_1 la plus fine et G_2 la plus grossière) sous le

même type de chargement, avec la même compacité, la même minéralogie et la même forme initiale des grains, et cherché quelles conditions assuraient un même taux de ruptures dans les deux milieux granulaires sous contraintes. Pour cela, une même probabilité de survie doit être assurée pour deux granulats homologues de G_1 et G_2 , et l'on dispose de la relation suivante pour la résistance à l'écrasement: $\sigma_{G2} = \sigma_{G1} (d_2/d_1)^{-nd/m}$ (d'après l'Eq.1) où σ_{G1} et σ_{G2} sont les résistances à l'écrasement des granulats de G_1 et G_2 , respectivement.

Frossard (2009) a ensuite montré que pour obtenir le même taux de rupture dans les matériaux granulaires sous cisaillement, la relation précédente doit être vérifiée par les états de contrainte $\{\tau, \sigma_n\}$ des enveloppes de cisaillement des deux assemblages granulaires homologues:

$$\tau_{G2} = \tau_{G1} \left(\frac{d_2}{d_1} \right)^{-n_d/m} \quad \text{et} \quad \sigma_{n-G2} = \sigma_{n-G1} \left(\frac{d_2}{d_1} \right)^{-n_d/m} \quad (2)$$

En faisant l'hypothèse, pour le matériau G_1 avec des paramètres calés A_{G1} et b_{G1} , d'une enveloppe de rupture telle que celle proposée par De Mello (1977) [$\tau = A \cdot \sigma_n^b$], une enveloppe de rupture peut être prédite pour G_2 en utilisant l'Eq.2 :

$$\tau_{G2} = A_{G1} \cdot \left(\frac{d_2}{d_1} \right)^{-\frac{n_d(1-b_{G1})}{m}} \cdot (\sigma_{n-G2})^{b_{G1}} \quad (3)$$

On remarque que dans l'Eq.2, l'effet d'échelle affecte le coefficient A , mais pas l'exposant b .

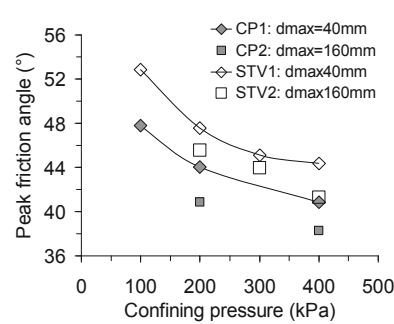


Figure 5. Angles de frottement au pic



Figure 6. Gros granulats de STV récupérés après essai triaxial.

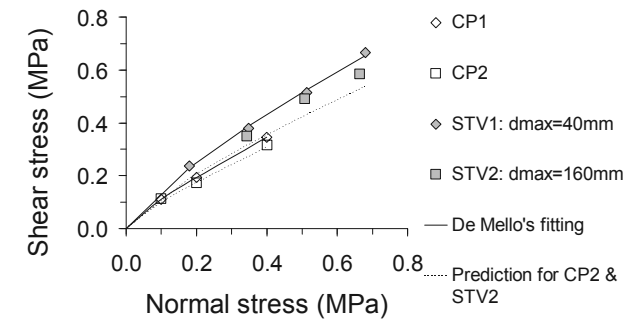


Figure 7. Effet d'échelle sur la résistance au cisaillement

La Figure 7 montre la comparaison entre mesures et prédictions pour les enveloppes de cisaillement pour les deux matériaux CP et STV : en partant du meilleur calage de l'expression de De Mello sur les mesure réalisées sur CP1 et STV1 ($d_{max}=40mm$), on a réalisé la prédiction issue de l'Eq. 2 pour CP2 et STV2

($d_{max}=160$ mm). En comparant aux points de mesures effectuées par ailleurs sur CP2 et STV2, on constate que la méthode de Frossard (2009) prédit correctement les effets d'échelle dans les deux cas, même si elle reste conservatrice dans le cas de STV2.

5 CONCLUSIONS

L'effet d'échelle sur la résistance à l'écrasement de granulats rocheux est responsable d'effets d'échelle sur le taux de rupture dans les assemblages d'enrochements, et de là sur la résistance au cisaillement d'ensemble des milieux granulaires correspondants. Même si l'on utilise de très grands appareils de cisaillement au laboratoire, l'échantillon sera toujours réduit d'un ordre de grandeur, ou plus, par rapport aux enrochements de barrage. Dans ce cas, une méthode rationnelle de prédiction devient nécessaire.

Les grandes lignes d'une telle méthode sont établies dans ce travail et largement présentées dans Frossard (2012). Cette référence détaille aussi par ailleurs les incidences sur la stabilité d'ouvrages en enrochements de cet effet d'échelle matériel sur la résistance au cisaillement, et de là les incidences sur la conception des ouvrages en enrochements.

6 REMERCIEMENTS

Ce travail fait partie du projet de recherche ECHO (Effets d'Echelle dans les Ouvrages en Remblais du Génie Civil), financé par l'Agence Nationale Française de la Recherche (ANR).

7 REFERENCES

- Bažant, Z.P., Planas, J. (1998): Fracture and size effect in concrete and other quasibrittle materials. CRC Press, USA.
- De Mello, V.F.B. (1977). Reflections on design decisions of practical significance to embankment dams. *Géotechnique*, Vol. 27, No.3, pp. 281-355
- Frossard, E. (2009): Scale effects in granular fill shear strength and in stability of large rockfill structures. *Proc. XVIIth ICSMFE*, Vol 1, 97-101, Alexandria, Egypt, 5-9 Oct., 2009
- Frossard, E., Hu, W., Dano, C., Hicher, P.Y. (2012): Rockfill shear strength evaluation: a rational method based on size effects. *Géotechnique* 62(5), 415-428
- Frossard, E. (2012): Granular materials in Civil Engineering: recent advances in the physics of their mechanical behavior, and applications to engineering works. *in Multiscale Geomechanics- From soil to Engineering Projects- Ed. by P.-Y. Hicher*, ISTE/Wiley, 35-81, Jan 2012, ISBN 978-1-84821 – 246-6
- Griffith, A.A. (1921): The phenomena of rupture and flow in solids. *Phil. Trans. R. Soc. Lond. A* 221, 163-198
- Hu, W., Dano, C., Hicher, P.-Y., Le Touzo, J.-Y., Derks, F., Merliot, E. (2011): Effect of sample size on the behavior of granular materials. *Geotech. Test. J.* 34(3), 186-197
- Marachi, N.D., Chan, C.K., Seed, H.B., Duncan, J.M. (1969): Strength and deformation characteristics of rockfills materials. Report No. TE-69-5, Dept. of Civil Engineering, Univ. of California, Berkeley.
- McDowell, G. R., Amon, A.: (2000) The application of Weibull statistics to the fracture of soil particles. *Soils Found.* 40(5), 133–141
- Weibull, W. (1939): A statistical theory of the strength of materials. *Proc. Roy. Swedish Inst. Eng. Res.* 151

Electro-osmotic consolidation: Laboratory tests and numerical simulation

Électro-osmotique de consolidation : les tests de laboratoire et simulation numérique

Hu L., Wu H., Wen Q.

*State Key Laboratory of Hydro-Science and Engineering, Department of Hydraulic Engineering,
Tsinghua University, Beijing, 100084 CHINA*

ABSTRACT: Electro-osmotic consolidation is an attractive soil improvement technique for soft clay. A review of research and practical applications of electro-osmosis was performed. The achievements and major bottleneck of the current technique were briefly discussed, and the potential areas for scientific research and engineering application were proposed. A laboratory testing facility was developed for the electro-osmotic consolidation, and the electrical voltage, soil mass displacement, water discharge and electrical current can be monitored during testing process by use of online transducers. Moreover, the microscopic phenomenon is investigated by use of Environmental Scanning Electron Microscopy (ESEM). A theoretical model for electro-osmotic consolidation is briefly introduced, which couples Biot's consolidation equation with the electro-osmotic flow and control equation for an electrical field, and incorporates the nonlinear variation of the mechanical, hydraulic and electrical properties. The FEM software was developed based on the theoretical model to describe the electro-osmotic consolidation process, and the model tests are simulated. The numerical results showed good agreement with the testing data in terms of ground settlement, indicating the rationality of the analytical model. It is also shown that the variations of electrical conductivity have a significant effect on the consolidation process. The developed software can predict the displacement behavior of soil mass and provide useful data for system design of electro-osmosis treatment.

RÉSUMÉ : La consolidation électro-osmotique est une technique attrayante d'amélioration de l'argile. Une révision des demandes de recherche et de pratique de l'électro-osmose a été effectuée. Les réussites et le goulot d'étranglement majeur de la technique actuelle ont été brièvement discutés, et les domaines potentiels de la recherche scientifique et l'application d'ingénierie ont été proposés. Un centre d'essais en laboratoire a été développé pour la consolidation électro-osmotique, et la tension électrique, le déplacement massif de sol, l'évacuation de l'eau et le courant électrique peuvent être contrôlés pendant le processus d'essai en utilisant des capteurs en ligne. En outre, le mécanisme microscopique de la migration de l'eau interstitielle et des ions est étudié par l'utilisation de microscopie électronique à balayage environnemental (ESEM). Un modèle théorique pour la consolidation électro-osmotique est brièvement présenté, qui couple l'équation de consolidation de Biot avec le flux électro-osmotique et l'équation de commande d'un champ électrique, et intègre la variation non linéaire des propriétés mécaniques, hydrauliques et électriques. Le logiciel FEM a été développé sur la base du modèle théorique pour décrire le processus de consolidation électro-osmotique, les essais sur modèle sont simulés. Les résultats numériques montrent un bon accord avec les données de test en termes de tassement du sol, indiquant la rationalité du modèle analytique. Il est également montré que les variations de la conductivité électrique ont un effet significatif sur le processus de consolidation. Le logiciel développé permet de prédire le comportement de déplacement de masse de sol et de fournir des données utiles pour la conception de systèmes d'électro-osmose.

KEYWORDS: electro-osmosis; axisymmetric test; scanning electron microscopy; theoretical model; numerical simulation

1 INTRODUCTION

Electro-osmotic consolidation provides an attractive soil improvement technique, during which flow of pore water occurs from the anode toward the cathode under the electric field in soils. The electro-osmotic consolidation technique has been used in various geotechnical engineering applications, including stabilization of slopes, excavations, and embankments, controlling groundwater flow, increasing pile capacity and the strength of clays, and dewatering tailings and sludge, meanwhile, numerous laboratory studies have been published, mostly based on soil column tests (Casagrande, 1948; Casagrande, 1983; Shang and Dunlap, 1998; Mitchell and Soga, 2005; Jones et al., 2008). Recently, the combination of electro-osmosis and vacuum preloading was investigated by means of axial-symmetric model tests (Li et al., 2009; Wu and Hu, 2012).

The theory for electro-osmosis was also developed to predict the soil behavior during consolidation. One-dimensional analytical solutions for electro-osmotic consolidation were proposed by previous researchers (Esrig, 1968; Wan and Mitchell, 1976). The analytical solutions for 2D problems were also developed (Shang, 1998; Su and Wang, 2003; Xu et al. 2011). Lewis and Humpheson (1973) formulated a finite element model to analyze the groundwater flow in two-dimensional electric fields. Rittirong and Shang (2008) proposed a 2D finite difference model to obtain excess pore-

water pressure during electro-osmosis, analyzing the subsurface settlement and undrained shear strength.

Up to now, most of the analytical solutions are based on the assumption of uniform electric field, which is not applicable for most field applications due to the complexity of electrode configuration, complicated boundary conditions and anisotropic soil properties. Furthermore, the previous model focused on predicting pore water pressure, while the ground settlement was not considered. A comprehensive numerical model is demanded to predict the soil behavior and provide data for system design of electro-osmosis treatment.

In this paper, a laboratory testing facility for axial-symmetric model was developed for the electro-osmotic consolidation, and the electrical voltage, soil mass displacement, water discharge and electrical current were monitored during testing process by use of online transducers, and the coupled effect of mechanical, hydraulic and electrical field was discussed. A theoretical model was briefly introduced (Hu et al., 2012), and FEM software was developed based on the theoretical model to describe the electro-osmotic consolidation process. The software was verified by comparison of numerical results and test data. The ESEM tests were conducted to observe the change of soil particles.

2 TEST ON ELECTRO-OSMOTIC CONSOLIDATION

An axial-symmetrical electro-osmosis apparatus made of plexiglas with a radius of 18.8 cm and a height of 20 cm shown in Fig. 1 is developed to conduct the electro-osmotic consolidation test on kaolin clay. The radius of the central cylindrical drainpipe with many small holes on it is 1.25 cm. After convolving a piece of geotextile that only water can pass through, an iron wire is then convolved on the geotextile in a spiral form to be the cathode. The anode consists of many vertical iron wires inserted in the soil sample and cling to the inner surface of the apparatus. A set of monitoring device is also developed to measure voltage distribution, water discharge, surface settlement and current during the electro-osmotic consolidation. A vacuum pump and a gas-water separation device are used to collect the water discharged from the soil sample. A multimeter is installed in the circuit to monitor the current.

The basic properties of the kaolin clay used in the test are listed in Table 1.

After filling the test device with saturated kaolin clay, an intermittent DC electric filed shown in Fig. 2 was applied to the soil sample. The total testing time was 100 hours and the conduction time was 57 hours.

Table 1 Basic property of kaolin clay

Initial water content, $w/\%$	Degree of saturation, $S/(\%)$	Liquid limit, $w_L/(\%)$	Plastic limit, $w_p/(\%)$	Specific gravity, G_s
100	100	73	31	2.61

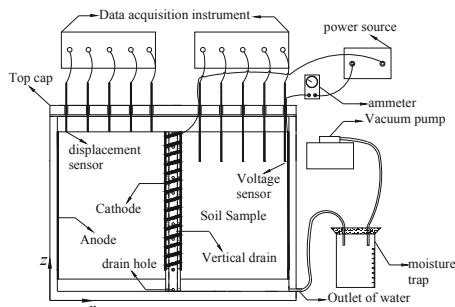


Figure 1. Testing facility for electro-osmotic consolidation

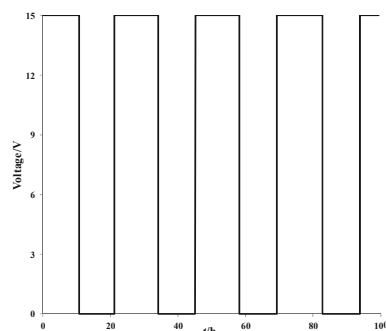


Figure 2. Voltage applied between the anode and the cathode

Fig. 3 shows the voltage distribution along the radial direction during the electro-osmotic consolidation. In most of the previous study, the voltage was supposed to be steady and linear. However, the test result illustrates that the voltage first increases with time with the largest increase near the cathode and the smallest increase near the anode. This is mainly caused by the change of soil resistance near the cathode. Along with the electro-osmosis, the soil near the cathode began to separate from the electrode so that the soil resistance near the cathode increased, which then resulted the increase of the voltage near

the cathode. After 10 hours, the voltage began to decrease and finally the voltage near the cathode was even smaller than that at the beginning. During the electro-osmotic consolidation, the pore water moved from the anode to the cathode and some cracks gradually generated near the anode, which made the soil resistance near the anode to increase, and this further resulted the decrease of the voltage near the cathode. Therefore, the electrical conductivity changes with time and position and it is important to consider this change in the theoretical and numerical analysis.

The current across the soil mass is shown in Fig. 4 with an initial value of 0.72 A, and decrease to about 0.21 A finally. The discharge of water is demonstrated in Fig. 5 and 2294 ml water was discharged in total. When the power was switched off, the current decreased to 0 and the drainage speed almost decreased to 0. When the power was once again turned on then, the movement of the pore water was accelerated and the current first increased and after it decreased again. This was because the pore water flowed back from the anode to the cathode under hydraulic gradient after the current was cut down, which made the moisture distribution to be more uniformity and the resistance therefore decreased.

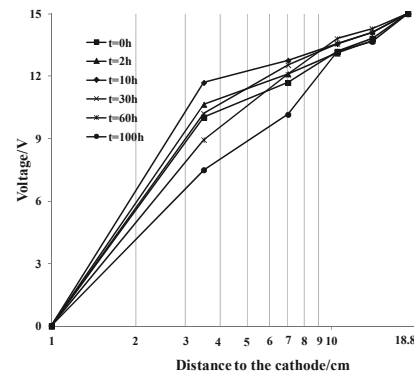


Figure 3. Distribution of the voltage along the radial direction

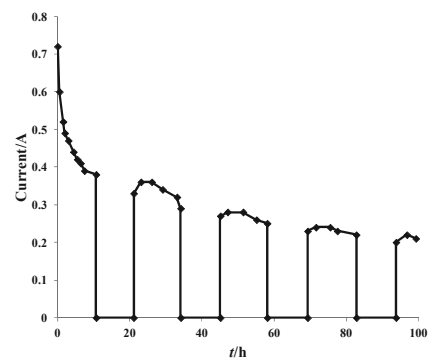


Figure 4. Current during electro-osmotic consolidation

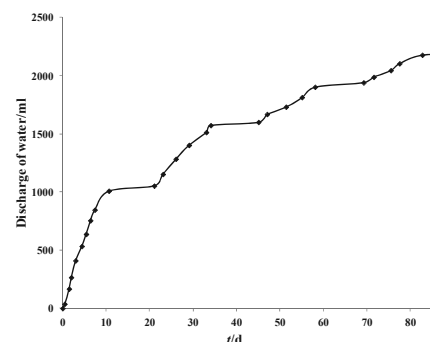


Figure 5. Discharge of water during electro-osmotic consolidation

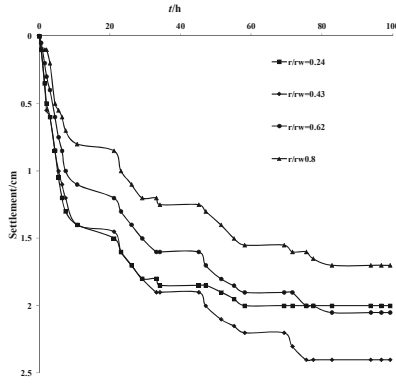


Figure 6. Surface settlements of the soil sample

The surface settlement is shown in Fig. 6. Esrig's theory about excess pore water pressure indicates that the largest settlement happens near the anode since the largest excess pore water pressure is occurred there. However, Fig. 6 shows that the largest surface settlement occurred in the middle of the two electrodes with a value of about 2.4 cm. The friction of the test device and the soil at the anode is the main reason for this phenomenon.

The test results shows that the soil parameters vary during the electro-osmotic consolidation processes, and the previous analytical solution could not adequately predict the soil behavior due to the complicated coupling effects. The multi-physics theoretical model was developed by the authors, by means of coupling soil deformation, pore water flow and electrical field (Hu et al., 2012).

3 THEORETICAL MODEL

During electro-osmotic consolidation, the pore-water flow, soil mass deformation, and electricity have a coupling effect on soil behavior.

3.1 Coupled pore water flow

Pore-water flow is due to the hydraulic and electrical gradients. The pore water velocity in radial and vertical directions can be described according to the Darcy's law and electro-osmotic flow theory (Esrig, 1968),

$$\begin{aligned} v_r &= -k_r \frac{\partial H}{\partial r} - k_{er} \frac{\partial V}{\partial r} \\ v_z &= -k_z \frac{\partial H}{\partial z} - k_{ez} \frac{\partial V}{\partial z} \end{aligned} \quad (1)$$

in which V and H are the electric potential and total head, respectively; v_r and v_z are the pore-water flow velocity; k_r and k_z are the hydraulic conductivity in the radial and the vertical direction; k_{er} and k_{ez} are the coefficient of electro-osmotic conductivity in the radial and the vertical direction, respectively.

For a saturated soil system with non-compressive pore-water and soil particles, the pore-water flow induces the volume strain of soil mass, i.e., consolidation of the soil skeleton. Using the law of conservation of mass for pore water, the following equation can be derived,

$$\frac{\partial v_r}{\partial r} + \frac{v_r}{r} + \frac{\partial v_z}{\partial z} = -\frac{\partial \varepsilon_v}{\partial t} \quad (2)$$

in which ε_v is the volume strain of soil mass.

Therefore the governing equation for the pore water movement in the soil mass can be obtained,

$$\begin{aligned} &\frac{1}{r} (k_r \frac{\partial H}{\partial r} + k_{er} \frac{\partial V}{\partial r}) + k_r \frac{\partial^2 H}{\partial^2 r} + k_{er} \frac{\partial^2 V}{\partial^2 r} + k_z \frac{\partial^2 H}{\partial^2 z} + k_{ez} \frac{\partial^2 V}{\partial^2 z} \\ &= -\frac{\partial}{\partial t} (\frac{\partial u^s}{\partial r} + \frac{\partial w^s}{\partial z}) \end{aligned} \quad (3)$$

3.2 Static equilibrium for soil mass

The elastic constitutive model was used to reflect the relationship of the stress and the strain of the soil skeleton. Therefore, the governing equations can be obtained from Biot's theory and the effective stress principle as,

$$\begin{cases} \frac{\partial}{\partial r} (c_1 \frac{\partial u^s}{\partial r} + c_2 \frac{\partial w^s}{\partial z}) + \frac{\partial}{\partial z} [c_3 (\frac{\partial u^s}{\partial z} + \frac{\partial w^s}{\partial r})] + \frac{c_3}{r} \frac{\partial u^s}{\partial r} = \gamma_w \frac{\partial H}{\partial r} \\ \frac{\partial}{\partial z} (c_1 \frac{\partial w^s}{\partial z} + c_2 \frac{\partial u^s}{\partial r}) + \frac{\partial}{\partial r} [c_3 (\frac{\partial u^s}{\partial z} + \frac{\partial w^s}{\partial r})] + \frac{c_3}{r} \frac{\partial w^s}{\partial r} = \gamma_w \frac{\partial H}{\partial z} + \gamma'_s \end{cases} \quad (4)$$

in which c_1 , c_2 , c_3 are the constant parameters only related to the young's modulus and the poisson's ratio; u^s and w^s are the radial and the vertical displacements; γ'_s denotes the submerged unit weight.

3.3 Conservation of electrical charge

According to the law of conservation of electrical charge the governing equation for the electric field can be represented by the following equation,

$$\begin{aligned} &\sigma_{er} (\frac{\partial^2 V}{\partial r^2} + \frac{1}{r} \frac{\partial V}{\partial r}) + \sigma_{ez} \frac{\partial^2 V}{\partial z^2} + \sigma_{hr} (\frac{\partial^2 H}{\partial r^2} + \frac{1}{r} \frac{\partial H}{\partial r}) \\ &+ \sigma_{hz} \frac{\partial^2 H}{\partial z^2} = C_p \frac{\partial V}{\partial t} \end{aligned} \quad (5)$$

in which C_p is the capacitance per unit volume; σ_{er} and σ_{ez} are the electric conductivity in the radial and vertical direction. σ_{hr} and σ_{hz} are the streaming electric conductivity in the radial and vertical direction, which denotes the current density caused by a unit hydraulic gradient.

4 NUMERICAL SIMULATION

Based on the equation (3) ~ (5), an axisymmetric electro-osmotic consolidation model was developed and the size is the same as the test apparatus. The relationship between the electrical conductivity and the void ratio is conducted from laboratory test results as (Wu, 2009),

$$\sigma_e = 1.016 \times \left(\frac{e}{1+e} - 0.349 \right) \quad (\text{S/m}) \quad (6)$$

The soil parameters used in the numerical model are adopted according to the basic physical properties tests on the soil sample and are shown in Table. 2 (Wu, 2009).

Table 2. Parameters adopted in the numerical model

Initial water content, w (%)	100
Saturation, S (%)	100
Hydraulic conductivity, k_r , k_z ($\text{m} \cdot \text{s}^{-1}$)	8×10^{-10}
Electro-osmosis conductivity, k_{er} , k_{ez} ($\text{m}^2 \cdot \text{s}^{-1} \cdot \text{V}^{-1}$)	8.5×10^{-9}
Electrical conductivity, σ_{er} , σ_{ez} ($1 \cdot \text{ohm}^{-1} \cdot \text{m}^{-1}$)	0.38
Young's module, E (kPa)	2×10^6
Poisson's ratio, ν	0.3

Fig. 7 shows the comparison of the surface settlement obtained from the numerical results and the experiment data. The surface settlement at the position of $r/r_e = 0.62$, in which r_e is

the radius of the model, is compared since the settlement at other positions are largely influenced by the test apparatus.

The settlement obtained from the numerical model which considered the variation of the electrical conductivity is smaller than that obtained from another numerical model in which the electrical conductivity keeps constant. The result of the model with variable electrical conductivity agrees better with the experimental data than the model with constant electrical conductivity during the consolidation process.

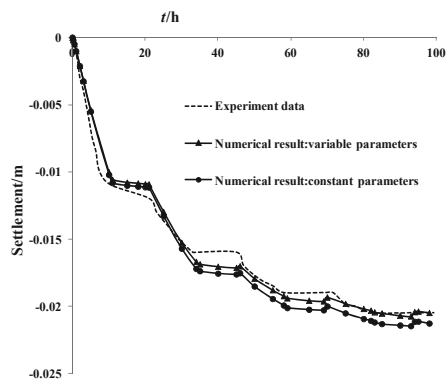
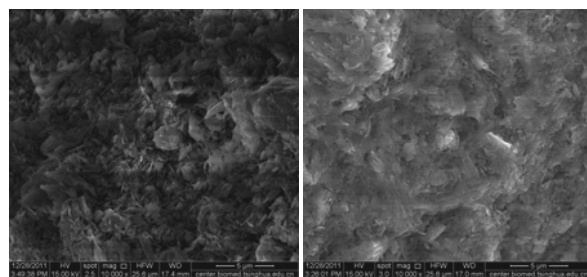


Figure 7. Comparison of the numerical model with the experimental data

5 MICROSCOPIC OBSERVATION

The scanning electron microscopy tests of the soil samples before and after electro-osmosis were conducted and the comparison was shown in Fig. 8.

Fig. 8 illustrates that after the electro-osmosis treatment, the shape of soil particles changed. The content of spherical particle and flaky particle decreased, and the content of acicular particle increased, which mean that under the application of the voltage, the soil particles broke from spherical particle or flaky particle to acicular particle.



(a) After electro-osmosis (b) Before electro-osmosis
Figure 8. Results of the scanning electron microscopy test.

6 SUMMARY AND CONCLUSIONS

The electro-osmotic consolidation process is investigated by means of laboratory test and numerical simulation. The following conclusions can be obtained based on this study.

A laboratory testing facility was developed for the electro-osmotic consolidation, and the electrical voltage distribution, soil mass displacement, water discharge and electrical current were monitored during the testing process. The coupled effect of mechanical, hydraulic and electrical field, as well as the nonlinear variation of soil resistance during electro-osmosis was observed.

A theoretical model for electro-osmotic consolidation is briefly introduced, and FEM software was developed based on

the theoretical model to describe the electro-osmotic consolidation process.

The numerical results showed good agreement with the testing data in terms of ground settlement, indicating the rationality of the analytical model. It is also shown that the variations of electrical conductivity have a significant effect on the consolidation process and the numerical model which considered the variation of the electrical conductivity agrees better with the experimental data than the model with constant electrical conductivity.

The developed software can predict the displacement behavior of soil mass and provide useful data for system design of electro-osmosis treatment.

The result of the ESEM test shows that the soil particles break from spherical particle or flaky particle to acicular particle because of the applied voltage.

ACKNOWLEDGEMENTS

Financial supports from National Natural Science Foundation of China (NSFC50978139), National Key Basic Research Program (2012CB719804), Tsinghua University (2010THZ02-1), and the State Key Laboratory of Hydrosience and Engineering (SKLHSE-2012-KY-1) are gratefully acknowledged.

REFERENCES

- Casagrande L. 1948. Electroosmosis in soils. *Geotechnique* 1: 159-177.
- Casagrande L. 1983. Stabilization of soils by means of electro-osmotic state-of-art. *J. Boston Civ. Eng. ASCE* 69, No. 3, 255-302.
- Esrig M.I. 1968. Pore pressures, consolidation and electro-kinetics. *Journal of the Soil Mechanics and Foundation Engineering Division, ASCE* 94(SM4), 899- 921.
- Hu L. M., Wu W. L. & Wu, H. 2012. Numerical model of electro-osmotic consolidation in clay. *Geotechnique* 62(6), 537-541.
- Jones C. J. F. P., Lamont-Black J., Glendinning S., Bergado D., Eng T., Fourie A., Hu L., Pugh C., Romantshuk M., Simpanen S. & Zhuang Y. 2008. Recent research and applications in the use of electro-kinetic geo-synthetics. Keynote paper, *Proc. 4th European Geo-synthetics Conf.*, Edinburgh, UK, Paper No 329, CDROM.
- Lewis R. W. & Humpheson C. 1973. Numerical analysis of electro-osmotic flow in soils. *Journal of the Soil Mechanics and Foundations Division, ASCE* 99(SM8), 603-616.
- Li Y., Gong X., Jiao D. & Liu Z. 2009. Experimental study on two-dimensional electro-osmotic consolidation of soft clay. *Chinese Journal of Rock Mechanics and Engineering* 28(2), 4034-4039. (in Chinese)
- Mitchell J. K. & Soga K. 2005. Fundamentals of soil behavior. 3rd Edition, John Wiley & Sons, USA.
- Rittirong A. & Shang J. Q. 2008. Numerical analysis for electro-osmotic consolidation in two-dimensional electric field. *Proc. 18th ISOPE, Vancouver, Canada*, 566-572.
- Shang J.Q. 1998. Electroosmotic enhanced preloading consolidation via vertical drains. *Can. Geo. J.* 35(3), 491-499.
- Shang J. Q. & Dunlap W. A. 1998. High voltage reinforcement of ground anchors. *J. Geotech. Geoenviron. Eng. ASCE* 124, No. 9, 840-845.
- Su J. Q. & Wang Z. 2003. Theory of two-dimensional electro-osmotic consolidation of soils. *Geotechnique* 53(8), 759-763.
- Wan T. Y. & Mitchell J. K. 1976. Electro-osmotic consolidation of soils. *J. Geotech. Eng. Div., ASCE* 102(5), 473-491.
- Wu H. & Hu L. M. 2012. Analytical models of the coupling of vacuum preloading and electro-osmotic consolidation for ground stabilization." *J. Tsinghua Univ. (Sci. and Tech.)* 52(2), 182-185. (in Chinese)
- Xu W., Liu S. H., Wang L. J. & Wang J. B. 2011. Analytical theory of soft ground consolidation under vacuum preloading combined with electro-osmosis. *Journal of Hohai University (Natural Sciences)* 39(2), 169-175. (in Chinese)

Creep characteristics of clay in one-dimensional compression with unloading/reloading cycles

Propriétés de fluage des argiles en compression unidimensionnelle avec cycles de charge/décharge.

Kawabe S., Tatsuoka F.
Tokyo University of Science, Japan

ABSTRACT: One-dimensional compression tests including sustained loading stages were performed on reconstituted specimens of two types of clay. In some tests, a number of sustained loading tests were performed during otherwise multiple unloading/reloading cycles with relatively large stress amplitudes. The creep strain taking place at respective sustained loading stages is a function of the ratio of the current sustained loading stress to the stress at which the latest reversal of loading direction was made, independent of previous loading history and current sustained loading stress. A new laboratory one-dimensional compression test method is proposed to evaluate the creep process at such very low strain rates as observed in the field. Creep processes until the creep strain rate becomes as small as about 10^{-10} %/sec can be observed by this method. This new method is much less time-consuming than sustained loading tests starting from the primary loading.

RÉSUMÉ : Des essais oedométriques avec fluages ont été réalisés sur deux types d'argiles remaniées. Pour certains essais, des fluages ont été appliqués lors de grands cycles de chargement comprenant de multiples charges/décharges. Les déformations de fluage observées sont fonction uniquement du rapport entre l'état de contrainte actuel appliqué et l'état de contrainte appliqué lors de la dernière inversion de chargement. Elles restent ainsi indépendantes de l'histoire précédant la dernière inversion et de la seule valeur de la contrainte actuelle. Une nouvelle procédure expérimentale est ainsi proposée pour étudier de manière plus précise les déformations de fluage, jusqu'à des valeurs extrêmement faibles (10^{-10} %/sec) telles que mesurées in situ. La procédure proposée a l'avantage d'écourter grandement la durée du chargement comparativement à des essais classiques de fluage réalisés lors d'une première charge.

KEYWORDS: Clay, One-dimensional compression test, Creep, Strain rate, Loading/unloading/reloading.

1 INTRODUCTION

Most of the previous studies on long-term residual compression of soft clay were focused on the behaviour during primary loading under the normally consolidated conditions. Despite strong geotechnical engineering needs, the study under general loading conditions, not only primary loading but also unloading (UL), reloading (RL) and cyclic loading (UL/RL), is very limited (e.g., Acosta Martínez et al. 2005; Kawabe et al., 2009). Besides, the concerned elapsed time under constant load after the end of construction in a full-scale soft clay deposit is usually very long. Therefore, the residual strain rates are very low, significantly lower than those in ordinary laboratory tests (Figure 1). Consequently, the field residual deformation can be predicted from ordinary laboratory test results only when based on a relevant theoretical framework that can simulate the creep behaviour at very low strain rates as in the field. However, laboratory tests to validate such a framework as above take extremely long time when sustained loading is performed until the strain rate reaches typical values in the field or when constant-rate-of-strain (CRS) tests are performed at typical strain rates in the field (Tanaka, 2005).

In view of the above, Kawabe et al. (2011) performed a series of one-dimensional (1D) compression tests including multiple unloading/reloading (UL/RL) cycles with many sustained loading (SL) stages on disc-shaped specimen (ϕ 60 mm $\times$ h 20 mm, Figure 2) made by trimming a large clay cake. The inside face of the ring was smeared with a grease layer to decrease the friction. Although the apparatus was relatively simple, CRS tests could be conducted by means of a feed-back and automated pneumatic loading system. Figure 3 summarises the relationships between the creep strain during sustained loading (SL) for three hours and the ratio of the stress at the start of unloading (UL) or reloading (RL) at an axial strain rate equal to ± 0.005 %/min, σ_{UL} or σ_{RL} , to the stress at SL, σ_{SL} . The creep strain rates during otherwise UL and RL are smaller

than those during otherwise primary loading and become very small or even negative depending on loading history. Besides, positive and negative creep strains are a unique function of, respectively, the loading stress ratio, σ_{UL}/σ_{SL} (i.e., over-consolidation ratio, OCR) and σ_{RL}/σ_{SL} , not by the stress difference, $\sigma_{UL} - \sigma_{SL}$ and $\sigma_{RL} - \sigma_{SL}$. This relation is also a function of other factors. For example, the creep strain increases with an increase in the initial strain rate (i.e., the strain rate during the CRS loading immediately before the concerned SL stage) and the elapsed time during the SL stage.

In this study, to confirm the generality of the empirical relations described in Figure 1, another series of 1D compression tests including multiple UL/RL cycles with many SL stages were performed. Based on the test results, a new test method to observe creep behaviours at very low strain rates in a relatively short time is proposed.

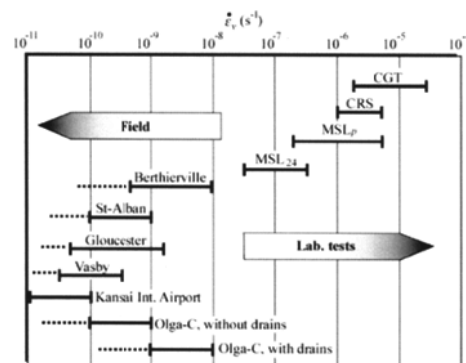


Figure 1. The strain rate measured in laboratory and in situ (Leroueil 2006)

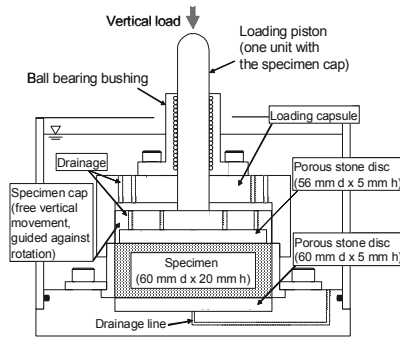


Figure 2. Oedometer used in the present study (Kongkitkul et al. 2011)

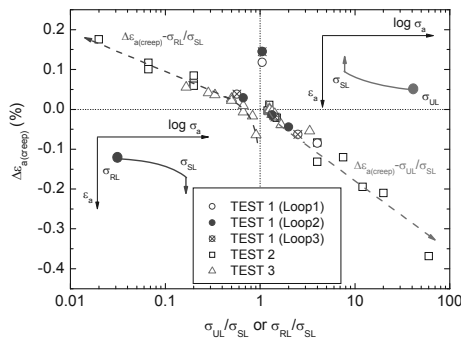


Figure 3. Relationship between the creep strain and loading stress level (Kawabe et al. 2011)

2 ONE-DIMENSIONAL COMPRESSION TESTS

The test method is basically the same as Kawabe et al. (2011). Large clay cakes of two types of reconstituted soft clay, Fujinomori clay ($w_L \approx 42\%$ and $PI \approx 15$) and kaolin clay ($w_L \approx 44\%$ and $PI \approx 16$), were produced by consolidating slurry prepared at a water content twice of the liquid limit at $\sigma = 100$ kPa in a 20 cm diameter cylinder for a week. After having applied a pre-load scheme to minimize effects of disturbance during specimen preparation, each specimen, drained at the top and bottom, was subjected to: monotonic recompression at $\dot{\epsilon}_a = 0.05\%/min$ to $\sigma_a = 100$ kPa; SL for one day; UL at $\dot{\epsilon}_a = -0.05\%/min$ to $\sigma_a = 100$ kPa; and SL for one day. Logarithmic axial strains are used throughout this paper. It was ensured that the specimen under the conditions described above is always essentially drained from the fact that the excess pore water pressure measured at the undrained bottom of the specimen with the drained top set in another apparatus was always less than 1 % of transient applied total axial stress (Kawabe et al. 2011). So, the measured total axial stress, σ_a , is regarded as the effective axial stress.

3 TEST RESULTS

3.1 Creep during multiple unloading/reloading cycles

In the test described in Figure 4a (test FJM1008), a number of SL stages for three hours were applied during multiple CSR UL/RL cycles changing the stress amplitude. Figures 4b and c show the time histories of σ_a and creep axial strain at each SL stage, $\Delta\epsilon_{a(creep)}$. In Figure 4c, the data sets for the same value of σ_{UL}/σ_{SL} or σ_{RL}/σ_{SL} are denoted by the same symbol. The creep characteristics at similar value of σ_{UL}/σ_{SL} or σ_{RL}/σ_{SL} are similar, independent of loading history and axial stress at SL stage. The similar test result is shown in Figure 5. It is to be noted that these relations change with changes in the elapsed time of SL.

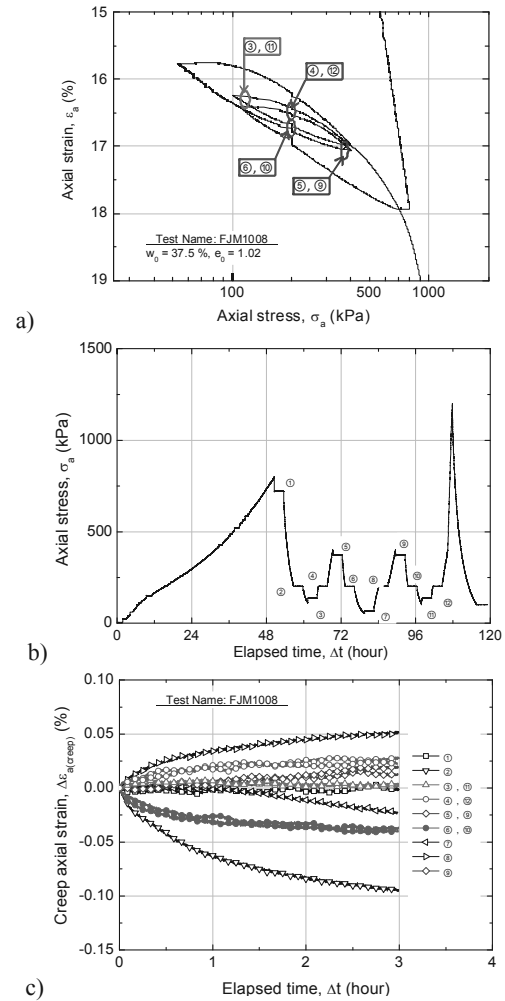
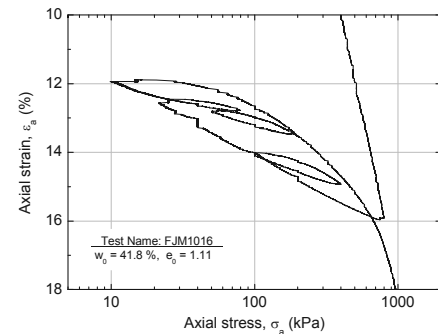
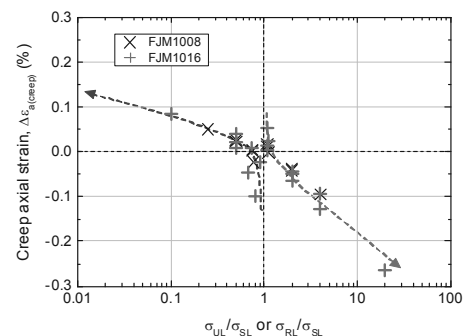

 Figure 4. a) Zoom-upped $\epsilon_a - \log \sigma_a$ relations; b) stress history; and c) time history of creep strain, test FJM1008

 Figure 5. Zoom-upped $\epsilon_a - \log \sigma_a$ relation, test FJM1016

 Figure 6. $\Delta\epsilon_{a(creep)} - \sigma_{UL}/\sigma_{SL}$ or σ_{RL}/σ_{SL} relations, tests FJM1008 and FJM1016

Figure 6 shows the relationship between $\Delta\epsilon_{a(creep)}$ for three hours and the stress ratio, σ_{UL}/σ_{SL} or σ_{RL}/σ_{SL} , at each SL

stage, plotted in the same way as Figure 3. Also in this case, the relation is independent of loading history and stress at SL stage, and the relations for the two tests are nearly the same. This result indicates that the empirical rule presented in Figure 3 is also valid for a wide variety of loading history, at least when the void ratio range is as small as in these tests.

The current creep strain rate is controlled by the instantaneous yield characteristics. As typically seen from Figures 4a and 5, the stress at which large-scale yielding starts is not a fixed value but controlled by loading history. Kawabe et al. (2011) showed that the hysteretic stress-irreversible strain relations during cyclic one-dimensional compression of clay can be adequately described by revising the proportional rule that was originally proposed for shear tests (Tatsuoka et al. 2003). This feature is referred to below when inferring very low creep strain rates during otherwise cyclic loading.

3.2 Very low creep strain rates

Many clay types exhibit the Isotach viscous properties in one-dimensional compression (e.g., Imai 1995; Niemunis and Krieg 1996; Leroueil et al. 1996; Leroueil and Marques 1996; Kawabe et al. 2011). In that case, at the ‘loading’ state in the sense that the irreversible axial strain rate, $\dot{\epsilon}_a^{ir}$, has been kept positive since the start of loading, a unique effective stress is defined for given irreversible strain and its rate. Then, the $\epsilon_a - \log \sigma_a$ curve of a CRS test at a lower $\dot{\epsilon}_a^{ir}$ value is located more left or lower in the plot presented in Figure 7. The $\dot{\epsilon}_a^{ir}$ value at the strain-stress state, (ϵ_a, σ_a) , that is ultimately reached in any SL is zero. These zero-strain-rate states form the stress-strain relation called the reference relation. The reference relation is the same for different loading histories keeping the same sign of $\dot{\epsilon}_a^{ir}$, but different for different loading histories changing the sign of $\dot{\epsilon}_a^{ir}$. In Figure 7, the reference relation at the ‘loading’ state and the one for the ‘first unloading’ state, where $\dot{\epsilon}_a^{ir}$ has become negative for the first time since the start of loading, are presented. Point B is located on the first unloading curve starting from point A (on the primary loading curve). During this unloading process, the total axial strain rate, $\dot{\epsilon}_a (= \dot{\epsilon}_a^e + \dot{\epsilon}_a^{ir})$ is negative, but $\dot{\epsilon}_a^{ir}$ is kept positive while the elastic axial strain rate $\dot{\epsilon}_a^e$ is kept negative with a negative axial stress rate. Note again that all the strain-stress states located right of, or above, the reference relation for ‘loading’, including points A, B and B’, are at the ‘loading’ state.

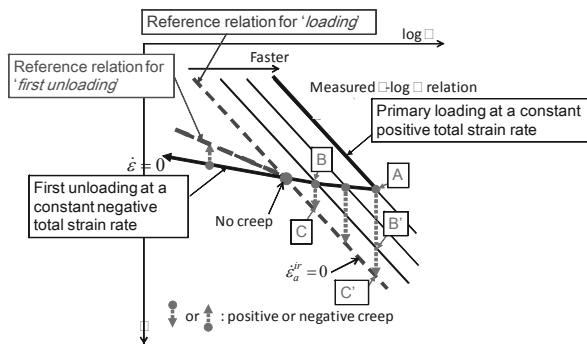


Figure 7 Illustration of $\epsilon_a - \log \sigma_a$ and creep in Isotach theory

In SL at the ‘loading’ state (where $\dot{\epsilon}_a^{ir} > 0$), as the initial strain rate at the start of SL becomes lower, the reference relation is reached faster while the residual creep strain rate after the same elapsed time since the start of SL becomes smaller. For example, the reference relation is reached much faster when SL starts from point B than when SL starts from point A. By taking advantage of this feature, in a relatively short period by starting after having made some unloading (such as point A to point B), we can reach the creep behaviour at very low strain rates that can be observed only after a very long period when SL starts during otherwise primary loading.

Figure 8a shows the $\epsilon_a - \log \sigma_a$ relation from a CRS test on kaolin clay, in which CRS primary loading was followed by: 1) a SL stage, SL1; 2) a small unloading; 3) another SL stage, SL2; and 4) a global RL another SL stage, SL4. Figure 8b is the zoom-up of the behaviours around the stages 1) – 3). Despite that the viscous properties of Fujinomori clay is slightly non-Isotach (Kawabe et al. 2009), the result from a similar test performed as the above (test FJM1104, cf. Figure 10) was analysed below in the framework of Isotach viscous properties.

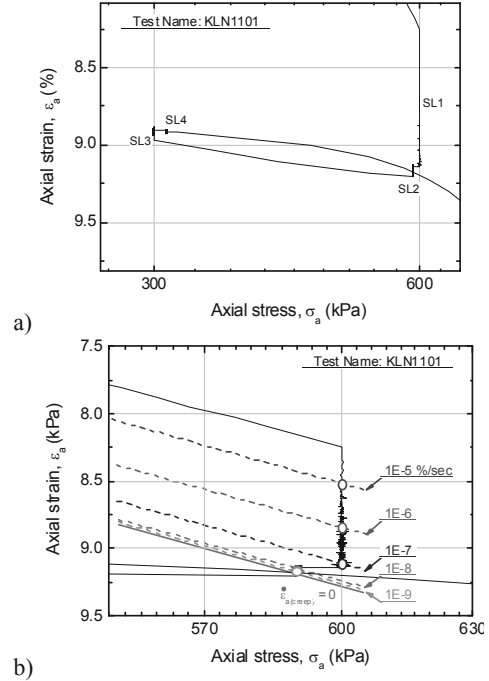


Figure 8. a) Zoom-upped $\epsilon_a - \log \sigma_a$ relation; and b) behaviours around SL1 and SL2 stages, test KLN1101

Figures 9a and 10a show the $\log \dot{\epsilon}_a^{(creep)} - \log \Delta t$ relations from tests KLN1101 and FJM1104, where SL1 starts during otherwise the primary loading and SL2 starts during otherwise the first unloading following SL1 (see Figure 8a). It may be seen that the creep strain rate, $\Delta \dot{\epsilon}_a^{(creep)}$, decreases linearly with the logarithm of the elapsed time, Δt , since the start of SL at both SL1 stages. The relations at SL1 and SL2 are different, showing that the elapsed time since the start of respective SL stages, Δt is not the parameter that controls the creep behaviours at different stresses, σ_{SL} . Figures 9b and 10b show the $\log \Delta \dot{\epsilon}_a^{(creep)} - \log \dot{\epsilon}_a^{(creep)}$ relations from these two tests. The two relations at SL1 are nearly straight until $\dot{\epsilon}_a^{(creep)}$ becomes about 10^{-7} %/sec. The relations at SL2 stage are utterly different from the above.

Referring to Figure 7, according to the Isotach theory, the creep behaviour from point B to point C is equivalent to the one from point B' to point C' at the last part of the SL that has started from point A at the primary loading state. The $\dot{\epsilon}_a^{ir}$ value is the same at points B and B' and at points C and C'. Correspondingly, in Figure 8b, the $\epsilon_a - \log \sigma_a$ relations for different $\dot{\epsilon}_a^{ir}$ values that are in parallel to each other have been depicted (i.e., broken lines). Each of these relations passes different (ϵ_a, σ_a) states where different strain rate was observed. Following this procedure, in Figures 9b and 10b, the measured relations of SL2 have been parallel-shifted upwards to be connected to the end part of the relation of the respective SL1 stages. The combined relations can be regarded as the continuous relations that would have been obtained if SL1 had continued longer than the actual tests.

The obtained relations exhibit a drastic decrease in the slope toward eventually zero $\dot{\epsilon}_a^{(creep)}$. This trend is consistent with

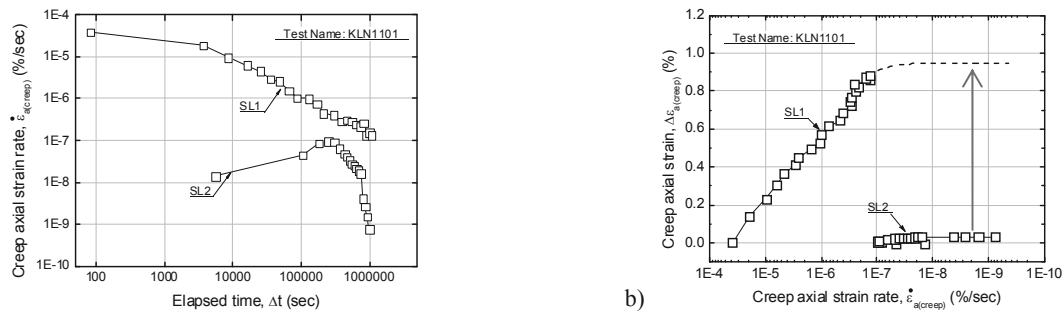


Figure 9. Test KLN1101 (SL1 and SL2): a) $\log \dot{\epsilon}_{a(creep)} - \log \Delta t$ relations; and b) $\Delta \epsilon_{a(creep)} - \log \dot{\epsilon}_{a(creep)}$ relations

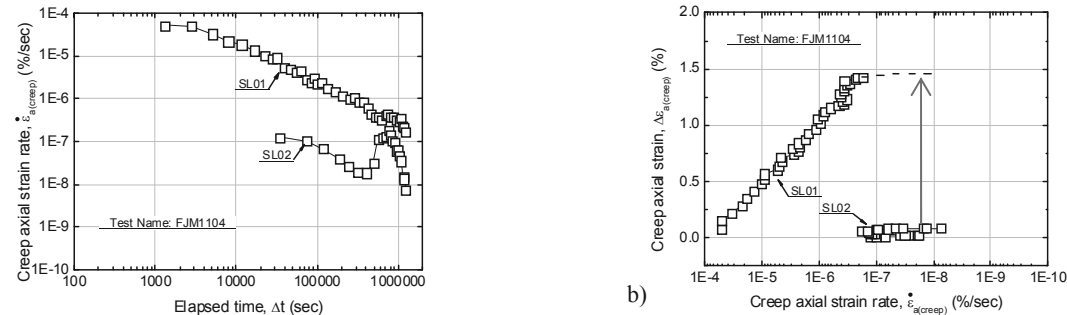


Figure 10. Test FJM1104 (SL1 and SL2): a) $\log \dot{\epsilon}_{a(creep)} - \log \Delta t$ relations; and b) $\Delta \epsilon_{a(creep)} - \log \dot{\epsilon}_{a(creep)}$ relations

the Isotach theory, by which the $\dot{\epsilon}_{a(creep)}$ value becomes ultimately zero in a given SL stage (i.e., points C and C' in Figure 7). This trend is seen in the test result presented in Figure 8b. In this figure, the reference relation, along with $\dot{\epsilon}_{a(creep)} = 0$, has been drawn based on the value of $\sigma_{UL} / (\sigma_{SL} \text{ at } \Delta \epsilon_{a(creep)} = 0) = 1.25$ that was obtained from a similar test result as those presented in Figure 3 and 6. The axial strain increment, $\Delta \epsilon_a$, between the relations for the $\dot{\epsilon}_a$ values different by a factor of 10 (i.e., one log. cycle of $\dot{\epsilon}_{a(creep)}$) is similar until $\dot{\epsilon}_{a(creep)}$ becomes about 10^{-7} %/sec, after which $\Delta \epsilon_a$ becomes smaller towards zero as the relation approaches the reference line.

The largest advantage of the method proposed above is that creep behaviours at very low strain rates can be observed in a relatively short. This method becomes reliable when the reference relation along $\dot{\epsilon}_{a(creep)} = 0$ is obtained from such relations as shown in Figures 3 and 6 by performing many SL during otherwise UL and RL. Extending the method described above, creep behaviours at very low strain rates (positive or negative) starting during otherwise UL/RL can be predicted by inferring the hysteresis reference relation.

4 CONCLUSIONS

The following conclusions can be derived:

- 1) The creep strain during normally or over-consolidated conditions after arbitrary loading history is controlled by the ratio of the stress at load reversal immediately before to the stress at sustained loading, irrespective of loading history and the stress at sustained loading.
- 2) With Isotach materials, creep behaviour at very low strain rates as those reached after a long period by sustained loading starting during otherwise primary loading can be observed in a relatively short period by performing sustained loading after a relevant amount of stress reversal.
- 3) The logarithm of creep strain rate decreases rather linearly with creep strain until the creep strain rate becomes a certain low value, followed a drastic decrease toward zero.

5 REFERENCES

- Acosta-Martínez H., Tatsuoka, F., and Li, J.-Z. 2005. Viscous property of clay in 1-D compression: evaluation and modelling. *Proc. 16th ICSMGE, Osaka*, 779-783.
- Imai, G. 1995. Analytical examination of the foundations to formulate consolidation phenomena with inherent time dependence. *Keynote Lecture, Proc. Int. Symp. on Compression and Consolidation of Clayey Soils, IS Hiroshima*, (2) 891-935. Rotterdam: Balkema.
- Kawabe, S., Kaihara, K. and Tatsuoka, F. 2009. Non-Isotach viscous property and its simulation in drained one-dimensional compression on clay. *Proc. of 44th Conf. on Geotechnical Eng., Japan Geotechnical Society, Yokohama*, 237-238. (in Japanese)
- Kawabe, S., Kongkitkul, W. & Tatsuoka, F. 2011. 1D Compression with Unload/Reload Cycles on Soft Clay and its Simulation. *Proc. 14th Asian Regional Conference on SMGE, Hong-Kong, CD*.
- Kongkitkul, W., Kawabe, S., Tatsuoka, F. and Hirakawa, D. 2011. A simple pneumatic loading system controlling stress and strain rates for one-dimensional compression of clay. *Soils and Foundations* 51 (1), 11-30.
- Leroueil, S., and Marques, M.E.S. 1996. Importance of strain rate and temperature effects in geotechnical engineering. *S-O-A Report, Measuring and Modeling Time Dependent Soil Behavior, ASCE Geotech. Special Publication*, 61, 1-60.
- Leroueil, S., Perret, D., and Locat J. 1996. Strain rate and structuring effects on the compressibility of a young clay. *Measuring and Modeling Time Dependent Soil Behavior, ASCE Geotech. Special Publication* 61, 137-150.
- Leroueil, S. 2006. Suklje's Memorial Lecture – The isotache approach: Where are we fifty years after its development by Professor Suklje?. *Proc. European-Danube Conference on Geotechnical Engineering, Ljubljana*, 1, 55-88.
- Niemunis, A., and Krieg, S. 1996. Viscous behaviour of soil under oedometric conditions. *Canadian Geotechnical Journal* 33, 159-168.
- Tanaka, H. 2005. Consolidation behaviour of natural soils around p_c value - long term consolidation test -. *Soils and Foundations*, 45(3) 83-95.
- Tatsuoka, F., Masuda, T. and Siddiquee, M.S.A. 2003. Modelling the stress-strain behaviour of sand in cyclic plane strain loading. *Geotechnical and Environmental Engineering, Journal of Geotechnical and Environmental Engineering, ASCE*, 129 (6) 450-467.

Comparison of the geotechnical properties of pumice sand from Japan and New Zealand

Comparaison des propriétés géotechniques de sables de pierre ponce du Japon et de Nouvelle-Zélande

Kikkawa N.

National Institute of Occupational Safety and Health, Japan

Pender M.J., Orense R.P.

University of Auckland, New Zealand

ABSTRACT: The geotechnical properties of pumice sands from Japan and New Zealand are compared. Both of these materials are characterised by the presence of particles that are easily crushed against a hard surface under fingernail pressure. The paper gives data on the changes in particle size distribution before and after hydrostatic compression and drained triaxial shear. It is apparent from the results of drained triaxial compression tests that the shear stress continues to climb with increasing shear strain, so that there is not a well-defined friction angle. However, the mobilised friction angle reaches quite large values when the mean principal effective stress is in the range of a few hundred kPa, but decreases as the mean principal effective stress increases further. A particularly interesting feature of the drained shear behaviour of the materials is that with a sufficiently large confining pressure they tend to deform in one-dimensional compression. In general terms, the properties of these two materials of different origin are quite similar.

RESUME: On compare les propriétés géotechniques de sables de pierre ponce provenant du Japon et de Nouvelle-Zélande. Ces deux types de matériaux sont caractérisés par la présence de particules qui sont facilement écrasées contre une surface dure sous la pression de l'ongle. Des données sont fournies sur les modifications de distribution granulométrique des particules avant et après compression hydrostatique et cisaillement triaxial drainé. Il ressort des résultats d'essais de compression triaxiale drainée que la contrainte de cisaillement continue d'augmenter avec l'augmentation de la déformation de cisaillement, de telle sorte qu'il n'y a pas d'angle de frottement bien défini. Toutefois, l'angle de frottement mobilisé atteint des valeurs très élevées lorsque la contrainte effective principale moyenne est de l'ordre de quelques centaines de kPa, mais diminue à mesure que la contrainte effective principale moyenne continue de croître. Une caractéristique particulièrement intéressante du comportement en cisaillement drainé de ces matériaux est qu'avec une pression de confinement suffisamment élevée, ils ont tendance à se déformer en compression unidimensionnelle. De manière générale, les propriétés de ces deux matériaux d'origines différentes sont tout à fait similaires.

KEYWORDS: pumice, particle crushing, shear strength mobilisation, lateral strain during drained shear

MOTS CLES : pierre ponce, écrasement de particules, mobilisation du cisaillement, déformation latérale en cisaillement drainé

1 INTRODUCTION

Pumice sands derived from volcanic activity are found in Japan and New Zealand. Both of these materials are characterised by the presence of particles that are easily crushed against a hard surface under fingernail pressure. In this paper we compare the geotechnical properties of these materials. The composition of both pumice sands is dominated by silica and aluminium oxide.

Pumice deposits are found in several areas in New Zealand. They originated from a series of volcanic eruptions centred in the Taupo and Rotorua regions of the central North Island. Although they do not cover wide areas, their concentration in river valleys and flood plains means they tend to coincide with areas of considerable human activity and development. Thus, they are frequently encountered in engineering projects and their evaluation is a matter of considerable geotechnical interest. Further information about the properties of New Zealand pumice sand are given by Wesley et al (1999), Pender et al (2006) and Naotaka et al (2011).

The Japanese pumice used in this study was sampled from the Kanoya-city located on the Osumi-peninsula in the Southern Kyushu.

Individual particles are not solid but have internal voids. This is the reason for the low unit weights presented in Table 1. The particle density is not well-defined because of the internal voids, hence only typical values are given in Table 1.

2 STRESS-STRAIN-STRENGTH BEHAVIOUR

The first comparison is given in Figure 1 where the change in volume of dry pumice sand specimens under hydrostatic compression is plotted. It is apparent that the Japanese pumice undergoes considerably more volume change than the NZ material; this may be a consequence of the different particle size range of the two samples, as shown in Figure 5, and the well-sorted size distribution of the Japanese pumice.

Figure 2 presents the deviator stress – axial strain plots for conventional drained triaxial tests on specimens of the sand after consolidation under hydrostatic pressure. Figure 3 has the volume change behaviour of these specimens; note that the de-

Table 1. Physical properties of the pumice sands used in this study.

	Grain size	Typical particle density	Minimum dry unit weight	Maximum dry unit weight	Test dry unit weight
	d mm	ρ_s kg/m ³	γ_{dmin} kN/m ³	γ_{dmax} kN/m ³	γ_d kN/m ³
NZ pumice	0.05 ~1.18	2340	5.65	6.95	5.42 ~5.87
Japanese pumice	0.15 ~0.30	2489	4.34	6.41	5.70 ~5.77

crease in volume means that the tests were done on loose material. Also marked on these plots is a line indicating one dimensional compression, that is the volumetric strain is equal to the axial strain, hence along this line the shearing occurs without change in diameter of the specimen.

Figure 4 plots contours of equal axial strain on a set of axes with deviator stress on the vertical axis and mean principal effective stress on the horizontal axis. (The deviator stress is: $q = \sigma'_1 - \sigma'_3$, and the mean principal stress is: $p' = (\sigma'_1 + 2\sigma'_3)/3$). The first, and most important observation from these plots, is that the mobilization of the drained shear resistance for both pumice sands continues to increase with increasing axial strain, also apparent from Figure 2. Note that this is not so for the undrained test also shown in Figure 2b. Any point on the plots in Figure 4 indicates a mobilized friction angle, as the ratio of q/p' can be used to calculate the mobilized friction angle at that point. Looking at the data in Figure 4a, one can draw a tangential envelope near the origin which defines a friction angle of about 41 degrees. Next taking the highest point on the 20% strain contour the q/p' ratio defines a friction angle of about 30 degrees. Thus although the shear resistance continues to increase with increasing axial strain the mobilized friction angle decreases. Similar comments can be made about the data for the New Zealand pumice shown in Figure 4b.

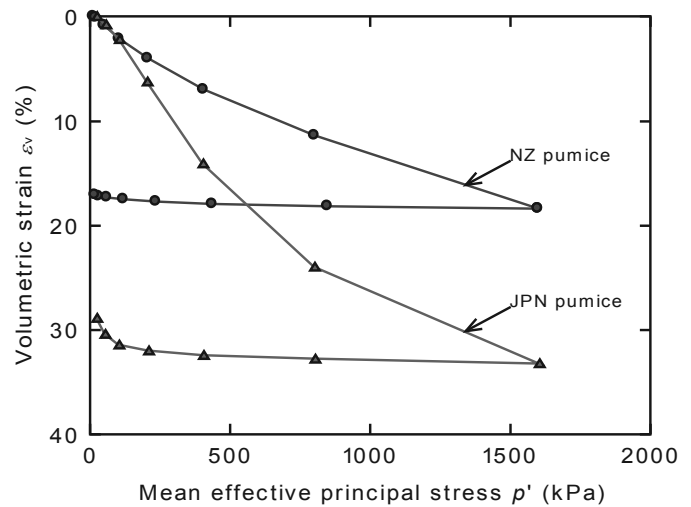


Figure 1. Volumetric strain under hydrostatic compression.

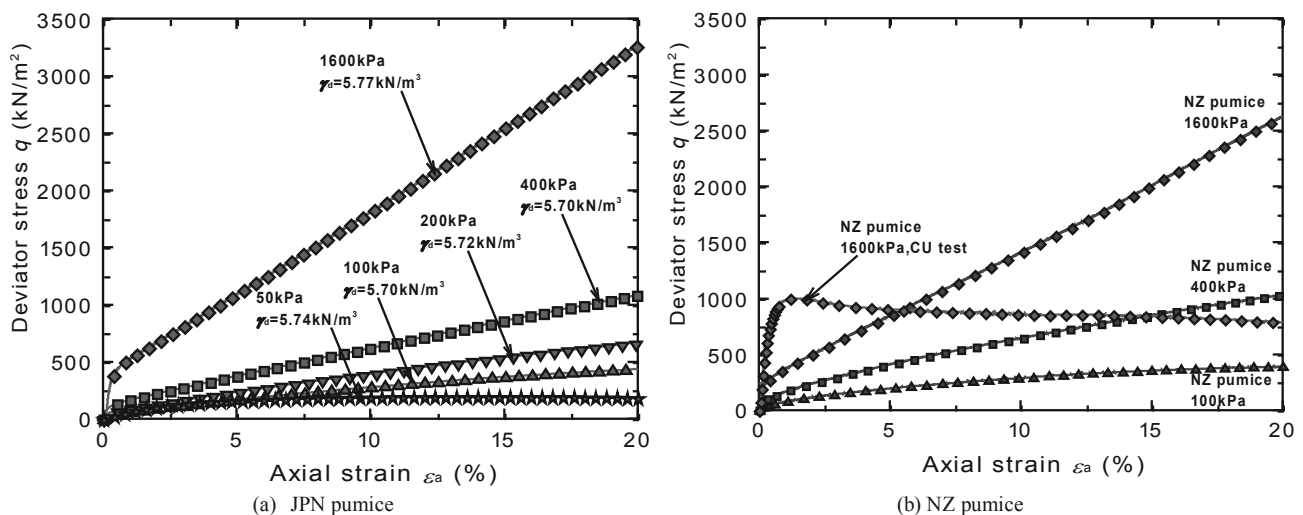


Figure 2. Triaxial test deviator stress versus axial strain.

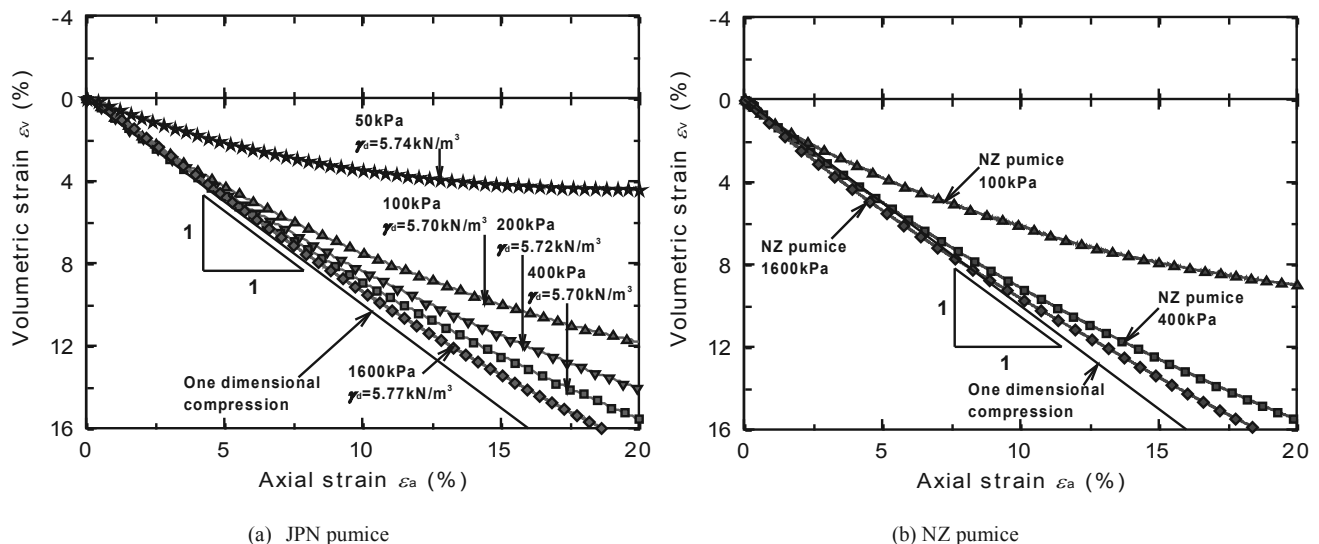
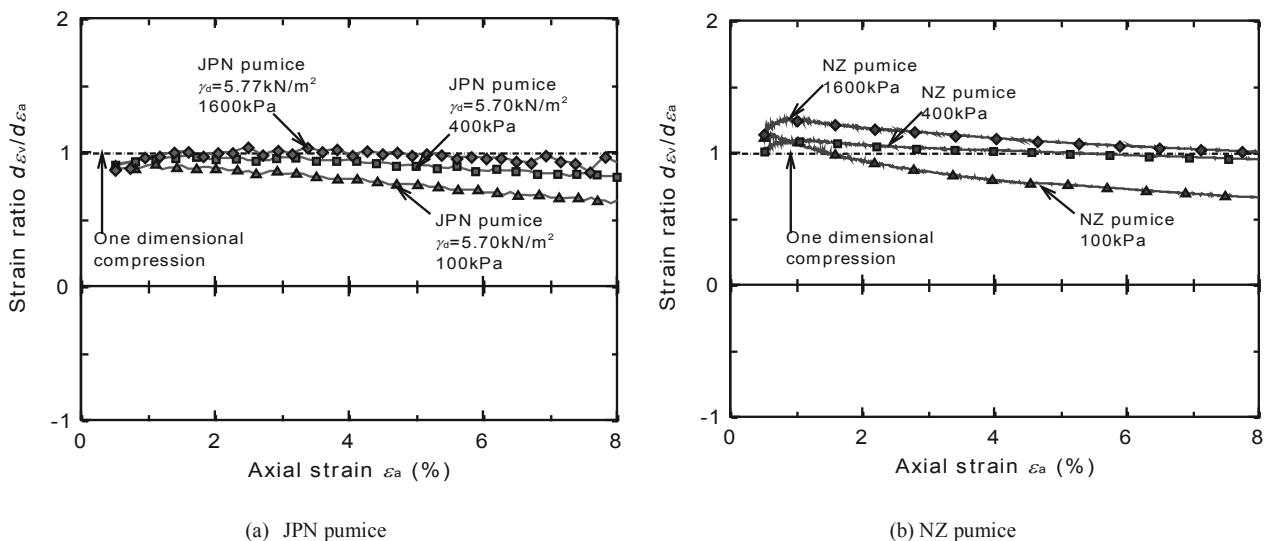
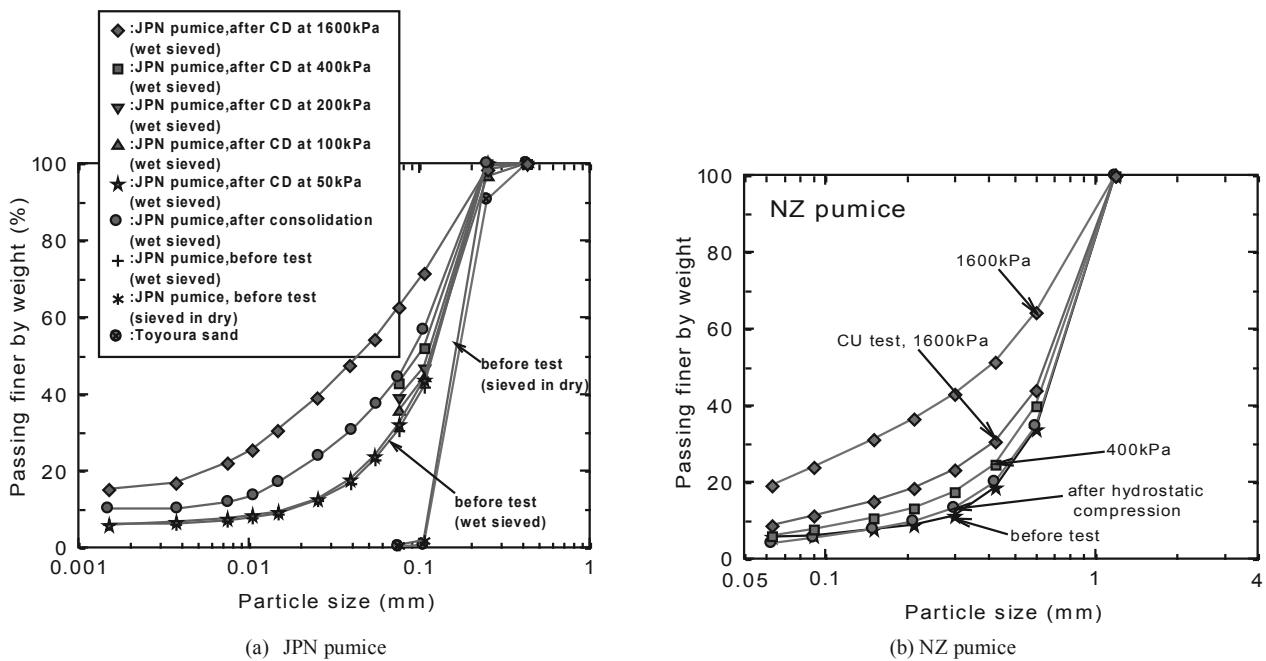
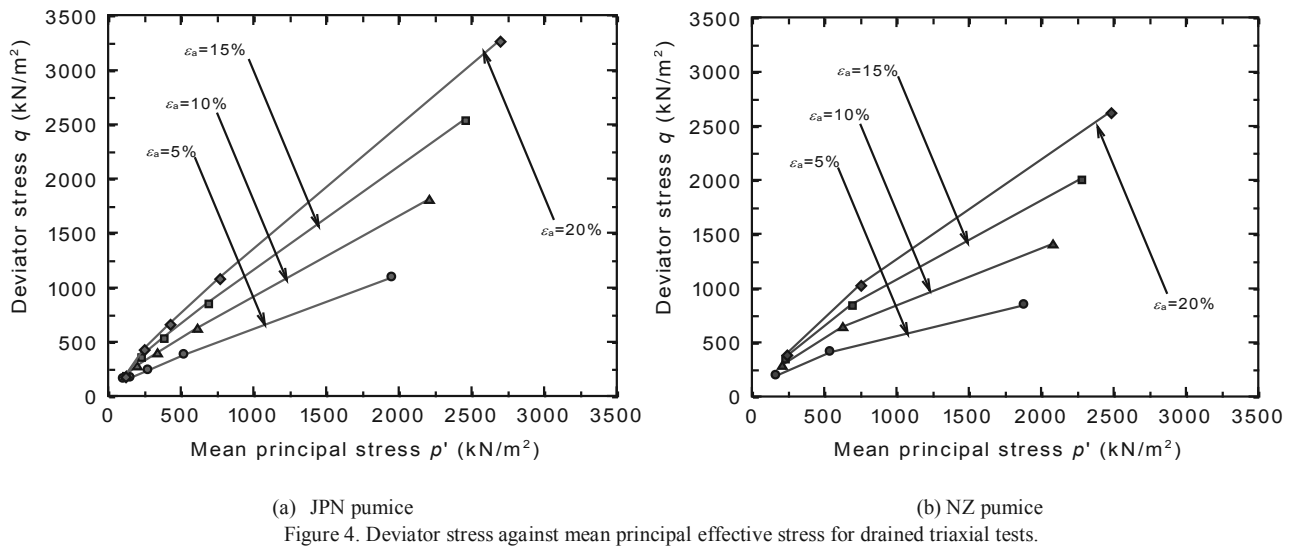


Figure 3. Drained triaxial test volumetric strain versus axial strain.



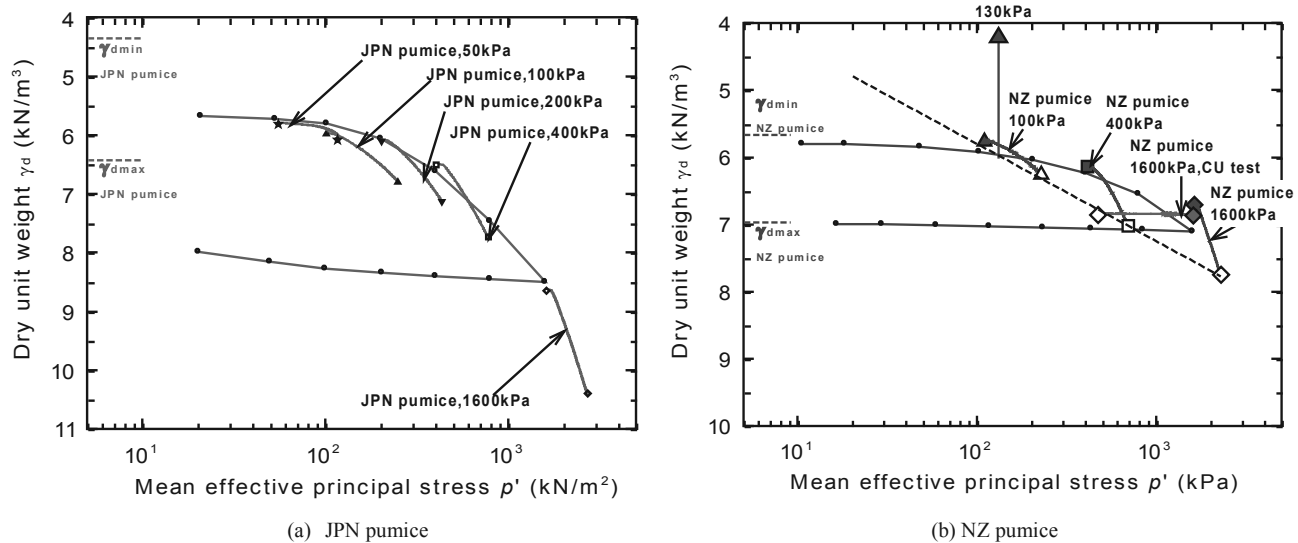


Figure 7. Paths of dry unit weight against mean principal effective stress for the hydrostatic compression and drained triaxial tests.

Figure 5 shows that for both pumice sands there is considerable particle breakage during the drained shearing and that this is progressively more severe as the confining pressure increases. Note that the Japanese pumice is much finer than the New Zealand material but the general features of particle crushing are similar for both materials. One interesting observation for the NZ material is that there is much less particle crushing for the undrained test following consolidation to 1600 kPa than for the drained test from the same consolidation pressure. The reason is because of the differing stress paths. For the drained test the stress paths for drained compression, if plotted on the set of axes in Figure 4, is a line of slope +3 upwards and towards the right. On the other hand since the specimens are loose the undrained effective stress path will move to the left and so subject the sand particles to lower effective stresses.

Figure 6 presents a surprising insight into the deformation behaviour of these materials. In the drained triaxial test the axial deformation and the volume change are measured. From this it is possible, with the usual assumption that the specimen deforms as a right circular cylinder, to determine the change in diameter of the specimen. The figure shows that for higher consolidation pressures there is zero or very little change in diameter of the specimen. In fact, for the New Zealand pumice for consolidation pressures of 400 and 1600 kPa it is seen that the diameter of the specimen actually decreases, that is there is a decrease in length because of the compression and also lateral compression. In our view this is a consequence of the large amount of particle crushing at these higher consolidation pressures. This observation was first made in the paper by Pender et al (2006) on the properties of New Zealand pumice sands.

Figure 7 plots the dry unit weight of the specimens tested on the vertical axis against the logarithm of the mean principal effective stress on the horizontal axis. In the Critical State literature diagrams such as this are plotted with the void ratio on the vertical axis. However, one of the difficulties of dealing with material having voids within the particles is that the value for the "operational" solid density is unclear. So in Table 1 the values given for the solid density are reasonable indications but not exact values. One way to overcome this difficulty is to use the dry unit weight rather than the void ratio, hence the vertical axes in Figure 7. The beginning and end points of the various shear tests are shown in Figure 7 along with the hydrostatic compression curves. For the New Zealand pumice results in Figure 7b a dotted straight line is drawn through the end points.

However, this cannot be regarded as a critical state line as Figures 2 and 3 show that the state of the specimens has not reached a condition with constant q and constant volume.

3 CONCLUSIONS

The main conclusions derived from this brief comparison of the properties of New Zealand and Japanese pumice sands are:

- The properties of the materials are very similar, despite their different origins and particle size distributions.
- In drained triaxial testing, neither the shear stress or the specimen volume reaches a steady value even at axial strains of 20% (Figs. 2 & 3).
- During drained shearing particle crushing is considerable (Fig. 5).
- There is no well-defined angle of shear resistance (Fig. 4).
- At high confining pressures the lateral deformation during drained shear indicates behaviour close to one-dimensional compression (Fig. 6).
- Figure 7 indicates that the materials have not reached a critical state condition.

4 ACKNOWLEDGEMENTS

We are very grateful to the Japan Society for the Promotion of Science (JSPS) for support. This research has been partially funded by a JSPS Grant-in-Aid for Young Scientists (B) (No.23760448).

5 REFERENCES

- Kikkawa, N., Orense, R.P. & Pender, M.J. (2011). "Mechanical behaviour of loose and heavily compacted pumice sand," *Proc., 14th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering*, Hong Kong, 23-27 May 2011, Paper 214. 6pp.
- Pender, M J, Wesley, L D, Larkin, T J and Pranjoto, S (2006) "Geotechnical properties of a pumice sand." *Soils and Foundations*, Vol. 46, No. 1, pp. 69-81.
- Wesley, L. D., Meyer, V. M., Pranjoto, S, Pender, M. J., Larkin, T. J. and Duske, G. C. (1999) "Engineering properties of pumice sand" *Proc. 8th Australia-NZ Conference on Geomechanics*, Hobart, Vol. 2, 901-908.

Evaluation of Consolidation Behavior of Soils under Radial Drainage Condition Using Digital Image Analysis

Évaluation du comportement de consolidation des sols sous des conditions de drainage radial à partir de l'analyse d'image numérique

Kim J.-Y., Chung C.-K.

Department of Civil and Environmental Engineering, Seoul National University

Cho N.-G.

Hyundai Engineering & Construction

Yune C.-Y.

Department of Civil Engineering, Gangneung-Wonju National University

ABSTRACT: Most of the analytical solutions for consolidation under radial drainage are based on the assumption that only vertical compression occurs. During consolidation, however, soils deform not only vertically but also radially. The radial deformation during consolidation may induce additional excess pore pressure in a soil mass and unexpectedly cause differential settlement and produce stresses in structures founded on consolidating ground. A new consolidation apparatus, which enables the deformation of a specimen to be visually observed, and the total vertical stress as well as the pore water pressure to be measured, was developed. Consolidation tests for reconstituted kaolinite specimens were performed using the new consolidation apparatus under radial drainage, along with test under vertical drainage. Digital image analysis was adopted to analyze the consolidation deformation of the specimen. Radial deformations as well as vertical deformations during consolidation were monitored and compared for the two different drainage condition. Non-uniformities of the specimen induced during horizontal drainage were confirmed by measured total vertical stress.

RÉSUMÉ : La plupart des solutions analytiques pour la consolidation avec écoulement radial sont fondées sur l'hypothèse qu'il n'y a que des compressions verticales. Cependant, pendant la consolidation, les sols se déforment non seulement verticalement mais aussi radialement. La déformation radiale pendant la consolidation peut induire des surpressions interstitielles additionnelles dans le sol, pouvant entraîner des tassements différentiels imprévus ainsi que des contraintes au sein des structures fondées sur le sol en cours de consolidation. Un nouvel appareil de consolidation, qui permet d'observer visuellement la déformation d'une éprouvette et de mesurer la contrainte verticale totale appliquée ainsi que la pression interstitielle, a été développé. Des essais de consolidation ont été réalisés sur des éprouvettes de kaolinite reconstituées en utilisant le nouvel appareil de consolidation avec drainage radial, ainsi que des essais avec drainage vertical. La technique d'analyse d'image numérique a été adoptée pour analyser les déformations de consolidation de l'éprouvette. Les déformations radiales ainsi que les déformations verticales ont été mesurées et comparées pour les deux conditions de drainage. Les hétérogénéités de l'éprouvette induites par le drainage horizontal ont été confirmées par la mesure de la contrainte verticale totale.

KEYWORDS: consolidation, vertical drain, horizontal deformation, digital image analysis

MOTS-CLÉS : consolidation, drainage vertical, déformation horizontale, analyse d'image numérique

1 INTRODUCTION

Soft clayey soil ground under preloading improvement with vertical drains has known to experience mostly vertical deformation with radial drainage. However, even though no radial displacement is expected in the present consolidation theory, progressive consolidation process from vertical drains induces void ratio variation in the radial direction, which is a strong evidence of radial inward displacements of soils toward drains (Yune 2005). These radial displacements during consolidation have already been recognized by many researchers. Pyrah and Tanaka (1999) and Atkinson et al. (1985) investigated possibility of horizontal movements of soils under radial drainage by measuring horizontal variations of water contents of soil specimens at the end of consolidation. Baek and Moriwaki (2004) measured directly the radial displacements during consolidation by monitoring the movements of magnets installed in the soil specimens.

Spatial variations of void ratio and excess pore pressure yield the stiffness variation of soil ground and result in vertical total stress change and excess pore pressure change under constant vertical preloading. Accordingly, monotonic dissipation of excess pore pressure is no longer possible, and consolidation coupled with total stress change and radial displacement will be the precise condition for soft ground under preloading with vertical drains.

To obtain the precise and detailed information on the soil behavior under vertical loading with radial drainage, physical model test equipment under plane strain condition to monitor the soil responses of entire zone under consideration throughout the whole consolidation process was developed. Soil movements during consolidation were determined by digital image analysis using photo images. Detailed strains and void ratios of entire soil specimen can be obtained. Also, variations of total stress and pore pressure were also measured in several locations in the specimen. Using reconstituted kaolinite specimen, a test with horizontal drainage under vertical loading was carried out, together with a test with vertical drainage. Results from both tests are comparatively analyzed.

2. EXPERIMENTAL PROGRAM

2.1. Test apparatus

The schematic diagram of the consolidation testing apparatus developed for this study is illustrated in Figure 1. The apparatus takes a rectangular parallelepiped specimen of 150 mm in height, 140 mm in width, and 40 mm in thickness, surrounded by four transparent acrylic walls which enable monitoring the internal movement of soils during consolidation. The photo images on the front wall are used for digital image analysis to find out the soil deformations during the entire consolidation process. On the opposite side, 6 pore pressure transducers at 2

different vertical locations and 3 different horizontal locations are installed to obtain the pore pressure distributions from the drain boundary. And 3 earth pressure gauges are mounted on the top rigid loading platen, which will provide the total vertical stress variations. A plate of porous plastics used for drain materials are placed on the top of the specimen for vertical drainage and at the side for horizontal drainage. The pressure lines were connected to top and side of model box, which allow saturation of the sample by applying back pressure and dissipate pore water.

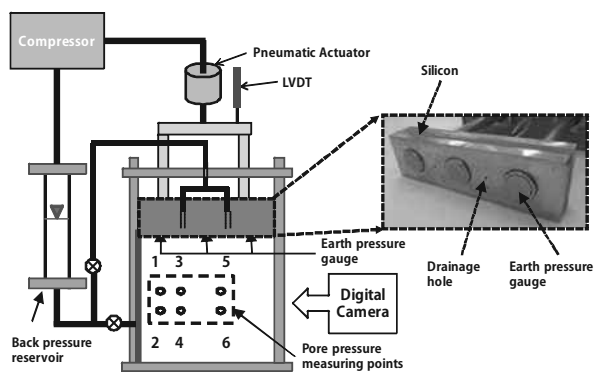


Figure 1. Schematic diagram of developed consolidation test apparatus

Although the axi-symmetric condition with vertical drains is in reality, the developed model box simulates plane strain condition to provide a more favorable environment to perform digital image analysis. So, if necessary, further study for modification of the interpretations of the results will be followed.

2.2. Soils

The reconstituted EPK Kaolinite was used for the test specimen. Its index properties are summarized in Table 1. The clay specimens were created using the slurry consolidation techniques described by Sheeran and Krizek (1971). Dried EPK Kaolinite powder was mixed with de-aired water until the water contents reach twice of the liquid limit of the Kaolinite. The prepared slurry was poured into a large consolidometer of which diameter is 0.3 m. The slurry was loaded with 10 kPa increments until the applied vertical pressure reaches 100 kPa. After reaching the target pressure, the vertical stress was kept constant for 7 to 10 days for the complete consolidation. After removing the vertical pressure, the sample was extruded carefully from the consolidometer, wrapped in plastic films, coated with paraffin wax, and stored in the controlled humidity and temperature storage room until ready for use. The maximum past pressures of the clay samples are determined as about 100 kPa, from the oedometer test results.

Table 1. Index properties of reconstituted kaolinite

Liquid limit (%)	64.4	USCS	MH
Plasticity index (%)	21.8	Initial void ratio	1.56
Percent finer than #200 sieve (%)	98	Initial water contents (%)	61.0
Specific gravity	2.55	Maximum past pressure (kPa)	100

2.3. Test procedure

The test procedure with the developed device for consolidation was summarized as; 1) trimming the reconstituted kaolinite sample, 2) creating image patterns on the surface of trimmed sample and 3) assembling test apparatus. Deformation analysis through image processing is basically done by using two

different images taken at two separate times. For this process, a random image pattern was created by spraying oil-based paint on one side of specimen as shown in Figure 2.

After assembling the apparatus with specimen, back pressure was applied for specimen saturation. Pore pressure parameter C value defined as the ratio of excess pore pressure increment to applied vertical total stress, higher than 0.97 was considered as full saturation. For the consolidation test, vertical stress increments by pneumatic pressure were applied on the top of the specimen. Consolidation process was achieved by applying vertical stress increment under undrained condition and then opening drainage line. 3 total stress increments (initial stress to 100 kPa, 100 kPa to 200 kPa, and 200 kPa to 300 kPa) were applied for both tests on horizontal drainage and vertical drainage condition. Initiating consolidation by opening the drainage valve, the values of pore water pressure and total vertical stress are stored via the data logger, and images are taken at regular time intervals, 10 seconds in the early stage of consolidation and increasing time steps as consolidation proceeds. A Nikon D90 digital camera, which has a resolution of 4288 x 2848 pixels, was used to get photo images.

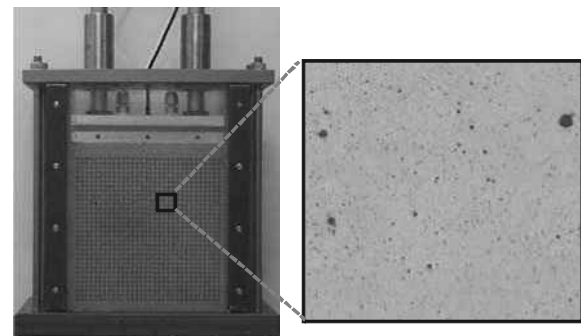


Figure 2. Prepared sample in the assembled test apparatus

The completion of consolidation was confirmed by both measuring pore pressure and interpreting time-settlement curve. After completion of all consolidation tests, water contents at 25 locations for both tests of drained direction were measured. In this study, to exclude the effect of stress history, final loading step (200 kPa to 300 kPa of vertical stress increment) was focused to be analyzed.

2.4. Digital image analysis

In the geotechnical application of the digital image analysis, two major techniques have been widely used; digital image correlation (Rechenmacher and Finno 2004) and particle image velocimetry (White et al. 2003). Herein, the Particle Image Velocimetry (PIV) technique was employed for the deformation measurement because of its better performance in convergence as well as the simplicity of the algorithm. To perform the image analysis, a flat object is supposed to deform 2-dimensionally in the plane of the image which can easily be satisfied in the developed consolidation apparatus which describes plane strain condition.

Kim et al. (2011) suggested a procedure determining optimum image processing condition by statistically analyzing the data of image analysis resulted from the original image of test specimen and manipulated one. Following this procedure, the lowest maximum error, which contains the value of accuracy and precision (Taylor 1999), of approximately 0.002 mm in 90% confidence level was obtained when using PIV with bi-square interpolation, 60 x 60 pixel subset size and 0.28 % of vertical strain interval. As shown in Figure 2, the adopted image in the test specimen is divided into 1024 (32x32) pixel subsets; thus, providing 1024 displacement vectors at the center points of the pixel subsets.

3. RESULTS AND DISCUSSION

3.1. Deformations

Figure 3 shows time-settlement curve for 200 kPa to 300 kPa vertical stress increment of both tests under vertical and horizontal drainages. Time for completion of consolidation in horizontal drainage based on excess pore pressure response is far shorter (3 times) than that for vertical drainage, as expected.

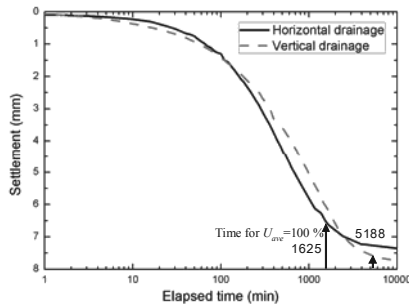


Figure 3. Time-Settlement curve of consolidation tests

The total consolidation process was divided into 18 and 20 stages for horizontal and vertical drainage condition to achieve the optimum condition of image analysis with a vertical strain interval of 0.28 %. The horizontal and vertical displacement increment contours were drawn for each section using 1024 displacement vectors resulting from image analysis. Interpretation results for horizontal drainage reveals 3 distinct stages based on inner soil displacements. Figure 4 to 6 show representative displacement increment contours under horizontal drainage for “early”, “intermediate” and “last” stages of consolidation, corresponding to 0 to 4%, 40 to 46%, and 78 to 82% of average degree of consolidation, U_{ave} , respectively.

In the early stage, significant horizontal displacements were observed, and they were increasing, approaching drained boundary. In the intermediate stage, only minor horizontal displacements were observed. However, in the last stage, even

though they were small, displacements to undrained boundary were observed. So, it is found out that significant horizontal displacements occurred in case of horizontal drainage condition under vertical compression and they were highly dependent on consolidation progress. Furthermore, horizontal displacement toward drained boundary is dominant throughout entire process of consolidation and it led into uneven void ratio distribution after completion of consolidation (decreasing void ratio as approaching drained boundary). Figure 7 indicating water content distribution after test provides its strong evidence, with lower values near drained boundary.

54.8	53	51.4	50.3	49.4
53.4	53.5	53.1	52.4	52.0
54.0	53.3	53.7	53.5	53.4
54.1	53.9	54.1	53.9	53.9
54.6	54.6	53.5	53.6	53.4

Figure 7. Final water content distribution of horizontal drainage

Horizontal displacements for horizontal drainage under vertical loading can be explained by progressive consolidation process from drained boundary. In the early stage, consolidation near drained boundary proceeds quickly with decreasing void ratio, and decrease of void ratio can be induced by horizontal soil displacements (i.e. soils imported from nearby zone) under equal vertical strain condition of test. In the intermediate stage, consolidation rate in the horizontal plane is not much different because of relatively uniform horizontal gradients of flow, and thus minor horizontal displacements are observed. But, in the last stage, dissipation of pore pressure remnant left in zone near undrained boundary invokes infinitesimal horizontal displacements inversely.

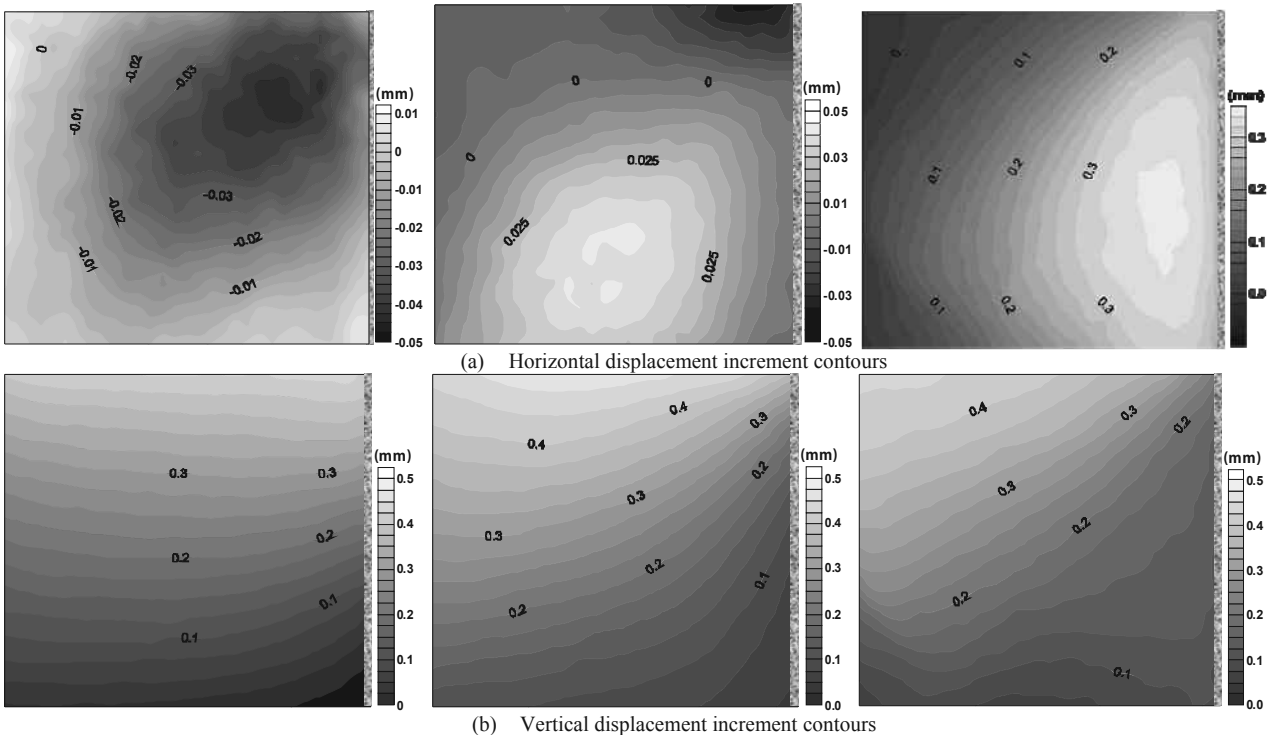


Figure 4. Early stage of consolidation ($U_{ave} = 0 \sim 4\%$)

Figure 5. Intermediate stage of consolidation ($U_{ave} = 40 \sim 46\%$)

Figure 6. Last stage of consolidation ($U_{ave} = 78 \sim 92\%$)

Meanwhile, for the vertical drainage, only minor scattering values of horizontal displacement increment less than 0.006 mm were observed, as shown in Figure 8. Furthermore, most of horizontal displacements were less than 0.002 mm, which is the maximum error in a 90% confidence level, and thus, it is estimated that horizontal displacements were hardly detectable under vertical drainage. Vertical displacement increment during consolidation also showed fairly uniform distribution along horizontal plane. Uniform vertical deformation and negligible horizontal deformation is maintained through the whole consolidation period.

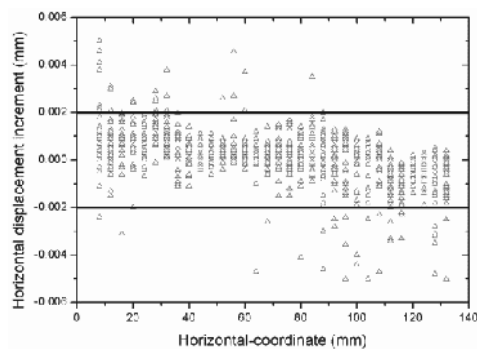


Figure 8. Horizontal displacement increment during consolidation under vertical drainage condition ($U_{ave} = 0 \sim 5\%$)

3.2. Total vertical stress

During the consolidation, the total vertical stresses were measured from 3 earth pressure gauges at different horizontal locations on the top of the specimen as shown in Figure 1. The variation of total vertical stress (measured total vertical stress minus initial total vertical stress, $\Delta\sigma_v$) with consolidation time is shown in Figure 9. In the case of vertical drainage, the vertical total stress maintains constant with minor scattering. On the other hand, in the case of horizontal drainage, total vertical stresses changed with consolidation time in a different manner for each location. At near drainage location, total vertical stress initially decreased and then increased at 35% of U_{ave} . At the end of consolidation, it reached net increase of total stress. Meanwhile, at mid-plane and far drainage locations, total stresses initially increase and then decrease. At the end, net decreases of total stresses were observed.

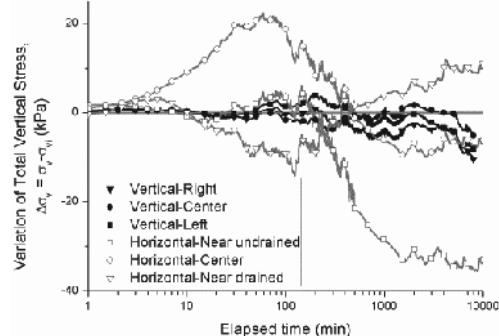


Figure 9. Variation of total vertical stress during consolidation

These total stress variations are strongly related to progressive consolidation process from drained boundary. As mentioned above, initial decrease of void ratio at near drainage induces increase of total stress nearby zone such as mid-plane location. And as consolidation progresses, void ratio of entire horizontal plane decreases and ideally reaches to the same void ratio value at completion of consolidation. In this case, even distribution of total vertical stress is expected. However, uneven void ratio distribution at the end (i.e.,

decreasing value of void ratio as approaching drained boundary) yields higher total vertical stress at near drainage and lower at far drainage.

4. CONCLUSIONS

In this study, consolidation tests were performed under the condition of horizontal and vertical drainage using the newly developed consolidation apparatus. The developed equipment can monitor inner soil deformations visually and measure the pore water pressure distributions inside the specimen.

Results of digital image analysis and vertical total stress measurement during the experiment are briefly presented as follows. Horizontal displacements for horizontal drainage under vertical loading were predominantly observed. In the early stage of consolidation, significant horizontal displacements to the drainage were observed, and thereafter it is decreasing as consolidation proceeds, and at the last stage of 70 % or more of U_{ave} , relatively small amount of horizontal displacements in opposite direction were monitored. On the other hand, only minor scattering values of horizontal displacement were measured for vertical drainage condition. The induced non-uniformity was confirmed by measuring water content at the end of the consolidation: as approaching to the drainage surface, the measured water contents were decreasing.

The total vertical stress in the horizontal plane measured by three earth pressure gauges showed constant values in vertical drainage conditions. In the horizontal drainage, total vertical stress near drainage boundary initially decreased and then increased, reaching net increase (12 %). Meanwhile, at mid-plane and far drainage locations, total stresses initially increased and then decreased, finally having net decrease. (7, 30% respectively)

5. REFERENCES

- Atkinson, J. H., Evans, J. S., and Ho, E. W. L. 1985. Non-uniformity of triaxial samples due to consolidation with radial drainage. *Geotechnique*, Vol. 35, No. 3, pp. 353-355.
- Baek, W. and Moriwaki, T. 2004. Internal behavior of clayey ground improved by vertical drains in 3D-consolidation process. *Soils and Foundations*, Vol. 44, No. 3, pp. 25-37.
- Kim, J. Y., Jang, E. R., and Chung, C. K. 2011. Evaluation of accuracy and optimization of digital image analysis technique for measuring deformation of soils. *Journal of KGS*, Vol. 27, No. 7, pp.5-16. Korean
- Pyrh , I. C., Smith, I. G. N., Hull, D., and Tanaka, Y. 1999. Non-uniform consolidation around vertical drains installed in soft ground. *Proc. Of 12th European Conference on Soil Mechanics and Geotechnical Engineering*, pp.1563-1569.
- Rechenmacher, A. L. and Finn , R. J. 2004. Digital image correlation to evaluate shear banding in dilative sands. *Geotechnical Testing Journal*, Vol. 27, No. 1, pp. 13-22.
- Sheeran, D. E. and Krizek, R. J., 1971. Preparation of homogeneous soil samples by slurry consolidometers. *Journal of Materials*, Vol. 6, No. 2, pp. 356-373.
- Taylor, J. R. 1999. *An introduction to error analysis: The study of uncertainties in physical measurements*. University Science Books, pp.128-129.
- White, D. J., Take, W. A., and Bolton, M. D. 2003. Soil deformation measurement using Particle Image Velocimetry (PIV) and photogrammetry. *Geotechnique*, Vol. 53, No. 7, pp. 619-631.
- Yune, C.Y. 2005. Influence of void ratio variation on consolidation behavior of clayey soil with vertical drains. Ph.D thesis, Seoul National University.

Mise au point d'un dispositif expérimental pour l'analyse du retrait-gonflement des sols argileux

Development of an experimental device for swelling-shrinkage analysis of clayey soils

Maison T.

Institut Polytechnique LaSalle Beauvais, Beauvais, France

Laouafa F., Delalain P.

INERIS, Verneuil-en-Halatte, France

RÉSUMÉ : L'article présente un nouveau dispositif expérimental, pouvant être incorporé dans un Microscope Electronique à Balayage Environnemental (MEBE). Il permet de mesurer simultanément l'évolution de la déformation surfacique d'agrégats argileux et l'évolution de leur teneur en eau. L'état initial des échantillons est sous forme de poudre, ce qui permet d'effectuer un cycle d'humidification-séchage complet en 24 heures, soit beaucoup plus rapide que les essais classiques à l'échelle mésoscopique. Les résultats obtenus semblent permettre de dissocier les argiles homogènes et/ou « industrielles » des argiles naturelles hétérogènes par l'évolution de leur déformation surfacique et de leur teneur en eau. Ces observations permettent d'envisager des applications concrètes telles que la corrélation des tassements d'un massif aux profils d'évolution de la teneur en eau (fonction de la profondeur) ou l'optimisation du choix des bentonites utilisées pour les scellements des stockages de déchets radioactifs.

ABSTRACT: The paper presents a new experimental device, which can be incorporated in an Environmental Scanning Electron Microscope (ESEM). It allows to measure simultaneously the evolution of the surface strain of clayey aggregates and the evolution of their water content. The initial state of samples is in powder form, what allows making a complete cycle of wetting-drying in 24 hours, faster than the classic tests at the mesoscopic scale. The obtained results seem dissociate homogeneous and/or "industrial" clays than heterogeneous natural clays with the evolution of their surface strain and their water content. These observations allow to envisage concrete applications such as the correlation of the collapses of a massif with the profiles of evolution of the water content (function of the depth) or the optimization of the choice of bentonites used for the sealings of the storages of radioactive waste.

MOTS-CLÉS : MEBE, sols, argiles, micro-pesée.

KEYWORDS : ESEM, soils, clays, micro-weighing.

1 INTRODUCTION

L'amélioration de la connaissance des mécanismes physico-chimiques mis en jeu à l'échelle locale lors du retrait et du gonflement des matériaux argileux est une étape incontournable pour la caractérisation de la sensibilité de ces sols et de leur prédisposition vis-à-vis du phénomène de retrait/gonflement.

Le retrait-gonflement des sols argileux exige, pour l'évaluation de ses effets sur une zone donnée, des paramètres obtenus par des essais en laboratoire, menés sur des échantillons de plusieurs centimètres cubes. La faible cinétique des sols argileux implique des durées de tests jusqu'à trois mois pour certains d'entre-eux.

Aujourd'hui, cet inconvénient peut être surmonté avec l'utilisation du Microscope Electronique à Balayage Environnemental (MEBE). Le contrôle des conditions hygrométriques dans la chambre du MEBE permet l'analyse de l'échantillon dans des conditions hygrométriques in situ (Romero et Simms 2008). Les échantillons peuvent être étudiés sous forme de poudre, ce qui diminue la durée du test à 24 heures pour les argiles.

Les analyses menées au MEBE ces dernières années ne concernaient que des argiles industrielles type bentonite MX80 (Montes-H. 2002). Cet article présente des observations au MEBE effectuées sur des argiles industrielles mais aussi sur des argiles naturelles (prélevées in situ), afin d'étudier le comportement du retrait-gonflement dans les sols argileux. Cinq argiles sont présentées : une argile naturelle homogène commercialisée et pouvant être utilisée dans l'industrie, la

montmorillonite grecque ; une argile « industrielle », la bentonite MX80 ; trois argiles naturelles hétérogènes, l'argile verte de Romainville, la marne de Mormoiron et l'argile des Flandres.

Le but de cet article est de proposer l'observation et l'analyse de l'évolution du retrait-gonflement de ces argiles en fonction de leur teneur en eau. A cette fin, un dispositif expérimental a été développé pour peser l'échantillon sous forme de poudre tout en permettant l'observation des agrégats d'argile. Via la mesure de l'évolution du poids et la teneur en eau initiale, l'évolution de la teneur en eau peut être obtenue et reliée à l'évolution de la déformation surfacique des agrégats argileux.

A partir de ces résultats, quelques applications peuvent être imaginées, telles que la corrélation avec des mesures d'évolution de la teneur en eau in situ en fonction de la profondeur. L'extrapolation de l'échelle microscopique du MEBE à l'échelle macroscopique des structures pourrait être établie pour l'évaluation du retrait-gonflement, par exemple sur une zone donnée. Une autre application, dans le cas des argiles « industrielles », serait l'étude et la modélisation du gonflement des bentonites dans le cadre du stockage des déchets radioactifs.

2 PRINCIPALES CARACTÉRISTIQUES DES ARGILES ÉTUDIÉES

La montmorillonite grecque (MG) provient de l'île de Milos, dans l'archipel des Cyclades, et présente une origine volcanique. Comme son nom l'indique, elle est composée principalement de smectite, une espèce argileuse gonflante.

La bentonite MX80 provient du laboratoire de recherche souterrain de Meuse/Haute-Marne de l'ANDRA qui l'utilise dans le cadre de ses recherches sur les scellements des centres de stockage de déchets radioactifs haute activité vie longue (HAVL). Elle est composée principalement de montmorillonite, mais également de quartz, de muscovite et d'albite en phases mineures.

L'argile verte de Romainville (AVR) provient de l'Est de Paris (France), et présente une origine sédimentaire. Les espèces argileuses en présence sont la kaolinite, l'illite, des interstratifiés illite/smectite et elle contient également des carbonates, du quartz et du feldspath.

La marne de Mormoiron (MM) provient de la commune de Mormoiron, dans le Sud-Est de la France, et présente une origine fluviale. Les espèces argileuses en présence sont la montmorillonite et des interstratifiés muscovite/illite et elle contient également des carbonates et du quartz.

L'argile des Flandres (AF) provient de la commune de Merville, dans le Nord de la France, et présente une origine fluviale. Les espèces argileuses en présence sont la montmorillonite, la kaolinite et l'illite, et elle contient également du quartz et des carbonates.

Les principales caractéristiques de ces argiles sont présentées dans le tableau 1. Les valeurs de teneur en eau initiales sont celles mesurées après broyage (< 200 µm) dans les conditions environnementales du laboratoire.

Tableau 1. Caractéristiques géotechniques des argiles étudiées (d'après Fleureau *et al.* 1993, Christidis *et al.* 1997, Audiguier *et al.* 2007, Makki *et al.* 2008, Montes-H. 2002, Plat *et al.* 2009).

	MG	MX80	AVR	MM	AF
Limite de liquidité, w_L , %	170	-	75	32,9	90
Limite de plasticité, w_P , %	60	-	40	14,5	45
Indice de plasticité, I_p	110	-	35	18,4	45
Surface spécifique, m^2/g	61,76	33	195	-	-
CEC, meq/100 g	104,4	-	39,5	-	-
Teneur en eau initiale (%)	12	7,67	0,04	3,96	2,27

3 ANALYSE DU RETRAIT/GONFLEMENT AU MICROSCOPE ELECTRONIQUE À BALAYAGE ENVIRONNEMENTAL (MEBE)

3.1 Présentation du MEBE

Le Microscope Electronique à Balayage Environnemental (MEBE) permet d'observer des échantillons dans des conditions environnementales (température, pression, hygrométrie) contrôlées. Le gaz, injecté dans la chambre du MEBE lors de nos expériences, est la vapeur d'eau, qui joue à la fois le rôle de gaz d'ionisation (Figure 1) pour la formation de l'image et de fluide d'hydratation de l'échantillon.

Le mode « environnemental » permet l'observation des échantillons sans aucune préparation préalable (métallisation à

l'or ou au carbone, indispensables en mode classique) et ainsi de préserver la structure des échantillons et leur teneur en eau naturelle. Par effet Peltier, il est possible d'imposer une température à l'échantillon et, par l'intermédiaire du contrôle de la pression dans la chambre d'observation, d'effectuer des analyses sous hygrométrie contrôlée. Le MEBE ne permet toutefois que des observations microscopiques de surface. Les essais réalisés peuvent être monotones (hydratation, séchage) avec ou sans palier, mais également cycliques.

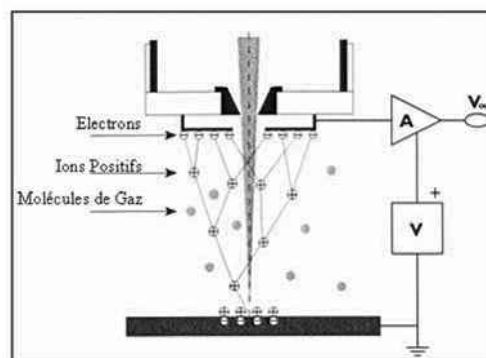


Figure 1. Représentation schématique de l'ionisation d'un gaz dans la chambre du MEBE (Montes-H. 2002).

3.2 Développement d'un dispositif expérimental

Le MEBE permet des observations microscopiques, des analyses chimiques et des mesures linéaires ou surfaciques d'éléments grâce à un logiciel de traitement d'image. Cependant, toute mesure physique sur l'échantillon (poids, teneur en eau...) est impossible. C'est pourquoi un nouveau dispositif expérimental, breveté, a été développé dans le MEBE, afin de mesurer la teneur en eau de l'échantillon et la déformation surfacique des agrégats simultanément (Kazmierczak *et al.* 2011). Le principe est de peser l'échantillon et d'obtenir une évolution et une intensité précises de la teneur en eau en fonction de l'humidité relative. Pour cela, le dispositif expérimental est constitué d'un capteur de force couplé à un bras de levier afin d'amplifier l'évolution du poids (Figure 2B). Le bras de levier est assez long pour obtenir une amplification suffisante et est réalisé en aluminium. Son extrémité contient une coupelle avec une cavité où le microéchantillon de sol (sous forme de poudre) de très petit volume (quelques millimètres cube) est déposé et un plateau pour la zone d'observation. Pour concentrer l'humidité relative sur l'échantillon, l'extrémité du bras de levier est entourée d'un doigt de confinement en cuivre qui est relié à la platine Peltier du MEBE (Figure 2A). Faute d'une place suffisante, une partie du dispositif doit être externalisée par rapport à la chambre du MEBE. A cet effet et pour maintenir les conditions environnementales (pression et température), une enceinte en aluminium est installée sur le MEBE (Figure 2C). Une centrale d'acquisition permet de suivre simultanément plusieurs paramètres dans la chambre du MEBE :

- la température à l'intérieur du doigt de confinement (capteur PT100) ;
- la température à l'intérieur de la chambre du MEBE (capteur PT100) ;
- la température de la platine Peltier (donnée par le MEBE) ;
- la pression à l'intérieur de la chambre du MEBE (jauge de pression) ;
- l'évolution de poids de l'échantillon (capteur de force) (Figure 3).

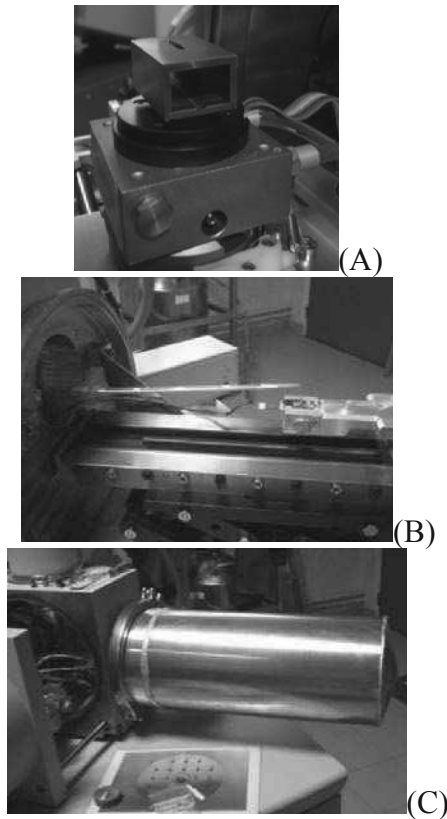


Figure 2. Configuration du dispositif expérimental dans la chambre du MEBE : doigt de confinement (A) ; capteur de force et bras de levier (B); fermé avec l'enceinte (C).

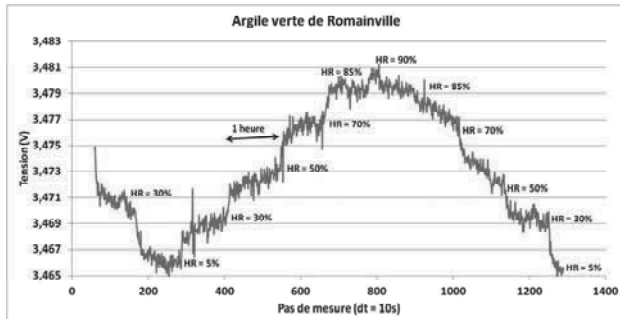


Figure 3. Mesure du capteur de force : évolution du poids de l'échantillon en fonction du temps. Exemple de l'argile verte de Romainville.

3.3 Protocole expérimental

Les argiles naturelles présentent une cinétique plus lente que les argiles industrielles car elles présentent une hétérogénéité minéralogique que ne présentent pas les argiles industrielles. De fait, un protocole expérimental a été établi afin de respecter cette cinétique naturelle.

Le protocole expérimental mis en œuvre lors de l'analyse de ces agrégats consiste à appliquer graduellement des niveaux d'humidité relative avec des paliers de temps d'équilibre régulier (de 10 à 60 minutes selon le type d'argile), afin de laisser suffisamment de temps pour que les processus d'hydratation et de déshydratation atteignent un état d'équilibre au sein des agrégats argileux. Le taux d'humidification est ainsi plus lent et régulier et semble plus proche des conditions in situ. Le protocole expérimental comporte deux phases : pendant la première, les paliers de temps sont appliqués pour différents incréments d'humidité relative (HR) (incrément de 10 % d'HR jusqu'à 85 % d'HR). Dans la seconde phase, les paliers de temps sont appliqués pour différents incréments de pression

(incrément de 10 Pa, 5 Pa et 1 Pa) (Figure 4). Avec un tel protocole, une bonne précision dans l'hydratation des agrégats argileux et dans les mesures de gonflement aux fortes humidités relatives, peut être obtenue. L'immersion de la zone d'observation est retardée grâce au taux d'hydratation modéré, rendant ainsi possible l'analyse des images acquises lors de l'essai.

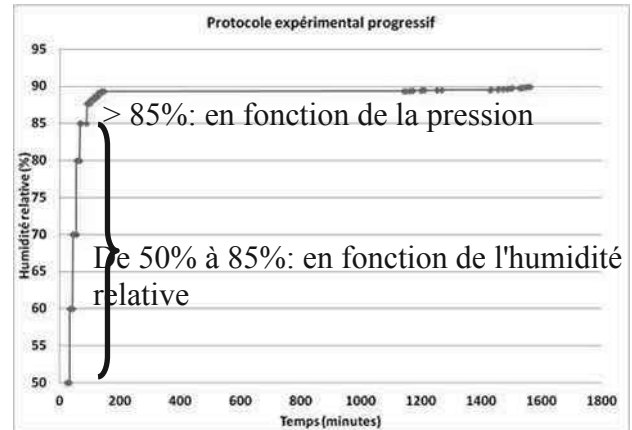


Figure 4. Humidité relative en fonction du temps : protocole expérimental utilisé pour les différentes argiles.

Les argiles sont analysées sous forme de poudre (granulométrie inférieure à 200 μm) et soumises à un cycle d'humidification-séchage. Afin d'effectuer des observations au MEBE, la poudre est déposée sur un scotch double face carbone pour éviter le déplacement des agrégats pendant l'humidification. La température est maintenue constante à 18°C à l'intérieur du doigt en cuivre et la pression est contrôlée pour obtenir l'humidité relative désirée. L'hygrométrie initiale dans la chambre du MEBE est d'environ 50 % (celle du laboratoire). Le cycle d'humidification-séchage est appliqué sur l'ensemble de l'échantillon mais seule une zone est observée.

L'évolution de la surface apparente des agrégats argileux est suivie en temps réel. La déformation surfacique induite par le retrait et le gonflement est déduite de l'analyse des images acquises (de 1500 à 2000 photos en moyenne) avec le logiciel Scandium (Soft Imaging System). Le terme « déformation surfacique » est utilisé pour définir le changement relatif de surface des agrégats au temps t_i par rapport à l'état initial (Figure 5) :

$$\varepsilon_s = \frac{S_i - S_0}{S_0} = \frac{\Delta S}{S_0} \quad (1)$$

où S_i = déformation surfacique au temps t_i ; S_0 = surface initiale de l'agrégat.

La déformation volumique ε_v est égale à :

$$\varepsilon_v = \alpha \varepsilon_s \quad (2)$$

où $\alpha > 1$ est fonction du degré d'isotropie ou d'anisotropie du gonflement ou du retrait.

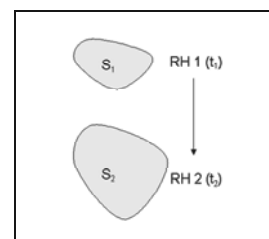


Figure 5. Définition de la déformation sur la base des paramètres du temps, de la surface et de l'humidité relative des agrégats argileux

4 ANALYSE EXPÉRIMENTALE

Ces expériences permettent donc de mesurer deux paramètres intimement liés : l'évolution de la déformation surfacique et l'évolution de la teneur en eau en fonction de l'humidité relative. La qualité de la relation dépend fortement de la quantité de sol, du temps d'équilibre des paliers et de la stabilité des conditions environnementales pendant l'expérience. Ces deux courbes peuvent être corrélées pour obtenir l'évolution de la déformation surfacique en fonction de la teneur en eau (Figure 6).

Sur la base de nos essais, nous observons que, pour la même taille d'agrégats argileux (50 μm) et un temps d'équilibre moyen de 30 minutes, le gonflement serait variable selon le type d'argile. Les argiles homogènes et/ou « industrielles » (MG et MX80) présenteraient une déformation surfacique et une teneur en eau élevées. En revanche, les argiles naturelles hétérogènes présenteraient une déformation surfacique et une teneur en eau plus faibles.

Rappelons que le premier groupe se caractérise par une composition principalement constituée de smectite et le deuxième de divers minéraux (kaolinite, illite, interstratifiés illite/smectite, carbonates, quartz, feldspath). Le gonflement peut se développer librement dans les argiles « industrielles » grâce à la smectite (ou montmorillonite). En revanche, le gonflement est probablement gêné dans les argiles naturelles hétérogènes à cause de deux paramètres : (i) la présence de kaolinite et illite, qui sont des espèces argileuses avec des liaisons fortes; (ii) les carbonates connus pour constituer une cimentation (Maison et al. 2010a). Cependant, l'allure générale des courbes paraît semblable, montrant que la majeure partie des déformations surfaciques se produirait aux fortes humidités relatives (supérieures à 75 % d'humidité relative). Il peut être souligné que les gonflements mesurés pour les argiles naturelles hétérogènes dans les observations au MEBE, semblent similaires aux gonflements mesurés avec des essais œdométriques de gonflement libre classiques (Maison et al. 2010b). Nous retrouvons la différence entre les deux types d'argiles avec la gamme de teneur en eau (jusqu'à 32 % pour la bentonite MX80; jusqu'à 12 % pour l'argile verte de Romainville) probablement en raison de la présence de smectite qui permettrait une évolution de la teneur en eau plus importante.

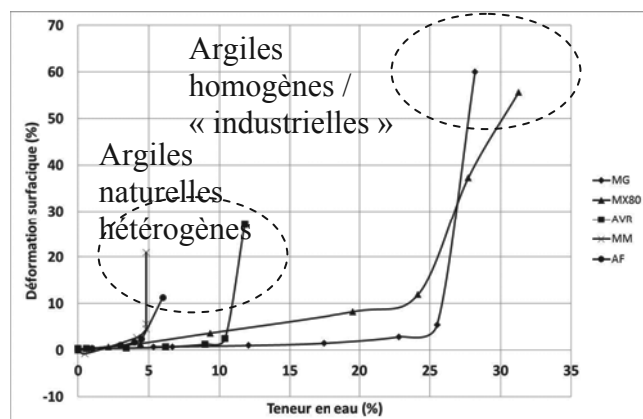


Figure 6. Courbe reliant la variation de la déformation surfacique à la teneur en eau pour les différentes argiles étudiées.

5 CONCLUSION ET PERSPECTIVES

L'étude du retrait-gonflement des sols argileux est réalisée de nos jours principalement par des essais en laboratoire qui requièrent plusieurs semaines pour obtenir des résultats. Nous proposons une approche différente qui consiste en l'étude des sols argileux dans un MEBE avec de petites quantités

d'échantillon. La durée de l'expérience est ainsi diminuée (jusqu'à 24 heures). Les essais consistent à mesurer l'évolution de la déformation surfacique en fonction de la teneur en eau grâce à un nouveau dispositif expérimental. Les résultats obtenus sur les argiles naturelles hétérogènes sont cohérents avec ceux obtenus à l'échelle mésoscopique (essais en laboratoire).

A partir de ces résultats, il est possible d'envisager quelques applications avec des approches simples afin d'extrapoler ces résultats à l'échelle macroscopique des structures. Une approche analytique, basée sur des mesures in situ d'évolution de la teneur en eau en fonction de la profondeur, pourrait permettre l'évaluation rapide du tassement différentiel sous une structure. Elle pourrait être utilisée par des laboratoires ou des bureaux d'étude comme une approche rapide qui peut être basée sur des données facilement accessibles. Une approche numérique, basée sur l'analogie avec le comportement thermomécanique, permettrait, quant à elle, de mener des simulations numériques sur plusieurs structures soumises au retrait-gonflement. Les argiles « industrielles », telles que la bentonite MX80, sont utilisées dans de nombreux domaines, notamment celui du stockage de déchets radioactifs en guise de scellements au niveau des alvéoles de stockage. Ces essais pourraient permettre d'optimiser le choix de la meilleure bentonite pour cet usage ainsi que l'influence de la chaleur dégagée par les déchets sur le comportement de la bentonite.

6 REFERENCES BIBLIOGRAPHIQUES

- Audiguier M., Geremew Z., Laribi S., Cojean R. 2007. Caractérisation au laboratoire de la sensibilité au retrait-gonflement des sols argileux. *Revue française de Géotechnique* 120-121, 67-82.
- Christidis G.E., Scott P.W., Dunham A.C. 1997. Acid activation and bleaching capacity of bentonites from the islands of Milos and Chios, Aegean, Greece. *Applied Clay Science* 12, 329-347.
- Fleureau J.M., Kheirbek-Saoud S., Soemito R., Taibi S. 1993. Behavior of clayey soils on drying-wetting paths. *Canadian Geotechnical Journal* 30, 287-296.
- Kazmierczak J.B., Maison T., Delalain P., Laouafa F. 2011. Dispositif et procédé de caractérisation d'un matériau. Brevet 10-54771, 14 juin 2011.
- Makki L., Duc M., Droniuc N., Reiffsteck P., Maloula A., Magnan J.P. 2008. Essai de retrait pour une meilleure classification de la sensibilité des sols à la sécheresse. SEC2008, Paris, France, 1-3 septembre, 257-264, Editions du LCPC.
- Maison T., Laouafa F., Fleureau J.M., Delalain P. 2010a. Analyse au niveau microscopique du comportement hydrique d'agrégats argileux. Journées nationales de géotechnique et de géologie de l'ingénieur (JNGG 2010), 7-9 juillet 2010, Grenoble.
- Maison T., Laouafa F., Fleureau J.M. 2010b. Volume changes of swelling clayey soils at microscopic scale level. UNSAT 2010, 6-8 septembre 2010, Barcelona, Spain.
- Montes-H. G. 2002. Etude expérimentale de la sorption d'eau et du gonflement des argiles par microscopie à balayage environnementale (ESEM) et l'analyse digitale d'images. Strasbourg, Ecole et observatoire des sciences de la terre.
- Plat E., Le Roy S. et Vincent M. (BRGM), Audiguier M., Cojean R., Geremew Z. et Laribi S. (Armines), Tang A.-M., Cui Y.-J., Ta A.-N. et Hemmati S. (Cermes), Fabre R. et Chrétien M.-C. (GHYMAC), Pantet A., Proust D. et Fontaine C. (Université de Poitiers), Masroufi F., Mrad M. et Nowamooz H. (LAEGO), Magnan J.P., Duc M. et Makki L. (LCPC), Fleureau J.-M., Guellati Z., Nguyen D. et Souli H. (LMSSMat), Djeran-Maire I. et Pothier C. (Insavalor), avec la collaboration de Logeais P. (2009) – Projet ARGIC (module B, tâches 16 et 17) – Caractérisation des formations géologiques régionales sujettes au retrait-gonflement étudiées dans le cadre du projet. Annexe B16 au rapport de synthèse du projet ARGIC. Rapport BRGM/RP-57223-FR, 204 p., 1 ill., 12 fiches.
- Romero E. et Simms P.H. 2008. Microstructure Investigation in Unsaturated Soils: A Review with Special Attention to Contribution of Mercury Intrusion Porosimetry and Environmental Scanning Electron Microscopy. *Geotechnical Geological Engineering* 26, 705-727.

Residual shear strength behavior of swelling soils

Comportement de résistance résiduelle au cisaillement des sols gonflants

Markou I.N.

Democritus University of Thrace, Xanthi, Greece

ABSTRACT: Three clay soils from North-Eastern Greece, presenting “high” or “very high” swelling potential, were used in this investigation. The residual strength behavior of these soils was evaluated by performing ring shear tests on remolded specimens. The specimens were prepared either with the optimum water contents resulted from Standard compaction test (26% - 31%) or with higher water contents (46% - 47%), derived from the specimen moistures at the end of the preceding tests with optimum water contents. Ring shear tests were performed using a Bromhead apparatus, according to the procedures described in BS 1377 – Part 7. The results obtained from the consolidation stage of the ring shear tests conducted in swelling clays, do not present the typical form attained in ordinary clays. Although the failure envelopes in residual state present curvature in some cases, they can be considered as linear for effective normal stresses up to 250 kPa. The resulting values of residual friction angle do not exceed 14° and, in some cases, are affected by the water content of the specimens used in the tests.

RÉSUMÉ: Trois sols argileux provenant du Nord-Est de la Grèce et présentant un potentiel de gonflement « haut » ou bien « très haut », ont été utilisés dans le cadre de ce travail. Le comportement de la force résiduelle de cisaillement a été évalué à l'aide des essais de cisaillement annulaire sur des échantillons remués. Les échantillons étaient préparés soit avec le pourcentage de contenu de l'eau optimal, résultant par l'essai de compactage standard (26 %-31 %), soit avec des pourcentages de contenu de l'eau plus élevés (46 %-47 %) qui dérivent par l'humidité des échantillons à la fin des essais précédents avec des contenus de l'eau optimaux. Les essais de cisaillement annulaire ont été réalisés à l'aide de l'appareil de Bromhead, selon la procédure décrite dans BS 1377-Partie 7. Les résultats obtenus par le stage de consolidation du cisaillement annulaire, conduits sur des argiles gonflantes, n'atteignent pas la forme classique des argiles ordinaires. Bien que l'enveloppe de rupture présente une courbe dans quelques cas, elles peuvent être considérées comme linéaires pour des contraintes normales effectives allant jusqu'à 250 kPa. Les valeurs résultantes de l'angle de friction résiduelle, n'excèdent pas 14° et, dans quelques cas, elles sont affectées par le contenu de l'eau des échantillons utilisés lors des essais.

KEYWORDS: soil behaviour, swelling soils, expansive clays, residual shear strength, laboratory investigation, ring shear tests

1 INTRODUCTION

The residual shear strength of cohesive soils plays a part in the stability of old landslips, in the assessment of the engineering properties of soil deposits which contain pre-existing shear surfaces, and in the assessment of the risk of progressive failure in stability problems in general. Extensive investigations on the residual strength have been carried out and directed towards: (a) developing suitable laboratory techniques for its measurement, (b) identifying the influence of geomechanical features on the residual strength, (c) understanding the basic mechanisms involved in the mobilization of shear strength during the residual stage of deformation and (d) establishing correlations with soil index properties. Although the residual strength has been studied in the laboratory using experimental techniques such as the triaxial test procedure proposed by Chandler (1966), the ring shear test and the multi-reversal direct shear test are widely used to measure the residual strength of soils. Published results (Lupini et al. 1981, Skempton 1985) show that residual shear behavior changes significantly as the clay content of cohesive soil increases, and that a change in shearing mechanism also occurs. Apart from the clay fraction, the mineralogy of the clay also has an effect on residual strength, especially when the clay fraction is large (Kaltefleiter 1993). A number of correlations between residual strength and clay fraction, plasticity index and liquid limit have been proposed (e.g. Skempton 1964, Lupini et al. 1981, Hawkins and Privett 1985, Skempton 1985, Mesri and Cepeda-Diaz 1986), but

Lupini et al. (1981) suggested that all these correlations can not be general.

Soils containing expansive clay minerals, called swelling or expansive soils, have created problems of uplift and instability on many structures, because there is an opportunity for water to become available and thus facilitate the expansion (swelling) of the clay minerals. Swelling clays are often subject to extreme changes in shear strength because of extreme moisture changes. In addition to the strength factors related to the minerals involved, the interrelation of moisture, density and load plays an important part in the strength (Gibbs et al. 1960). Swelling soils also exist in Greece and have created a number of problems and/or failures in projects (Christodoulas and Gasios 1987, Stamatopoulos et al. 1989). Therefore, the properties and the swelling characteristics of swelling soils from Greece were investigated (Xeidakis 1993, Tsiambaos and Tsiligopoulos 1995, Kollaros and Athanasopoulou 1997), preventive and corrective measures against swelling were applied (Christodoulas and Gasios 1987, Stamatopoulos et al. 1989) and the treatment of swelling soils using various methods for reducing swell potential and increasing strength was examined (Stamatopoulos et al. 1992). It is, therefore, of merit to investigate the residual shear strength behavior of swelling soils. Toward this end, a laboratory investigation was conducted in order to evaluate the residual shear strength parameters of selected swelling soils having different moisture contents and the results obtained and observations made, are reported herein.

2 SOIL PROPERTIES

Three soils from the region of Thrace (North-Eastern Greece) were used in this investigation because of their swelling characteristics. According to the properties presented in Table 1, all three soils can be considered as clay soils since the clay fraction (grain sizes <0.002 mm) ranges from 70% to 80%. The P1-S2 and P2-S2 soils are classified as CH, while the P2-S1 soil is classified as MH in accordance with the Unified Soil Classification System. The values of maximum dry unit weight, γ_{dmax} , and optimum moisture content, w_{opt} , were obtained by conducting compaction tests with standard compaction effort.

All three soils present “high” or “very high” swelling potential, according to the known correlations of soil index properties with swelling characteristics (Papakyriakopoulos and Koudoumakis 2001). One-dimensional swell tests (ASTM D4546, Method A) were conducted using laboratory-compacted specimens (ASTM D698) of these soils (Koudoumakis 2000). The results obtained from specimens with initial moisture contents similar to the optimum moisture contents of the soils (Table 1), are presented in Table 2. It can be observed that the swell pressure ranges from 170 kPa to 820 kPa and the free swell ranges from 11% to 19%. These values are indicative of the swelling potential of the soils used in this investigation.

3 EXPERIMENTAL PROCEDURES

The residual shear strength behavior of the soils was evaluated by performing ring shear tests on remolded specimens. The use of remolded specimens allowed the adequate control of the specimen moisture content. The specimens were prepared with the moisture contents shown in Table 3. At first, the optimum moisture content, resulted from the Standard compaction test (Table 1), was used for each soil. The following specimens were prepared with larger moisture content, obtained for each soil as the average value of the specimen moisture contents at the end of the preceding tests started with optimum water content. The specimens were placed in the cell of the ring shear apparatus by kneading the soil with the desired moisture content evenly to fill

Table 1. Properties of soils.

Soil designation		P1-S2	P2-S1	P2-S2
Sampling depth (m)		1.5-3.0	0.0-1.6	1.6-3.0
Specific gravity G_s		2.82	2.68	2.73
Grain size analysis	Sand (%)	11.5	5.6	3.8
	Silt (%)	9.3	23.9	25.8
	Clay (%)	79.2	70.5	70.4
Atterberg limits	Liquid limit w_L	86	82	87
	Plasticity index I_p	63	42	55
Compaction characteristics	Maximum dry unit weight γ_{dmax} (kN/m ³)	13.80	13.95	14.27
	Optimum moisture content w_{opt} (%)	31.2	27.0	25.8

Table 2. Typical results of one-dimensional swell tests.

Soil designation		P1-S2	P2-S1	P2-S2
Initial moisture content w_0 (%)		30.63	31.13	24.02
Initial void ratio e_0		0.990	0.946	1.054
Initial degree of saturation S_{r0} (%)		87.25	88.19	61.08
Dry unit weight γ_d (kN/m ³)		13.87	13.52	12.81
Swell pressure P_s (kPa)		820	335	170
Specimen height increase Δh (mm)		3.85	2.20	2.52
Free swell $\Delta h/h_0$ (%)		19.23	11.60	13.28

Table 3. Testing program.

Soil	Moisture content w (%)	Effective normal stress σ'_n (kPa)			
		25	75	250	800
P1-S2	31.2	25	75	250	800
	45.8	25	50	100	200
P2-S1	27.0	25	75	200	600
	47.1	25	50	100	200
P2-S2	25.8	25	75	200	600
	46.4	25	50	100	200

the annular cavity between the confining rings of the cell, using a small spatula (BS 1377 – Part 7).

Ring shear testing was based on the procedure described in BS 1377 – Part 7. The tests were conducted using a Bromhead ring shear apparatus (Bromhead 1979) and annular specimens of 5 mm thickness with internal and external diameters of 70 mm and 100 mm, respectively. The specimens were consolidated for a period of 24 hours under the effective normal stresses, σ'_n , presented in Table 3 and, subsequently, were sheared at a constant rate of angular displacement equal to 0.048 degrees/min. The selection of this rate of angular displacement was dictated by the unconventional results of the consolidation stage of the tests, described in the next section, and was based on the fact that this rate has been found satisfactory for a large range of soils (BS 1377 – Part 7). One of the objectives of the present study was to investigate the residual strength behavior of the soils for a wide range of effective normal stresses reaching or even exceeding the values of swell pressure shown in Table 2. Although this goal was accomplished for the specimens prepared with the optimum moisture contents (Table 3), the use of effective normal stresses larger than 200 kPa was not feasible in the tests performed with soil moisture contents ranging from 46% to 47% because of excessive specimen loss during testing.

4 RESULTS AND DISCUSSION

Typical “specimen length change” – “log time” curves obtained from the consolidation stage of ring shear tests conducted with optimum moisture content and with moisture content ranging from 46% to 47%, are shown in Figures 1a and 1b, respectively. It can be observed (Figure 1a) that, in several cases, the curves do not present the usual form attained in ordinary clays, either due to the expansion of specimens or because the consolidation was not completed within the predetermined period of 24 hours. It can also be observed (Figure 1b) that, in general, the classic type of curves appears in the test performed under the highest effective normal stress of each test series. The overall behavior of the three soils in the consolidation stage of the ring shear tests is summarized in Table 4. More specifically, the tests are divided in those exhibited specimen expansion and those demonstrated specimen compression during consolidation. It is evident that specimen expansion occurred almost in all series of tests and that the interchange of expansion and compression takes place at higher values of effective normal stress in the tests conducted with optimum water contents. This can be attributed to the denser condition of the specimens in these tests, since their values of initial dry unit weight, γ_{d0} , range from 1.24 gr/cm³ to 1.59 gr/cm³ and are larger than those (γ_{d0} = 1.10 gr/cm³ – 1.27 gr/cm³) in the tests conducted with moisture contents ranging from 46% to 47%.

The residual shear stress, τ_r , is the minimum constant value of shear stress, determined at the end of the shearing stage of each test and used in the drawing of the residual failure envelopes of soils. As typically shown in Figure 2a, the residual failure envelopes resulted from the ring shear tests conducted on specimens with optimum water contents are curved, in agreement with the observation of Stark and Eid (1994) that the non-linearity of failure envelopes is significant for soils with a

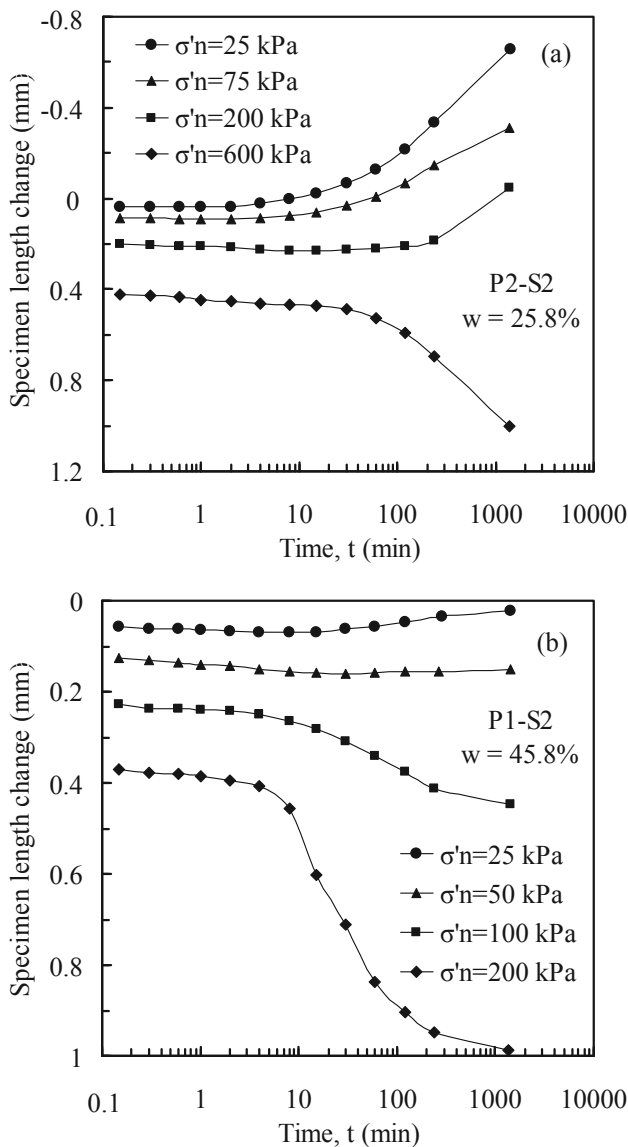


Figure 1. Typical results from the consolidation stage of ring shear tests.

Table 4. Soil behavior during the consolidation stage of ring shear tests.

Soil	Moisture content w (%)	Expansion	Compression
P1-S2	31.2	25, 75 ^a	250, 800 ^a
	45.8	25, 50	100, 200
P2-S1	27.0	25, 75, 200	600
	47.1	-----	25, 50, 100, 200
P2-S2	25.8	25, 75, 200	600
	46.4	25	50, 100, 200

^a Values of effective normal stress σ'_n (kPa) used in the tests

clay fraction $> 50\%$ and a liquid limit between 60 and 220. The fitting of the same experimental data with a linear failure envelope is applied in Figure 2b, without considering the measurement corresponding to the maximum effective normal stress. As a result, a very high correlation coefficient, R^2 , is obtained indicating that the failure envelopes of the soils with optimum water contents can be considered as linear for effective normal stresses up to 250 kPa. Typical residual failure envelope obtained from specimens prepared with moisture contents ranging from 46% to 47%, is also presented in Figure 2b. It can be stated with confidence that these failure envelopes

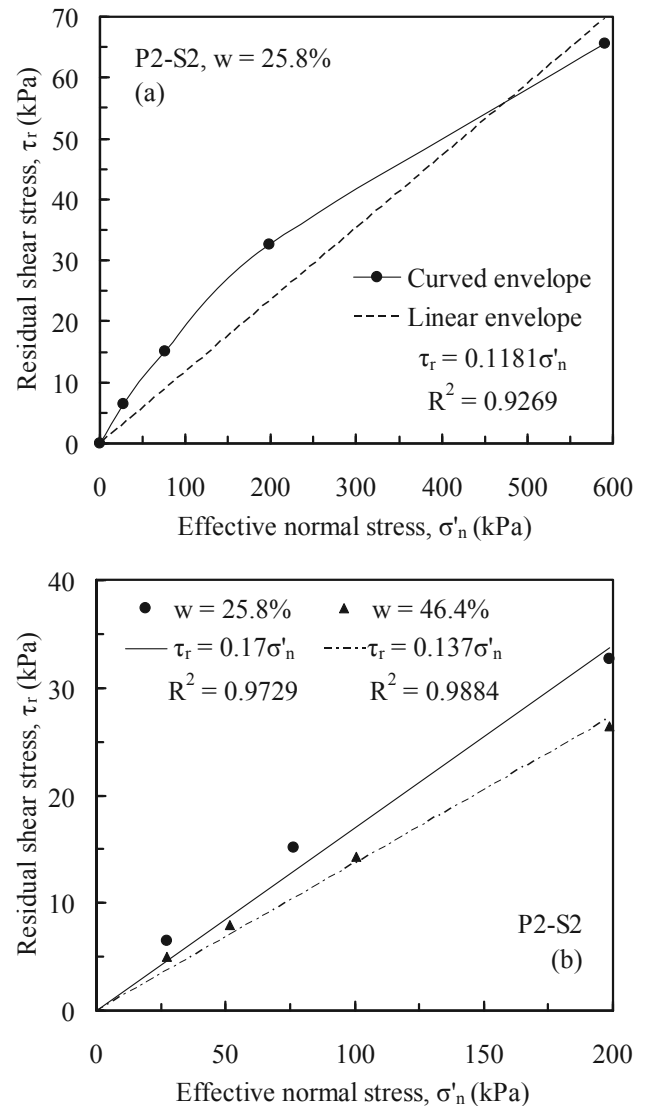


Figure 2. Typical residual failure envelopes from ring shear testing.

 Table 5. Values of residual friction angle ϕ'_R (degrees).

Soil	w_{opt}	w_{opt}	$w = 46\% - 47\%$	Difference (%)
P1-S2	7.3 ^a	8.8 ^b	8.8	17.0 ^c 0.0 ^d
P2-S1	8.4 ^a	9.1 ^b	8.7	7.7 ^c 4.4 ^d
P2-S2	6.7 ^a	9.6 ^b	7.8	30.2 ^c 18.8 ^d

^a For all values of effective normal stress used in the tests

^b For values of effective normal stress up to 250 kPa

^c Between the two angle values determined for w_{opt}

^d Between angles determined for w_{opt} ($\sigma'_n \leq 250$ kPa) and $w = 46\% - 47\%$

are also linear because the resulting correlation coefficients range from 0.98 to 0.99. Linear residual failure envelopes were also obtained from ring shear tests conducted on Greek clayey soils with effective normal stresses ranging from 50 to 400 kPa (Kaltefleiter 1993).

The values of the residual friction angle, ϕ'_R , obtained in this investigation, are summarized in Table 5. These values are similar to those reported by other researchers (Bishop et al. 1971) and were determined after it was ascertained that the values of residual cohesion, c'_R , are negligible and can be set equal to zero. If the curvature of the failure envelopes for optimum water contents is not taken into consideration and all experimental data are fitted with a linear failure envelope (Figure 2a), the correlation coefficients are satisfactory ($R^2 > 0.92$) but the resulting ϕ'_R values are even by 30% lower than

the values obtained for effective normal stresses up to 250 kPa (Table 5). The relatively low difference (7.7%) in P2-S1 soil is attributed to the not so pronounced curvature of its failure envelope. Xeidakis (1993) has reported that the residual friction angle decreases as the moisture content of swelling soils increases. This effect of moisture content on the residual friction angle was verified in the present research only for P2-S2 soil (Table 5), probably because the variation of moisture content used, ranges from 14% to 21% and is low, compared to the variation of 35% used by Xeidakis (1993).

The results of residual shear strength tests are often presented by plotting the values of residual friction coefficient, τ_r/σ'_n , against the corresponding values of effective normal stress, σ'_n (Lupini et al. 1981, Hawkins and Privett 1985). Thus, the “complete residual failure envelopes” (Hawkins and Privett 1985) can be obtained and the effect of effective normal stress on residual shear strength can be evaluated. The residual friction angle can be expressed as (Hawkins and Privett 1985):

$$\phi'_R = \tan^{-1} \frac{\tau_r}{\sigma'_n} \quad (1)$$

The results of ring shear tests conducted on specimens with optimum water contents were analyzed using Equation 1 and the resultant values of residual friction angle are presented in Figure 3. It is observed that the residual friction angle decreases with increasing effective normal stress and that P2-S2 soil presents the most pronounced curvature of failure envelope and, as a result, the maximum variation of residual friction angle. Finally, it appears that the “lowest constant residual strength” (Hawkins and Privett 1985) was not reached by the tested soils for the range of effective normal stresses used in this study.

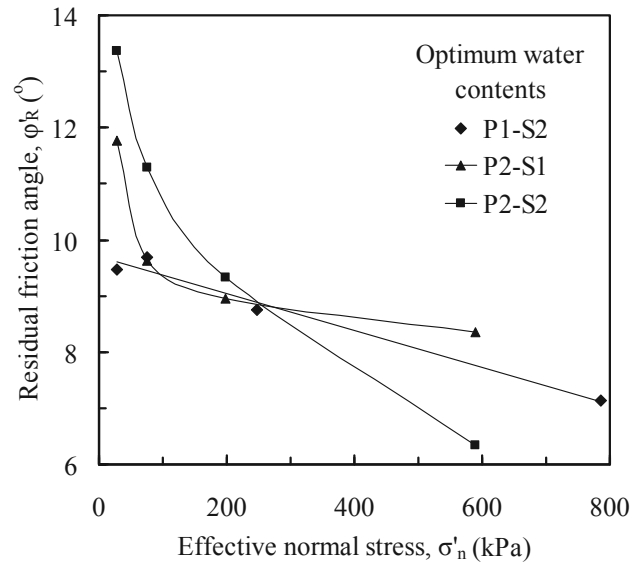


Figure 3. Complete residual failure envelopes of swelling soils tested with optimum water contents.

5 CONCLUSIONS

Based on the results of this investigation and within the limitations posed by the soils used and the number of tests conducted, the following conclusions may be advanced:

- The behavior of swelling soils in the consolidation stage of ring shear tests depends on the specimen moisture content and the effective normal stress used.
- The residual failure envelopes obtained for swelling soils, tested with the optimum moisture contents resulted from the Standard compaction test, are curved. Consequently, the residual friction angle decreases with increasing effective

normal stress and does not attain a minimum constant value for the range of effective normal stresses used in this study.

- All residual failure envelopes obtained in this investigation can be considered as linear for effective normal stresses up to 250 kPa, regardless of the moisture content of soils.
- The residual friction angle does not always decrease as the moisture content of soil increases.

6 ACKNOWLEDGEMENTS

The ring shear tests described in this paper were conducted at the Soil Mechanics & Foundation Engineering Laboratory of Democritus University of Thrace by the students M. Georgiadou and Ch. Trigousi, whose careful work is acknowledged.

7 REFERENCES

- ASTM D698 1998. Standard test method for laboratory compaction characteristics of soil using Standard effort (12,400 ft-lbf/ft³ (600 kN-m/m³)). *Annual Book of ASTM Standards* 04.08 (1), 77-84.
- ASTM D4546 1998. Standard test methods for one-dimensional swell or settlement potential of cohesive soils. *Annual Book of ASTM Standards* 04.08 (1), 663-669.
- BS 1377 1990. *Soils for Civil Engineering Purposes – Part 7: Shear Strength Tests (Total Stress)*. British Standards Institution, London.
- Bishop A.W., Green G.E., Garga V.K., Andresen A. and Brown J.D. 1971. A new ring shear apparatus and its application to the measurement of residual strength. *Géotechnique* 21 (4), 273-328.
- Bromhead E.N. 1979. A simple ring shear apparatus. *Ground Engineering* 12 (5), 40-44.
- Chandler R.J. 1966. The measurement of residual strength in triaxial compression. *Géotechnique* 16 (3), 181-186.
- Christodoulakis J. and Gasios E. 1987. Investigation on the motorway damage due to expansive soil in Greece. *Proc. 6th Int. Conf. on Expansive Soils*, New Delhi, 241-245.
- Gibbs H.J., Hilf J.W., Holtz W.G. and Walker F.C. 1960. Shear strength of cohesive soils. *Proc. Research Conf. on Shear Strength of Cohesive Soils*, Boulder, 33-162.
- Hawkins A.B. and Privett K.D. 1985. Measurement and use of residual shear strength of cohesive soils. *Ground Engineering* 18 (8), 22-29.
- Kalteziotis N. 1993. The residual shear strength of some Hellenic clayey soils. *Geotechnical and Geological Engineering* 11, 125-145.
- Kollaros G. and Athanasopoulou A. 1997. The character and identification of swelling soils in road construction projects. *Proc. Engineering Geology and the Environment*, Marinos et al. (eds.), Balkema, 187-192.
- Koudoumakis P. 2000. *Personal communication*. Democritus University of Thrace, Xanthi, Greece.
- Lupini J.F., Skinner A.E. and Vaughan P.R. 1981. The drained residual strength of cohesive soils. *Géotechnique* 31 (2), 181-213.
- Mesri G. and Cepeda – Diaz A.F. 1986. Residual shear strength of clays and shales. *Géotechnique* 36 (2), 269-274.
- Papakyriakopoulos P. and Koudoumakis P. 2001. Swelling soils in the area of Thrace. *Proc. 4th Hellenic Conf. on Geotechnical and Geoenvironmental Engineering* 1, Athens, 163-170. (in Greek)
- Skempton A.W. 1964. Long-term stability of clay slopes. *Géotechnique* 14 (2), 75-101.
- Skempton A.W. 1985. Residual strength of clays in landslides, folded strata and the laboratory. *Géotechnique* 35 (1), 3-18.
- Stamatopoulos A., Gassios E., Christodoulakis J. and Giannaros H. 1989. Recent experiences with swelling soils. *Proc. 12th Int. Conf. on Soil Mechanics and Foundation Engineering* 2, Rio de Janeiro, 655-658.
- Stamatopoulos A., Christodoulakis J. and Giannaros H. 1992. Treatment of expansive soils for reducing swell potential and increasing strength. *Quarterly Journal of Engineering Geology* 25, 301-312.
- Stark T.D. and Eid H.T. 1994. Drained residual strength of cohesive soils. *Journal of Geotechnical Engineering* 120 (5), 856-871.
- Tsiambaos G. and Tsaligopoulos Ch. 1995. A proposed method of estimating the swelling characteristics of soils: some examples from Greece. *Bulletin of the International Association of Engineering Geology* 52, 109-115.
- Xeidakis G. 1993. Swelling soils in Thrace, Northern Greece: origins and properties. *Proc. Geotechnical Engineering of Hard Soils – Soft Rocks* 1, Anagnostopoulos et al. (eds.), Balkema, 863-870.

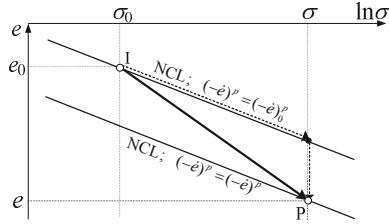


Figure 2. Explanation of non-stationary flow surface model

indicate the initial ($\sigma=\sigma_0$, $e=e_0$) and the current ($\sigma=\sigma$, $e=e$) states, respectively. The plastic change in void ratio from the initial state to the current state can be expressed by

$$\begin{aligned} (-\Delta e)^p &= e_0 - e - (-\Delta e)^e = F + \lambda_\alpha \ln \frac{t}{t_0} \\ &= F + \lambda_\alpha \ln \frac{(-\dot{e})_0^p}{(-\dot{e})^p} \quad \left(\text{where, } F = (\lambda - \kappa) \ln \frac{\sigma}{\sigma_0} \right) \end{aligned} \quad (1)$$

Here, λ and κ denote compression and swelling indices. By solving this ordinary differential equation with variables of time and plastic void ratio change (plastic strain), the plastic change in void ratio is given in the following form as a function of stress term F and time t (Sekiguchi 1977):

$$\begin{aligned} (-\Delta e)^p &= \lambda_\alpha \ln \left\{ \frac{(-\dot{e})_0^p \cdot t}{\lambda_\alpha} \exp \left(\frac{F}{\lambda_\alpha} \right) + 1 \right\} \\ &= \lambda_\alpha \ln S \quad \left(\text{where, } S = \frac{(-\dot{e})_0^p \cdot t}{\lambda_\alpha} \exp \left(\frac{F}{\lambda_\alpha} \right) + 1 \right) \end{aligned} \quad (2)$$

Then, the increment of plastic void ratio is calculated as

$$\begin{aligned} d(-e)^p &= \frac{\partial(-\Delta e)^p}{\partial \sigma} d\sigma + \frac{\partial(-\Delta e)^p}{\partial t} dt \\ &= (\lambda - \kappa) \left(1 - \frac{1}{S} \right) \frac{1}{\sigma} d\sigma + \lambda_\alpha \left(1 - \frac{1}{S} \right) \frac{1}{t} dt \end{aligned} \quad (3)$$

The elastic increment of plastic void ratio change is given by

$$d(-e)^e = \kappa \frac{1}{\sigma} d\sigma \quad (4)$$

Total increment is given by a summation of the plastic component of Equation (3) and the elastic component of Equation (4). As is seen from Equation (3), the non-stationary flow surface model contains time variable t , which is not objective. The multi-dimensional model in which the stress term F is replaced by the yield function of the Cam clay using (p, q) corresponds to so-called Sekiguchi model (1977).

2.2 Over-stress type model

As is seen from Figure 3, viscoplastic strain rate of the Bingham body is given by

$$\dot{\epsilon}^p = \frac{1}{\mu} \langle \sigma - \sigma_c \rangle = \frac{\sigma_c}{\mu} \left\langle \frac{\sigma}{\sigma_c} - 1 \right\rangle \quad (5)$$

Here, the symbol $\langle \rangle$ denotes the Macaulay bracket, i.e. $\langle A \rangle = A$ if $A > 0$; otherwise $\langle A \rangle = 0$, and σ_c and μ are the yield stress and the coefficient of viscosity. The over-stress type of viscoplasticity gives the viscoplastic strain rate as follows generalizing the Bingham viscosity:

$$\begin{aligned} \dot{\epsilon}^p &= \gamma \langle \Phi(\xi) \rangle \\ (\text{where } \langle \Phi(\xi) \rangle &= \Phi(\xi) \text{ if } \xi > 0, \quad \langle \Phi(\xi) \rangle = 0 \text{ if } \xi \leq 0) \end{aligned} \quad (6)$$

The straight line with slope of λ in Figure 4 indicates NCL at reference state in which the strain rate is sufficiently small. The condition on this line is called “static”. Now, it is assumed that clay at point P in Figure 4 satisfies the creep behavior in Figure 1. From Figure 4, the difference between the current void ratio and that on NCL ($-\rho$) at the same stress is expressed as

$$-\rho = e - e_{ref} = (\lambda - \kappa) \ln \frac{\sigma}{\sigma_s} \quad (7)$$

Also, the following equation is obtained from the interpolated diagram in Figure 1:

$$-\rho = \lambda_\alpha \ln \frac{t_{ref}}{t} = \lambda_\alpha \ln \frac{(-\dot{e})^p}{(-\dot{e})_{ref}^p} \quad (8)$$

From Equations (7) and (8), the rate of plastic void ratio change is expressed as

$$(-\dot{e})^p = (-\dot{e})_{ref}^p \exp \left(\frac{-\rho}{\lambda_\alpha} \right) = (-\dot{e})_{ref}^p \exp \left(\frac{\lambda - \kappa}{\lambda_\alpha} \ln \frac{\sigma}{\sigma_s} \right) = (-\dot{e})_{ref}^p \left(\frac{\sigma}{\sigma_s} \right)^{\frac{\lambda - \kappa}{\lambda_\alpha}} \quad (9)$$

Here, the current stress σ is called “dynamic stress”, and the stress which is determined from the current void ratio σ_s is called “static stress”. These correspond to “dynamic yield function” and “static yield function” in multi-dimensional over-stress type viscoplastic model. If it is assumed that Equation (9) holds not only under creep condition but also under other conditions, the increment of plastic void ratio change is given by

$$d(-e)^p = (-\dot{e})_{ref}^p \left\langle \left(\xi + 1 \right)^{\frac{\lambda - \kappa}{\lambda_\alpha}} \right\rangle dt \quad \left(\text{where } \xi = \frac{\sigma}{\sigma_s} - 1 \right) \quad (10)$$

The elastic component is given by Equation (4). it is also shown by Mimura and Sekiguchi (1985) that Equation (10) corresponds to Equation (3) in which the term of stress increment is eliminated. Since the plastic strain increment is related with time alone, there is no loading condition in this type of models.

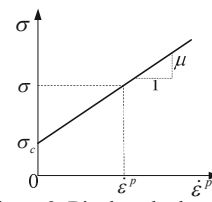


Figure 3. Bingham body

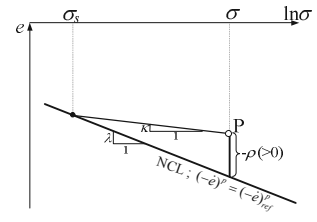


Figure 4. Explanation of over-stress type model

3 NEW TYPE OF TIME-DEPENDENT MODEL

As mentioned above, NCL shifts depending on the strain rate. The two straight lines in Figure 5 indicate NCLs corresponding to the initial state (point I; $\sigma=\sigma_0$, $e=e_0$, $(-\dot{e})^p = (-\dot{e})_0^p$) and the current state (point P; $\sigma=\sigma$, $e=e$, $(-\dot{e})^p = (-\dot{e})^p$). The difference of these two lines are expressed as $(\psi - \psi_0)$. To model not only normally consolidated clay but also over consolidated clay, the state variable ρ ($=e_N - e$) which is the difference between the current void ratio and that on NCL is introduced. Furthermore, to describe the behavior of structured clay such as naturally deposited clay, another state variable ω , which represent an imaginary increase of density due to bonding effect, is employed. In the case of $\rho=0$ and $\omega=0$, Figure 5 results in Figure 2. From Figure 5, the plastic change of void ratio is expressed as

$$\begin{aligned} (-\Delta e)^p &= (-\Delta e) - (-\Delta e)^e \\ &= \{ (e_{N0} - e_N) - (\rho_0 - \rho) - (\psi_0 - \psi) \} - (-\Delta e)^e \\ &= \lambda \ln \frac{\sigma}{\sigma_0} - (\rho_0 - \rho) - (\psi_0 - \psi) - \kappa \ln \frac{\sigma}{\sigma_0} \end{aligned} \quad (11)$$

Denoting $F = (\lambda - \kappa) \ln(\sigma / \sigma_0)$ in the same way as Equation (1) and $H = (-\Delta e)^p$, the following equation (1D yield function) is obtained:

$$f = F - \{ H + (\rho_0 - \rho) + (\psi_0 - \psi) \} = 0 \quad (12)$$

From the consistency condition ($df=0$),

$$df = dF - \{ dH + d\rho + d\psi \} = 0 \quad (13)$$

Now, it can be considered that the evolution rule of ρ with the development of plastic deformation for structured soil is determined not only by the state variable ρ related to real

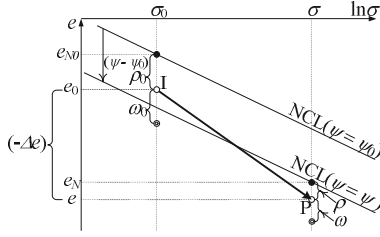


Figure 5. Explanation of the proposed model

density but also by the state variable ω related to the imaginary increase of density (bonding effect). Larger values of the state variables ρ and ω can be assumed to have more effects for the degradation of ρ and ω . Then, the evolution rules of ρ and ω can be given in the following form, using increasing functions $G(\rho)$ and $Q(\omega)$ which satisfy $G(0)=0$ and $Q(0)=0$, respectively:

$$d\rho = -\{G(\rho) + Q(\omega)\} \cdot d(-e)^p \quad (14)$$

Simply, the evolution rule of ω is also given using $Q(\omega)$ by

$$d\omega = -Q(\omega) \cdot d(-e)^p \quad (15)$$

The distance $(\psi - \psi_0)$ of the two NCLs in Figure 5 is expressed as a function of the elapsed time t or the rate of plastic change in void ratio $(-e)^p$, referring to the interpolated diagram in Figure 1.

$$\psi - \psi_0 = \lambda_\alpha \ln \frac{t}{t_0} = \lambda_\alpha \ln \frac{(-\dot{e})_0^p}{(-\dot{e})^p} = \{-\lambda_\alpha \ln(-\dot{e})^p\} - \{-\lambda_\alpha \ln(-\dot{e})_0^p\} \quad (16)$$

Its increment is given by

$$d\psi = \frac{\partial \psi}{\partial t} dt = \lambda_\alpha \frac{1}{t} dt = (-\dot{e})^p dt \quad (17)$$

Now, it is assumed that Equations (16) and (17) hold not only for normally consolidated soils but also for over consolidated soils and naturally deposited soils. Substituting Equations (14) and (17) into Equation (13), the increment of the plastic void ratio can be obtained as

$$d(-e)^p = \frac{(\lambda - \kappa) \frac{1}{\sigma} d\sigma + (-\dot{e})^p \cdot dt}{1 + G(\rho) + Q(\omega)} \cong \frac{(\lambda - \kappa) \frac{1}{\sigma} d\sigma + (-\dot{e})^{p*} \cdot dt}{1 + G(\rho) + Q(\omega)} \quad (18)$$

Here, $(-\dot{e})^{p*}$ denotes the rate of the plastic void ratio change at the step immediately before the current calculation step. Finally, the total increment of void ratio is given by the following equation:

$$d(-e) = d(-e)^p + d(-e)^e = \left(\frac{\lambda - \kappa}{1 + G(\rho) + Q(\omega)} + \kappa \right) \frac{d\sigma}{\sigma} + \frac{(-\dot{e})^{p*}}{1 + G(\rho) + Q(\omega)} dt \quad (19)$$

In order to simplify the numerical calculations, the known rate $(-\dot{e})^{p*}$ in the previous calculation step can be used, instead of the current rate, as described. The error caused by using the previous known rate is automatically corrected by the subloading surface concept and is negligible in the calculations, because an incremental method with small steps is used in non-linear analysis. Time variable t which is not objective is not included, and the loading condition is given as $d(-e)^p > 0$. The details of the present modeling are described in Nakai et al. (2011a) and Nakai (2012).

4 SIMULATIONS OF TIME-DEPENDENT BEHAVIOR

4.1 Analysis of normally consolidated clay by three models

Table 1 shows material parameters for Fujinomori clay, which are common to the three models. The initial rate of plastic void ratio change is assumed as $(-\dot{e})_0^p = (-\dot{e})_{ref}^p = 1.0 \times 10^{-7}/\text{min}$. Linear increasing functions, $G(\rho) = 100\rho$ and $Q(\omega) = 40\omega$, are employed for the case to consider density and bonding effects.

Figure 6 shows the results of one-dimensional compression behavior of normally consolidated (NC) clay for different strain

Table 1. Material parameters for clay

λ	0.104
κ	0.010
$N(e_N \text{ at } \sigma = 98 \text{ kPa})$	0.83
λ_α	0.003
$(-\dot{e})_{ref}^p$	$1 \times 10^{-7}/\text{min}$

N.B. N is the void ratio of NCL at $(-\dot{e})^p = (-\dot{e})_{ref}^p$

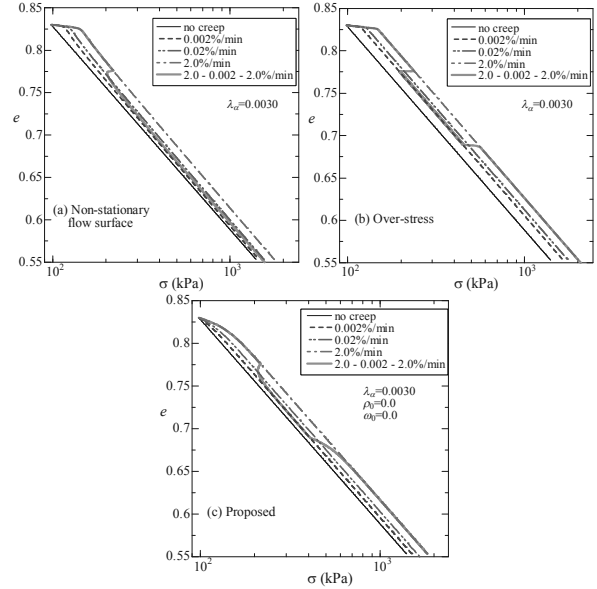


Figure 6. Simulations of strain rate effect of NC clay

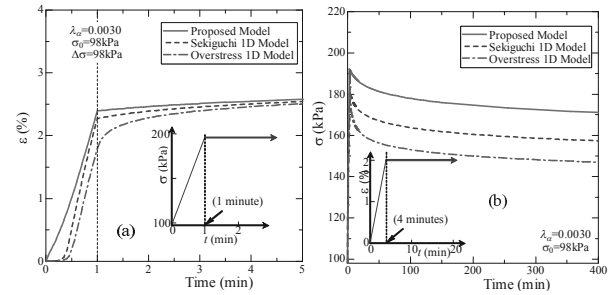


Figure 7. Simulations of creep and stress relaxation of NC clay

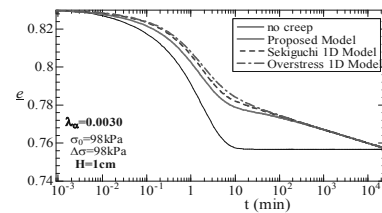


Figure 8. Simulations of oedometer test on NC clay

rates, arranged in terms of the e - $\log \sigma$ relation. In the figure, the solid straight lines (no creep) show the relation without time effect, and the thick lines show the results in which strain rate decreases and increases again along the way. In every model, the lines of constant strain rate are parallel to each other, which is a good agreement with published experimental results (e.g., Bjerrum, 1967). It is also seen that when the strain rate decreases at a certain point, the curve follows exactly the same path which is supposed to follow for the new strain rate. However, when the strain rate increases again to the previous rate, the result of the non-stationary flow surface model does not return to the target curve corresponding to the respective strain rate which is called the phenomenon of 'isotache'. The result of the over-stress model shows sudden change of stress,

because the plastic strain increment does not related to the stress increment as described before. The result of the proposed model describes properly the strain rate effects known experimentally. Figure 7 shows the results of (a) creep behavior after loading with constant stress rate, and (b) stress relaxation behavior after loading with constant strain rate (loading processes are indicated in the interpolated diagram in each figure). Figure 8 shows the computed e -log t relation of conventional oedometer test by 1D soil-water coupled finite element analysis. Here, σ_0 is the initial stress, and $\Delta\sigma$ is the instantaneous stress increment. It can be seen that the three models describe the well-known consolidation behavior of normally consolidated clay including the secondary compression.

4.2 Analysis of over consolidated clay and naturally deposited clay by the proposed model

The non-stationary flow surface model and the over-stress type model described in the above section are applicable to normally consolidated soil alone. On the other hand, as can be seen from the derivation process, the proposed model is valid for over consolidated soil and structured soil as well. In this subsection, some applications of the proposed model to over consolidated clay and structured clay are shown.

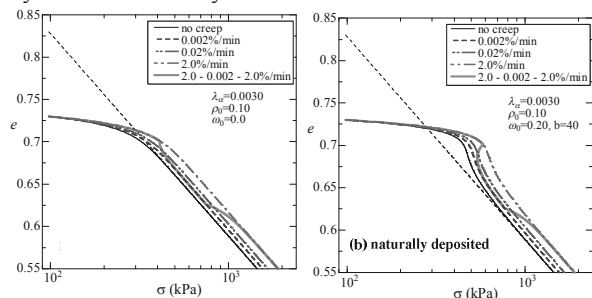


Figure 9. Simulations of strain rate effect

Figure 9 shows the results of one-dimensional compression behavior of these clays in the same way as Figure 6. It can be seen that the present model describe well the typical feature of e -ln σ relation for these clays and the time effect such that the quasi-yield stress becomes large with increasing strain rate and the phenomenon of isotache. Figure 10 shows the computed results of oedometer tests on non-structured ($\omega_0=0.0$) and structured ($\omega_0=0.20$) clays with two kinds of initial void ratios ($\rho_0=0.0$ and 0.10), arranged with the same manner as Figure 8. Here, thin curves and thick curves indicate ones without bonding ($\omega_0=0.0$) and ones with bonding ($\omega_0=0.20$). Diagrams (a), (b) and (c) shows the results of the ratio of stress increment to initial stress: $\Delta\sigma/\sigma_0=1$, 4 and 8, respectively. It is seen from diagrams (a) and (b) that although the behavior of the normally consolidated structured clay ($\rho_0=0.0$) is different from that of the normally consolidated non-structured clay under small stress increment ($\Delta\sigma/\sigma_0=1$), there is not much difference between them under large stress increment ($\Delta\sigma/\sigma_0=4$). On the other hand, the behavior of over consolidated clays ($\rho_0=0.10$) is highly influenced by the effect of structure (bonding) not under small stress increment but under large stress increment. It is also seen that from diagram (c) that when the stress increment is extremely large ($\Delta\sigma/\sigma_0=8$), there is not much difference between non-structured clay and structured clay regardless of

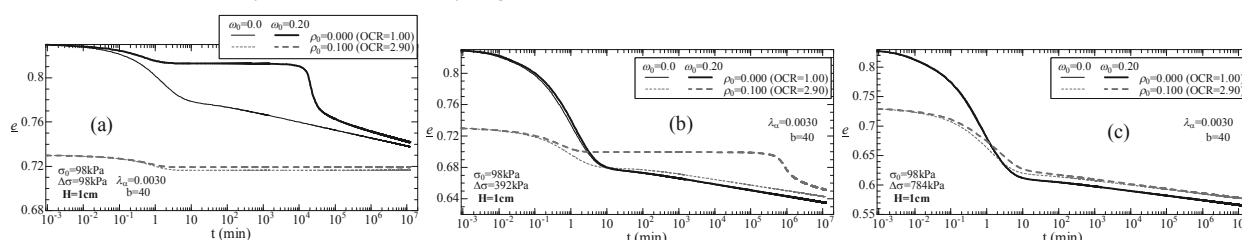


Figure 10. Simulations of oedometer tests on non-structured and structured clays with different initial void ratios

the initial void ratio. Figure 11 shows the stress strain relation of these structured clays under large and small strain rate. The delayed settlements of structured clays may occur when the quasi-yield stress, which becomes small with decreasing strain rate, moves through the stress of the corresponding element (indicated by vertical dotted lines).

5 CONCLUSIONS

The feature of the ordinary viscoplastic models for clay are explained in 1D condition for easy understanding. A new approach to model without using ordinary viscoplasticity is also shown. The new model is applicable not only to normally consolidated soil but also to over consolidated soil and naturally deposited soil. The applicability of these models are discussed through the simulations of various time-dependent behavior of clays. Using the t_{ij} concept (Nakai and Mihara 1984), the present 1D model can be extended to 3D one (see Nakai et al. 2011b, Nakai 2012).

6 REFERENCES

- Adachi T. and Oka F. 1982. Constitutive equation for normally consolidated clays based on elasto/viscoplasticity. *Soils and Foundations*, 22(4), 57-70.
- Bjerrum L. 1967. Engineering geology of Norwegian normally consolidated marine clays as related to settlements of buildings, *Geotechnique*, 17(2), 81-118.
- Hashiguchi K. 1980. Constitutive equation of elastoplastic materials with elasto-plastic transition. *Jour. of Appl. Mech., ASME*, 102(2), 266-272.
- Mimura M. and Sekiguchi H. 1985. A review of elasto-viscoplastic models with particular emphasis on stress-rate effect. *Proc of 20th Annual Meeting of JGS*, 1, 403-406 (in Japanese).
- Nakai T. and Mihara Y. (1984): A new mechanical quantity for soils and its application to elastoplastic constitutive models, *Soils and Foundations*, 24(2), 82-94.
- Nakai T., Shahin H.M., Kikumoto M., Kyokawa H., Zhang F. and Farias, M.M. 2011a: A simple and unified one-dimensional model to describe various characteristics of soils. *Soils and Foundations*, 51(6), 1129-1148.
- Nakai T., Shahin H.M., Kikumoto M., Kyokawa H., Zhang F. and Farias, M.M. 2011b. A simple and unified three-dimensional model to describe various characteristics of soil. *Soils and Foundations*, 51(6), 1149-1168.
- Nakai T. 2012. *Constitutive Modeling of Geomaterials: Principles and Applications*. CRC Press.
- Perzyna P. 1963. The constitutive equations for rate sensitive plastic materials, *Quart. Appl. Math.*, 20(4), 321-332.
- Sekiguchi H. 1977. Rheological characteristics of clays. *Proc. of 9th ICSMFE*, Tokyo, 1, 289-292.

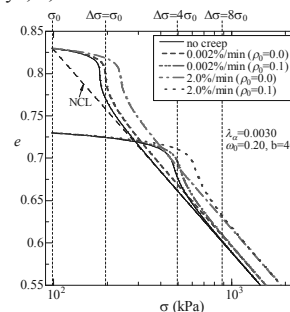


Figure 11. Stress-strain behavior of structured clays

Quantification des gonflements des sols surconsolidés en fond de déblai

Swelling quantification of overconsolidated soils at excavation base

Petit G.

Arcadis, Toulouse, France

Virollet M., Bernard Ph.

Consultants indépendants, Toulouse, France

RÉSUMÉ : La problématique du gonflement différé des sols surconsolidés en fond de déblais est apparue progressivement avec la mise en service de grandes infrastructures routières conduisant à la création de tranchées de plus en plus profondes. Pour la réalisation de nouvelles infrastructures ferroviaires à grande vitesse, cette problématique peut avoir des répercussions bien plus fortes tant en termes d'exploitation (problématique des équipements ferroviaires) qu'économique (plus fortes contraintes géométriques, répercussions sur un long linéaire). Cet article présente une méthode de quantification de ce type de gonflement à partir de l'analyse d'essais œdométriques de déchargement et en se basant sur le modèle Ylight. L'existence d'une phase élastique réversible et d'une phase irréversible sont ainsi mises en évidence.

ABSTRACT : The delayed swelling of overconsolidated soils at excavation base has progressively become an important issue with the development of major road infrastructures resulting in the creation of deeper and deeper excavations and trenches. As far as new high speed railway infrastructures projects are concerned, this issue may have major consequences in terms of serviceability (sensitive railway equipments) and economics (stronger geometric constraints and backlash over long railway portions).

This paper presents a quantification method for this type of swelling, based on the analysis of specific unloading oedometer tests, on the basis of the Ylight model of behavior. This approach allows to show the existence of a reversible elastic phase and of an irreversible phase.

MOTS-CLÉS: gonflement, sols surconsolidés, oedomètre, modèle Ylight

KEYWORDS: Swelling, overconsolidated soils, oedometer, Ylight model.

1 INTRODUCTION

A partir d'une approche mécanique simplifiée, basée sur l'utilisation de résultats d'essais œdométriques, associée à une analyse en chemins de contraintes selon le modèle Ylight (Leroueil, Magnan & Tavenas, 1985), une étude du comportement des sols en déchargement est effectuée. En premier lieu, le phénomène du gonflement lié à un déchargement généralisé de type érosion est abordé à l'aide du modèle Ylight applicable aux sols surconsolidés (Magnan & Serratrice, 1995) puis, ensuite, une analyse du fond de déblai est menée afin d'évaluer les amplitudes de gonflement.

2 COMPORTEMENT DES SOLS AU DÉCHARGEMENT GÉNÉRALISÉ

2.1. Variation de l'OCR avec la profondeur

Si l'on considère à l'état actuel un sol qui a été érodé sur une hauteur h_e et qui de ce fait est surconsolidé, l'indice de surconsolidation (OCR), à une profondeur z donnée comptée à partir de la surface du sol après érosion, est défini comme égal à :

$$OCR = \frac{h_e \gamma + z\gamma}{z\gamma}$$

Cet indice décroît alors lorsque la profondeur croît pour tendre vers un sol normalement consolidé ($OCR = 1$).

2.2. Chemins de contraintes dans l'espace de Lambe

Les essais réalisés par Magnan et Serratrice (1995) sur une marne fortement surconsolidée, montrent que dans le domaine du déchargement, la surface d'état limite peut être définie dans le modèle Ylight en la représentant comme une ellipse dans l'espace $[s', t]$ avec :

$$s' = \frac{\sigma'_1 + \sigma'_3}{2} ; t = \frac{\sigma'_1 - \sigma'_3}{2}$$

Description du modèle Ylight

Le modèle Ylight, représenté ci-dessous en adimensionnel, fait apparaître plusieurs domaines.

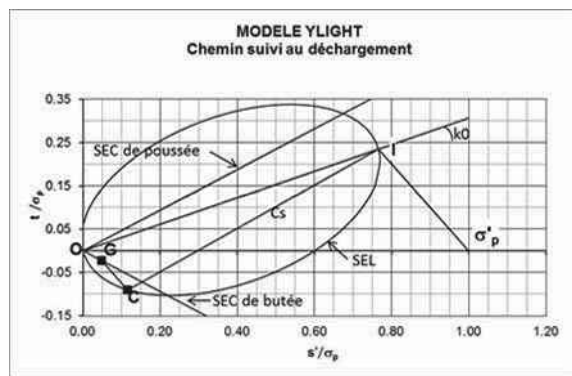


Figure 1. Modèle Ylight représenté dans le plan (s', t)

Un domaine élastique situé à l'intérieur de l'ellipse dont l'axe principal est la droite déduite de $k_0 = \sigma'_3/\sigma'_1$.

Le point I qui fixe sa longueur est obtenu à partir de la pression de consolidation σ'_p . Cette ellipse, appelée SEL, représente la projection de la Surface d'Etat Limite dans ce plan. A l'intérieur de l'ellipse les déformations sont élastiques réversibles. Cette ellipse projetée dans un plan perpendiculaire au plan de représentation de l'œdomètre $[e, \log \sigma]$ s'appuie sur la droite C_s . Les points d'état ne peuvent franchir la SEL.

Un domaine plastique. Lorsque le point représentatif de l'état de contraintes considéré atteint la surface de rupture, caractérisée par les droites limites de Mohr-Coulomb (SEC, Surface d'Etat Critique), il y a rupture du sol avec des phénomènes de radoucissement.

Étude du déchargement à l'aide du modèle Ylight

Dans la phase de déchargement généralisé (érosion), les déformations horizontales sont nulles : $\Delta \varepsilon_3 = 0$.

En fin de chaque phase de déchargement, une fois l'équilibre atteint, les pressions interstitielles sont supposées constantes et en équilibre avec celle du niveau considéré.

Dans le domaine élastique

Dans ce domaine, il est possible d'écrire :

$$\Delta \sigma_3 = \Delta \sigma_1 \frac{\nu}{1-\nu} \quad (1)$$

où ν représente le coefficient de Poisson. Il vient alors :

$$\Delta \sigma_1 = \gamma h_e \quad \text{et} \quad \Delta \sigma_3 = \gamma h_e \frac{\nu}{1-\nu} \quad (2)$$

Dans le plan $[s', t]$, cela se traduit par :

$$\Delta s' = \gamma h_e \frac{1}{2(1-\nu)} \quad \text{et} \quad \Delta t = \gamma h_e \frac{1-2\nu}{2(1-\nu)} \quad (3)$$

La pente de la droite représentant le déchargement élastique du sol est $p=1-2\nu$. Au point I l'état de contrainte s'écrit :

$$s' = \frac{\sigma'_p(1+k_0)}{2} \quad \text{et} \quad t = \frac{\sigma'_p(1-k_0)}{2} \quad (4)$$

En tenant compte des équations (2), (3) et (4), l'équation de la droite de déchargement dans l'espace $[s', t]$ est :

$$t = (1-2\nu)s' + \sigma'_p[\nu(1+k_0) - k_0] \quad (5)$$

Celle-ci intercepte la SEL au point C. A partir de ce point d'équilibre correspondant à la contrainte verticale σ'_{v0} qui à une profondeur donnée est constante ($\sigma'_{v0} = \gamma z$), le déchargement par excavation (déblai) s'effectue suivant un chemin de contraintes (s', t) correspondant à une droite de pente -1 , qui coupe la courbe d'état critique au point G.

Dans le domaine plastique

Lorsque le déchargement se poursuit le chemin de contrainte suivi passe sur la droite GO.

Le point G représente le passage entre le comportement élastique caractérisé par l'indice de gonflement C_s et le comportement plastique caractérisé par le coefficient de gonflement C_g .

3. ANALYSE DES PHÉNOMÈNES DE GONFLEMENT A L'ŒDOMÈTRE

3.1. Essai de gonflement (norme XP P94-091)

Cet essai à l'œdomètre consiste à mesurer, sur quatre échantillons prélevés à la même profondeur, les gonflements obtenus lors de la saturation sous quatre contraintes différentes, la contrainte de gonflement donnant un gonflement nul.

En fait, cette contrainte devrait correspondre à la contrainte effective verticale obtenue à la profondeur z (profondeur de prélèvement des échantillons) : $\sigma'_{v0} = \gamma z$.

Dans ces conditions l'exploitation des mesures consiste à reporter sur un diagramme logarithmique les valeurs finales des différentes déformations mesurées en fonction

du rapport $\frac{\sigma_g}{\sigma}$ dans l'espace $\left[\frac{\Delta H_f}{H_i}, \log \sigma \right]$. Ces différents

points s'alignent sur une droite, dont la valeur absolue de la pente est appelée rapport de gonflement : R_g .

Nous avons la relation :

$$\Delta H_f / H_i = -R_g \log(\sigma_g / \sigma) \quad (6)$$

σ_g étant la contrainte qui correspond à une déformation nulle, soit la contrainte effective correspondant au poids des terres.

3.2. Essai œdométrique classique adapté

La norme XP P94-090-1 a été adaptée pour la circonstance afin de détailler le gonflement des sols surconsolidés au déchargement.

La courbe de déchargement mesurée dans un œdomètre à partir de la pression de consolidation maximale est obtenue en déchargeant par paliers successifs l'échantillon imbibé à partir de cette pression. Elle fait apparaître typiquement deux branches de droites (figure 2) :

La première, de pente C_s dans la représentation œdométrique $[e, \log(\sigma)]$;

La seconde, de pente donnée par l'équation (6), que nous noterons $C_g = R_g / (1 + e_i)$, e_i étant l'indice des vides avant déchargement.

Ces deux droites se coupent en un point qui correspond à la contrainte notée σ_G , pour la distinguer de la contrainte σ_{Gg} définie ci-dessus.

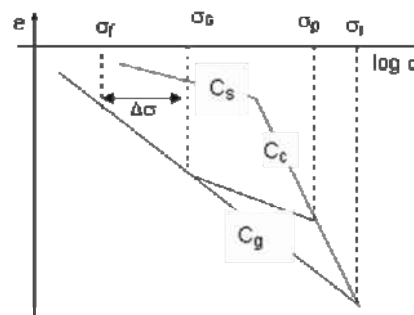


Figure 2. Courbe de déchargement à l'œdomètre

3.3. Elaboration d'un modèle de calcul

Le modèle Ylight nous a permis de montrer que la contrainte σ_G correspondait à la transition entre le comportement élastique du sol et le comportement irréversible de caractéristiques respectives C_s et C_g .

Avant déchargement généralisé $\Delta\sigma$, la transition σ_i entre ces deux comportements se situe à la profondeur correspondant à l'intersection entre les droites C_g et C_c

$$\text{soit } H_i = \frac{\sigma_i}{\gamma}$$

La valeur de σ_i est donnée par la relation :

$$\sigma_i = 10 \frac{(C_c - C_g) \log \sigma_p - (C_g - C_s) \log \sigma_G}{C_c - C_s} \quad (7)$$

Après érosion sur une hauteur h_e , la nouvelle contrainte

σ_i' doit être recalculée, comme suit :

$$\sigma_i' = \sigma_i 10^{\left[\frac{C_c - C_s}{C_c - C_g} \log \frac{\sigma_p + \gamma h_e}{\sigma_{p0}} - \frac{C_g - C_s}{C_c - C_g} \log \frac{\gamma(z + h_e)}{\gamma z} \right]} \quad (8)$$

La profondeur d'intersection est alors donnée par :

$$H_i' = \frac{\sigma_i'}{\gamma} - h_e \quad (9)$$

4. ÉTABLISSEMENT D'UN MODÈLE DE CALCUL DE DÉCHARGEMENT GÉNÉRALISÉ

Dans le paragraphe précédent, nous avons montré que la contrainte σ_i définissait la hauteur de la zone de

déformation irréversible $H_i = \frac{\sigma_i}{\gamma}$ et que cette contrainte

σ_i devait être corrigée pour ramener l'épaisseur par rapport à la nouvelle surface obtenue après érosion : $H_i = \frac{\sigma_i}{\gamma} - h_e$

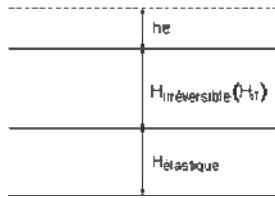


Figure 3. Définition de la zone de déformation irréversible

4.1. Modèle de calcul

Les gonflements sont par la suite notés s , s_{el} pour sa partie élastique et s_{irr} pour sa partie irréversible.

4.1.1. Gonflement élastique

Méthode analytique

Pour une couche d'épaisseur dz , à la profondeur z comprise entre H_i et $H = H_i + H_{élastique}$, nous obtenons

$$\frac{ds}{dz} = -0,434 \frac{C_s}{1 + e_0} \ln \frac{\gamma z}{\gamma z + \gamma h_e} \quad (9)$$

$$s_{el} = -0,434 \frac{C_s}{1 + e_0} [(z + h_e) \ln(\gamma z + \gamma h_e) - h_e - z \ln(\gamma z)]$$

avec e_0 indice des vides pour la contrainte σ .

Nous avons pris : $e_0 = e_G - C_s \log \frac{\gamma H}{\gamma H_i}$

Méthode numérique

Nous écrirons l'équation (9) pour une variation Δz finie. Il vient alors :

$$\Delta s_{el} = -0,434 \frac{C_s}{1 + e_0} \ln \frac{\gamma z}{\gamma z + \gamma h_e} \Delta z$$

$$\Delta s_{el} = \frac{C_s}{1 + e_0} \log \frac{H_i}{\gamma(z + h_e)} \Delta z \quad (10)$$

dans ce calcul, la variation de e_0 est prise en compte à chaque incrément de Δz .

4.4.2. Gonflement irréversible

Méthode analytique

Nous procédons de la même manière que ci-dessus en faisant varier la profondeur z entre 0,5m et H_i . Nous obtenons ainsi :

$$\frac{ds}{dz} = -0,434 \frac{C_g}{1 + e_G} \ln \frac{\gamma z + \gamma h_e}{\gamma H_i} \quad (11)$$

$$s_{irr} = -0,434 \frac{C_g}{1 + e_0} [(z + h_e) \ln(\gamma z + \gamma h_e) - h_e - z \ln(\gamma z)] \quad (12)$$

La borne minimale d'intégration a été prise égale à 0,5m afin de permettre la convergence du calcul numérique.

Méthode numérique

Nous écrirons l'équation (12) pour une variation Δz sous la forme :

$$\Delta s_{irr} = \frac{C_g}{1 + e_0} \log \frac{\gamma z + \gamma h_e}{\gamma z} \Delta z \quad (13)$$

4.3. Application et résultats des calculs

4.3.1. Caractéristiques du sol retenu

Nous avons adopté les caractéristiques mesurées sur l'essai œdométrique présenté sur le graphique et données dans le tableau ci-dessous.

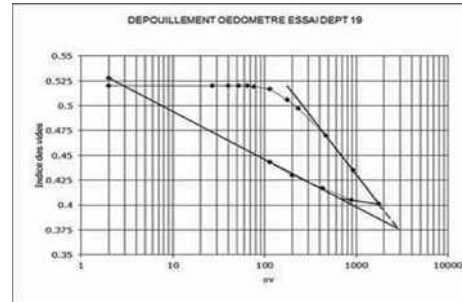


Figure 4. Essai œdométrique utilisé

Tableau 1. Valeurs œdométriques utilisées

C_s	C_c	C_g	σ'_{p0} kPa	σ'_g kPa	e_g
0,0135	0,1192	0,0471	1752	750	0,406

4.3.2. Variation des gonflements en fonction de la hauteur d'érosion

Les résultats des calculs sont représentés sur le graphique de la figure 5. Pour plus de lisibilité, l'échelle des profondeurs a été limitée à 120m. Il est rappelé à ce propos que pour des hauteurs d'érosion de 20 et 40m les profondeurs d'origine du gonflement sont plus importantes.

Les résultats montrent que les gonflements élastiques sont faibles avec $0,01 \leq s_{el} \leq 0,02$ m

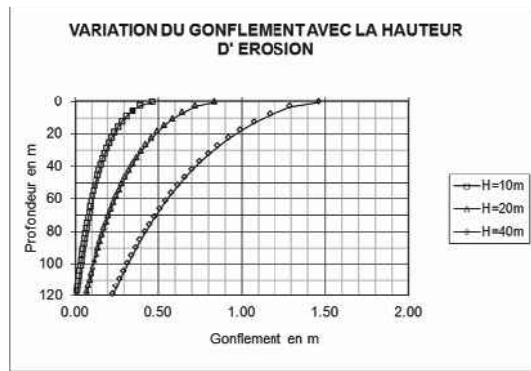


Figure 5. Résultats des calculs de gonflement pour différentes hauteurs H d'érosion.

5. ÉTABLISSEMENT D'UN MODÈLE DE CALCUL POUR UN DÉBLAI

Dans le cas d'un déblai, le terrassement n'est pas infini. De ce fait il se produit un amortissement des contraintes avec la profondeur, comme indiqué ci-après.

5.1. Effet de la profondeur

Nous allons considérer une charge sur un milieu semi-infini :

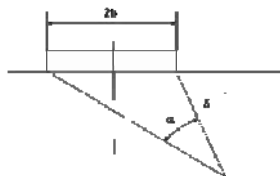


Figure 6. Méthode de Poulos & Davis

La contrainte verticale s'écrit (Poulos & Davis 1974) :

$$\sigma_z = \frac{P}{\pi} [\alpha + \sin \alpha \cos(\alpha + 2\delta)] \quad (14)$$

Dans le cas d'un déblai, le terrain peut être considéré comme infini de part et d'autre de la tranchée. Cela entraîne donc :

$$\text{quand } l \rightarrow \infty \text{ alors } \alpha + 2\delta \rightarrow \frac{\pi}{2}.$$

A partir de cette relation, il est alors possible de calculer la variation de contrainte dans l'axe du déblai en prenant en compte l'influence des deux parties situées de part et d'autre de ce dernier.

5.2. Résultats des calculs

Au préalable, il convient de noter que dans ce qui suit, les calculs ne peuvent être réalisés qu'avec la méthode numérique.

Ils ne sont réalisés que dans le domaine irréversible compte tenu des profondeurs importantes à atteindre pour avoir un comportement élastique.

Les caractéristiques géométriques du talus prises en compte sont définies ci-dessous :

Tableau 2. caractéristiques de déblai prises en compte

Hauteur déblai (m)	Pente talus $\tan \beta$	Largeur plateforme (m)	Largeur équivalente (m)
10	1/2	20	40
20	1/2	20	60
40	1/2	20	100

Les résultats des calculs obtenus sont représentés sur le graphique de la figure 7 ci-dessous.

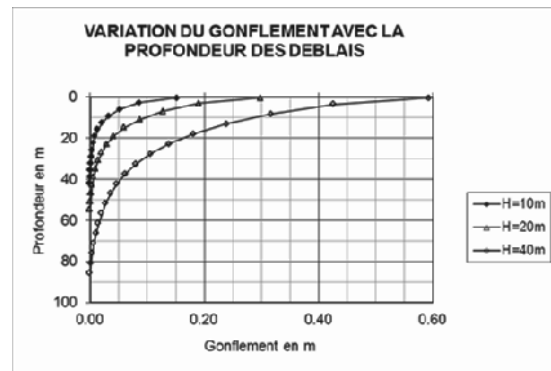


Figure 7. Résultats des calculs de gonflement pour différentes profondeurs H de déblai

Ces résultats montrent que la profondeur d'influence s'accroît avec la profondeur du déblai. Les amplitudes de gonflements calculées en fond de déblai pour ces configurations varient de 0,15 à 0,60 m.

Il est à noter qu'afin d'améliorer la prise en compte des talus du déblai, il est possible de décomposer le déblai en plusieurs charges rectangulaires épousant la pente du déblai. Les exemples présentés ici n'intègrent pas cette décomposition.

6. CONCLUSION

Les essais œdométriques spécifiques analysés, permettent de caractériser plus précisément le comportement des sols surconsolidés au déchargement. Celui-ci se traduit successivement dans l'espace œdométrique $[e; \log \sigma]$ par deux droites de pentes respectives C_s (indice de gonflement) puis C_g (coefficient de gonflement).

Ces deux droites se coupent en un point défini par $\sigma_G = \gamma z$.

A partir de ces essais, il est alors possible de calculer les gonflements résultant d'un déchargement généralisé (érosion) ou de la réalisation d'un déblai.

Dans le cas de déblai, où la géométrie du déchargement n'est pas infinie, il convient de tenir compte de l'amortissement des contraintes avec la profondeur, se traduisant par des gonflements beaucoup plus faibles que pour un déchargement généralisé.

Le cas de sols très fortement surconsolidés, présentant des pressions de préconsolidation trop élevées pour être correctement mesurées avec un essai œdométrique classique, reste encore à étudier et nécessite la mise au point de méthodes expérimentales spécifiques.

7. REMERCIEMENTS

Remerciements à Guilhem Teulade qui a été à l'origine de cette réflexion.

8. REFERENCES

- LEROUEL S., MAGNAN J.P. & TAVENAS F. remblais sur argile molle. Lavoisier 1985
- MAGNAN J.P. & SERRATRICE J.F. Détermination de la courbe d'état limite d'une marnes. Séminaire « De la Géologie au calcul des Ouvrages ». Grenoble 6-10 novembre 1995.
- POULOS & DAVIS. Elastic Solutions for Soils and Rocks. J. Wiley 1974.

Rate effects at varying strain levels in fine grained soils

Effets de vitesse de déformation à niveaux de déformation variant en sols à grains fins

Robinson S., Brown M.J.

Division of Civil Engineering, University of Dundee, Dundee, Scotland, UK.

ABSTRACT: There is a need for an improved understanding of rate effects over a wide range of strain rates in order to improve the modelling and analysis of high strain rate activities. Using triaxial testing on reconstituted kaolin over a wide strain rate range this paper examines the impact of strain rate on the aspects of soil response which are important in many areas of geotechnical engineering. It is demonstrated that shear strength, small strain stiffness and the elastic shear strain threshold are rate dependent.

RÉSUMÉ : Il est besoin pour une meilleure compréhension des effets la vitesse de déformation sur une grande gamme de vitesses de déformation en vue d'améliorer la modélisation et l'analyse des activités à vitesses de déformation haute. Par l'utilisation tests triaxiaux sur kaolin reconstitué sur une grande gamme de vitesses de déformation, présent document examine l'impact de vitesse de déformation sur les aspects de réponse du sol qui sont importants dans nombreux domaines d'ingénierie géotechnique. Il est démontré que résistance au cisaillement, rigidité à faible déformation et seuil de déformation élastique sont dépendant la vitesse de déformation.

KEYWORDS: rate effects, strain rate, strain level, small strain, elastic strain threshold, shear modulus, dynamic tests, triaxial, kaolin

1 INTRODUCTION

Rate effects are an important consideration in many areas of geotechnical engineering as they influence the strength and stiffness of the soil, however, the impact of rate effects is difficult to determine and analyse. Because of this, there is a need for a better understanding of rate effects and how they influence soil response.

One aspect that requires further study in order to achieve this is the relationship between rate effects and strain level, as this varies throughout the course of geotechnical activities. Being better able to consider rate effects throughout the entire strain range raises the opportunity to improve both accuracy and efficiency.

This paper considers the strain-rate dependence of rate effects over a wide range of strain magnitudes at shear strain rates from 0.333 to 60,000 %/hr.

1.1 The influence of strain rate

The most commonly used assumption in terms of rate effects is that of Kulhawy and Mayne (1990) which states that for every log cycle (tenfold) increase in strain rate, a ten percent increase in shear strength is expected. However, this has been shown to be variable, with the rate effects observed ranging from 9.5 to 20 % (Bea 1982). Another key consideration is that this assumption does not consider any variation of rate effects with strain level (Brown 2009), as it is based on a correlation using only peak shear strengths. Similarly, only undrained testing was used, meaning that the impact of drainage on the determination of the rate effect was not dealt with.

Drainage effects are significant as the strain rate influences whether there is time for drainage to occur. This means that rate effects tend to follow a “U-shaped” curve as shown in Figure 1. At extremely slow rates, there is time for pore pressure developed during shearing to fully dissipate. As the strain rate increases, the reducing time for drainage means that increasing pore pressures and hence lower shear strengths are observed in the partially drained region. As the strain rate is increased

further, the soil response becomes almost fully undrained, where pore pressures are at their maximum. After this point, viscous effects cause the observed shear strength to increase with strain rate up to a potential viscous limit (Chow and Airey 2011). The form of this shear strength-strain rate relationship has been verified by studies using a wide range of test methods from cone penetrometer testing to large scale shear boxes. (Steenfelt 1993, Lehane et al 2009)

Shibuya et al (1996) showed that the initial small strain stiffness, E_{max} , is rate independent and constant for the range of strain rates investigated (0.6 to 84 %/hr). It was also found that this linearity continued up to a strain defined as the elastic strain threshold, ϵ_{EL} , which was itself rate dependent, increasing with increasing strain rate. These findings were confirmed by Mukabi and Tatsuoaka (1999).

Lo Presti et al (1996) found that soil stiffness is rate independent below 0.001 % shear strain, and increases with strain rate beyond this point. To allow comparison of relative stiffnesses at different strains and strain rates, Lo Presti et al (1996) defined the strain rate coefficient, α .

There are numerous geotechnical applications where rate effects are an important consideration, including free falling penetrometer tests where velocities can reach up to 12 m/s

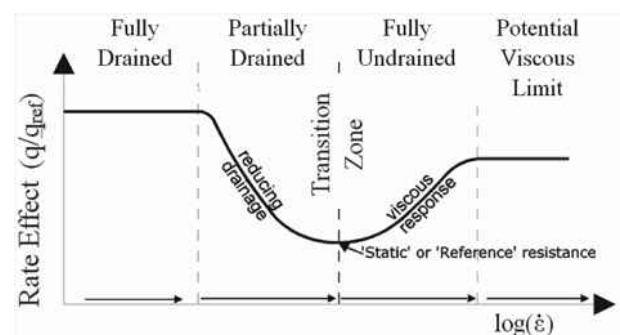


Figure 1. U-shaped curve showing the idealised variation of rate effect with strain rate (Quinn and Brown 2011)

(Chow and Airey 2011) and Statnamic pile tests (Brown and Hyde 2008) where the pile is displaced at rates of the order of 1 m/s. In these cases, rate effects cause the capacities derived from the tests to exceed static values; an issue which is usually dealt with by the inclusion of damping co-efficients in the analyses. (Brown and Powell 2013) These damping co-efficients are not always uniform throughout each test, but may vary with strain level. This highlights the importance of understanding the strain level dependence of rate effects in order to improve the accuracy of dynamic testing in the field.

2 MATERIAL TESTING AND PROCEDURES

The tests were carried out on reconstituted speswhite kaolin, the properties of which are shown in Table 1.

Table 1. Properties of the speswhite kaolin used

Property	Value
Plastic limit, w_p (%)	32.5
Liquid limit, w_L (%)	65.0
Plasticity index, PI (%)	32.5
Clay fraction (%)	80
Activity (%)	40.6
Specific surface area (m^2/g)*	36.7
Permeability, k (mm/s) [#]	1.17×10^{-6}
c_v ($m^2/year$) ⁺	23.52
MCSL	0.9
λ	0.101
N	2.678

* Determined from methylene blue spot testing

[#] Determined at an effective stress of 300 kPa

⁺ Determined for a 100 kPa stress increment

The samples were first prepared as slurry with a moisture content of 120 % using de-aired, de-ionised water before being one dimensionally consolidated to an effective stress of 180 kPa for three days. These were then trimmed to 200 mm length and 100 mm diameter to create triaxial samples. Once installed in the triaxial apparatus, the sample was saturated to an effective stress of 50 kPa at a back pressure of 300 kPa and then reconsolidated to an effective stress of 300 kPa to restore isotropic conditions. Sample drainage was facilitated by using vertical filter paper drains on the surface of the sample, connected to both the top and bottom drainage valves. These were required as the use of lubricated end platens in the testing meant that conventional drainage was not possible, and had the additional benefit of significantly reducing consolidation times.

2.1 Testing apparatus

The tests were carried out in a GDS advanced electromechanical dynamic triaxial rig specially modified to carry out high speed monotonic tests. The rig is capable of axial displacement rates of 100 mm/s, and during high speed testing it is controlled by a GDS digital system capable of controlling the axial displacement within a time interval of 0.1 milliseconds. Both the back and cell pressures were provided by GDS pressure controllers. Lubricated end platens of a similar design to those proposed by Rowe and Barden (1964) were used in order to minimise the inhomogeneity caused by end restraint

conditions. As these can introduce errors into the measurement of small strains using external methods, these were measured using Hall effect transducers, two axial and one radial, mounted directly on the sample providing a resolution of 1×10^{-6} % strain. Pore pressures were monitored using a mid-height pore pressure transducer mounted on the surface of the sample.

2.2 Testing programme

The testing programme consisted of triaxial tests at shear strain rates from 0.333 to 60,000 %/hr in order to investigate strain rate effects over as large a range as possible. These were carried out at a comparatively low effective stress of 300 kPa as previous studies have shown that greater rate effects are observed at higher moisture contents. (Bea 1982, Brown and Hyde 2008, Chow and Airey 2011) Throughout the testing programme, the samples were allowed to drain through the filter paper drains in order to allow rate effects due to consolidation to be investigated.

3 RESULTS AND DISCUSSION

3.1 Observed rate effects and their modelling

The rate effects observed at the various strain rates are shown in Figure 2, with a shear strain rate of 100 %/hr taken as the reference rate. In order to allow comparison with other studies using differing materials and sample sizes, the strain rates have been converted into the normalised dimensionless velocities used by Randolph and Hope (2004) as shown in Equation 1.

$$V = \frac{vd}{c_v} \quad (1)$$

where V is the normalised dimensionless velocity, v is the strain rate applied (in m/year), d is the sample diameter (in m) and c_v is the co-efficient of consolidation in $m^2/year$. As can be seen, the curve follows the behaviour expected with time for consolidation effects dominating up to $V = 11$, after which undrained viscous effects are significant.

In order to quantitatively assess the data, the rate effects model proposed by Randolph and Hope (2004) shown in Equation 2 has been fitted to the data using least mean square regression.

$$\frac{q}{q_{ref}} = \left(1 + \frac{b}{1 + cV^d} \right) \left\{ 1 + \frac{\lambda}{\ln(10)} \left[\sinh^{-1} \left(\frac{V}{V_0} \right) - \sinh^{-1} \left(\frac{V_{ref}}{V_0} \right) \right] \right\} \quad (2)$$

where b , c , and d are curve fitting parameters used to model the time for consolidation effects and λ is the rate effect per log cycle increase in strain rate used to model viscous effects. V_{ref} is the normalised velocity associated with chosen reference rate and V_0 is the point after which time for consolidation effects are negligible. This process was also repeated for the measured rate effects at selected strains to identify the variation in rate effects at different strain levels as shown in Figure 3. The parameters obtained from the curve fitting process are shown in Table 2.

The fitting parameters at peak strength (which occurs at varying strain levels) show that the rate effect per log cycle was found to be 22.5% which is higher than previous studies. This highlights the need to develop a framework to predict rate effects based on the current soil state and properties of the material in question.

Table 2. Parameters used to fit the model by Randolph & Hope (2004) to the data at each shear strain level

Property	0.17%	0.33%	0.67%	1.67%	2.67%	Peak
b	0	0	0.153	0.505	0.888	1.387
c	26.52	25.55	24.61	23.11	23.53	25.09
d	1.865	1.859	2.163	2.589	2.556	2.309
λ	0.275	0.251	0.244	0.212	0.181	0.225
V_0	18.17	21.08	17.76	14.10	6.81	23.39
V_{ref}	11.17	11.17	11.17	11.17	11.17	11.17

3.2 Strain level dependence of rate effects

Figure 3 shows that in the partially drained domain rate effects increase with strain level, which is intuitive as at greater strains there will be more time for consolidation to occur. Additionally, at the point at which the transition to undrained behaviour occurs, the rate effects were found to be relatively independent of the strain level. Beyond this point the rate effect per log cycle reduces from 27.5 % at 0.17 % shear strain to 18.1 % at 2.67 % strain, indicating that undrained rate effects reduce with strain level, which is important in areas such as rapid load pile testing (RLT) and assessment of static pile working loads.

Lo Presti et al (1996) developed a method for comparing rate effects at different shear strain levels by defining a strain rate coefficient, α (Equation 3). This is the change in shear modulus at a given strain over the log cycle increase in shear strain rate normalised by the shear modulus at a reference shear strain rate.

$$\alpha(\gamma) = \frac{\Delta G(\gamma)}{\Delta(\log(\dot{\gamma})) \cdot G(\gamma, \dot{\gamma}_{ref})} \quad (3)$$

This method was used to further investigate the apparent dependence of rate effects on strain level. Figure 4 shows the variation of the strain rate co-efficient, α , with shear strain for the shear strain rates considered. This confirms the earlier findings that for rates where there is time for consolidation, the rate effects observed increase with increasing strain. Similarly, the rate effects reduced with strain for the undrained tests. However, the graph highlights a significant difference in the strain rate co-efficient between the partially drained and undrained tests at extremely small strains.

3.3 Small strain behaviour

To further investigate the effect of strain level on rate effects the small strain data from the Hall effect transducers was examined. Figure 5 shows the variation of the shear modulus, G , for each of the rates. Of particular interest is the fact that the initial small strain shear modulus, G_0 , appears to vary with shear strain rate, reducing according to the correlation shown in Figure 6. One possible explanation for this is that at slower rates, there is more time for sample drainage at a micro-scale, causing an increase in the shear modulus at low shear strain rates. However, this does not explain the increased initial strain rate co-efficient at low strains in the fast tests.

Figure 7 shows the variation of the elastic shear strain threshold and this confirms that as found by Mukabi and Tatsuoka (1999), the elastic shear strain threshold increases with rate. A best fit was applied to this data and found that for the soil properties and stress conditions used, γ_{EL} varies as in equation 4.

$$\gamma_{EL} = 0.003 \dot{\gamma}^{0.2} \quad (4)$$

Considered in conjunction with Figure 5, this shows that at the initial strain levels considered in Figure 4 ($\gamma = 0.2$ %), the faster tests exhibited a greater stiffness than the slower rate tests, explaining the discrepancy in the initial values of the strain rate coefficients. This is due to the delay in stiffness degradation caused by the increase in the elastic shear strain threshold.

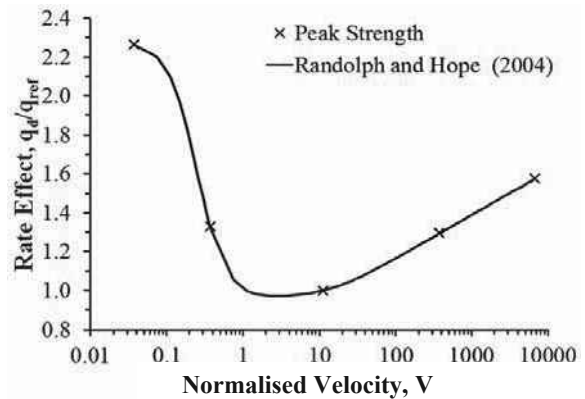


Figure 2. Graph of rate effect against normalised velocity at peak strength with the model by Randolph and Hope (2004) using a shear strain rate of 100%/hr as the reference rate

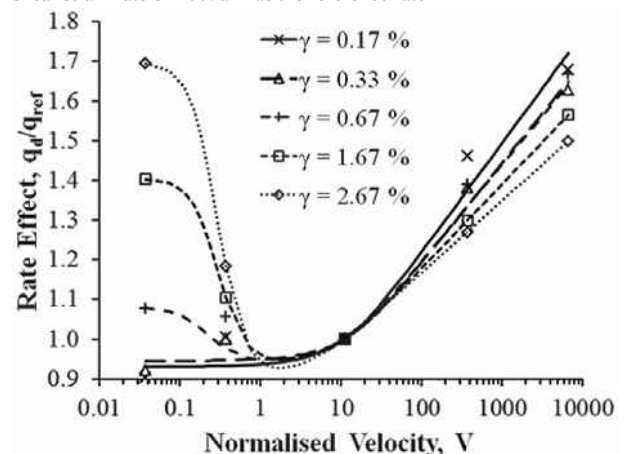


Figure 3. Graph of rate effect against normalised velocity at various shear strain levels shown with the model proposed by Randolph and Hope (2004)

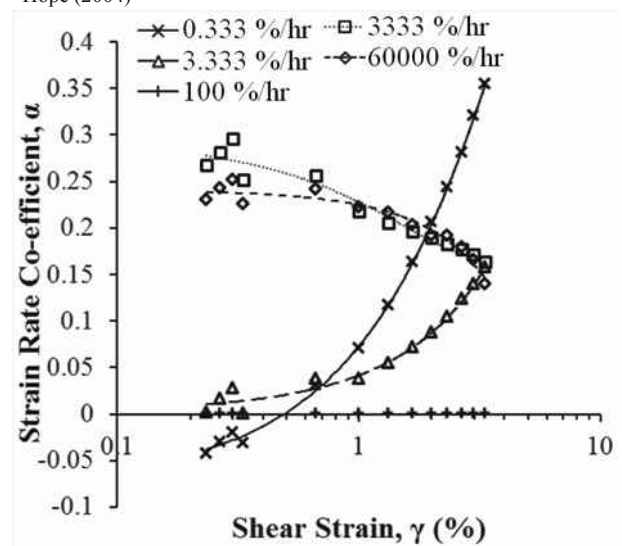


Figure 4. Graph of strain rate co-efficient against shear strain for various shear strain rates using a shear strain rate of 100 %/hr as a reference

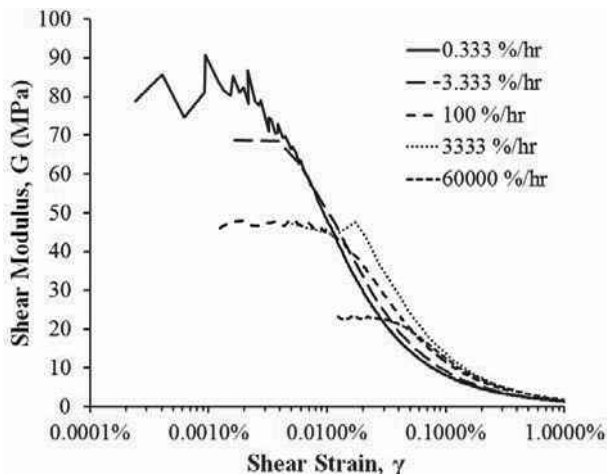


Figure 5. Graph of shear modulus against shear strain for various shear strain rates from Hall effect small strain transducer data

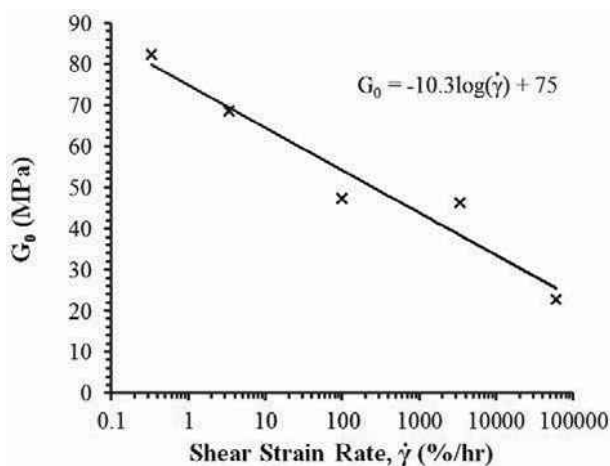


Figure 6. Graph of G_0 against shear strain rate

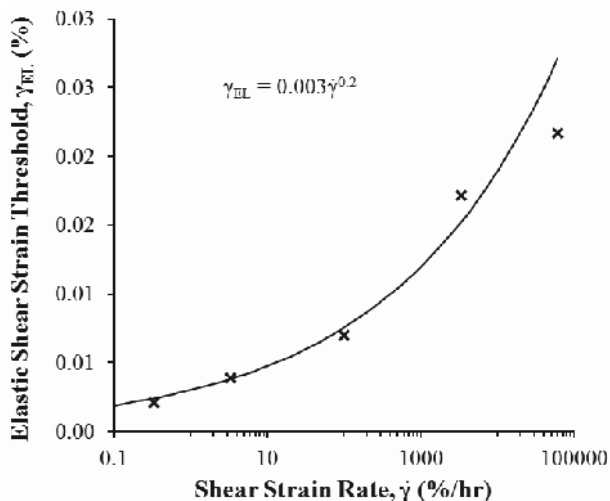


Figure 7. Graph of elastic shear strain threshold against shear strain rate

4 CONCLUSIONS

Based on triaxial tests on reconstituted kaolin at strain rates over more than 5 orders of magnitude, it has been shown that rate effects have numerous effects on the response of the soil, ranging from increased deviatoric strength to changes in small strain behaviour which are strain level dependent.

- Peak strength rate effects in the partially drained domain have been shown to be greater than undrained rate effects due to time for consolidation.
- The undrained rate effects observed have been shown to be higher than those reported by other studies at 22.5% per log cycle for peak strength.
- The importance of strain level in relation to rate effects has been highlighted, with rate effects increasing with strain when partially drained and reducing with strain when undrained.
- The applied strain rate was found to have a significant impact on the small strain response, with the elastic shear strain threshold, γ_{EL} , increasing with rate and G_0 appearing to reduce with rate.

It is hoped that the results and discussion in this paper will be of use in rate effect analysis and in the associated applications.

5 ACKNOWLEDGEMENTS

This work is supported by funding from the University of Dundee Division of Civil Engineering and the Institution of Civil Engineers Research and Development Enabling Fund, which is gratefully acknowledged.

6 REFERENCES

- Bea, R.G. 1982. Soil strain rate effects on axial pile capacity. *Proc. 2nd Int. Conf. on Numerical Methods in Offshore Eng.* 107-132.
- Brown, M.J. and Hyde, A.F.L. 2008. Rate effects from pile shaft resistance measurements. *Canadian Geotechnical Journal*, 45 (3), 425-431.
- Brown, M. 2009. Recommendations for Statnamic use and interpretation of piles installed in clay. *Rapid Load Testing on Piles*, Taylor and Francis, London, 23-36.
- Brown, M.J. and Powell, J.J.M. 2013. Comparison of rapid load test analysis techniques in clay soils. *ASCE Journal of Geotechnical and Geoenvironmental Engineering*. Available online 21/03/12.
- Chow, S.H. and Airey, D.W. 2011. Rate effects in free falling penetrometer tests. *Proc. Int. Symp. on Deformation Characteristics of Geomaterials*, 1-3 September, Seoul, 592-599.
- Krieg, S. and Goldscheider, M. 1998. Bodenviskosität und ihr Einfluß auf das Tragverhalten von Pfählen. *Bautechnik* 75, Ernst und Sohn.
- Kulhawy, F.H. and Mayne, P.W. 1990. Manual on estimating soil properties for foundation design. *Report EL-6800*, Electric Power Research Institute, Palo Alto.
- Lehane, B.N., O'Loughlin, C.D., Gaudin, C. and Randolph, M.F. 2009. Rate effect on penetrometer resistance in kaolin. *Geotechnique* 59, 41-52.
- Lo Presti, D.C.F., Jamiolkowski, M., Pallara, O. and Cavallaro, A. 1996. Rate and creep effect on the stiffness of soils. *ASCE GSP* 61, 166-180.
- Mukabi, J.N. and Tatsuoka, F. 1999. Influence of reconsolidation stress history and strain rate on the behavior of kaolin over a wide range of strain. *Geotechnics for developing Africa*, Balkema, 365-377.
- Quinn, T.A.C. and Brown, M.J. 2011. Effect of strain rate on isotropically consolidated kaolin over a wide range of strain rates in the triaxial apparatus. *Proc. Int. Symp. on Deformation Characteristics of Geomaterials*, 1-3 September, Seoul, 607-613.
- Randolph, M.F. and Hope, S. 2004. Effect of cone velocity on cone resistance and excess pore pressures. *Proc. Int. Symp. Eng. Practice and Performance of Soft Deposits*, 147-152.
- Rowe, P.W. and Barden, L. 1964. Importance of free ends in triaxial testing. *ASCE journal of the soil mechanics and foundations division* 90 (SM1), 1-27.
- Shibuya, S., Mitachi, T., Hosomi, A. and Hwang, S.C. 1996. Strain rate effects on stress-strain behavior as observed in monotonic and cyclic triaxial tests. *ASCE GSP: Measuring and modelling time dependent soil behavior* 61, 214-227.
- Steenfelt, J.S. 1993. Sliding resistance for foundations on clay till. *Predictive soil mechanics*, Thomas Telford, London, 664-684.

Comparison of Stress-Strain Behaviour of Carbonate and Silicate Sediments

Comparaison de la réponse contrainte-déformation de sédiments carbonatés et siliceux

Safinus S., Hossain M.S., Randolph M.F.

Centre for Offshore Foundation Systems, The University of Western Australia, Perth, Australia

ABSTRACT: Compared to silica sand, carbonate sand has considerably higher angularity, lower grain hardness and higher intra-particle porosity, which result in high friction angles and compressibility. The corresponding dilatancy is affected strongly by the confining stress. Thus, even for low relative densities, dilation occurs at low confining stresses, reflecting the greater particle interlocking compared to silica sand. However, with the increase of confining stress, the dilatancy is suppressed quickly, and finally diminishes completely at a relatively low stress level, due to particle degradation. This distinctive characteristic significantly influences the behaviour of continuously penetrating spudcan foundations in calcareous sediments. Centrifuge tests were carried out on spudcan foundations penetrating multi-layer soils with an interbedded strong layer composed with either carbonate or silica sand. All measures of spudcan punch-through severity were significantly lower for interbedded carbonate sand despite its higher friction angle ($\phi_{crit} = 40^\circ$) compared to silica sand ($\phi_{crit} = 34^\circ$). For the spudcan penetration through the sand layer to the lower clay layer, the soil failure mechanisms quantified by particle image velocimetry (PIV) analysis allowed for identifying the differences in the evolution of sand frustum beneath the advancing spudcan. The spreading angle of the frustum, which determines the size of the projected bearing area, was found to be proportional to the mobilised dilatancy.

RÉSUMÉ : Comparativement au sable siliceux, le sable carbonaté a une angularité considérablement plus élevée, une plus faible dureté de grain et une porosité intra-particulaire plus élevée, ce qui a pour effet de produire un angle de frottement et une compressibilité élevés. La dilatace de ce dernier est fortement affectée par la contrainte de confinement. Ainsi, même pour de faibles densités relatives, le comportement dilatat peut se produire pour des contraintes de confinement faibles, reflétant une tendance à l'imbrication des particules plus élevée par rapport au sable siliceux. Cependant, la dilatace est rapidement réprimée lorsque la contrainte de confinement augmente, et finalement disparaît complètement pour des niveaux de contrainte relativement faibles, du fait de la dégradation des particules. Cette caractéristique particulière influence de manière significative le comportement des fondations 'spudcan' lors de leur pénétration dans des couches de sédiments calcaires. Des essais en centrifugeuse ont été réalisés sur des fondations 'spudcan' pénétrant des sols multi-couches comprenant une couche intermédiaire composée soit de sable carbonaté, soit de sable siliceux. Toutes les mesures de sévérité du risque de pénétration du « spudcan » étaient significativement plus faibles pour le cas d'une couche intermédiaire de sable carbonaté, en dépit du fait que l'angle de frottement soit plus élevé ($\phi_{crit} = 40^\circ$), par rapport au sable siliceux ($\phi_{crit} = 34^\circ$). Pour la pénétration du « spudcan » à travers la couche de sable jusqu'à la couche sous-jacente d'argile, les mécanismes de rupture du sol quantifiés par vélocimétrie d'image de particule (PIV) ont révélé des différences d'évolution du tronc de sable en dessous du « spudcan ». L'angle d'ouverture du tronc de sable, qui détermine la taille de la surface portante projetée, s'est révélé être proportionnel à la dilatace mobilisée.

KEYWORDS: carbonate, silicate, dilation, spudcan foundations.

1 INTRODUCTION

Carbonate sediments are prevalent in Australian waters and in the Caspian Sea, Arabian Gulf, South China Sea, offshore Qatar and offshore Florida. Standard geotechnical analysis models were generally developed for silica sediment. Extreme care should be exercised when applying those models for carbonate sediments and indeed predictions using routine bearing capacity methods linked to the friction angle have been shown to be inappropriate. This is exacerbated for continuous penetration of spudcan foundations due to the gradually rising stress levels (SNAME 2008, InSafeJIP 2010). Discrepancies between the predicted and measured behaviour can be significant, especially in cases involving loose sand or high stresses. This results mainly because of the critical characteristics of calcareous sediments such as crushable particles, high in-situ void ratios and compressibility. With increasing stress level, grain particles are crushed, which alters the stress-strain behaviour.

Many studies have been undertaken in the last decades to improve understanding of the stress-strain behaviour of carbonate sediments (Datta et al. 1980, Evans 1987, Golightly

and Hyde 1988, Semple 1988, Coop 1990, Al-Dhouri and Poulos 1992, Randolph et al. 1999, Desrosiers and Silva 2002). Bioclastic carbonate sediments comprising skeletal and shell fragments usually have very angular grains, and hence high friction angles and low particle crushing strength parameter, Q (see Table 1). The use of friction angle as the sole strength indicator for sand often results in excessive overestimation of bearing capacity and underestimation of penetration depth (Overly 2012). Dutt et al. (1985) reported a much lower apparent friction angle (19°), through back analysis of the measured spudcan penetration response, compared to the value obtained from a direct shear test ($\phi_{crit} = 50^\circ$). Semple (1988) recorded relatively large settlements of offshore jack-up footings in carbonate sediments, which was attributed to the high compressibility of the soil. Current offshore design guidelines SNAME (2008) and InSafeJIP (2010) recommend using a reduced design friction angle (by as much as 25°) and a mobilisation (reduction) factor of ~ 0.25 , respectively, for assessing spudcan penetration resistance in carbonate sands.

In stratified sediments, with interbedded sand layers, the problem is even more complex. The likelihood and severity of a

foundation punch-through failure depends on the operative friction angle and associated dilation angle, both of which reduce with increasing stress level.

This paper reports the results from a series of basic characterisation tests conducted on reconstituted samples of carbonate sand to understand its behaviour. Centrifuge tests were also carried out on spudcan foundations penetrating four-layer deposits, with an interbedded carbonate or silica sand layer for direct comparison.

Table 1. Values of Q and ϕ_{crit} derived from triaxial compression tests (after Randolph et al. 2004, InSafeJIP 2010).

Sand	Mineralogy	Q	ϕ_{crit}	Reference
Ticino	Siliceous	10.8	33.5	Jamiolkowski
Toyoura	Quartz	9.8	32	et al. (2003)
Hokksund	Siliceous	9.2	34	
Mol	Quartz	10	31.6	Yoon (1991)
Kenya	Calcareous	8.5	40.2	Jamiolkowski
Quiou	Calcareous	7.5	41.7	et al. (2003)

2 STRESS-STRAIN BEHAVIOR

Simple shear tests with a Berkeley type apparatus were performed on uncemented skeletal carbonate sand recovered from the seabed of Australian North-West Shelf (NWS). Particles smaller than 75 μm and larger than 2.36 mm were removed by washing and sieving prior to testing. The achieved median grain size and coefficient of uniformity were $d_{50} = 0.22$ mm and $C_u = 2.3$, respectively. The high grain angularity and intra-particle void resulted in a high void ratio with minimum and maximum value of 0.91 and 1.36 respectively.

Drained tests with a lateral stress ratio $K = 0.4$ were performed on loose and medium dense sand to obtain the stress-strain behaviour. The results are shown in Figures 1 and 2, highlighting a strong dependency of the volumetric dilatancy on the confining stress. The values of relative densities (I_D) shown in the figures represent the condition just before shearing. Dilative volume change occurred even in loose sand at a vertical stress $\sigma_v = 200$ kPa (see Figure 1). This dilative response is not unusual owing to the particle angularity and interlocking. For most tests, shearing ended in dilative volume state, except two at higher stresses with $\sigma_v \geq 400$ kPa. Interestingly, for dense sand subjected to $\sigma_v \geq 300$ kPa, dilative response at intermediate strains turned to contraction close to the end of shearing, indicating the influence of continual particle breakage. A transient dilation at the highest stress of $\sigma_v = 700$ kPa was also noticed at shear strain levels of 15 to 20%.

The transition from dilative to contractive behaviour occurred at a lower stress level, $\sigma_v < 400$ kPa or mean stress $p' < 240$ kPa, compared to silica sand.

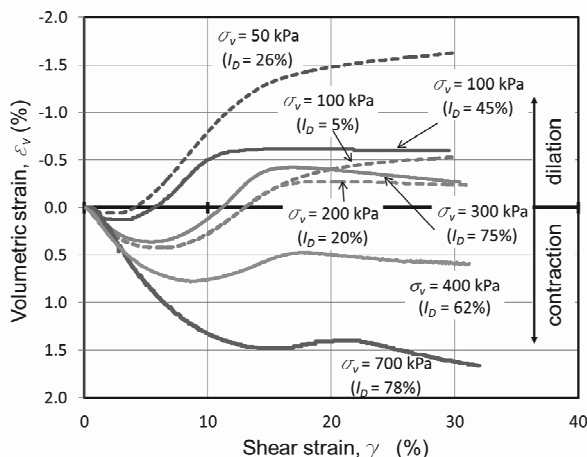


Figure 1. Volumetric change of carbonate sand in drained simple shear test with lateral stress ratio $K = 0.4$.

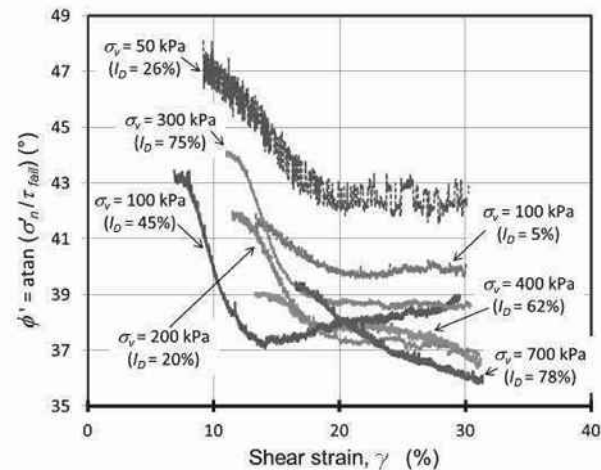


Figure 2. Effective friction angle of carbonate sand in drained simple shear test with lateral stress ratio $K = 0.4$ (post-peak condition only).

Peak dilation angle ψ_{peak} can be estimated using Bolton's (1986) empirical correlation

$$\phi'_{peak} - \phi_{crit} = 0.8\psi_{peak} = mI_R \quad (1)$$

$$I_R = I_D(Q - \ln p') - 1 \quad (2)$$

where m is a constant, taken as 3 for failure under triaxial or general loading conditions and 5 under plane-strain conditions, and I_R is the relative dilatancy. Some reported values for Q for siliceous grains range from 9.2 to 10.8, while lower values of 7.5 to 8.5 are reported for calcareous grains (see Table 1). Assuming that the dilation angle of the NWS carbonate sand turned to zero at a mean stress $p' = 240$ kPa, a value for Q can be calculated as 5.5.

The friction angle was interpreted using the AG method (Joer et al. 2011) by considering the actual observed shear plane. The normal and shear stresses were calculated for the diagonal shear plane and used to determine the friction angle. This method gave more realistic values compared to the traditional interpretation, which assumes a complementary shear stress on the vertical sample boundary. The calculated peak friction angle ϕ'_{peak} ranges from 39.5° to 48°, while the residual friction angles ϕ'_{res} from 35.8° to 42.9° (see Figure 2). No uniform steady state can be identified, rather a tendency of decreasing ϕ'_{res} with increasing confining stress is evident.

3 EFFECT OF PARTICLE DEGRADATION

In carbonate sands, high crushability and compressibility are led by the high intra-particle porosity, as discussed previously. Datta et al. (1980) reported the effect of grain crushing during shearing and found direct correlations between crushing and reduction of maximum principal effective stress ratio, change from dilative to contractive behaviour, more plastic stress-strain relation, and increase of failure strain.

Golightly and Hyde (1988) performed comprehensive isotropic drained triaxial (CID) tests on three different skeletal carbonate sands, all with a relative density of 97%. They reported results in terms of friction angle ϕ'_f , calculating according to $\phi'_f = \phi'_{peak} - \psi$, as shown in Figure 3. The dilation angles of the tested carbonate sands were found lower than those of the silica sand. The critical confining stress at which dilation was suppressed was also shown to be very low compared to silica sand. For instance, the dilation angle of Dogs Bay sand, which is mainly composed of skeletal mollusc fragments, decreased to zero at a confining stress of only 370 kPa. The siliceous Leighton Buzzard sand, on the other hand, has a constant dilation angle of around 9° to 10° for all tested confining stresses (< 1000 kPa).

A similar tendency can be found from the experimental results reported by Desrosiers and Silva (2002). A direct comparison was made between the behaviour of carbonate sand from South Australia and silica sand from the United Kingdom. At steady state shearing, the carbonate sand was already in contraction under 500 kPa confining stress, while the silica sand was still in dilation under 1000 kPa. The early transition to contractive behaviour attributes the fact of particle degradation of carbonate sand. The low grain hardness caused the grain to start crushing at relatively low stresses and hindered the development of potential maximum dilation. This behaviour agrees well with that observed on the NWS carbonate sand.

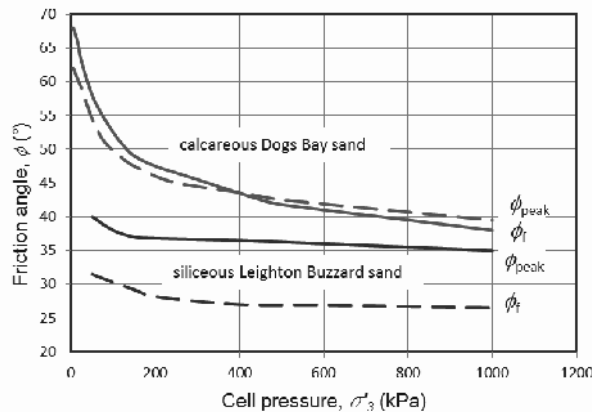


Figure 3. Comparison of dilative behaviour between calcareous and siliceous sand under various cell pressures (Golightly and Hyde 1988).

4 PRACTICAL APPLICATION: SPUDCAN RESPONSE

In order to examine the influence of this disparate characteristic of carbonate and silica sands on practical applications, model tests were carried out on spudcan foundations penetrating through four-layer soils, with a carbonate or silica sand layer interbedded in soft clay. The experimental program was carried out at 200 g in a drum centrifuge. The soil was confined within a purpose designed strongbox to facilitate producing multi-layer specimens, with the box mounted within the drum channel (Hossain and Randolph 2012).

Spudcan penetration tests were performed using a half-spudcan (HS) and a full-spudcan (FS) model of 60 mm (12 m prototype) diameter. The models were made from duraluminium and included a 13° shallow conical underside profile (included angle of 154°) and a 76° protruding spigot. The half-spudcan was designed to penetrate adjacent to the strongbox window, permitting the soil deformation to be captured by a camera. Separate full-spudcan penetration tests were performed away from the edges of the box to measure the load-penetration response, avoiding frictional resistance from the window.

Table 2 provides a summary of all centrifuge tests reported. Four tests encompassed two different four-layer profiles: (i) soft clay-carbonate sand-soft clay-stiff clay; (ii) soft clay-silica sand-soft clay-stiff clay. These multi-layer clay samples were prepared off the centrifuge. Two samples of uniform strength were prepared by consolidating thoroughly mixed, and then de-aired, kaolin slurry at 1 g in separate cells. Two different final pressures were used to obtain comparatively strong and soft samples. Each clay layer, as detailed in Table 2, was then cut to size of the strongbox. The bottom two (3rd and 4th) clay layers were amassed in the strongbox. A layer of water was poured into the strongbox. Dry super fine silica sand (or carbonate sand) was then air-pluviated into the strongbox on top of the placed lower layers. A loose to medium dense layer was deposited by raining the sand maintaining a relatively small sand drop height of about 100 mm. The sand surface was carefully levelled and the top clay layer was placed.

Table 2. Summary of centrifuge tests reported ($D = 12$ m).

Test	Layer 1		Layer 2		Layer 3		Layer 4	
	t_1/D	Soil	t_2/D	Soil	t_3/D	Soil	t_4/D	Soil
FS1				Carbonate sand				
HS1	0.25	Soft clay	0.5	Silica sand	0.96	Soft clay	0.33	Stiff clay
FS2				Silica sand				
HS2				sand				

Commercially available kaolin clay and super fine silica sand are commonly used for centrifuge model tests at UWA and an abundance of reliable data exists regarding the geotechnical properties (e.g. Stewart 1992, Cheong 2002). The carbonate sand was dredged directly from the North-West Shelf of Australia, as discussed previously. The critical state friction angles of the silica and carbonate sands were 34° and 40°, respectively.

The densities of the sand layers, which were determined by measuring the total added sand weight and the volume formed for all cases, corresponded to an average relative density, I_D , of 44%. For the clay beds, characterisation tests were carried out using a T-bar penetrometer, of diameter 5 mm and length 20 mm (model scale).

Figures 4 and 5 show the results from full-spudcan and half-spudcan tests, respectively. The load-penetration responses (see Figure 4) are presented in terms of ultimate bearing pressure, $q_u = P/A$ (where P is the penetration resistance and A is the largest plan area of the spudcan), as a function of normalised penetration depth, d/D . The potential for punch-through failure, with a local maximum in penetration resistance followed by some reduction, occurred for all cases investigated. The severity of failure is conventionally quantified by (a) the degree of post-peak reduction in resistance and (b) the ‘additional penetration’ before the peak resistance is re-established. By comparing the penetration resistance profiles for Test FS1 and Test FS2, on identical soil profiles with identical sand relative density, the measures of punch-through severity were significantly higher for the sandwiched silica sand despite its lower friction angle ($\phi_{crit} = 34^\circ$ compared to 40°). This is due to the behaviour of carbonate sand, as discussed previously and also described below.

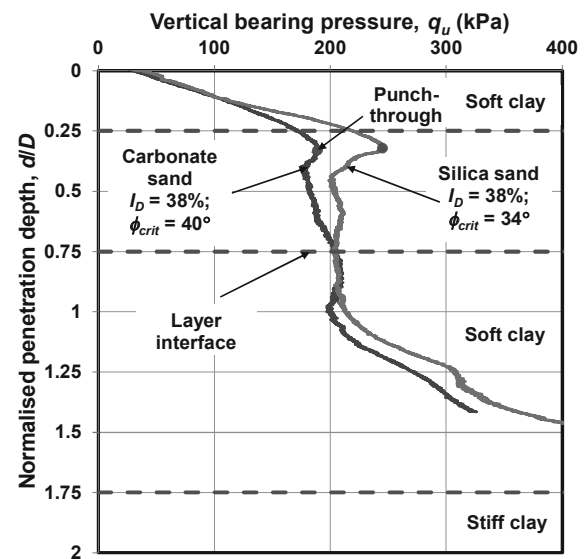


Figure 4. Effect of interbedded sand mineralogy on load penetration response: severity of punch-through (Tests FS1 and FS2, Table 2).

The accompanying soil deformation patterns are shown in Figure 5 by means of contours of the incremental absolute soil flow velocity v normalised by the foundation speed v_{spud} . The ratio v/v_{spud} of unity indicates that the soil moves with a speed

equivalent to that of the spudcan. The soil deformations were directed predominantly vertically down in the 2nd layer and laterally out in the lower (3rd) soft layer. The soil around the spudcan edges just started to flow back into the cavity formed above the spudcan. It can be seen that, under this relatively high confining stress in an embedded layer, the load spread angle is about 8° in carbonate sand and 19° in silica sand. The load spread angle is sometimes taken as the dilation angle (Lee et al. 2009; Teh et al. 2009). As such, it can be concluded that the interbedded carbonate sand layer showed less dilatancy. Furthermore, the trapped plug height (and hence the bearing base) is slightly lower for carbonate sand.

In both deposits, with the progress of penetration, the dilatancy was suppressed quickly and hence a plug with the shape of an inverted truncated cone, bounded by clear shear planes, was formed in the stronger (2nd) layer and moved down with the spudcan. Continual backflow provided a seal above the advancing spudcan and limited the cavity depth.

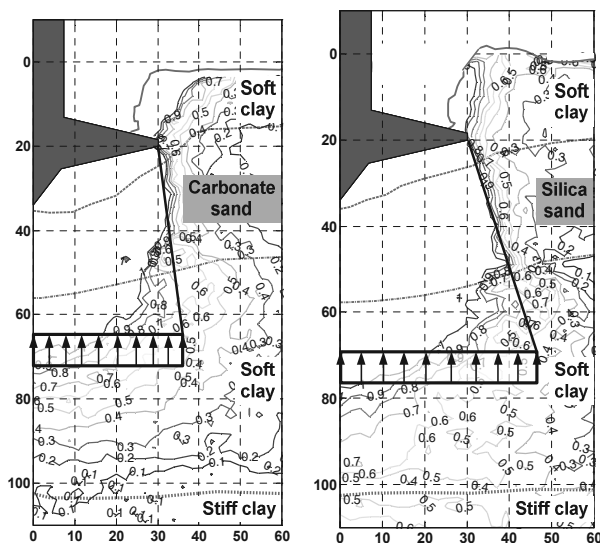


Figure 5. Effect of interbedded sand mineralogy on dilation and load spread angle (Tests HS1 and HS2, Table 2).

5 CONCLUDING REMARKS

This paper reported results from a series of simple shear tests for characterising carbonate sand dredged directly from Australian North-West Shelf. The stress-strain behaviour was compared with those of silica sand, focusing particularly on dilatancy. To examine the influence of dilatancy on foundation performance, a series of centrifuge model tests were carried out on spudcan foundations penetrating four-layer soils, with a carbonate or silica sand layer interbedded in soft clay layers. The following key conclusions can be drawn from the results presented in the paper.

1. The dilatancy of carbonate sand was affected strongly by the confining stress. Even for relative density as low as 5%, in contrast to silica sand, dilative behaviour was shown to occur, reflecting the greater interlocking compared to silica sand.
2. With the increase of confining stress, dilatancy of carbonate sand was suppressed quickly, and eventually diminished completely at a relatively low stress level, due to particle degradation. In contrast, silica sand showed dilatant behaviour at stresses > 1000 kPa.
3. This distinctive characteristic influenced the behaviour of continuously penetrating spudcan foundations, causing a less severe punch-through failure in an interbedded carbonate sand compared to that in silica sand layer, with significantly lower bearing capacity.

6 ACKNOWLEDGEMENTS

The research presented here was undertaken with support from the Australian Research Council through the Linkage Project LP110100174. The work forms part of the activities of the Centre for Offshore Foundation Systems (COFS), currently supported as a node of the Australian Research Council Centre of Excellence for Geotechnical Science and Engineering and in partnership with The Lloyd's Register Educational Trust. This support is gratefully acknowledged, as is the assistance of the drum centrifuge technician, Mr. Bart Thompson and soil technician, Mrs. Satoko Ishigami.

7 REFERENCES

- Al-Dhouri R.H. and Poulos H.G. 1992. Static and cyclic direct shear tests on carbonate sands. *Geotechnical Testing Journal*, *GTJODJ* 15 (2), 138-157.
- Bolton M.D. 1986. The strength and dilatancy of sands. *Géotechnique* 36(1), 65-78.
- Cheong J. 2002. *Physical testing of jack-up footings on sand subjected to torsion*. Honours Thesis, The University of Western Australia.
- Coop M.R. 1990. The mechanics of uncemented carbonate sands. *Géotechnique* 40 (4), 607-626.
- Datta M., Gulhati S.K., and Rao G.V. 1980. Crushing of carbonate sands during shear. *Offshore Technology Conference*, Houston.
- Desrosiers R. and Silva A.J. 2002. Strength behavior of marine sands at elevated confining stresses. *Marine Georesources and Geotechnology* 20: 1-19.
- Dutt R.N., Moore J.E., Mudd R.W., and Rees, T. E. 1985. Behavior of piles in granular carbonate sediments from offshore Philippines. *Offshore Technology Conference*, Houston.
- Evans K.M. 1987. A model study of the end bearing capacity of piles in layered carbonate soils. PhD Thesis, University of Oxford, UK.
- Golightly C.R. and Hyde A.F.L. 1988. Some fundamental properties of carbonate sands. *Engineering for Carbonate Sediments*. Balkema, Rotterdam.
- Hossain M.S. and Randolph M.F. 2012. Spudcan foundations on multi-layered soils with interbedded sand and stiff clay layers. *Int. J. Offshore and Polar Engineering*, 22(3), 248-255.
- InSafeJIP 2010. *Improved guidelines for the prediction of geotechnical performance of spudcan foundations during installation and removal of jack-up units*. Joint Industry Funded Project.
- Jamiolkowski M.B., Lo Presti D.C.F. and Manassero M. 2003. Evaluation of relative density and shear strength of sands from cone penetration test (CPT) and flat dilatometer (DMT). *Soil Behaviour and Soft Ground Construction*, Eds. J.T. Germain, T.C. Sheahan and R.V. Whitman, ASCE, GSP 119, 201-238.
- Joer H.A., Erbrich C.T. and Sharma S.S. 2011. A new interpretation if the simple shear test. *Proc. Int. Symp. on Frontiers in Offshore Geotechnics*, Perth.
- Lee K.K., Randolph M.F., and Cassidy M.J. 2009. New simplified conceptual model for spudcan foundations on sand overlying clay soils. *Offshore Technology Conference*, Houston.
- Overy R. 2012. Predicting spudcan penetration in loose sand from measured site soil parameters. *Proc. 7th Int. Conf. Offshore Site Investigation and Geotechnics*, Society for Underwater Technology, London, 589-596.
- Randolph M.F., Jamiolkowski M.B. and Zdravković L. 2004. Load carrying capacity of foundations. *Proc. Skempton Memorial Conf.*, London, Vol. 1, 207-240.
- Randolph M.F., Watson P.G. and Fahey M. 1999. *An integrated study of foundation systems in carbonate sediments*. MERIWA Project No. 268.
- Semple R.M. 1988. State of the art reports: The mechanical properties of carbonate soils. *Proc. Int. Conf. on Calcareous Sediments*, Perth, Australia, 2, 807-836.
- SNAME 2008. *Recommended practice for site specific assessment of mobile jack-up units*. T and R Bulletin 5-5A, 1st Edition – Rev. 3, Society of Naval Architects and Marine Engineers, New Jersey.
- Stewart D.P. 1992. *Lateral loading of piled bridge abutments due to embankment construction*. PhD Thesis, Univ. of Western Australia.
- Teh K.L., Leung C.F., and Chow Y.K. 2009. Prediction of punch-through for spudcan penetration in sand overlying clay. *Offshore Technology Conference*, Houston.
- Yoon Y. 1991. Static and dynamic behaviour of crushable and non-crushable sands. *PhD Thesis*, Ghent University.

A new test field in sulphide clay with test embankments for study of compression properties

Un nouveau essai sur le terrain d'argile sulfatée en mettant en place des remblais d'essai pour l'étude des propriétés de compression

Westerberg B., Andersson M.

Swedish Geotechnical Institute / Luleå University of Technology, Sweden

ABSTRACT: In this paper on-going research of compression properties of Swedish fine-grained sulphide soils is presented. In the paper, the establishment of two test embankments founded on sulphide clay at a new test field is described. Some selected results from the project are presented. The characteristics of the particular types of sulphide soils investigated are shortly described. The overall purpose of the research project is to improve the possibilities to predict long term settlements of structures founded on sulphide soils.

RÉSUMÉ : Cet article présente des recherches en cours sur des propriétés de compression des sols à granulométrie fine suédois. L'article décrit l'établissement de deux remblais d'essai fondés sur argile sulfatée, dans un nouveau essai sur le terrain. Quelques résultats choisis du projet sont présentés. Les caractéristiques des types particuliers de sols sulfatés enquêtés sur le terrain sont brièvement décrites. L'objectif général du projet de recherche est d'améliorer les possibilités de prédire des tassements à long terme des structures fondées sur les sols sulfatés.

KEYWORDS: clay, embankments, sulphide, organic, compression, creep, settlements, geotechnical engineering.

1 INTRODUCTION

There is only limited knowledge concerning compression properties of sulphide soils. There is thus a need for improved tools to better predict settlements including creep settlements in sulphide soils. Most often the predicted settlements of an construction founded on sulphide soils deviates significantly from those measured, and normally the predicted settlements are too small.

Sulphide soils, as designated in this paper, are found and common along the coast line of the Gulf of Bothnia, i.e. in north-eastern Sweden over a distance of about 900 km and north-western Finland, figure 1. Sulphide soils in Sweden are not uniform, but properties like grain size distribution, water content, and density vary as for other fine-grained soils with location and often with depth (Westerberg and Andersson 2009). Sulphide soils in Sweden are most often designated as organic silt to organic silty clay and in cases with higher organic contents as silty or clayey gyttja (Larsson et al. 2007). In sulphide soils, the structure is often relatively porous and the voids between the mineral grains and clay particles are filled with pore water, organic material and iron sulphide (Pusch, 1973; Eriksson et al. 2000). The organic matter and iron sulphide are believed to contribute to the open structure, low bulk density and high water content. The sulphide soils are normally coloured black or varved with black bands and the black colour comes from the iron sulphide (FeS). Sulphide soils have normally low undrained shear strength, typically 10-20 kPa, and are in general very compressible and show significant creep behaviour (Westerberg et al. 2005). These properties are accentuated when the temperature is raised from in situ ground temperature to room temperature in the laboratory (Eriksson 1992).

In this paper a few results are presented from an on-going research project at the Swedish Geotechnical Institute (SGI) concerning compression properties and settlement predictions of sulphide soil (Andersson 2012). The results have been obtained in connection with two test embankments founded on sulphide

clay in a new test field at Lampen outside the city of Kalix (see figure 1) and next to the new railroad Haparandabanan. The main purpose of the project is to improve the knowledge of compression properties of sulphide soils, with focus on the creep properties. Another aim is to establish extensively instrumented test embankments where settlements and pore pressures can be measured during many decades to come.



Figure 1. The approximate location of sulphide soils (shaded area) in north-eastern Sweden and north-western Finland (Schwab 1976).

2 FIELD AND LABORATORY INVESTIGATIONS

Field investigations of the properties of the sulphide soil in the test field at Lampen have been performed mainly by cone penetration tests, field vane tests and Swedish piston sampling. An extensive program of laboratory tests, mainly oedometer tests, both by incremental loading and constant rate of strain, creep tests in oedometers, permeability tests and undrained direct simple shear tests, was conducted for the determination of compression and strength properties. In figure 2 bulk density, liquid limit and water content of the sulphide soil in the test field are presented. For the sulphide soil the clay content varies between 25-36 %, the organic content between 2.7-5.1 % of dry weight, the iron content between 2.9-4.3 % of dry weight and the sulphur content between 0.5-1.6 % of dry weight. The soil is designated as organic sulphide clay. Below the test embankments the depth of the sulphide soil is in general about 7-9 m and overlaying a moraine. At the ground surface there is a layer of peat down to about 0.4 m overlaying a layer of about 0.6 m relatively soft dry crust of sulphide soil.

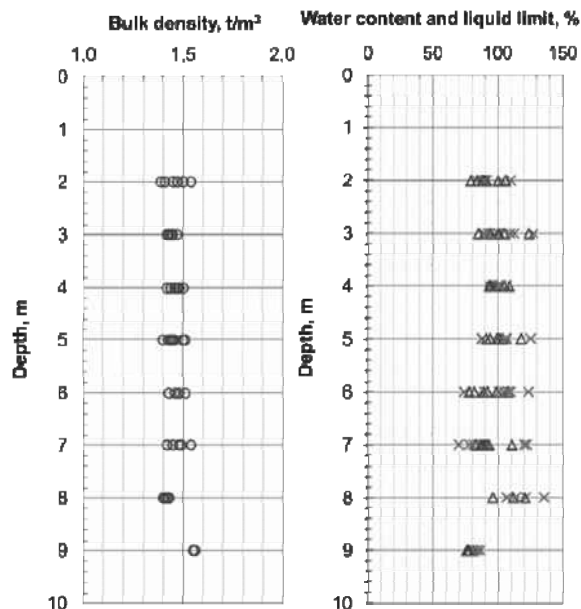


Figure 2 Bulk density, water content (triangle symbol) and liquid limit (cross symbol) with depth in the test field at Lampen.

Evaluated preconsolidation pressures from CRS and incremental oedometer tests conducted at approximately soil temperature and using the evaluation methods from Sällfors (1975) and Casagrande (1936) respectively are presented in figure 3. In the upper part of the soil profile, the two tests give about the same results, and in the lower part the incremental loading tests give higher values of preconsolidation pressure. There is a significant overconsolidation with respect to the current in situ stresses in the whole soil profile, figure 3.

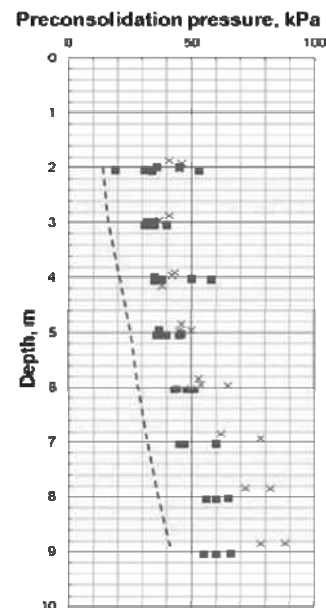


Figure 3 Preconsolidation pressures at the test field at Lampen evaluated from CRS oedometer tests (square symbol) and incremental oedometer tests (cross symbol), and estimated effective vertical in situ stresses (dotted line) with depth.

3 TEST EMBANKMENTS AND INSTRUMENTATION

Two test embankments with square bases of $30 \times 30 m^2$, one with the final height 2.0 m (embankment 1), and the other with 1.5 m height (embankment 2) have been constructed the year 2010 up to 1.5 m, and raised 2011 to 2.0 m (embankment 1), figures 4-6. A fine-grained moraine, with an average compacted bulk density of $2.0 t/m^3$, was used as construction material for the embankments, leading to pressures of about 40 kPa and 30 kPa from the two embankments respectively.



Figure 4 Cautious construction of embankment around the measuring equipment.



Figure 5 Construction up to height 1.5 m for embankment 1.



Figure 6 Finished construction of embankment 2 (1.5 m height). Photograph taken about two years after finished the construction.

Equipment for measuring movements (deformations), pore pressures and soil temperatures have been installed below and beside the two embankments, see figure 7 for embankment 2. For measuring horizontal movements inclinometer tubes of PVC with a square cross-section were used. This equipment has been designed for measurements in soft clays but had never before been tested in sulphide soils, figure 8.

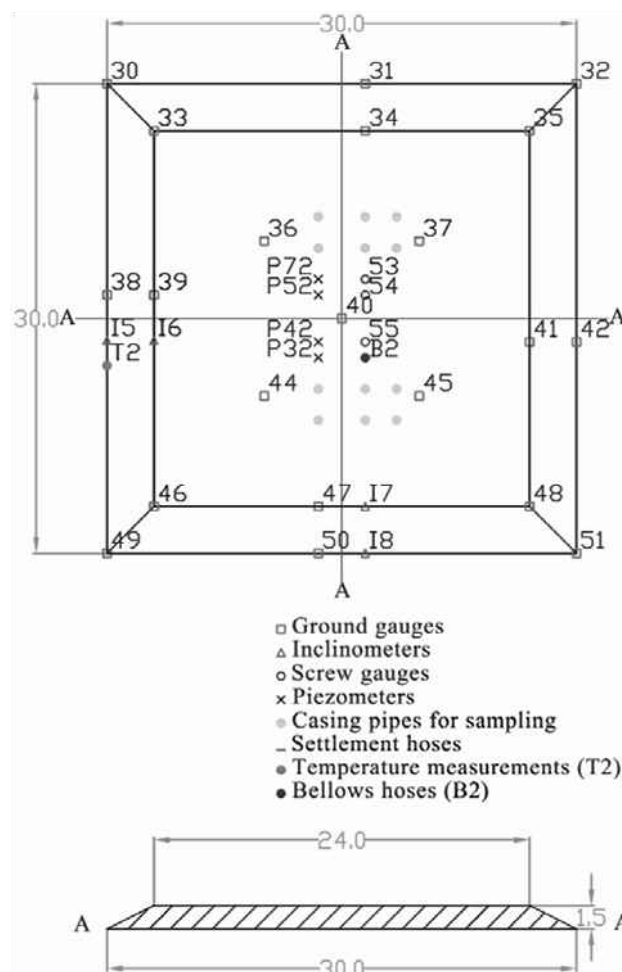


Figure 7 Plan of measuring equipment and cross-section of embankment 2 (in meters).



Figure 8 Inclinometer tube of PVC with square cross-section with steel point at the end.

4 FIELD MEASUREMENTS

In figure 9 is presented one example of measured vertical movements from settlement hoses and settlement gauges, embankment 2 about 1.2 years after construction. The settlements are similar when comparing the two measuring methods.

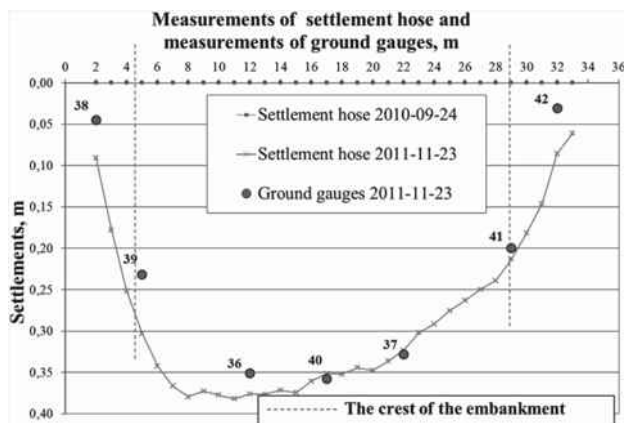


Figure 9 Measured settlements by settlement hose and ground gauges, embankment 2 about 1.2 years after construction.

5 CONCLUSIONS

Two test embankments, one with the height 1.5 m and the other with 2.0 m, have been constructed on sulphide clay at a new test field. Extensive measuring equipment has been installed to measure movements and pore pressures in the underlying sulphide clay. The construction of embankments and installation of equipment has been successful and the measured deformations obtained from the different equipments are in general agreement.

6 ACKNOWLEDGEMENT

The research project is supported financially by the former Swedish Road Administration, the former Swedish Railway Administration, both organisations now joined as the Swedish Transport Administration (Trafikverket), the Development Fund of the Swedish Construction Industry (SBUF), the Swedish Geotechnical Institute, and Luleå University of Technology. The research project is conducted in cooperation between the Swedish Geotechnical Institute and Luleå University of Technology.

7 REFERENCES

- Andersson M. 2012. Kompressionsegenskaper hos sulfidjordar, En fält- och laboratoriestudie av provbankar. Licentiate thesis, Division of Mining and Geotechnical Engineering, Luleå University of Technology, Luleå. (in Swedish)
- Casagrande A. 1936. The determination of the Preconsolidation Load and its Practical Significance. 1st International conference on Soil Mechanics and Foundation Engineering, Cambridge, Massachusetts, Vol. 3, p. 60.
- Eriksson L. G. 1992. Sulfidjordars kompressionsegenskaper, Inverkan av tid och temperatur. Licentiate thesis, 1992:08L, Division of Soil Mechanics and Foundation Engineering, Luleå University of Technology, Luleå. (in Swedish)
- Eriksson L. G., Mácsik J., Pousette K. and Jacobsson, A. 2000. Sulfidjord – en problemjord längs Norrlandskusten. Bygg & Teknik, Vol. 92, No. 1. (in Swedish)
- Larsson R., Westerberg B., Albing D., Knutsson S. and Carlsson E. 2007. Sulfidjord – geoteknisk klassificering och odränerad skjuvhållfasthet. (Sulphide soil – geotechnical classification and undrained shear strength). Research report, 2007:15, Luleå University of Technology, Luleå / Swedish Geotechnical Institute, Report No. 69, Linköping. (in Swedish)
- Pusch R. 1973. Influence of organic matter on the geotechnical properties of clay. National Swedish Building Research, Document 11:1973, Stockholm.
- Schwab E. 1976. Bearing capacity, strength and deformation behaviour of soft organic sulphide soils. Ph.D. Thesis, Department of Soil and Rock Mechanics, Royal Institute of Technology, Stockholm.
- Sällfors 1975. Preconsolidation pressure of soft high plastic clays. Ph.D. Thesis, Department of Geotechnical Engineering, Chalmers University of Technology, Gothenburg.
- Westerberg B., Albing D. and Larsson R. 2005. Research on strength and deformation properties of Swedish fine-grained sulphide soils. Osaka: Proceedings, International conference on soil mechanics and geotechnical engineering, 16, vol 2, pp 623-626.
- Westerberg B. and Andersson M. 2009. Undrained shear strength and compression properties of Swedish fine-grained sulphide soils. Alexandria: Proceedings, International conference on soil mechanics and geotechnical engineering, 17, vol 1, pp 72-75.

Laboratory testing issues related to crushable sands

Questions concernant des essais de laboratoire sur les sables écrasables

Wils L., Van Impe W.F., Haegeman W.
Ghent University

Van Impe P.O.
AGE (Geotechnical and geo-environmental consultancy)

ABSTRACT: Working with crushable sands, laboratory test issues arise related to their crushable nature and specific morphology. Complications occur already when determining the minimum and maximum density. The major issue is erosion of the sand during transportation and flow at testing, resulting in an ever increasing maximum density, affecting the definition of relative density, D_r . In addition, as with any other granular material at the initial stages of compression, a crushable sand densifies when interparticle voids decrease; yet when at a relatively low stress level particles also start to crush, their so-called intraparticle voids – typical for sands from bioclastic origin – gain importance. Other issues influencing the crushable sand matrix are: polishing of angular grains into more rounded particles; larger shells sheltering smaller grains; apparent cohesion due to interlocking of the angular particles, creating sandclusters with the appearance of larger particles; etc. During any kind of treatment of the crushable sand samples, attrition of the particles must be addressed with great care and granulometric properties should be closely monitored throughout.

RÉSUMÉ : Pendant les essais de laboratoire sur le sable écrasable, beaucoup de questions se posent liées à sa nature déformable et sa morphologie spécifique. Des complications surviennent déjà lors de la détermination de la densité minimale et maximale. Le problème majeur est l'érosion du sable pendant le transport, ce qui augmente continuellement la densité maximale, et influence la densité relative, D_r . En outre, comme dans autres sols granulaires dans les premières étapes de compression, un sable déformable densifie par diminution des vides interparticulaires; pourtant, à partir d'un niveau de contrainte relativement bas les particules commencent aussi à s'écraser, puis les vides dits intraparticulaires – trait des sables bioclastiques – rendent important. D'autres questions aussi influent la matrice des grains écrasables : le polissage des grains anguleux; grandes coquilles abritant des petits grains de sable; la cohésion apparente par l'emboîtement des particules angulaires, ainsi créant des clusters qui ressemblent à de grosses particules; etc. Pendant tout type de traitement des sables écrasables, l'attrition des particules doit être abordée avec soin et les propriétés granulométriques doivent être surveillés.

KEYWORDS: Crushable sands ; Relative density ; Sieving ; Breakage.

1 INTRODUCTION

At the Ghent University Laboratory of Geotechnics, an ongoing research program focuses on the stress-strain behaviour of crushable sands. Several laboratory test issues inherent to the brittle nature and specific morphology of these grains complicate the performing and interpretation of experiments and the deduction of soil mechanical properties. Tests on crushable sands need an appropriate approach that differs from non-crushable, silica sands.

2 PREPARATION OF SAMPLES FOR LABORATORY TESTING

Since tests in soil mechanics laboratories represent geotechnical situations in the field, samples need to be prepared at a relevant density. Usually the minimum and maximum dry density of a sand are determined according to standard methods, and a sample is prepared at the density of interest, and possibly saturated if it is to resemble offshore conditions.

The minimum density in the research on crushable sands is determined following the ASTM D4254. Method A, herein described consists of pouring a mass of sand into a mould with known volume through a funnel. The pouring spout of the funnel has an inside diameter of either 12.7 mm or 25.4 mm, depending on the grain size of the sand. Should the grains be small enough, the small funnel can be used and this will yield the lowest density owing to blocking of the sand and a very

slight sand stream. However, for a fine calcareous sand the small funnel gets blocked due to a cohesive arch formed by the angular particles. Jamming can only be avoided by gradually filling the funnel, along with the progression of the sand through the outlet. Therefore, contrary to what is normally the case, lower densities are obtained using the large funnel and pouring sprout.

For determining the maximum density of a crushable sand, the usual Proctor compaction is replaced by a less invasive vibratory table densification process as described in ASTM D4253. In this procedure a known mass and volume of sand is compacted by fixing the mould onto a table that vibrates at a frequency of 50 Hz for 12 minutes. A surcharge of 13.8 kPa keeps the sand grains from segregating. The test is repeated at different peak-to-peak amplitudes between 0.30 and 0.91 mm, where the optimal amplitude is the one where the energy transfer of vibration to compaction is most efficient, providing the maximum density by definition. This procedure only holds true as long as there is no crushing during vibration. Occurrence of crushing was evaluated by comparison of grain size distributions before and after the densification. Instead of having to divide the whole ± 3.5 kg of sand into manageable portions to sieve, it was originally assumed that crushing would occur mostly in the top layer; hence a 120 g sample was taken from there. The granulometry then showed no evidence of crushing and the test could therefore be continued at higher

amplitude reusing the tested sand. It was noticed (as shown in Figure 1) that the density increased with higher amplitudes, however for every refill of the mould extra sand was needed, indicating that there had been erosion indeed. This had gone by unnoticed in the sand portion from the top because segregation during vibration had caused the crushed particles to flow downwards in the mould. When the entire ± 3.5 kg sand mass was mixed and representative portions of 120 g were taken and sieved, crushing was eventually evidenced (Figure 2).

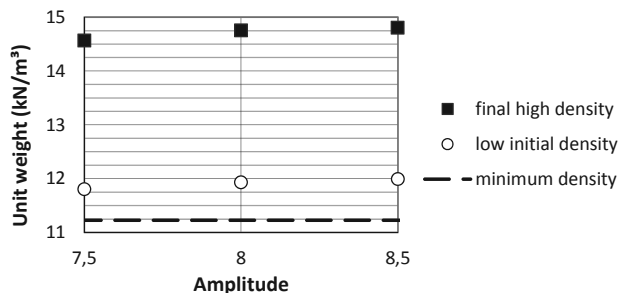


Figure 1. Vibratory table densification of S1-sand: Reusing the same sand causes crushing, which increases both minimum and maximum density. Note: The dimensionless “amplitudes” in this figure simply denote the positions (on a scale of 10) on the rheostat controlling the force of vibration, they are not the actual amplitudes.

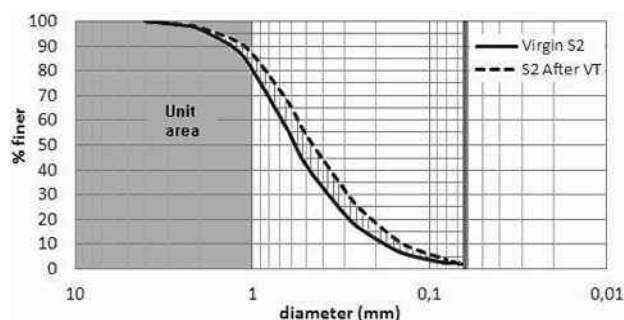


Figure 2. The granulometric distribution of S2-sand before and after vibratory table test at “amplitude” 6.5. Crushing can be quantified by Hardin’s relative breakage factor, here $B_r = 0.1$ (see below).

High amplitude shaking causes crushing of the calcareous sand particles, altering the sand and leading to an ever increasing minimum and maximum density, the increase being greater with increasing amplitude. It is important to define single correct lower and upper values of the density in calculations of the relative density since the definition of $D_r = (e_{max} - e)/(e_{max} - e_{min}) \times 100$ – a ratio of two small numbers – makes it vulnerable to errors. Moreover, e_{min} should be obtained without crushing since the interpretation of any geotechnical test on crushable sand is accompanied by the degree of crushing during the test, and during the test only; any crushing that occurs during preparation of the sample at a certain density is unwanted.

To some extent however, there is always a change in grain size distribution when handling crushable sands. Youd (1973) offers two determining factors by stating that a test has caused “no crushing” when the increase of particles passing the N°200 sieve ($63 \mu m$) is less than 1.5% of the total specimen weight, and the maximum increase of particles passing any sieve is less than 2%.

Still, even when there is no discernible change in grain size distribution, the densification process causes breaking of asperities of the angular sand grains. Particles become more rounded and very small dust particles are released. Both effects leave the granulometric distribution unchanged: the diameter (intermediate dimension) of a polished particle is still as before, and the fines lack substance to appear in the sieve fractions

(especially when weighted with the total sample mass) or they get lost due to the electrostatic effect during dry sieving. Yet these small morphological and granulometrical alterations do affect the structure and the packing (Figure 3) – and hence the behaviour of the sand to a considerable degree.

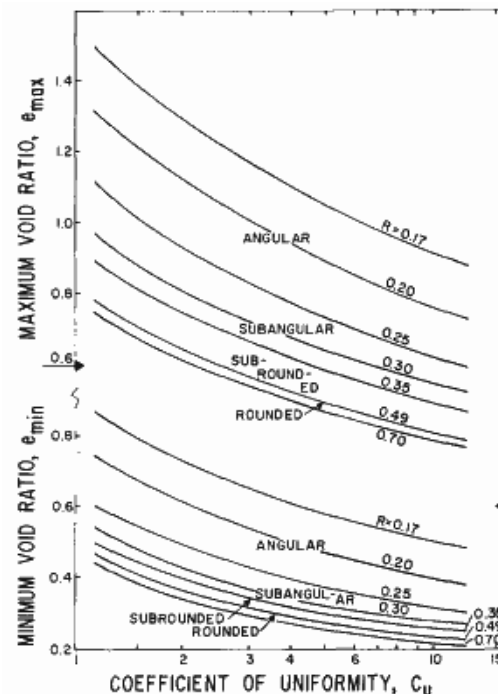


Figure 3. Gradation and particle shape control the possible packing configuration of sand (Youd 1973).

The ASTM D4253 notes that the sides of the mould may be struck a few times using a rubber hammer “to settle the soil so that the surcharge base plate can be easily placed into position and there is no surge of air from the mold when vibration is initiated”. Although the amount and intensity of the hammerblows is not specified, a few (e.g. 3) strikes in small-scale densification tests on calcareous sand already prove effective for further densification. The impact of the hammer causes the initially unstable sand structure (with its many “bridges”) to collapse into a more stable configuration that allows further settlement by vibration rather easily and without crushing. The standard method to obtain the maximum density of a sand according to the Japanese Geotechnical Society (1992) is based on this idea of densification by shear stress: a $\varnothing 40$ mm mould is filled with sand in 10 layers. After pouring each layer, the mould is impacted sideways with 100 hammerblows. Some questions remain with this method: the boundaries of the small mould affect the densification process and the obtained maximum density value, and the absence of a top plate enables segregation of the sand grains.

The intent of the formulation of relative density D_r is to evaluate the potential for the assembly of sand particles to form a compressible structure. Yet when the maximum density increases due to crushing, the upper limit in D_r changes. The particular compaction behaviour of crushable sands (being totally different from that of silica sands) might call for a reviewed test method for e_{min} . Most papers disregard this problem of a correct determination of D_r of crushable soils. At best they urge to keep in mind that the real D_r -value can differ from the laboratory one, when using it for engineering purposes. Mostly the problem is avoided by using the natural void ratio e_0 as the primary variable and potential index to compressibility, without relating it to e_{max} and e_{min} . For example Semple (1988,

Figure 4) states that there is continuity of response of bioclastic and silica sands in their consolidation behaviour, with the initial void ratio as the controlling factor that differs only because of the increased grain angularity of the bioclastic soil. Crushing is therefore not perceived as a consequence of the mineralogy, but as a consequence of the higher stress concentrations at fewer interparticle contacts.

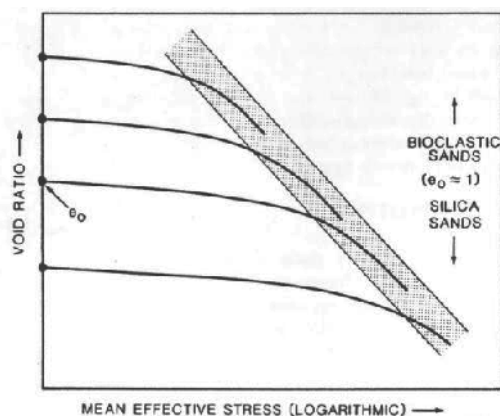


Figure 4. Schematic representation of the generalised compression behaviour of bioclastic and silica sands: virgin compression occurs in a diffuse band (Semple 1988).

In large-scale model testing, calibration chambers are filled with a homogeneous sand mass by pluvial deposition using a hopper and upward moving diffuser sieves. For calcareous sands this method has proved not useful (Nutt 1993), due to the irregularity of the grain shapes and the dependence of the method on a fluent sand rain for a homogeneous density. A small-scale variant of this method is therefore certainly not possible as a means of sample preparation.

Therefore, for the on-going study on crushable sand, homogeneous samples are prepared at the desired void ratio by sideways hammerblows using a small surcharge to avoid segregation. For triaxial specimen – where there are no sidewalls to blow against – satisfying homogeneity is obtained by pouring and tamping the sample in layers in accordance with the undercompaction method introduced by Ladd (1978).

In laboratory testing, not all densities can be obtained for crushable sand samples. A relative density of $D_r = 60\%$ seems both practical in the lab and relevant on site after deposition. Higher densities are hard to come by without crushing, lower values result in both structural collapse (obscuring the progression of the test) and irreproducible samples - two sand samples at the same high void ratio can have a different structure.

Especially when saturating the samples with water by means of flushing, a careful approach must assure that the grain structure remains intact. At low stresses this is of particular importance, since at low stress the sand structure is the controlling parameter of its behaviour.

Bioclastic sand samples contain the skeletal remains of marine organisms. Many of these shells are hollow, but in their complete state a high capillarity initially prevents water from penetrating the shells and reaching the enclosed “intraparticle voids”. Yet, upon crushing of the particles the voids open up, introducing air in the once saturated sample. For S2-sand, the different specific gravities from pycnometer tests on complete sand grains and on grinded powder ($\rho_{s,grains} = 2.82 \text{ Mg/m}^3$, $\rho_{s,powder} = 2.88 \text{ Mg/m}^3$) are indicative of the amount of pores enclosed within the shells (2.4% on average).

Regarding the dimensions of small-scale test set-ups for calcareous sand, it is difficult to follow standard recommended ratios. For example, according to the ASTM D2435 standard for

one-dimensional consolidation tests, the minimum initial height of the oedometer specimen shall not be less than ten times the maximum particle diameter. However, due to calcareous particles often being quite elongated, a presieved sample whose particles all passed the sieve with aperture size 4 mm will still contain particles measuring 10 mm and more, as is demonstrated in Figure 5. Although in this case a $\varnothing 40 \text{ mm}$ mould would satisfy the dimensional requirements that were formulated in the standard, such a mould might influence the calcareous sand matrix beyond what is the case for more rounded sand particles.

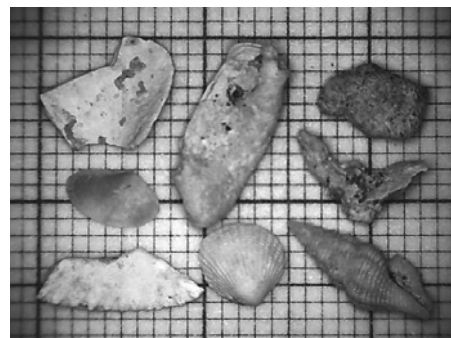


Figure 5. Sieved S2-sand particles with $D < 4 \text{ mm}$. From placement on millimeter-paper, it is clear that they have dimensions exceeding 4 mm.

3 INTERPRETATION OF LABORATORY TEST DATA

Sieving is an essential part of the research on crushable sands. Every test is accompanied by two sievings: one before and one after the test. From the shift in the granulometric distribution, the degree of crushing that occurred during the test is quantified by means of a certain “breakage factor”, aiding the interpretation of the test data.

An alternative to sieving beforehand, is to assemble virgin samples following a preset granulometric distribution. Thus, there is no need to take statistical granulometric variances into account, as the smallest shift in the grain size distribution will unquestionably indicate crushing. Additionally, when specimens share the same initial granulometry they have identical “breakage potential”, defined by Hardin (1985) as the area above the virgin granulometric curve. This renders comparison of Hardin’s “total breakage” after the test (the difference between the areas above the curve before and after testing) more straightforward, with Hardin’s “relative breakage factor” (the ratio between the total breakage and the breakage potential) reserved for comparing different sands.

Such manual assembly of a sample is however not feasible for larger samples. Moreover, after testing of a large sample, only a small portion can be sieved, lest the sieves get clogged. How and where within the large specimen a representative sample must be taken depends on the interest of the research and the kind of test (e.g. after shearing in a shear box, one might want to sample near the shear plane whereas this wouldn’t normally be achievable after a triaxial test; after vibratory table densification of sand, segregation has taken place within the mould, etc.).

After testing S2-sand under loads in the order of 1 – 8 MPa, recovering of the sand from the mould is problematic because of an apparent cementation. The only way to empty the mould without further damaging the sand is by wetting the sample, thus weakening the particle bonds. After drying the sample in the oven at 105°C , the sand mass is covered in a thin cemented crust made of salt crystals, sticking the small particles together (Figure 6a). Before sieving, the agglomerates must therefore be manually broken into separate particles (Figure 6b), otherwise they would create the illusion of larger particles. On the other hand, the impression of crushing would be formed when fine

particles that are initially hidden in larger shells come loose after manipulation.

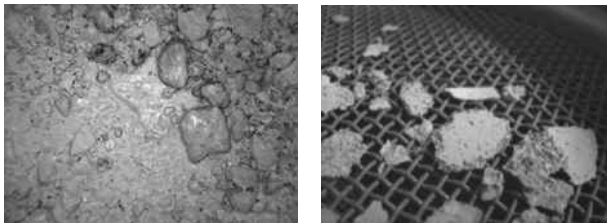


Figure 6a and 6b. S2-sand after wetting and drying in the oven.

Even though crushing is most often evaluated through granulometric distribution of equivalent grain diameters, microscopic analysis of the sand grains is a useful addition to fully quantify the effects of crushing as well as less invasive abrasion and polishing. Microscopy allows for exact measurement of grain dimensions, as opposed to a system of sieves that is based on the assumption that particles are perfectly spherical.

On a sample scale, Scanning Electron Microscopy might be a useful tool for viewing the sand matrix and the intergranular contacts, thus offering more insight in the effective stresses.

4 DEDUCTION OF SOIL MECHANICAL PROPERTIES

Findings from the laboratory tests must finally be translated to geotechnical situations with crushable sand. On site conditions are generally different from the laboratory: The very angular shape of the grains causes an anisotropic behaviour; water and temperature environment will be different; there is the scale effect of the small-scale samples; ageing can cause cementation that is hard to simulate in the lab. Crushing is a time effect and therefore has more time to establish in the field than in the lab. Corresponding to a continuous creep with time, the behaviour of crushable sands also depends on the strain rate (Nutt and Houlsby 1991), i.e. the possibility to form new structures without crushing.

Correlations between geotechnical parameters that are based on in situ experience with non-crushable sands, omit the specific behaviour of crushable materials. As a solution, Wehr (2005) links the differing best-fits between cone resistance and relative density for silica and calcareous sand, through a “shell correction factor” – without however revising the entire correlation.

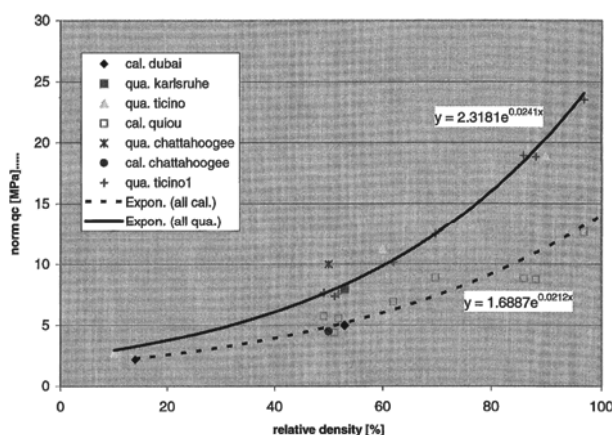


Figure 7. The correlations between relative density and cone resistance for quartz and calcareous sands can be related through a “shell correction factor” that depends on the relative density (Wehr 2005).

5 CONCLUSIONS

Just as research on sand differs from research on clay, crushable sands also need a different approach from non-crushable,

cohesionless sands. Sample preparation, test procedure and interpretation need revising to accommodate for the crushability and the angularity of the grains. The phenomenon of crushing, which causes the sand-grains and sand-structure to alter significantly during a test, complicates testing and the used sand should be closely monitored throughout.

Usage of a crushable sand causes grain degradation, which manifests as erosion or breakage, anyhow altering the minimum and maximum density of the sand. Consequently, unlike silica sands, geotechnical parameters of crushable sands cannot be derived through correlations with the calculated “relative density”, which is sensitive to errors in the limit density determinations and which has no real meaning for crushable sand. By the same token, crushing cannot be evaluated through sieving alone, a closer look at the grain morphology by means of microscopy is required to fully understand the structure and thus, the behaviour of crushable sands.

6 ACKNOWLEDGEMENTS

The authors acknowledge the Belgian Dredging company DEME, for providing the sands used in this study.

7 REFERENCES

- ASTM Standard D4254, 2000, “Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density,” ASTM International, West Conshohocken, PA, 2006, DOI: 10.1520/D4254-00, www.astm.org.
- ASTM Standard D425, 2000 (2006), “Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table,” ASTM International, West Conshohocken, PA, 2006, DOI: 10.1520/D4253-00R06, www.astm.org.
- ASTM Standard D2435, 1996, “Standard Test Methods for One-Dimensional Consolidation Properties of Soils,” ASTM International, West Conshohocken, PA, DOI: 10.1520/D2435-96, www.astm.org.
- ASTM Standard D422, 1963 (1998), “Standard Test Method for Particle-Size Analysis of Soils,” ASTM International, West Conshohocken, PA, DOI: 10.1520/D0422-63R98, www.astm.org.
- Hardin B.O. 1985, Crushing of Soil Particles, *Journal of Geotechnical Engineering*, ASCE, 111(10), 1177-1192.
- JIS A 1224 Standard, 1992 (2009), Test Method for Minimum and Maximum Densities of Sands, Japanese Geotechnical Society, www.jiban.or.jp (in Japanese).
- Ladd R.S. 1978, Preparing Test Specimens Using Undercompaction, *Geotechnical Testing Journal*, ASCE, 1(1), 16-23.
- Nutt N.R.F. 1993, Development of the Cone Pressuremeter, DPhil, University of Oxford.
- Nutt N.R.F., Houlsby G.T. 1991, Calibration Tests on the Cone Pressuremeter in Carbonate Sand, *Proceedings of the First International Conference on Calibration Testing*, Potsdam, New York, 265-276.
- Sample R.M. 1988, The mechanical properties of carbonate soils. *Proceedings of the International Conference on Calcareous Sediments*, Perth, Australia, Vol. 2, 397-406.
- Wehr J. 2005, Influence of the Carbonate Content of Sand on Vibro Compaction, *Proceedings of the 6th International Conference on Ground Improvement Techniques*, Coimbra, Portugal, 625-632.
- Youd T.L. 1973, Factors Controlling Maximum and Minimum Densities of Sands, *Evaluation of Relative Density and Its Role in Geotechnical Projects Involving Cohesionless Soils*, ASTM STP 523, American Society for Testing and Materials, 98-112.

Non-coaxial behaviour of sand in drained rotational shear

Comportement non-coaxial de sable drainé en cisaillement rotationnel

Yang L.-T., Yu H.-S., Wanatowski D., Li X.

Nottingham Centre for Geomechanics, University of Nottingham, Nottingham, United Kingdom

ABSTRACT: This paper presents an experimental investigation into the non-coaxial behaviour of saturated sand in rotational shear. A series of drained rotational shear tests has been performed on dense Leighton Buzzard sand in a hollow cylinder apparatus. The degrees of non-coaxiality defined as the deviation between the major principal stress direction and the major principal strain increment direction were analyzed. The test results demonstrate that the mechanical behaviour of sand under rotational shear is generally non-coaxial, and the variation of the non-coaxiality degree shows a periodic trend during the tests. It was also observed that the effective stress ratio has a significant effect on the non-coaxiality of sand. The larger the stress ratio, the lower degree of non-coaxiality is induced. The results also show that the volumetric strain of sand induced by cyclic rotation of principal stress axes is mainly contractive. Most of the volumetric change occurs during the first few rotation cycles and its accumulative rate tends to decrease as the number of cycles increases.

RÉSUMÉ : Cet article détaille une analyse expérimentale du comportement non-coaxial de sable saturé en cisaillement rotationnel. Une série de tests de cisaillement rotationnel drainé sur un sable Leighton Buzzard dense dans une presse triaxiale à cylindre creux a été réalisée. Les degrés de non-coaxialité calculés comme l'écart entre la direction de la contrainte principale majeure et la direction de l'incrément de la déformation principale ont été analysés. Les résultats des tests montrent que le comportement mécanique du sable sous cisaillement rotationnel est généralement non-coaxial, et que la variation du degré de non-coaxialité présente une certaine périodicité au cours des tests. Un impact non-négligeable du rapport de contrainte effective sur la non-coaxialité du sable a également été observé. Plus le rapport de contrainte est grand, plus le degré de non-coaxialité induit est bas. Les résultats montrent également que la déformation volumique du sable induite par rotation des axes principaux de contrainte est essentiellement contractante. La plupart du changement volumique se produit au cours des quelques premiers cycles de rotation et son taux d'accumulation tend à diminuer lorsque le nombre de cycles augmente.

KEYWORDS: non-coaxiality, HCA, drained rotational shear, sand

1 INTRODUCTION.

Non-coaxial behaviour refers to the non-coincidence of principal stress directions and principal plastic strain rate directions (Yu 2008). Evidenced by many laboratory observations, granular materials, like sand, often exhibit non-coaxial behaviour (Roscoe et al. 1967, Symes et al. 1984, Arthur et al. 1986, Gutierrez et al. 1991). In many practical situations, ignoring the effects of soil non-coaxiality may be unsafe (Yu and Yuan 2006).

Many loading situations such as those induced by earthquakes, traffic loading and sea waves involve cyclic rotation of principal stress axes. Extensive experimental studies on principal stress rotation have been carried out on granular soils over the past few decades (e.g., Ishihara and Towhata 1983, Symes et al. 1984, Nakata et al. 1998, Yang et al. 2007, Tong et al. 2010). Several test results have shown the significant effect of principal stress rotation on the response of soil specimens. Plastic deformations as well as non-coaxiality between the principal directions of stress and strain increment can be induced due to principal stress axes' rotation.

Until now, most studies on soils undergoing cyclic rotation of principal stress axes have mainly focused on the undrained strength, flow deformation and liquefaction behaviour of soils during rotational shear. The cyclic behaviour under drained conditions has not attracted much interest. Very little literature directly addresses the non-coaxial behaviour of sand under cyclic rotation of principal stress axes.

This paper presents an experimental investigation into the non-coaxial behaviour of saturated sand in drained rotational shear. All the tests were performed in a hollow cylinder apparatus (HCA) which allows an independent control of the magnitude of the three principal stresses and the inclination of the major principal stress axis. The degrees of non-coaxiality calculated as the deviation between the major principal stress direction and the principal strain increment direction at different stress ratios during rotational shear were analyzed.

2 TESTING ARRANGMENT AND PROCEDURES

2.1 Hollow cylinder apparatus

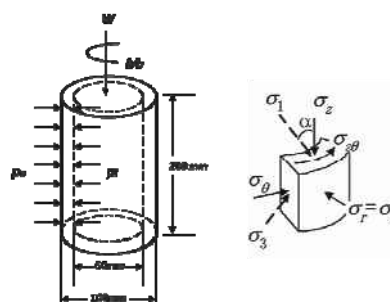


Figure 1. Applied loads and stress components in a hollow cylindrical specimen.

In this study, the HCA, developed by GDS Instruments Ltd, was used. For the details of the testing system, see Cai (2010). The hollow cylindrical specimen had an inner radius of 30 mm, outer radius of 50 mm and height of 200 mm. As shown in Figure 1, the loading of the specimen consisted of an axial load W , torque M_T , inner cell pressure P_i and outer cell pressure P_o . The application of these stress components enables the control of axial stress σ_z , radial stress σ_r , circumferential stress σ_θ , and shear stress $\sigma_{\theta z}$, on an element in the wall of the hollow cylindrical specimen.

Since the stresses are not uniformly distributed across the sample wall especially with unequal inner and outer cell pressures, it is necessary to compute average values. The equations used to calculate the average stress and strain components are listed in Table 1.

Table 1. Equations used to calculate stresses and strains (Hight et al. 1983)

Direction	Stress	Strain
Vertical	$\bar{\sigma}_z = \frac{W}{\pi(r_o^2 - r_i^2)} + \frac{P_o r_o^2 - P_i r_i^2}{r_o^2 - r_i^2}$	$\bar{\epsilon}_z = \frac{w}{H}$
Circumferential	$\bar{\sigma}_\theta = \frac{P_o r_o - P_i r_i}{r_o - r_i}$	$\bar{\epsilon}_\theta = -\frac{u_o + u_i}{r_o + r_i}$
Radial	$\bar{\sigma}_r = \frac{P_o r_o + P_i r_i}{r_o + r_i}$	$\bar{\epsilon}_r = -\frac{u_o - u_i}{r_o - r_i}$
Shear	$\bar{\sigma}_{\theta z} = \frac{3M_T}{2\pi(r_o^3 - r_i^3)}$	$\bar{\gamma}_{\theta z} = \frac{2\theta(r_o^3 - r_i^3)}{3H(r_o^2 - r_i^2)}$

r_o : outer radius, r_i : inner radius, H : height of specimen, w : axial displacement, u_o : outer radius displacement, u_i : inner radius displacement, θ : torsional angle.

The major principal stress, intermediate principal stress and minor principal stress are calculated by using Eqs. (1a)-(1c).

$$\sigma_1 = \frac{\sigma_z + \sigma_\theta}{2} + \sqrt{\left(\frac{\sigma_z - \sigma_\theta}{2}\right)^2 + \sigma_{\theta z}^2} \quad (1a)$$

$$\sigma_2 = \sigma_r \quad (1b)$$

$$\sigma_3 = \frac{\sigma_z + \sigma_\theta}{2} - \sqrt{\left(\frac{\sigma_z - \sigma_\theta}{2}\right)^2 + \sigma_{\theta z}^2} \quad (1c)$$

The stress path in drained tests can be characterized by four independent parameters, namely the mean principal effective stress p' , deviatoric stress q , intermediate principal stress parameter b and the angle α_σ between the major principal stress σ_1 and the vertical direction. These parameters are defined in Eqs. (2a)-(2d).

$$p' = \frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3} \quad (2a)$$

$$q = \sqrt{\frac{1}{2} \left[(\sigma'_1 - \sigma'_2)^2 + (\sigma'_1 - \sigma'_3)^2 + (\sigma'_2 - \sigma'_3)^2 \right]} \quad (2b)$$

$$b = \frac{\sigma'_2 - \sigma'_3}{\sigma'_1 - \sigma'_3} \quad (2c)$$

$$\alpha_\sigma = \frac{1}{2} \tan^{-1} \left(\frac{2\sigma_{\theta z}}{\sigma_z - \sigma_\theta} \right) \quad (2d)$$

2.2 Sample preparation method

The tests were performed on dense Leighton Buzzard (Fraction B) sand, with an initial relative density of $D_r \approx 70\%$. Leighton Buzzard sand is made up of sub-rounded particles and mainly composed of quartz. It has a specific gravity of 2.65, mean particle size of 0.62 mm, minimum void ratio of 0.52 and maximum void ratio of 0.79 (Cai 2010). The water sedimentation method was employed to prepare all the samples. After saturation, with a Skempton's B value greater than 0.96,

specimens were consolidated isotropically under an effective confining stress of $p' = 200$ kPa.

2.3 Stress paths of drained rotational shear tests

The stress paths in $q - p'$ and deviatoric stress planes are shown in Figure 2. In the deviatoric stress plane, the vector from the origin has a length equal to the magnitude of the deviatoric stress q and makes an angle of 2α , which is twice the angle of the major principal stress σ_1 makes with the vertical direction. During the tests, the samples were first subjected to monotonic shearing in the vertical direction up to a specified stress ratio ($\eta = 0.8, 0.9, 1.0, 1.1$) while keeping the effective mean stress p' constant (A $\rightarrow$ B). After that the principal stress axes were rotated counter clockwise (B $\rightarrow$ C $\rightarrow$ D $\rightarrow$ E $\rightarrow$ B) under drained condition, while keeping the deviatoric stress constant and maintaining the effective mean stress $p' = 200$ kPa and the intermediate principal stress parameter $b = 0$. To ensure full discharge of water from the specimen, the major principal stress direction α_σ was rotated at a slow rate of $2^\circ/\text{min}$.

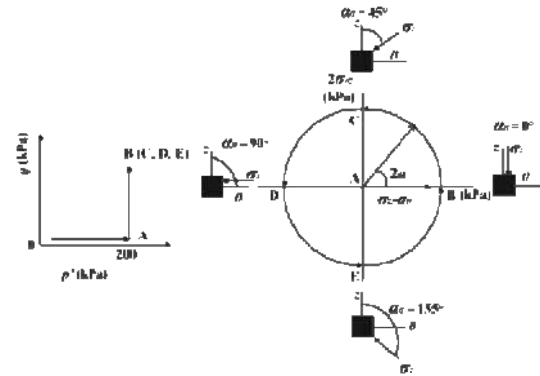


Figure 2. Stress paths for rotational shear test (Nakata et al. 1998).

3 TEST RESULTS AND DISCUSSIONS

3.1 Development of volumetric strain

The evolutions of the volumetric strain ϵ_v with the increasing number of cycles for rotational shear tests are shown in Figure 3. A positive value along the vertical axis indicates contraction and negative indicates dilation. Although the magnitudes of principal stresses were maintained constant during each test, contractive volumetric strain accumulated due to the rotation of principal stress directions. It can be observed that most of the volumetric strain occurs during the first few cycles and its accumulation rate tends to decrease as the number of cycles increases. It is clear that the effect of stress ratio on the development of the volumetric strain is significant under otherwise identical conditions. For all the four tests, the amount of the contractive volumetric strain at the same number of cycles increases with the increase in the stress ratio η .

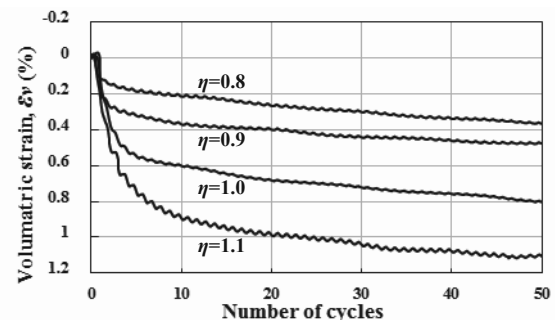


Figure 3. Stress paths for rotational shear test.

3.2 Non-coaxiality

The degree of non-coaxiality as measured by the deviation between the major principal stress direction and the principal strain increment direction, $(\alpha_{de}-\alpha_{\sigma})$ for tests with different stress ratios are plotted in Figure 4(a) - (d). As elastic strain increment takes a much smaller proportion in the total strain increment compared to that of the plastic strain increment (Gutierrez et al. 1991), the total strain increment instead of the plastic strain increment is used in the following analysis. In general, the variation trend of the non-coaxiality degree shows an obvious oscillation during the tests. Lower degrees of non-coaxiality are observed in the first few cycles. When the rotational shear continues, the strain increment direction becomes closer to the stress increment direction and higher degrees of non-coaxiality are observed. After about 20 rotation cycles, the variation of the non-coaxiality degree appeared to be stabilized. It is clear that the increasing trend of the non-coaxiality degree at the initial stage is more obvious for tests with higher stress ratios.

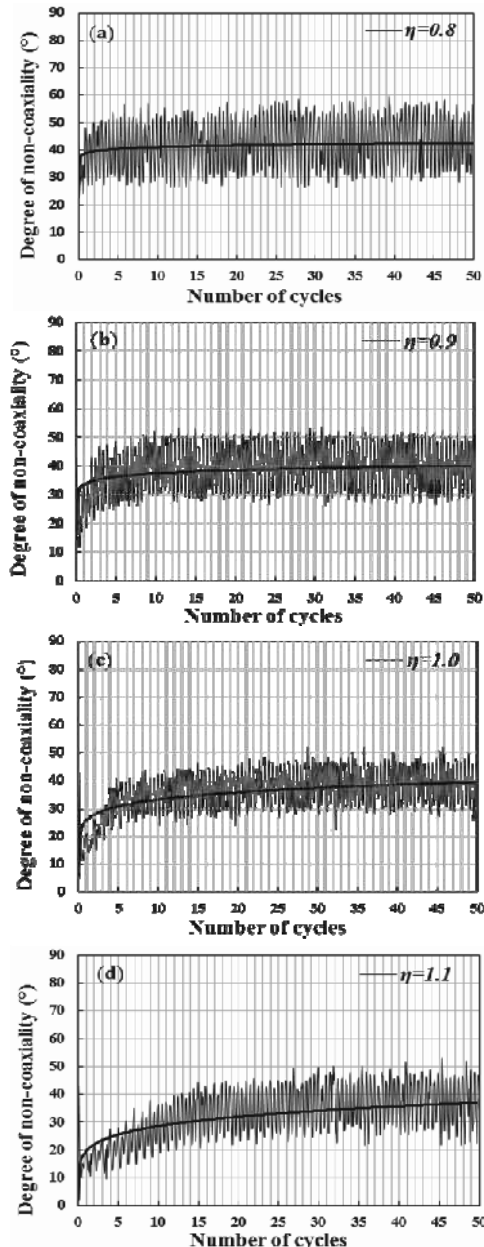


Figure 4. Degree of non-coaxiality against number of cycles for test with: (a) $\eta=0.8$; (b) $\eta=0.9$; (c) $\eta=1.0$; (d) $\eta=1.1$.

The black solid lines in Figure 4 are the trend lines of the non-coaxiality degree. With a lower stress ratio ($\eta=0.8$), $(\alpha_{de}-\alpha_{\sigma})$ is closer to 45° , indicating the strain increment direction is closer to the stress increment direction. At the same number of cycles, the degree of non-coaxiality decreased with the increase of the stress ratio. This observation agrees well with the laboratory results presented by Gutierrez et al. (1991) and numerical results obtained by Li and Yu (2009) based on DEM simulations.

As described above, the variation trend of the non-coaxiality degree shows an obvious periodicity during the tests. To have a better view, the relationship of the non-coaxiality degree with the major principal stress direction α_{σ} at the cycle numbers $N=1$ and $N=20$ are presented in Figure 5(a) and 5(b). Figure 6 shows the corresponding stress paths and strain increments in these tests. It is clear that the variation of the non-coaxiality degree differs significantly at the two stages. At the initial stage when $N=1$ the degree of non-coaxiality lies approximately in the range of 10 to 40° , as shown in Figure 5(a). It tends to decrease when α_{σ} rotates from 0 to 90° and then increase during the latter half cycle's rotation. However, at $N=20$ (Figure 5b), the fluctuation of the non-coaxiality degree exhibits two periods of a sine wave displaced at 180 degree intervals. Meanwhile, as mentioned above, the stress ratio has a significant effect on the non-coaxiality degree. The larger the stress ratio, the lower non-coaxial degree between the directions of the strain increment and stress is induced.

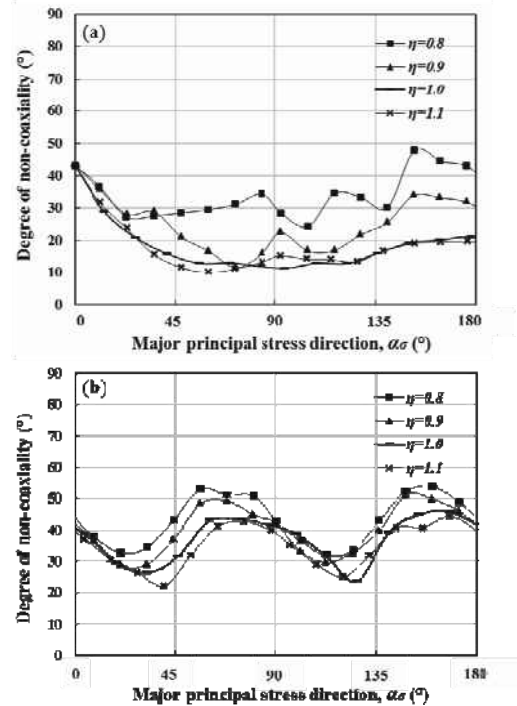


Figure 5. Degree of non-coaxiality for rotational shear tests at: (a) $N=1$; and (b) $N=20$.

As shown in Figure 6, for both $N=1$ and $N=20$, the directions of the strain increments are gradually enlarged with the increase of the stress ratio. It is interesting to see that for $N=20$ the strain increment differs significantly in different sections. When the principal stress axes rotate along the stress paths of DA and BC, which correspond to the major principal stress direction α_{σ} in ranges of $[45^\circ, 90^\circ]$ and $[135^\circ, 180^\circ]$, the strain increment direction almost coincides with the stress increment direction. Similar results have been reported by Tong et al. (2010). For lower stress ratios such as $\eta=0.8$ and 0.9 in Figure 6, several arrows pointed inside the cycle, indicating the strain increment

direction is larger than the stress increment direction. However, when the principal stress axes rotate along the stress paths of AB and CD, corresponds to α_σ in ranges of $[0^\circ, 45^\circ]$ and $[90^\circ, 135^\circ]$, the directions of the strain increments are larger and the non-coaxiality between the directions of the strain increment and stress are induced.

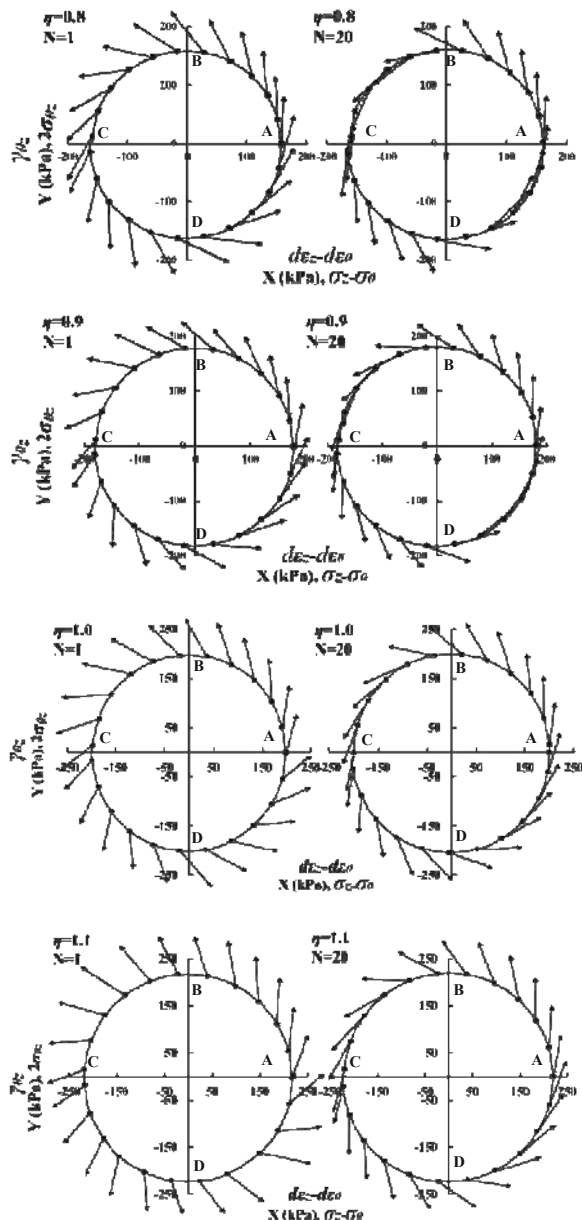


Figure 6. Stress paths and strain increments in rotational shear tests at $N=1$ and $N=20$.

4 CONCLUSIONS

This paper presents an experimental investigation of drained behaviour of saturated Leighton Buzzard sand in rotational shear. The samples were subjected to cyclic rotation of principal stress axes while the magnitudes of mean stress and shear stress were maintained constant. A special attention of the investigation has been placed on the non-coaxiality of granular soil subjected to the continuous rotation of major principal stress axis at different stress ratios. The conclusions drawn are as follows:

- The volumetric strain induced by cyclic rotation of principal stress axes is mainly contractive. Most of the contractive volumetric strain occurs during the first few cycles and its accumulation rate tends to decrease as the number of cycles increases. The accumulated volumetric strain increases with the increase in the stress ratio.
- The mechanical behaviour of sand under rotational shear is generally non-coaxial, and the variation trend of the non-coaxiality degree shows an obvious periodicity during the tests.
- Lower degrees of non-coaxiality are observed in the first few cycles. When the rotational shear continues, the strain increment direction becomes closer to the stress increment direction and higher degrees of non-coaxiality are observed. The variation of the non-coaxial degree appeared to be stabilized after approximately 20 cycles of shearing.
- It was also observed that the stress ratio has a significant effect on the non-coaxiality. The larger the stress ratio, the lower degree of non-coaxiality between the directions of the strain increment and stress is induced.

5 REFERENCES

- Arthur, J. R. F, Koenders M. A. and Wong R. K. S. 1986. Anisotropy in particle contacts associated with shearing in granular media. *Acta Mechanica*, 64, 20-29.
- Cai Y. 2010. An experimental study of non-coaxial soil behaviour using hollow cylinder testing. Ph.D thesis. University of Nottingham, UK.
- Gutierrez M., Ishihara K., and Towhata I. 1991. Flow theory for sand during rotation of principal stress direction. *Soils Found.*, 31 (4), 121-132.
- Hight D. W., Gens A., and Symes M. J. 1983. The development of a new hollow cylinder apparatus for investigating the effects of principal stress rotation in soils. *Geotechnique*, 33 (4), 355-383.
- Ishihara K. and Towhata K. 1983. Sand response to cyclic rotation of principal stress directions as induced by wave loads. *Soils Found.* 23 (4), 11-26.
- Roscoe, K. H., Bassett, R. H., and Cole, E. R. L. 1967. Principal axes observed during simple shear of a sand. *Proc. 4th Eur. Conf. Soil Mech. Found. Eng., Oslo*, 231-237.
- Symes, M. J., Gens, A., and Hight, D. W. 1984. Undrained anisotropy and principal stress rotation in saturated sand. *Géotechnique*, 34 (1), 11-27.
- Li X., Yu H.S. 2010. Numerical investigation of granular material behaviour under rotation shear. *Geotechnique*, 60 (5), 381-394.
- Nakata Y., Hyodo M., Murata H. & Yasufuku N. 1998. Flow deformation of sands subjected to principal stress rotation. *Soils Found.* 38 (2), 115-128.
- Tong Z. X., Zhang J-M, Yu Y. L., Zhang G. 2010. Drained deformation behavior of anisotropic sands during cyclic rotation of stress principal axes. *J. Geotechnical and Geoenvironmental Engineering*. ASCE. 136 (11), 1509-1518.
- Yang Z. X., Li X. S. and Yang J. 2007. Undrained anisotropy and rotational shear in granular soil. *Geotechnique*, 57 (4), 371- 384.
- Yu, H. S. 2008. Non-coaxial theories of plasticity for granular materials. *Proc. 12th Int. Conf. of Int. Assoc. Comp. Meth. Adv. Geom. (IACMAG)*, Goa, India, 361-378.
- Yu, H. S., and Yuan, X. 2006. On a class of non-coaxial plasticity models for granular soils. *Proc. Royal Soc. A*, 462, 725-748.

Test study and constitutive modelling of the time-dependent stress-strain behavior of soils

Caractérisation expérimentale et modélisation du comportement contrainte-déformation à dépendance du temps

Yin J.-H., Tong F.

Department of Civil and Environmental Engineering, The Hong Kong Polytechnic University, Hong Kong, China

ABSTRACT: This paper reports firstly the experimental data from one-dimensional (1D) straining oedometer tests and 2D consolidated undrained compression (CU) tests on a bentonite (clay mineral montmorillonite) mixed with different percentages of silicon sand. The 1D tests include (i) multi-staged loading tests with unloading/reloading and with creep and swelling and (ii) step-changed constant rate of strain compression tests with unloading/reloading in 1D straining as well. From the test data, creep, swelling and strain rate effects are observed and discussed. This paper introduces a new 1D Elastic Visco-Plastic model considering both creep and swelling (called 1D EVPS) for soils in 1D straining based on the previous 1D Elastic Visco-Plastic (1D EVP) models. The 1D EVPS model is used to simulate a step-changed constant rate of strain compression test with unloading/reloading in 1D straining with comparison with measured data. In addition, the 1D EVPS model is used to simulate single-staged constant rate of strain CU tests and relaxation tests. The typical results from a selected 2D CU test on a bentonite mixed 70% silicon sand under effective cell pressure of 100 kPa in a triaxial state compressed with step-changed constant rate of strain and unloading-reloading are presented and discussed in the paper. The 2D CU test data show clearly strain rate effects and unloading-reloading loops. The main conclusions from the study are: (a) the stress-strain behaviour of a bentonite mixed with different percentages of silicon sand exhibits strong dependence on time and strain rate; (b) in 1D straining condition, there exists a creep region when stress-strain state closer to the normal consolidation line (NCL) and a swelling region when far away from the NCL; (c) the 1D EVPS model can re-produce well the time-dependent stress-strain behaviour of the soil including creep, swelling, strain effects, relaxation, and unloading-reloading loops in 1D straining condition; and (d) the data from 2D consolidated undrained triaxial compression tests on the bentonite-sand mixture also show the strong strain effects and unloading-reloading loops.

RÉSUMÉ : Cet article rend compte d'une part les données expérimentales provenant unidimensionnel (1D) rude épreuve essais oedométriques et 2D consolidés non drainés de compression (CU) des tests sur une bentonite (montmorillonite minérale) mélangé avec des pourcentages différents de sable de silicium. Les tests 1D comprennent: (i) multi-étagées essais de charge avec déchargement / rechargement et avec un taux constant de fluage et gonflement et (ii) étape changé d'essais de compression de déformation avec le déchargement / rechargement en 1D forcer ainsi. D'après les données d'essai, les effets du taux de fluage, gonflement et la tension sont observées et discutées. Cet article présente une nouvelle 1D élastique visco-plastique modèle considérant à la fois au fluage et gonflement (appelé EVP 1D) pour les sols en 1D égouttage sur la base de l'élastique 1D précédente visco-plastique (1D EVP) des modèles. Le modèle 1D EVP est utilisé pour simuler une étape changé taux a changé constante de l'essai de compression souche avec déchargement / rechargement en 1D forcer la comparaison avec les données mesurées. En outre, le modèle 1D EVP est utilisé pour simuler une seule mise en scène constante de vitesse de déformation essais CU et des tests de relaxation. Les résultats typiques d'un test sélectionné CU 2D sur une bentonite mélangée sable de silicium 70% sous la pression de cellule effective de 100 kPa dans un état comprimé triaxial avec sa belle-changé vitesse de déformation constante et le déchargement-rechargement sont présentés et discutés dans le document. Les données 2D essais CU montrent clairement la souche effets de taux et de déchargement-rechargement des boucles. Les principales conclusions de l'étude sont les suivants: (a) le comportement contrainte-déformation d'une bentonite mélangée avec différents pourcentages de sable de silicium présente une forte dépendance du temps et de la vitesse de déformation, (b) dans un état tendu 1D, il existe une région de fluage lorsque le stress de souche état plus proche de la ligne de consolidation normale (NCL) et une zone d'expansion quand loin de la NCL, (c) le modèle 1D EVP peut re-produire et dépendant du temps comportement contrainte-déformation du sol, y compris au fluage, de l'enflure, les effets de déformation, de relaxation et de déchargement-rechargement des boucles dans un état tendu 1D et (d) les données provenant 2D consolidés non drainés des essais de compression triaxiale sur le mélange de bentonite et de sable montrent également les effets de contrainte forts et de déchargement-rechargement des boucles.

KEYWORDS: Stress-strain, time-dependent, creep, swelling, relaxation, strain-rate, visco-plastic.

1. INTRODUCTION

The stress-strain behavior of all geomaterials is time- (or rate-) dependent. The time-dependence of the stress-strain may be neglected for some geomaterials such as hard rock and sand, but cannot be ignored for some geomaterials such as soft rock and clayey soils. The phenomena of the time-dependence include creep, relaxation, the increase of the pre-consolidation pressure with strain rates, swelling, *etc.* Among these, creep is the compression with time under a constant stress; while swelling is expansion with time under a constant stress, opposite to the

creep. It is found that some clayey soils exhibit both creep and swelling such as soils containing clay mineral montmorillonite. In this paper, the authors will report first the experimental data from one-dimensional (1D) straining oedometer tests and consolidated 2D undrained compression tests on a bentonite mixed with different silicon sand contents. The 1D tests include (i) multi-staged loading tests with unloading/reloading and with creep and swelling and (ii) step-changed constant rate of strain compression tests with unloading/reloading in 1D straining as well. From the test data, creep, swelling and strain rate effects are observed and discussed. The 2D tests include (i) multi-

staged isotropic loading tests with unloading/reloading and with creep and swelling, and (ii) step-changed constant rate of strain consolidated undrained compression tests (CU tests) in triaxial state. From these test data, creep, swelling, relaxation, and strain rate effects are investigated and discussed.

Based on the 1D Elastic Visco-Plastic (1D EVP) models developed by Yin and Graham (1989, 1994, 1999), the authors have proposed a new 1D Elastic Visco-Plastic model considering both creep and swelling (called 1D EVPS) (Yin and Tong 2011). The data from the multi-staged oedometer tests are used to calibrate the new 1D EVPS model. After this, 1D EVPS model is used to simulate the step-changed constant rate of strain compression tests in 1D straining and other types of tests and make comparisons.

2. OEDOMETER TESTS ON BENTONITE MIXED WITH DIFFERENT SAND CONTENTS AND RESULTS

The authors have done a series of oedometer tests on a mixture of bentone and silicon sand with silicon sand contents of 50%, 60%, 70%, 80% and 90%. Table 1 gives a simmay of basis properties of the mixtue with 60%, 70%, and 80% (Tong and Yin 2011).

Table 1 Basic properties for different bentonite-sand mixed samples

Sand proportion	50%	60%	70%
Water Content (%)	211.5	159.0	124.5
Liquid Limit W_L (%)	146.0	106.0	82.6
Plastic Limit W_P (%)	29.9	28.7	27.4
Plastic Index I_P (%)	121.1	77.3	55.2

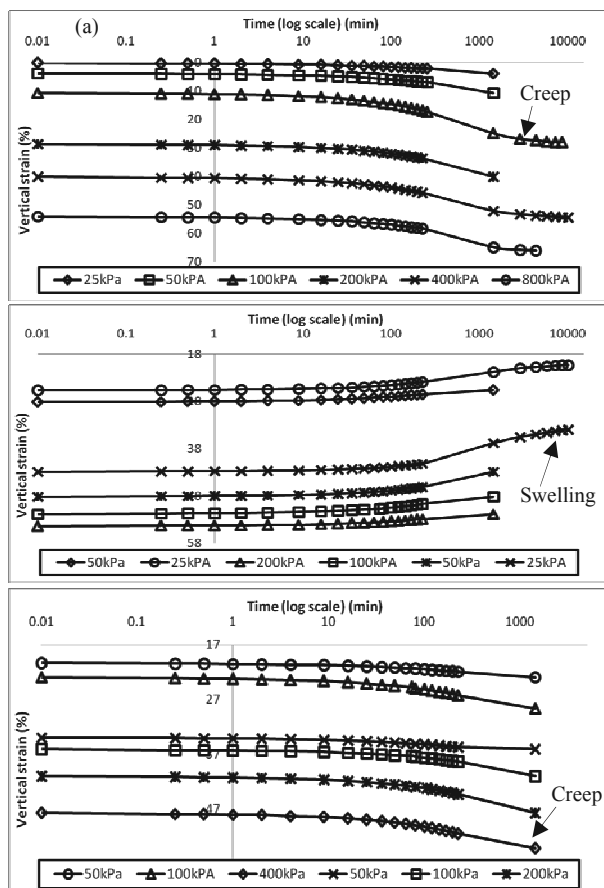


Figure 1. Vertical strain against time in log scale for 50% sand mixed with 50% bentonite (a) loading, (b) unloading, and (c) reloading

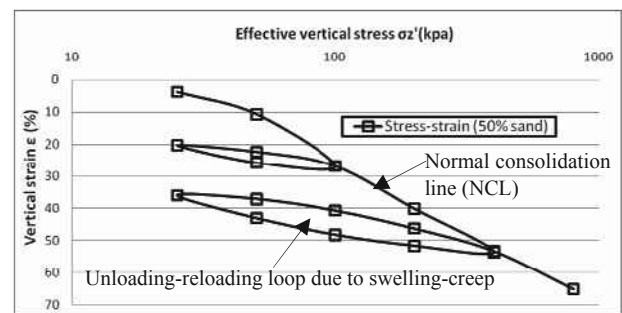


Figure 2. Vertical strain against vertical effective stress in log scale from tests on bentonite-sand mixture (50% sand content)

The curves of $\log(\text{time})$ and vertical strains for a mixture of 50% bentone and 50% sand are shown in Figure 1. The oedometer test was carried out in stages and with unloading-reloading.

It is clearly seen from Figure 1(a) and (b) that the mixture exhibits both time-dependent creep and swelling. It is noted that “swelling” here is neither the rebound due to unloading, or the expansion of an unsaturated clay due to absorbing water. Instead, the “swelling” here is defined expansion of a saturated clay due to swelling potnetial of the clay udner a constant vertical effective stress. The “swelling” here is oposite to teh creep.

Figure 2 shows the curve of vertical strain against vertical effective stress in log scale from tests on bentonite-sand mixture (50% sand content) with duration of 24 hours for each loading. It is seen from the figure that there are two unloading-reloading loops. The authors belive that the swelling and creep contribute to the loops. This is because that when the specimen is unloaded to very stress-strain state far from the normal consolidaiton line (NCL), the swelling poential of the mixture has caused the clay to expand and caused the time-depepdeint reduction of strain. When the specimen is re-loaded to the stress-strain state closer to the NCL, the mixtuer will have creep compression. The authors also consider that the partial movements of the mixture may also contribute to the formation of the loops.

It is noted that the curves of $\log(\text{time})$ and vertical strains and the curves of vertical strain against vertical effective stress in log scale of other mixtures are similar to those in Figures 1 and 2. From all test data, the authors have detrmained values of compression index C_c , rebound index C_r , creep coefficient $C_\square$ and swelling coefficient C_s . The definitions of those parameters are:

$$C_c = \frac{-\Delta e}{\Delta \log \sigma'_z}; \quad C_r = \frac{\Delta e}{\Delta \log \sigma'_z} \quad (1)$$

$$C_a = \frac{-\Delta e}{\Delta \log t}; \quad C_s = \frac{\Delta e}{\Delta \log t}$$

Values of C_c , C_r , $C_\square$ and C_s are presented in Table 1. In Table 1, the ratio of C_a/C_c and C_s/C_r are also given. The following ovservations can be obtained grom Table 2:

- Both the compression index C_c and recound index C_r decrease with the increase of the sand content. This means that the more the sand content, the less compression of the mixture.
- In generally speaking, the creep coefficient $C_\square$ decreases with the increase of the sand content. This means that the more the sand content, the less creep of the mixture. In addition, the coefficient $C_\square$ also decreases with the increase of the vertical effective stress.
- In generally speaking, the swelling coefficient C_s decreases with the increase of the sand content. This means that the more the sand content, the less swelling of the mixture. In addition, the coefficient C_s increases with the decrease of the vertical effective stress.

- The ratios of C_a/C_c and C_s/C_c also decrease with the sand content.

Table 2 Creep and swelling indexes for different bentonite samples

Sand proportion	50%	60%	70%	80%	90%
Compression index C_c	0.315	0.296	0.260	0.243	0.104
Rebounding index C_r	0.130	0.113	0.092	0.054	0.021
Creep coefficient C_{a1} ($\sigma_z=100\text{kPa}$)	0.0304	0.0263	0.0227	0.0062	0.0025
Creep coefficient C_{a2} ($\sigma_z=400\text{kPa}$)	0.0191	0.0141	0.0120	0.0051	0.0023
C_a/C_c ($\sigma_z=100\text{kPa}$)	0.0965	0.0889	0.0873	0.0255	0.0240
C_a/C_c ($\sigma_z=400\text{kPa}$)	0.0606	0.0476	0.0462	0.0210	0.0221
Swelling coefficient C_{s1} ($\sigma_z=10\text{kPa}$)	0.0284	0.0251	0.0160	0.0085	0.0033
Swelling coefficient C_{s2} ($\sigma_z=50\text{kPa}$)	0.0173	0.0131	0.0124	0.0062	0.0024
C_{s1}/C_r ($\sigma_z=100\text{kPa}$)	0.2185	0.2221	0.1739	0.1574	0.1571
C_{s2}/C_r ($\sigma_z=400\text{kPa}$)	0.1331	0.1159	0.1048	0.1148	0.1143
C_{s1}/C_c ($\sigma_z=100\text{kPa}$)	0.0902	0.0848	0.0615	0.0350	0.0317
C_{s2}/C_c ($\sigma_z=400\text{kPa}$)	0.0549	0.0443	0.0477	0.0255	0.0230

3. A NEW 1D ELASTIC VISCO-PLASTIC MODEL FOR SOILS EXHIBITING BOTH CREEP AND SWELLING

Yin and Tong (2012) has developed a new Elastic Visco-Plastic model considering both creep and swelling of the clays in one-dimensional straining, called 1D EVPS model. This new model is based on the previous 1D EVP models proposed by the first author and his co-workers (Yin 1990; Yin and Graham 1989, 1994). Important concepts of “elastic” time line, “reference” time lines in both creep and swelling regions, and “equivalent” time lines are shown in Fig.3 in the plot of the vertical effective stress in $\ln$ -scale and vertical strain. The “elastic” line has a slope κ/V and the two “reference” time lines in both creep and swelling regions have the same slope λ/V where $V=1+e_o$ in which e_o is the initial void ratio. Other parameters are also shown in the figure such $(\sigma'_{z0}, \epsilon'_{z0})$ and $(\sigma'^{rc}_{z0}, \epsilon'^{rc}_{z0})$. Yin and Tong (2012) used the natural logarithmic functions to fit the “elastic” time line, “reference” time lines, and creep compression using equivalent time concept. The 1D EVPS model can be derived and expressed as:

$$\dot{\epsilon}_z = \frac{\kappa}{V} \frac{\dot{\sigma}'_z}{\sigma'_z} + \frac{\psi^c}{V} \frac{1}{t_o^c} \exp\left[-\left(\epsilon_z - \epsilon'^{rc}_{z0}\right) \frac{V}{\psi^c} \left(\frac{\sigma'_z}{\sigma'^{rc}_{z0}}\right)^{\frac{\lambda}{\psi^c}}\right] - \frac{\psi^s}{V} \frac{1}{t_o^s} \exp\left[\left(\epsilon_z - \epsilon'^{rs}_{z0}\right) \frac{V}{\psi^s} \left(\frac{\sigma'_z}{\sigma'^{rs}_{z0}}\right)^{\frac{\lambda}{\psi^s}}\right] \quad (2)$$

where the parameters (ψ^c, t_o^c) are for creep compression and (ψ^s, t_o^s) are for swelling of the clays. Eq.(2) is a general constitutive model for the time-dependent stress-strain behaviour of soils exhibiting both creep and swelling in 1D straining. This model is valid for all loading conditions such as constant rate of strain (CRSN) loading, relaxation, unloading, reloading etc.

Yin and Tong (2012) has used data of a multi-staged oedometer test on a bentonite mixed with 70% silicon silica sand mixture (SMB) to determine values of all parameters in Eq.(2). The basic properties can be found in Table 1. The initial void ratio e_o is 5.5 (after pre-consolidation but before oedometer testing). All values of these parameters are presented in Table 3.

This 1D EVP model has been used to simulate the stress-strain behavior of the same clay under step-changed constant-

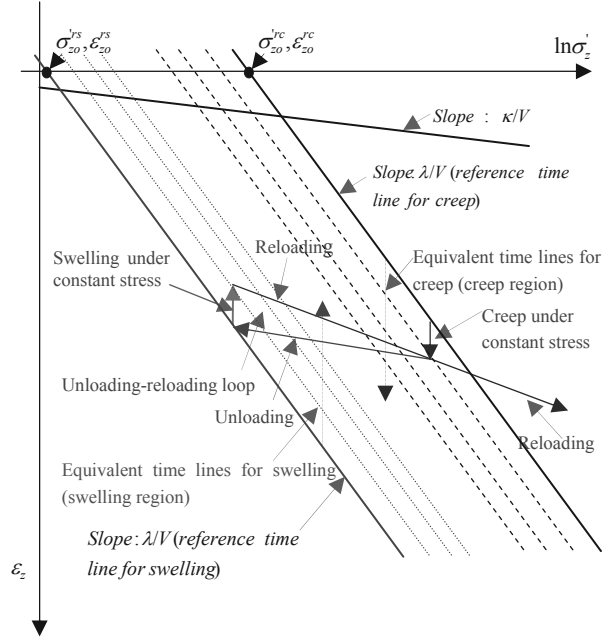


Figure 3. Conceptual illustration of creep, swelling, unloading-reloading loop, “elastic” line, “reference” time line, and “equivalent” time lines (extended from Yin 1990; Yin and Graham 1989, 1994, 1999)

rate of strain (CSR) loading with comparison with measured data as shown in Figure 3. It is seen from this figure that the 1D EBVPS model can simulate the strain effects, unloading/reloading loops, and the non-linear stress-strain behavior.

Table 3. Values of parameters in 1D EVPS model

Parameters	Bentonite with 70% sand
Elastic parameter κ/V	0.0542
Plastic parameter λ/V	0.1367
Creep parameter ψ^c	0.01956
Creep parameter t_o^c	24 hours
Swelling parameter ψ^s	0.01554
Swelling parameter t_o^s	24 hours
Interception stress σ'^{rc}_{z0}	50 kPa
Interception strain ϵ'^{rc}_{z0}	0
Interception stress σ'^{rs}_{z0}	16 kPa
Interception strain ϵ'^{rs}_{z0}	0

This 1D EVP model has also been used to simulate the stress-strain behavior of the same clay under staged constant-rate of strain (CSR) loading with CSR of $\dot{\epsilon}_z = 10^{-4}/\text{sec}, 10^{-5}/\text{sec}, 10^{-6}/\text{sec}, 10^{-7}/\text{sec}$. It is seen from the figure that the higher the strain rate, the larger the effective stress. The pre-consolidation pressure increases with the strain rate.

In addition, the 1D EVP model has also been used to simulate the stress decreasing with time (stress relaxation) in the creep region and the stress increasing with time (also called stress relaxation) in the swelling region when the vertical strain is kept a constant (Yin and Tong 2012). When the initial stress-strain state point is in the swelling region (far away from the normal consolidation line, the clay has a swelling potential so that the stress will increase with time when the specimen is confined in vertical thickness (strain is constant).

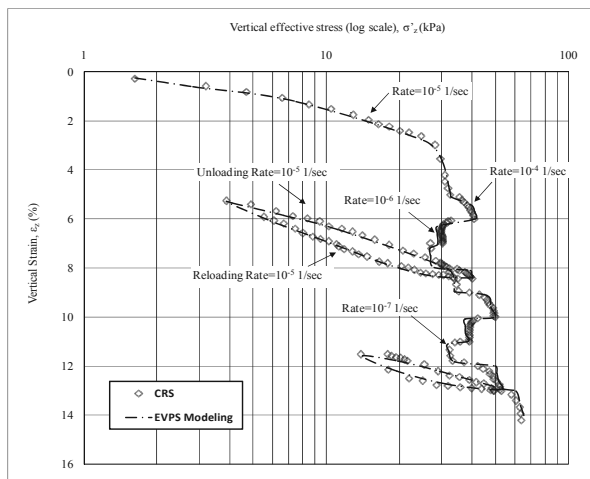


Figure 4. Comparison of modelling results with measured data from a CRS test with step-changed strain rates and with unloading and reloading

4. CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TESTS ON BENTONITE MIXED WITH SAND AND RESULTS

The authors have done consolidated undrained triaxial tests on the same bentonite mixed with different ratios of silicon sand with effective confining pressure of 50 kPa, 100 kPa and 200 kPa. Curves of (a) deviator stress versus axial strain, (b) the excess porewater pressure versus axial strain, and (c) the deviator stress versus effective (or total) mean stress of a CU test on bentonite mixed with 70% of sand and effective cell pressure of 100 kPa are shown in Figure 6. It is seen from Figure 6 that the effects of strain rates are very significant. The unloading-reloading loops are also evident.

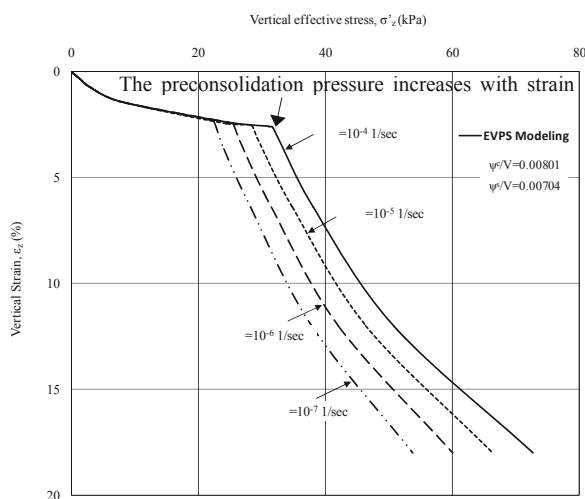


Figure 5. Simulation for strain rate dependent stress-strain behavior in CSR tests using the EVPS model

5. CONCLUSIONS

From the above presentation and study, the following conclusions can be drawn:

- The stress-strain behavior of a bentonite mixed with different percentages of silicon sand exhibits strong dependence on time and strain rate.
- Creep, swelling, relaxation and strain effects are clearly observed and are significant.
- In 1D straining condition, there exists a creep region with stress-strain state closer to the normal consolidation line (NCL) and a swelling region far away from the NCL.

- The 1D EVPS model can re-produce well the time-dependent stress-strain behavior of the soil including creep, swelling, strain effects, relaxation, and unloading-reloading loops in 1D straining condition.
- The data from 2D consolidated undrained triaxial compression tests on the bentonite-sand mixture also show the strain effects and unloading-reloading loops.

6. ACKNOWLEDGEMENTS

Financial supports (G-U663 and G-YG60) by The Hong Kong Polytechnic University are acknowledged.

7. REFERENCES

- Yin, J.-H. and Graham, J. 1989. Visco-elastic-plastic modeling of one-dimensional time-dependent behaviour of clays. *Canadian Geotechnical Journal*, 26(3), 199-209.
- Yin, J.-H. and Graham, J. 1994. Equivalent times and one-dimensional elastic visco-plastic modeling of time-dependent stress-strain behavior of clays. *Canadian Geotechnical Journal*, 31(2), 42-52.
- Yin, J.-H. and Graham, J. 1999. "Elastic visco-plastic modelling of the time-dependent stress-strain behavior of soils". *Canadian Geotechnical Journal*, 36(4), 736-745.
- Yin, J.-H. and Tong, F. 2011. Constitutive modeling of the time-dependent stress-strain behaviour of saturated soils exhibiting both creep and swelling. *Canadian Geot. J.*, 48(12), 1870-1885.
- Gibson R.E. and Henkel D.J. 1954. Influence of duration of tests at constant rate of strain on measured "drained" strength. *Géotechnique* 4 (1), 6-15.
- Tong, F and Yin, JH (2011). "Nonlinear Creep and Swelling Behavior of Bentonite Mixed with Different Sand Contents under Oedometric Condition". In the *Journal of Marine Georesources and Geotechnology*, Volume 29 , 346~363.

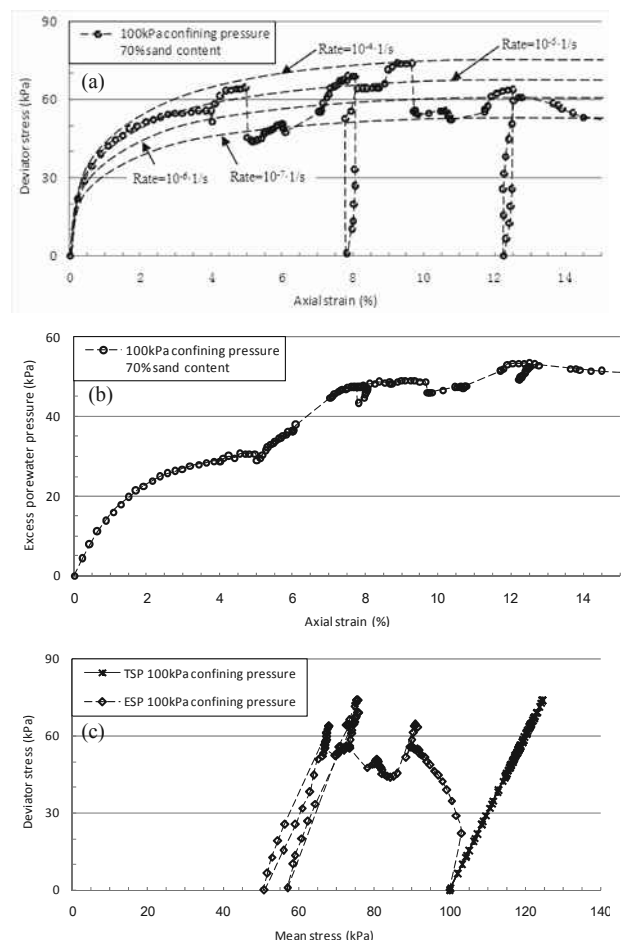


Figure 6. (a) Deviator stress versus axial strain, (b) the excess porewater pressure versus axial strain, and (c) the deviator stress versus effective (or total) mean stress – CU test on bentonite mixed with 70% of sand and effective cell pressure of 100 kPa

Special Features of Creep of Clayey Soils

Particularités du fluage des sols argileux

Zhakulin A.S., Zhakulina A.A., Orazaly E.E.
Karaganda State Technical University, Karaganda, Kazakhstan

Orazalin Z.Y.
Massachusetts Institute of Technology, Cambridge, USA

ABSTRACT: The problem of settlement predictions in clays involves the solution of complex engineering challenges associated with the determination of many experimental parameters (modulus of deformation, lateral expansion ratio, coefficients of permeability and consolidation, as well as rheological characteristics). The full analysis of ground behavior during consolidation and creep of the solid soil particles at the time of the construction and operation is only possible if the site investigation and laboratory results are reliable. In this paper the authors present the results of the long-term settlement observations (over 50 years) of large industrial facilities of Karaganda Metallurgical Complex and the soil properties of these sites based on the thorough site investigation program. The underlying soils have sufficient bearing capacity, relatively high values of density and low void ratios. The creep deformations at the constant loads in the saturated state exhibited gradually decreasing character. The actual settlements of the structures and the maximum allowable settlements (by the building codes) are compared. The conclusion is made on the need to amend the recognized formula given by the building codes for the ground analysis by the second group of limiting states.

RÉSUMÉ : Le problème de la prédiction de la base de sédiments est réduite à la résolution des problèmes d'ingénierie complexes associés à la détermination des paramètres expérimentaux (module de déformation, le coefficient de coefficients de dilatation latéraux, du filtre et la consolidation, de fluage et de la rhéologie caractéristiques) des sols argileux. Une analyse complète du comportement de la base pour la consolidation et le fluage des particules solides du sol avec le temps au cours de la construction et l'exploitation n'est possible que si les résultats expérimentaux sont fiables et exacts. Les auteurs de cet article établissent que les propriétés physiques des sols argileux du site de construction dans l'ouest du Kazakhstan se caractérisent par une hétérogénéité et la propagation en assez grand nombre de valeurs de caractéristiques. Les résultats des observations de règlement à long terme des grandes installations industrielles de Karaganda Complexe Métallurgique sont présentés dans le document. Les colonies réelles des structures et les colonies maximales admissibles (codes du bâtiment) sont comparées. La conclusion se fait sur la nécessité de modifier la formule communément reconnue pour l'analyse du sol par le deuxième groupe d'états limitatifs.

KEYWORDS: settlements, foundations, laboratory and in-situ characterization, creep.

1 INTRODUCTION

The long-term settlements of large industrial facilities of Karaganda Metallurgical Complex have been monitored for more than 50 years by geodesic surveying. In order to record the settlements, the survey points were installed during the construction of the blast furnace, charcoal oven battery and 250 meters tall chimney structures of the complex.

It is known that the primary settlements occur during the construction period and after several years of building operation. The settlement stabilization in clays occurs over time. In the central part of Kazakhstan the clayey soils (clay loam) with mostly semi-solid consistency are dominant which turn into a plastic state if the moisture content is increased.

In this regard, it is important to monitor the ground deformations from the moment of foundation installation and application of the loads, as well as to conduct the observations of changes in soil water content and groundwater levels (Tsytoich N.A. et al. 1967, Tsytoich N.A. and Ter-Martirosjan Z.T. 1981, Ukhov S.B. et al. 2002.)

2 PHYSICAL - MECHANICAL PROPERTIES OF SOILS

The physical and mechanical properties of the soils underlying the industrial facilities of Karaganda Metallurgical Complex are presented in Table 1.

Table 1. Physical- mechanical properties of the soils

Properties	Units	Facilities				
		Furnace	Oven battery	Chimney		
				H= 250 m	H= 100 m	H= 150 m
γ_o	kN/m ³	20,0	21,0	21,1	20,6	20,3
γ_s	kN/m ³	26,8	27,0	27,0	26,9	27,2
ω	%	12,5	14,2	18	19	17
J_l		0,25	0,24	0,26	0,21	0,26
e		0,52	0,76	0,49	0,66	0,52
ϕ	deg.	21	24	21	22	19
c	MPa	0,08	0,06	0,06	0,08	0,06
E_o	MPa	19,5	14,1	15,0	16,9	17,2

These geotechnical investigations and laboratory testing results show that the soils have relatively high density and low void ratios. The laboratory studies confirm an increase in water content of soils and the transition from a semi-solid to a low-plasticity consistency.

3 STRUCTURAL FEATURES OF FOUNDATIONS

The foundation of the blast furnace facilities was designed as a monolithic mass of reinforced concrete with the plan area of 27.0 x 27.0 m. The reinforcement of the foundation consisted of welded wire mesh made of steel class A – II. There are three

horizontal grids with spacing of 200 mm in both directions in the lower zone of the foundation. In the upper zone there is the grid reinforcement with spacing of 400 mm. The depth of the foundation is 4.0 m. The bearing pressure is 300 t/m².

The foundations of the charcoal battery structures consist of box sections with the plan dimensions of 15.0 x 80.0 m. The depth of the foundation is 3.5 m. Each foundation has 4 exhaust ducts and 1 vent groove. The foundations were reinforced with longitudinal and transverse bars with diameter 20 mm, class A - II. The bearing pressure is 18.0 t/m².

The foundations of the 250 meters tall chimney structures are made of monolithic reinforced concrete. The foundation base is round with the diameter of 40.0 m and height of 4.0 m. The design load at the base of the chimney is approximately 21.1 t/m². The adopted concrete grade for all types of foundations was M300 on the Portland cement. Based on the site-investigation to a depth 30.0 m, the groundwater was not revealed.

4 LABORATORY STUDIES OF CREEP BEHAVIOR

The volumetric creep behavior of clayey soils under hydrostatic compression was investigated on the triaxial testing devices by applying a constant load (Tatsuoka F., Shibuya S. and Kuwano R. 2001).

The results of the studies show that the structural strength of the samples of clay with natural moisture and density content ranged from 25 ÷ 100 kPa; it was established experimentally that the consolidation process of the intact samples taken at different depths is significantly different with respect to time and deformation modes. The intensity of consolidation for the investigated soils under axial stresses of 25 ÷ 100 kPa is practically unchanged. It was mainly determined by the destruction process of the soil structure and the development of creep in the soil skeleton over time at the axial stresses of 100 ÷ 400 kPa. The existence of transient creep in shear has been established experimentally. The shear creep deformations at the given loads in the saturated state exhibited gradually decreasing character (Zhakulin A.S. and Zhakulina A.A., Kropachev P.A. 2011, Nakai T., Shahin H., Kyokawa H., Miyahara Y. 2011.).

The following plots were obtained based on the investigation:

- volumetric strains -time;
- relationship between the hydrostatic pressure and volumetric strain;
- pore pressure-time.

Figure 1 shows the test data volume creep of clay. The values of the hydrostatic pressure in the experiments were: $\sigma_m = 0.10; 0.20; 0.3$ MPa.

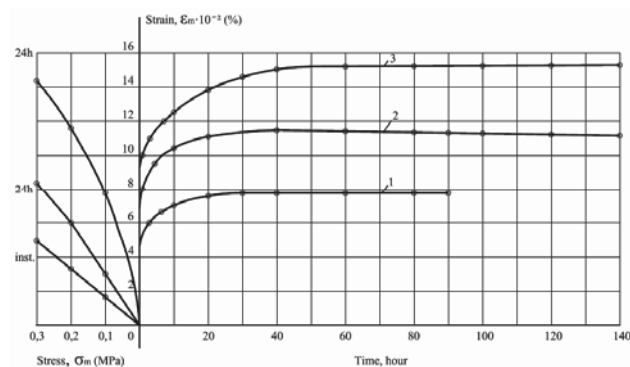


Figure 1. Volumetric creep curves of clay. 1 – at $\sigma_m = 0.1$ MPa, 2 – at $\sigma_m = 0.2$ MPa, 3 – at $\sigma_m = 0.3$ MPa

The results revealed that with the increasing σ_m , the stabilization time of the volumetric strain is increasing. It should be noted that the experiments on the intact samples with

natural moisture-density content showed that the volumetric strains were gradually decreasing for 1.5-2.0 days. When σ_m was changed from 0.1 to 0.3 MPa the volumetric strains increased by 2.1 times. During the study of the volumetric deformations of the saturated clay samples, an increase in the time of stabilization with an increase in the hydrostatic pressures was observed. Therefore, at $\sigma_m = 0.1$ MPa the full stabilization of volumetric creep deformations was achieved in 3-3.5 days, while at $\sigma_m = 0.3$ MPa this time increased to 5-7 days.

The values of volume strains when tested on clay samples for volumetric creep upon application of step-increasing loads were slightly less than upon the step application. This indicates that the step-increasing loading contributes to hardening of soil at each step and causes less strain than applying the same magnitude of a constant load.

However, the deviation was not more than 12.5%, which allows the use of the Boltzmann deformation imposition law. The laboratory results indicate that the soils exhibit gradually decreasing creep behavior (Vyalov S.S. 1978, Bulychov N.S. 1989).

5 SETTLEMENTS OBSERVATION RESULTS AND THEIR ANALYSIS

The elastic deformations of the magnitude of 12 mm were recorded during the dismantling of the blast furnace in 1985. The settlements stabilized in 2 months after the erection of the blast furnace. A similar pattern was observed in the reconstruction of the most blast furnaces.

The graph of the settlements of the blast furnace in time is shown in Figure 2. Upon the complete replacement of the oven batteries in 1986, the elastic deformations of 9 mm were observed which represented the one fifth of the total settlement. The graphs of settlements over time for charcoal oven batteries are shown in Figure 3.

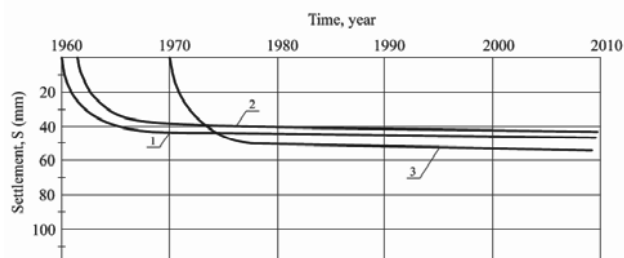


Figure 2. Settlements of blast furnaces over time. 1 - furnace #1, 2 - furnace #2, 3 - furnace #3.

The foundation base of the chimney is underlying by red-brown clay with semi-solid consistency. The stabilization of foundation settlement occurred in 5 years after the construction of the reinforced concrete stem.

The graphs of settlements of concrete chimneys in time are shown in Figure 4. It was found that the deformations of the foundations of blast furnaces and oven batteries were occurring for 2-3 years after their commissioning. The foundation settlements of the chimney structures faded immediately after the completion of construction work.

The comparison of the theoretical solutions and the long-term field settlement observations for 50 years provides a great deal of scientific and practical interest. In order to compare the design values with the actual and maximum allowable values of settlements (by building codes), the settlements of the foundations of the blast furnaces, charcoal oven batteries and 250 meters tall chimneys were re-analyzed.

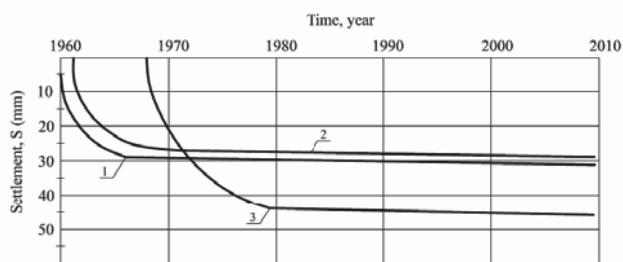


Figure 3. Settlements of charcoal oven batteries over time. 1 – battery #1, 2 – battery #2, 3 – battery #3.

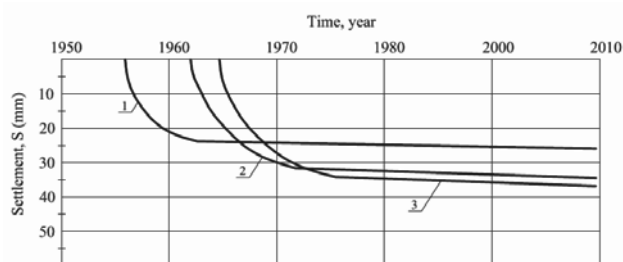


Figure 4. Settlements of chimneys over time. 1 - Chimney H=100m, 2 - Chimney H=250m, 3 - Chimney Power Station.

The settlements were calculated based on the design load: for the blast furnace - 300 t/m^2 , for the chimney - 21.1 t/m^2 , for charcoal oven battery - 18.0 t/m^2 at the upper edge of the foundation. The settlements were calculated by the method of summing the elementary layers and the method of the equivalent layer in accordance with the building code RK 5.01-01-2002 "Foundations of buildings and structures." Table 2 provides the comparison of design, actual, and maximum allowable (by the building codes) settlements.

According to the data provided, the actual foundation settlements of the industrial projects were several times less than the design values. As can be seen from Table 2, the actual settlements of the industrial objects are much lower than the design and maximum allowable ones: for the blast furnaces $6 \div 17$ times; for the oven batteries $2.5 \div 7.0$ times; for the Chimney in $2.5 \div 3.0$ times.

It should be noted that the building codes do not take into account the processes of consolidation and creep of the soil skeleton over time. The analysis by the method of summing the layers is based on the half-space elastic models.

The Eurocode 7 "Geotechnical Design" assumes taking into account the elastic, consolidation and creep components of the foundation settlements.

The settlements analysis that takes into account creep is the following:

$$S = S_0 + S_1 + S_2 \quad (1)$$

where S – total settlements; S_0 – instantaneous-elastic settlements; S_1 – consolidation settlements; S_2 – creep settlements (secondary);

Table 1. Comparison of design, actual, and maximum allowable (by building codes) settlements

Facility	Settlements, s, mm			
	Actual	Design		s_u
		Method #1 (layer summing)	Method #2 (equivalent layer)	
Blast furnace #1	45	156	210	400
Blast furnace #2	43	163	310	400
Blast furnace #3	55	178	200	400
Oven battery #1	27	138	144	250
Oven battery #2	26	120	132	250
Oven battery #3	44	140	149	250
Chimney (Power Station)	34	92	103	200
Chimney H=250 m	32	149	210	200
Chimney H=100 m	23	83	90	200

The total instantaneous foundation settlement can be determined by using the elastic theory:

$$s_0 = pbf / E_m \quad (2)$$

where E_m – elastic modulus for calculations;

f – influence factor;

p – linearly distributed bearing pressure.

The components of the total settlements that take into account consolidation and creep:

$$s = s_1 + s_2 = Hm_v p + Hb_k \ln(t/t_v) \quad (3)$$

To characterize the intensity of soil creep the creep parameter b_k is used; it is determined by the compression tests from the formula:

$$b_k = \Delta \varepsilon_{kt} / \ln(t/t_v) \quad (4)$$

where $\Delta \varepsilon_{kt} = \Delta s_{kt} / h$ – creep deformation; Δs_{kt} – creep settlement accumulated during the time $(t-t_v)$, cm; h – sample height, cm; t – compression time of the sample from the beginning of its loading, days; t_v – end of consolidation, days.

During the construction period $(t < t_v)$ the applied load is varying over time. It is appropriate to use the theory of hereditary creep of Boltzmann-Voltaire for the prediction of creep settlements under the variable loads (Vyalov S.S. 1978, Tsytoich N.A. & Ter-Martirosjan Z.T. 1981)

For the complex load growth pattern and complex core type of creep, it is convenient to solve the integral equations of the theory of hereditary creep by the numerical method of Krylov-Bogoliubov (Ukhov S.B. et al. 2002, Tsytoich N.A. & Ter-Martirosjan Z.T. 1981, Z.G.Ter-Martirosyan. 1990, Zaretskyi Y.K.1988, Fadeev A.B. 1987, Zhusupbekov A.Z, Zhakulin A.S, Orazaly E.E, Popov. V.N. 2001).

6 CONCLUSIONS

1. The long-term field settlement observation for 50 years provides a great deal of scientific and practical interest.
2. The geotechnical investigations and laboratory testing results show that the soils underlying the site have relatively high density and low void ratios. The laboratory studies confirm an increase in water content of soils and the transition from a semi-solid to a low-plasticity consistency.
3. The laboratory results indicate that the soils exhibit gradually decreasing creep behavior.
4. The actual foundation settlements of the industrial projects were up to 2.5-17 times less than the design and maximum allowable values based on the building codes.
5. The settlement analysis should take into account the elastic, consolidation and creep components of the settlements, as well as make use of elasto-plastic models of the ground.

6. The results show the need for additional research to collect and systematize data and new amendments to generally accepted formula in the building codes for determining the foundation settlements with creep. The amendments to the basic calculation formulae may bring the theoretically derived results to more practical values on the depth of the main distribution of compressive stresses, settlement values and the intensity of creep.

1 REFERENCES

- Tsyтович N.A. & Ter-Martirosjan Z.T. 1981. *Bases of applied geomechanics in construction*. Moscow: VS, 382 p.
- Tsyтович N.A. et al. 1967. *Prediction of settlements rates of buildings and structures*. Moscow: Stroizdat, 239 p.
- Bulychov N.S. 1989. *Mechanics of underground structures in the examples and problems: A study manual*. Moscow: Nedra, 270 p.
- Ukhov S.B. et al. 2002. *Soil Mechanics, Foundations*. Moscow: VS, 566 p.
- Building Code RK 5.01-01-2002 «*Foundations of buildings and structures*».
- Vyalov S.S. 1978. *Rheological principles of soil mechanics*. Moscow: Higher Schools Publishing House, 448 p.
- Tatsuoka F., Shibuya S. and Kuwano R. 2001. *Advanced laboratory stress-strain testing of geomaterials*. Netherlands: A.A Balkema Publishers. Tokyo. 329 p.
- Zhakulin A.S. and Zhakulina A.A., Kropachev P.A. 2011. *Water-saturated soil stress-strain characteristics*. The 14th Asian Regional Conference Soil Mechanics and Geotechnical Engineering. Soil Mechanics and Geotechnical Engineering: Challenges and Solutions. Hong Kong. China, p.151-152.
- Nakai T., Shahin H., Kyokawa H., Miyahara Y. 2011. *Modeling of one-dimensional consolidation behavior for clays*. The 14th Asian Regional Conference Soil Mechanics and Geotechnical Engineering. Soil Mechanics and Geotechnical Engineering: Challenges and Solutions. Hong Kong. China, p.128-129.
- Z.G. Ter-Martirosyan. 1990. *Reologic Parameters of the Grounds and Calculations of the Construction Foundations*. Moscow: Stroyizdat. 291 p.
- Zaretskyi Y.K. 1988. *The Ground Ductility and Plasticity and Calculations of the Construction*. Moscow: Stroyizdat. 352 p.
- Fadeev A.B. 1987. *The Method of Final Elements in Geomechanics*. Moscow: Nedra. 221 p.
- Zhusupbekov A.Z., Zhakulin A.S., Orazaly E.E., Popov. V.N. 2001. *Definition of parameters of consolidation and creep in saturated soils*. Proceedings of the 15th International Conference Soil Mechanics and Geotechnical Engineering. Istanbul. Vol.1, p.817-820.

Technical Committee 101
**Laboratory Stress Strain Strength Testing
of Geomaterials**

Session II - Strength properties and treated soils

Comité technique 101
***Caractérisation en laboratoire du comportement
des géomatériaux***

Session II - Propriétés de résistance et sols traités

General Report - Session II

Laboratory Testing of Geomaterials: Strength Properties and Treated Soil

Rapport général - Session II

Essais de laboratoire des géomatériaux : propriétés mécaniques et sols traités

Kim D.-S.

Korea Advanced Institute of Science and Technology (KAIST)

ABSTRACT: This report presents a brief review of 30 papers submitted for the session on Laboratory Testing of Geomaterials II: strength properties and treated soils. The paper topics were very diverse but the papers were separated into four broad categories : 1) shear strength, 2) treated soil, 3) testing method and 4) miscellaneous. Some papers include valuable database of important strength parameters for design and some of the tested soils include special soils. The report includes discussions related to the categories, summarizes the papers, and highlights their major conclusions.

RÉSUMÉ : Ce rapport présente un bref résumé des 30 contributions soumises à la session Essais de laboratoire des Géomatériaux II: propriétés mécaniques et sols traités. Les sujets traités ont été variés, mais finalement les contributions écrites ont été groupées en quatre grandes catégories: 1) résistance au cisaillement, 2) sols traités, 3) méthode d'essai et 4) sujets divers. Certains articles comprennent des bases de données pertinentes de paramètres importants de résistance pour la conception d'ouvrages et certains sols testés font partie des sols spéciaux. Le rapport comprend des discussions liées aux catégories, synthétise les résultats et met en évidence leurs principales conclusions.

KEYWORDS: laboratory test, shear strength, treated soil, testing method, database

1 INTRODUCTION

The theme of this session focuses on Laboratory Testing of Geomaterials II: strength properties and treated soils. The 30 papers submitted to this session are reviewed and summarized in this report. To guide readers to the most relevant papers, Table 1 is provided in the appendix that shows the paper ID and main focus of each paper, subdivided into four broad categories: a) shear strength, b) treated soil, c) testing method, and d) miscellaneous. The first session of shear strength was subdivided by three topics: 1) database of strength and stiffness properties, 2) gas-hydrate bearing soil and mining by-products, and 3) soil fabric and particle characteristics.

The diversity of authors and tested soils truly highlights the international nature of our profession. Authors are from Australia, Brazil, Chile, Denmark, France, Greece, Hong Kong, Hungary, India, Iran, Iraq, Japan, Korea, Mexico, South Africa, Sweden, Taiwan, Thailand, United Kingdom, and United States, based on the first author. Some of the papers include some valuable database such as anisotropic strength ratio of natural clays (2173), drained strength parameters from New Orleans area (2280), over-consolidated Danish clays (2399), soft and stiff Bangkok clays (2872), and rockfill materials (3011). Some of the tested soils include special soils such as gas-hydrate bearing ground in the Lake Baikal (2001), hydrophobic (non-wettable) sand (2178), soil with diatom microfossils in Mexico City (2239), coal mine debris (2782), and copper mine tailings (2800). The report includes discussions related to the categories, summarizes the papers, and highlights their major conclusions.

2 SHEAR STRENGTH

2.1. Database of Strength and Stiffness Properties

In the design of geotechnical structures, strength and stiffness parameters obtained by well-documented database and/or correlation with index properties are valuably utilized in the early stage of design. This session introduces test database

including anisotropic undrained strength ratio, the comparison of drained strength parameters obtained by triaxial and direct shear tests, drained strength parameters on OC Danish clay, stiffness and strength parameters for Bangkok clays and rockfill materials. For cohesive soils, strength parameters are correlated with plasticity index and compared with the well-known empirical correlations by Brooker and Ireland (1965), Berre and Bjerrum (1973), and Ladd et al. (1977). Strength and stiffness parameters based on Duncan and Chang (1970) model were obtained for Bangkok clay, Baghdad soil, and rockfill materials. Six papers are in this topic and each paper is summarized as below.

In paper 2137, Won developed the relation between anisotropic strength ratio ($K_s = S_{ue}/S_{uc}$) and plasticity index using the collected database of 203 pairs of triaxial tests performed on NC natural clays from 14 countries. For a consistent comparison, data selection criteria were carefully established. He found that the anisotropy was strongly influenced by the definition of failure in extension test, and no general trend of anisotropy with plasticity were observed, once the anisotropy data are grouped into their depositional environments (Fig.1). It is interesting to note that this is opposite to the well-known trend that anisotropy decreases with plasticity index and he emphasized the importance of careful consideration of site specific characteristics, spatial variability, depositional and post-depositional environments of the clay.

In geotechnical projects, triaxial and direct shear tests are often used interchangeably to determine drained shear strength parameters without regard to the potential difference. In paper 2280, Castellanos and Brandon performed a series of 63 consolidated undrained (CU) triaxial tests and 146 consolidated drained (CD) direct shear tests on undisturbed samples from New Orleans area to compare the shear strength parameters. They showed that drained friction angles obtained from the CU triaxial tests were considerably higher than those obtained from CD direct shear tests in New Orleans area (Fig 2) because

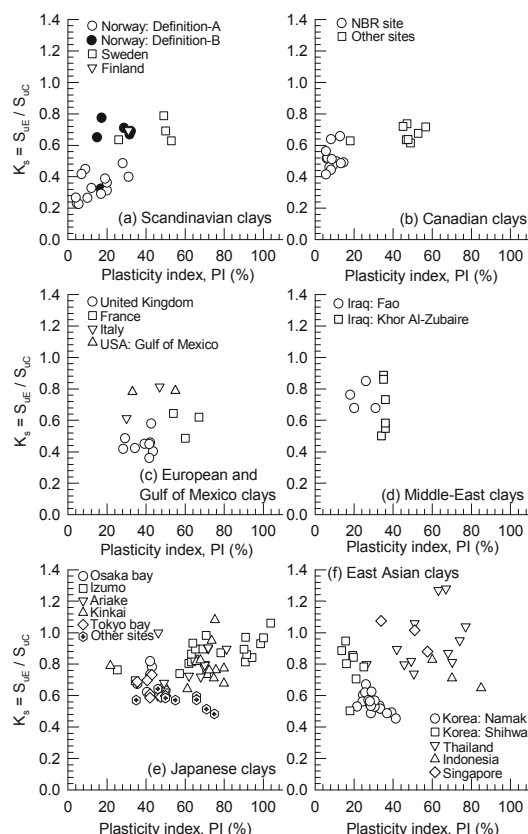


Fig. 1 Anisotropic strength ratio versus plasticity index for different depositional environments from J.Y. Won, paper 2137.

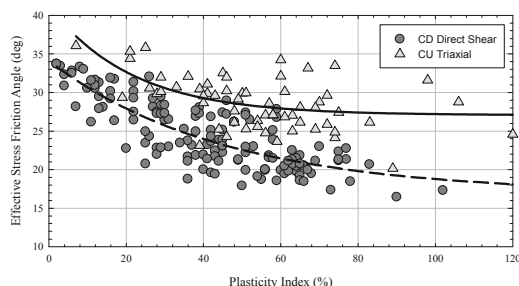


Fig. 2 Relationship between effective stress friction angle and plasticity index for CD direct shear tests and CU triaxial tests on undisturbed samples from Castellanos et al., paper 2280.

natural soils can have a preferred particle orientation or anisotropic fabric based on the deposition of the soil. The difference was much less when remolded specimens were tested.

Empirical correlations between the index properties and the strength and deformation properties of cohesive soils are useful in geotechnical engineering practice. In [paper 2399](#), Sorensen and Okkels have suggested simple correlations between plasticity index and drained peak strength parameters in terms of ϕ'_{oc} and c'_{oc} . The database includes triaxial compression test results on undisturbed OC Danish clays from very low to extremely high plasticity obtained from recent projects including Great Belt bridge, Fehmarnbelt investigation, and Esbjerg Harbour. The proposed correlations give cautious lower bound values of drained strength parameters, which can be used as a first approximation for use in the preliminary design (Fig. 3 and 4).

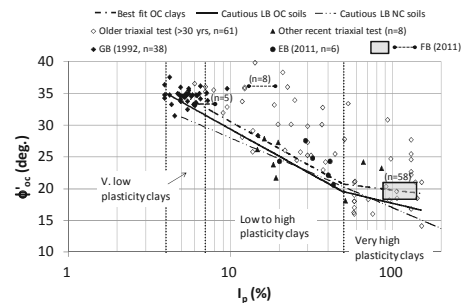


Fig. 3 Relationship between peak angle of shearing resistance ϕ'_{oc} and plasticity index I_p for overconsolidated undisturbed clays from Sorensen et al., paper 2399.

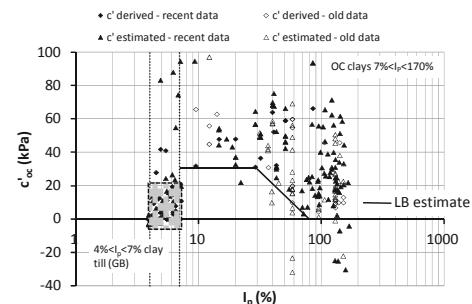


Fig. 4. Relationship between effective cohesion c'_{oc} and plasticity index I_p for overconsolidated undisturbed clays from Sorensen et al., paper 2399.

In [paper 2639](#), al-Damluji et al. provided the bounding surface plasticity model parameters for Baghdad soils by K_0 -consolidated compression and extension triaxial undrained tests. They mentioned these parameters can be used in the foundation design in the central Baghdad.

In [paper 2872](#), Likitlersuang et al. re-analysed the stress-strain data of soft and stiff Bangkok clays carried out at Asian Institute of Technology (AIT). Several series of isotropic consolidated drained and undrained compression (CID, CIU) and extension (CIUE, CIDE) tests were carried out at AIT. The finite element software PLAXIS contains the hardening soil model as an extension of the Duncan-Chang hyperbolic stress-strain model. The stiffness and strength parameters required for the hardening soil model to model undrained and drained behaviors were obtained.

In [paper 3011](#), Aghadam and Soroush studied mechanical behavior of thirty types of rockfill materials based on the hyperbolic model under triaxial compression. The rockfill materials are categorized as three types: highly angular, angular, and rounded. The exponent number was found to be dependent on confining pressure due to the particle breakage. The correlations estimating initial Young's modulus and friction angle were suggested based on particle shape, confining pressure, and uniformity.

2.2. Gas-hydrate Bearing Soil and Mining By-Products

Energy resource development is a facing problem of mankind. Gas-hydrate is an attractive energy source but the production is challenging due to following uncertainties such as changes in stress condition due to pore pressure changes during phase transition and sediment softening and volume contraction due to loss of hydrate bonding (Lee et al., 2011). Safe and economical storage of by-products in the coal and copper mines is an important geotechnical problem. Three papers in this topic and each paper is summarized as below.

Gas hydrates (GH) are attracting an attention as a next generation energy source but there are concerns that dissociation of GH and exsolution of dissolved gas can reduce the stability of seabed and may induce seafloor landslides. In [paper 2001](#), Yamashita et al. studied the effects of sample disturbance due to the exsolution of dissolved gas in gas-hydrate bearing deep lake bottom sediments in Lake Baikal. The hand vane shear and cone penetration tests were performed for core samples. The laboratory simulation of stress relief caused by bringing samples to the surface was also performed by CO₂ gas. They showed that the effects of the sample disturbance become larger and the strength is lower with the increase of gas concentration.

The overburden materials in the coalfields of South East Queensland are dominated by uncemented rocks, which rapidly break down on excavation to extract coal and bulk up to a very loose density. In [paper 2782](#), Williams and Kho performed the direct shear test and staged creep compression test on scalped specimens under dry and wet conditions. They found that wetting-up causes a substantial reduction in shear strength and they quantifies the settlement and shear strength of uncemented overburden materials excavated from open pit mining in the coalfields.

Both the increased production in the copper mining industry and the decrease of available space for the construction of tailing deposits led to the design of dam having unprecedented height above 250m. In [paper 2800](#), Campana et al. performed the drained and undrained triaxial tests on four different tailing sands from copper mines located in Chile and Peru by applying confining pressure up to 3 Mpa. The deformation moduli, static and cyclic shear strength under drained and undrained conditions, densities and fine contents have been obtained. It was found that tailing sands have shear resistance and shear modulus values greater than expected in natural sands.

2.3. Soil Fabric and Particle Characteristics

Fabric is a collective term to describe the geometric arrangement of grains and voids, and the distribution of inter-particle contacts, and the influence of initial fabric on stress-strain-strength responses received great attention and the uniqueness of critical state has been great challenged (Negussey and Islam, 1994). The particle characteristics such as particle shape, surface roughness, and angularity can affect inter-particle behaviors. Soils in nature often become hydrophobic (non-wettable) due to organic pollutants, wild fire, oil spill, and the difference in surface properties result in clear difference in flow and mechanical behaviour even in macro scale (Kim et al., 2010). Five papers are in this session and two of those are studied by numerical simulations.

The strength parameters are very important to conduct the stability analysis of earth structures but test results using reconstituted specimens change easily depending on the sample preparation method. In [paper 2043](#), Kotaka et al. studied the effects of initial water content during sample preparation on undrained shear behaviour in gravel-mixed sand. They thought that initial suction in the specimen produce various soil structure and verified using numerical simulation by the SYS Cam-clay model. It was found that numerical simulation can reproduce the various types of experimental shear behaviours of the gravel-mixed sand derived from the different soil structures.

The Jacky's K_0 equation is commonly used for the estimation of K_0 but uncertain aspect still exists which value of friction angle be adopted because the friction angle is a state-dependent variable. In [paper 2166](#), Lee et al. investigated the effects of particle shape, surface roughness, and angularity on Jacky's ϕ - K_0 relation using normal sand, glass bead, and etched glass bead

in the oedometer type thin-wall K_0 test. They found that the measured K_0 values for loose sand were close to the calculated values using inter-particle friction angle whereas for dense sand, critical state friction angle produced close match to the measured one, implying that the application of peak friction angle may underestimate K_0 value. For glass beads, calculated K_0 values using critical state angle were in good agreement with measured results in loose and dense states.

In [paper 2178](#), Kim et al. studied the hydraulic and geomechanical properties of non-wettable sands using artificially synthesized hydrophobic sands. Sands without any treatment and with chemical treatment were used. Fig 5 shows water distribution in hydrophilic and hydrophobic sands observed by an optical microscope. They found that surface modification at nano-scale determine the spatial configuration of water phase in pore space and its impact on fluid flow and strength with varying degree of saturation prevails.

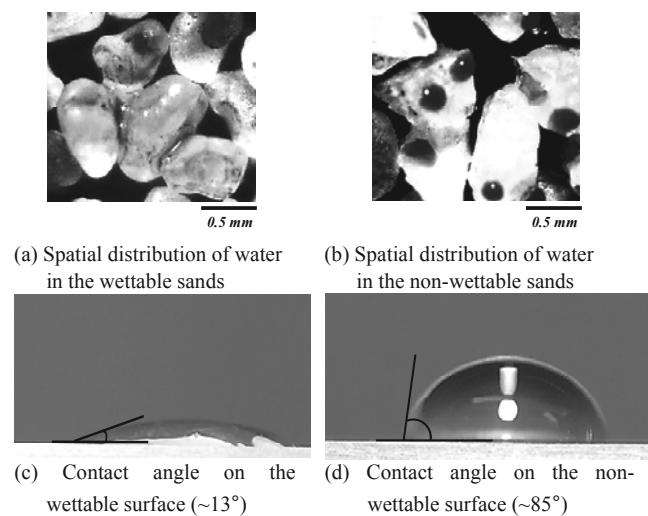


Fig. 5 Optical observations of water droplet formed on the wettable and non-wettable samples in the micro scale from Kim et al., [paper 2178](#).

Non-coaxial behavior of saturated sands, which refers to the non-coincidence of the principal stress directions and the principal plastic strain rate directions, were studied using hollow cylinder apparatus in [paper 2489](#). Yang et al. found that the effective stress ratio has a significant effect on the non-coaxiality of sand. The volumetric strain of sand induced by cyclic rotation of principal stress axes was mainly contractive and it occurs during first few cycles.

Critical state refers to a state where material undergoes continued distortion at constant volume and constant stresses, and the advances in modern laboratory tests have initiated the discussion on the effect of fabric on critical state and thus its uniqueness. In [paper 2960](#), Yan and Zhang investigated the fabric evolution of idealized two-dimensional assemblages having different initial fabrics subject to numerical biaxial shearing. It was found that a unique fabric of particle orientation and void space is achieved at very large strains where the stresses and volume of the assemblages are constant.

3. TREATED SOILS

Soft soils are prominently found in coastal regions and low land areas where many important infrastructures are located. Various admixtures are currently used to enhance the mechanical and flow properties of clay and sand soils. Historically, Portland cement and lime have been used for this purpose (Tim Newson, 2009). Recently, cement-mixed gravelly soil is used to construct bridge abutments for high speed trains in Japan and lightweight

cemented clays are used as a backfill for quay walls and bridge abutments. This session deals with various kinds of treated soils including lime treated soil, cement mixed soft clay, clay with diatom microfossils, cement mixed gravelly soil, asphalt aggregate, and lightweight cemented clay.

In [paper 1831](#), Hibouche et al. studied the small strain behaviours of lime treated soil using small strain triaxial tests equipped with two types of sensors to measure local strains: strain gauges and Hall effect sensors. Both measurements were compared. The elastic moduli derived from cycles in strains less than 10^{-4} were compared with those deduced from the elastic wave speed. Relations between the modulus, the strain level, and the hardening were established.

Compared to other methods of improving the soft ground, deep cement mixing and mass stabilization with binders are rapid techniques of ground improvement. In [paper 2174](#), the interaction of binders with the soft clay and improvement in strength were studied based on water/cement ratio, water content of clay, and curing time. The used binders include cement, lime, fly ash, slag and Sodium Silicate. The unconfined strength increased with decreasing water/cement ratio irrespective of initial water content of clay and the addition of Sodium Silicate to cement enhance the strength significantly.

There are several sites in the world where diatom microfossils have been detected in the soil deposits. Those soil deposits have singular physical and mechanical properties that do not follow the well-established empirical relations. In [paper 2239](#), a series of oedometer tests was performed on artificially prepared mixtures of diatom microfossils and kaolin and it was observed that the presence of diatom microfossils substantially alters the index properties as well as compressibility.

The cement-mixed gravelly soil (CMG) is widely used in geotechnical engineering and it is needed to develop the design and construction procedures. In [paper 2287](#), a series of drained triaxial compression tests were performed on laboratory prepared specimens and rotary core samples from the field. To better correlate the strength and deformation characteristics of CMG, two independent parameters are postulated; the soil skeleton porosity, n_s (representing the structure of the skeleton of gravelly soil particles only), and the cement void ratio, C_r (representing the fraction of the void of the soil skeleton occupied by cement). An empirical equation to predict the compressive strength is proposed using soil skeleton porosity, which controls initial compressive strength, and the cement void ratio which controls the increasing manner of $q_{\max}$ with curing time. The effects of grading characteristics on the strength were not significant but the effects of specimen volume were significant.

The soil cement technique has been used in pavement base layers, slope protection for earth dams, as a base layer for shallow foundations, and to prevent sand liquefaction. In [paper 2579](#), the elastic moduli of soil-cement mixtures in terms of shear and constrained moduli at small strain were measured with time based on wave propagation by Consoli et al. Significant increase in stiffness was observed compared to the uncemented sands as curing process continues. The unique relationships linking modulus values with porosity/cement ratio and curing time was developed.

The behaviour of asphalt under slow rates of loading and of the role of the aggregate skeleton is important when pavement is subject to subsidence. In [paper 2851](#), Airey and Prathapa performed a series of conventional drained and undrained triaxial tests on two types of asphalt, stone mastic asphalt(SMA) and dense asphaltic concrete(DAC). Tests without asphalt binder have also been conducted. The behaviour of asphalt was

observed sensitive to the details of the aggregate grading. For DAC, the behaviour was controlled by the aggregate particles but little affected by bitumen, whereas the bitumen has a significant effect on stress-strain-strength behaviour for SMA.

Lightweight cemented clays have wide applications as a backfill to reduce the earth pressure, as a fill on soft soil to reduce overburden pressure, and as a method of reducing pressure on the tunnel lining. In [paper 3040](#), Horpibulsuk et al. found that the void/cement ratio, V/C , is the prime parameter governing the strength and compression characteristics. The yield stress in K_0 -consolidation and compressive strength increase as V/C decreases and a relationship between strength, void/cement ratio for a particular water content and curing time was proposed.

4. TESTING METHOD

This section describes six papers that review the testing methods for soil properties. The measured soil properties include compaction, particle size distribution, tensile strength of soil, permeability, dispersive soil, and dry density of soil. The importance to understand proper mechanism, reliability of test results, source of errors and the need to develop optimum procedure without ambiguity are discussed in this session.

In [paper 2210](#), Perez et al. studied gyratory compactor which is developed to simulate the field compaction mechanism produced by the sheep foot roller. The controlled variables were gyration angle, vertical pressure, and number of gyrations. The optimum water content was reduced and dry density was increased as the vertical pressure increases, but the compaction curves are similar regardless of gyration angle and rate. It was suggested that using heavy equipment rather than many passes is more effective to increase the dry density.

The particle size distribution of coarse-grained soils is traditionally determined by sieve tests. In [paper 2248](#), Ohm and Hryciw discussed two image-based systems, the Sedimaging (for 2.0 mm to 0.075 mm particles) and the Translucent Segregation Table (TST) (for 75 mm to 2 mm particles). These tests do not require that particles be physically detached from one another unlike previous image-based methods. Testing setup of TST is shown in Fig. 6. Sedimaging uses mathematical wavelets to determine particle sizes and requires a camera magnification that provides at least 3 pixels per particle (PPD=3). The TST uses watershed analysis to digitally detach particles and requires PPD=9. The minimum PPDs must be achieved while capturing entire specimens in the camera's field of view. Extension of the systems to silt sized particles is explored.

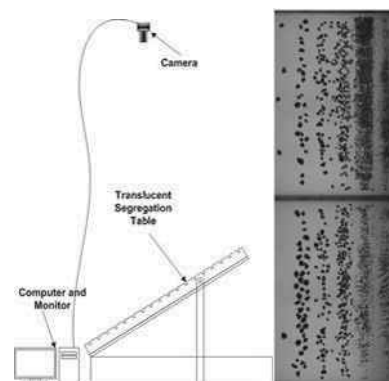


Fig 6. Translucent Segregation Table (TST) from Ohm & Hryciw et al., paper 2248.

The tensile strength of soil is an important parameter in the design of geo-systems, where tensile cracks contribute to the progressive erosion or landslides in excavation, slopes, dams,

riverbanks and other earth structures. In [paper 2572](#), Ge and Yang developed indirect indentation method based on an upper bound solution to a split tension failure in limit analysis and performed a series of indentation tests on lightly cemented sand. The limitation of this method is that a certain level of brittleness of the specimen is required so that a split tension failure would occur.

Coefficient of permeability can vary over an order of magnitude depending on the testing method even in a relatively homogeneous layer. In [paper 2930](#), Nagy et al. determined and compared permeability coefficients of a sandy silt and a silty sand obtained by Khafagi probe, Menard probe, water filtration method, constant head and falling head laboratory tests. As shown in Fig. 7, the result showed higher scattering than expected which has a relative error of one order of magnitude. They suggested that it is not practical to use different measurement methods in a single borehole because swapping may result in measured findings showing greater differences than normally accepted.

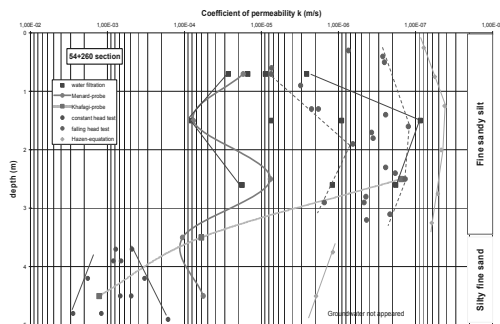


Fig. 7 Measured permeability coefficients from Nagy et al., paper 2930.

The reliable identification of dispersive soil is crucial but recent testing suggests that many shortcomings related to testing of dispersive soils have been overlooked during past routine investigations. In [paper 2976](#), a comparative study involving the testing of three samples using one standard test, the SCS double hydrometer test, was carried out. Maharaj and Paige-Green observed that the variability of the results appears to be the cause of many of the ambiguities and discrepancies in the classification system and stressed the needs to develop optimum procedure which is simple and have as few ambiguities as possible so that no misinterpretation can occur.

In [paper 3084](#), Imre et al. performed a statistical analysis based on the results of two doctoral programs concerning the dry density of sands (Kabai 1974, Lorincz, 1986). They showed that the different dimension of the mold has some impact on the measurement of minimum dry density and the $e_{\max}$ test of the German DIN may be biased because of the arching due to the too small ratio of the diameter and height of the mold.

5. MISCELLANEOUS

Hardening due to thixotropy is important mechanism for the very soft dredged soils with high water contents. In [paper 2446](#), Tanaka and Seng studied the thixotropic hardening by measuring change in stiffness and strength with time using bender element and vane shear tests. The effect of thixotropy was found to be significant at around the liquid limit state and the increment of shear modulus due to thixotropy appeared noticeably higher than that in the secondary consolidation.

In [paper 2494](#), Athanasopoulos et al. studied a 30m high slope failure at the Xerolakka municipal solid waste landfill. Field investigations including Lidar survey and shear wave velocity measurements to characterize the MSW materials were

performed and numerical analysis was followed. The results indicated that failure was caused by a combination of factors including inappropriate waste disposal practices and compaction, leachate and gas pressure generation and increased steepening of the landfill slopes.

In [paper 2705](#), Sarma and Sarma described the influence of minerals on the elastic behaviour of cohesive soil. They mentioned that the pattern is unique for cohesive soil and is under the possible domination of the existence of a common mineral and extent of its weathering process.

6. REFERENCES

Papers in the session of Laboratory Testing of Geomaterials II: Strength Properties and Treated Soil (please see Appendix-Table 1)

ADDITIONAL REFERENCES

- Brooker E. W. and Ireland H.O. (1965). Earth pressure at rest related to stress history. *Canadian geotechnical journal*, vol. 2, no. 1, pp.1-15
- Berre T. and Bjerrum L. (1973). Shear strength of normally consolidated clays. *Proc. 8th ICSMFE*, Moscow 1, pp.39-49.
- Duncan, J.M. and Chang, C.M. (1970). Nonlinear analysis of stress and strain in soils. *Journal of Soil Mechanics and Foundations Division, ASCE*, 96(SM5), pp.1629-1653.
- Kabai, I. (1974). The effect of grading on the compactibility of coarse grained soils. *Periodica Polytechnica*.18 (4), pp.255-275.
- Kim, D.H., Kim, Y.J., Lee, J.-S., Yun, T.S., (2010). Thermal and electrical response of unsaturated hydrophilic and hydrophobic granular materials. *Geotechnical Testing Journal* 34(5), pp.562-570.
- Ladd C.C., Foott R., Ishihara K., Schlosser F. and Poulos H.G. (1977). Stress deformation and strength characteristics. *Proc. 9th ICSMFE*, Tokyo 2, pp.421-494.
- Lórinicz, J (1986). Grading entropy of soils Doctoral Thesis, Technical Sciences, TU of Budapest.
- Negussey, D. and Islam, M. S. (1994). Uniqueness of steady state and liquefaction potential. *Can. Geotech. J.*, 31(1), pp.132-139.
- Lee J.Y., Ryu B.J., Yun T.S., Lee J.H. and Cho G.C. (2011). Review on the gas hydrate development and production as a new energy resource. *KSCE*, 15(4), pp.689-696

APPENDIX

Table 1. List of papers in Laboratory Testing of Geomaterials II: Strength Properties and Treated Soil

Title	Author(s)	Test soil	Testing Method		Main Topics
1. Strength					
1.1 Database of strength and stiffness properties					
Anisotropic strength ratio and plasticity Index of natural clays (No. 2137)	J. Y. Won (U.S.A)	Natural clay	CK ₀ UC, C K ₀ UE 203 Data Base from 14 countries		The well-known trend between $\frac{s_{vc}}{s_{vc}}$ & PI cannot be justified Careful consideration of site specific characteristics
A comparison between the shear strength measured with direct shear and triaxial devices on undisturbed and remolded soils (No. 2280)	Castellanos & Brandon (U.S.A)	Undisturbed and remolded clays in New Orleans	CU TX test CD DS test		Comparison between drained strengths measured by direct shear and triaxial test Importance of preferred particle orientation based on horizontal deposition
Correlation between drained shear strength and plasticity index of undisturbed overconsolidated clays (No. 2399)	Sorensen & Okkels (Denmark)	Undisturbed OC Danish clays	TXC test (305 data collection from old & recent projects)		Drained peak strength vs Plasticity index Lower bound ϕ' and c' for preliminary design
Bounding surface plasticity model parameters for Baghdad soils (No. 2639)	Omar al-Farouk Salem al-Damluji et al. (Iraq)	Undisturbed and disturbed soil sample	K ₀ & isotropically consolidated TXC & TXE		Parameters for bounding surface plasticity model for Baghdad soil
Duncan-Chang parameters for hyperbolic stress strain behaviour of soft Bangkok clay (No. 2872)	Suched Likitlersuang et al. (Thailand)	Soft and stiff Bangkok clay	Drained & undrained TXC and TXE Data Base from AIT		Undrained & drained strength & stiffness parameters for hardening soil model of Bangkok clay
Interpretation of stress-dependent mechanical behaviour of rockfill materials (No. 3011)	Jannati Aghdam & Soroush (Iran)	Rockfill materials	TXC test 30 types of database		Variations in deformation and strength parameters of rockfill material with confining pressure
1.2 Gas-hydrate bearing soil and mining by-products					
Evaluation of sample disturbance due to the exsolution of dissolved gas in the pore water of deep lake bottom sediments (No. 2001)	Yamashita & Miura Kitami (Japan)	Lake Baikal, Russia gas hydrate-bearing sediment	Field	VST, CPT	Effects of exsolution of dissolved gas in the pore water on strength properties Reduction of strength with the increase of gas concentration
			Lab	UC Simulation of exsolution of dissolved gas	
Settlement and shear strength of uncemented coal mine overburden materials placed loosed under dry and wet conditions (No. 2782)	Williams & Kho (Australia)	Overburden materials in coal mine	DS test Staged creep compression test		Settlement of overburden material due to self-weight, collapse on wetting and degradation
Shear strength and deformation modulus of tailing sands under high pressures (No. 2800)	Campana & Bard (Chile)	4 tailing sands from copper mines	Drained & undrained TX tests with confining pressure up to 3MPa		Compare strength, modulus, and cyclic shear resistance of tailing sands with natural sands
1.3 Soil fabric and particle characteristics					
Soil structure in gravel-mixed sand specimen and its influence on mechanical behaviour (No. 2043)	Kodaka et al. (Japan)	Gravel-mixed sand	Undrained TX test		Effect of initial suction during specimen preparation on soil structure and strength
			Numerical simulation		
Effect of particle characteristics on K₀ behavior for granular materials (No. 2166)	Hun Hwan Lee et al. (Korea)	Jumunjin sand glass beads etched glass beads	Oedometer test to measure K ₀		Effects of particle shape, surface roughness and angularity on ϕ' -K ₀ relation
Non-coaxial behaviour of sand in drained rotational shear (No. 2489)	Yang et al. (U.K)	Leighton Bazzand sand	Drained rotational shear test with hollow cylinder apparatus		Non-coincidence of principal stress directions and principal strain increment directions
Characterization of geomechanical and hydraulic properties of non-wettable sands (No. 2178)	Dae hyun Kim et al. (Korea)	Jumunjin sand	Hydraulic tests DS test		Hydraulic and mechanical properties of non-wettable sands
Fabric and critical state of granular materials (No. 2960)	Yan & Zhang (Hong Kong)	Mono-sized pill shape rigid particle	Numerical biaxial test (discrete element)		Particle orientations and void space at large strains

Title	Author(s)	Test soil	Testing Method		Content
2. Treated soils					
Small strain behaviour of a lime-treated soil (No. 1831)	Hibouche et al. (France)	Lime treated soil	Small-strain TX test		Relations between small strain modulus, strain level hardening time
Mechanisms of binder interactions and their role in strengthening Kuttanad clay (No. 2174)	Suganya & Sivapullaiah (India)	Organic clay Binder : cement, lime, slag, fly ash	UC test		Interaction of binders with soft clay and its effect on strength
Influence of diatom microfossils on soil compressibility (No. 2239)	Diaz-Rodriguez & Gonzalez-Rodriguez (Mexico)	Kaolin with diatomite	Oedometer test		Compressibility of kaolins increase sharply with addition of diatomite
Strength properties of densely compacted cement-mixed gravelly soil (No. 2287)	Ezaoui et al. (France)	Cement mixed gravelly soil	TXC test		Effects of degree of compaction, cement content, curing time, gradation, specimen volume on q_{max}
Experimental analysis of the mechanical properties of artificially cemented soils and their evolution in time (No. 2579)	Consoli et al. (Brazil)	Cemented sand	BE using S and P waves		Characterization of soil-cement mixtures in terms of G_0, M_0 with porosity/cement ratio and curing time
Triaxial testing of Asphalt (No. 2851)	Airey & Prathapa (Australia)	Stone mastic asphalt (SMA) & dense asphaltic concrete (DAC)	TX test		Asphalt response under slow rate loading and role of aggregate skeleton
A key parameter for strength control of lightweight cemented clays (No. 3040)	Horpibulsuk et al. (Thailand)	Lightweight cemented clay	Oedometer test, UC test		Void/Cement ratio as the prime parameter for strength & deformation
3. Testing method					
Behaviour of fine-grained soils compacted with high shear stresses (No. 2210)	Perez et al. (Mexico)	Clay, silt, sand	Gyratory compactor Standard & modified proctor		Effect of vertical pressure, angle of gyration and number of gyrations on compaction curve
Enhanced soil characterization through advances in imaging technology (No. 2248)	Ohm & Hryciw (U.S.A)	Coarse grained soil	Sedimaging Translucent segregation table		Particle size distribution with imaging technology
Tensile strength of lightly cemented sand through indentation tests (No. 2572)	Ge & Yang (Taiwan)	Lightly cemented sand	Indentation test (Tensile strength)		Indirect method for determining tensile strength of lightly cemented sand
Comparison of permeability testing methods (No. 2930)	Nagy et al. (Hungary)	Sandy silt and silty sand	Field	Menard probe, Khafagi probe, water filtration	Comparison and reliability of various permeability methods
			Lab	Const. or falling head	
The SCS double hydrometer test in dispersive soil identification (No. 2976)	Maharaj & Paige-Green (South Africa)	Dispersive soil	Double hydrometer test		Ambiguity and discrepancies in the classification system
Some notes concerning the dry density testing standards (No. 3084)	IMRE et al. (Hungary)	Danube and Bochum sands	Density test for e_{min} and e_{max}		Biased in e_{max} measurement due to arching
4. Miscellaneous					
Hardening process of clayey soils with high water content due to thixotropy effect (No. 2446)	Tanaka & Seng (Japan)	Soft clays	BE VST		Thixotropic hardening of clays with high water content
The December 29 th 2010 xerolakka municipal solid waste landfill failure (No. 2494)	George Athanasopoulos et al. (Greece)	Municipal solid waste (MSW)	Field	LIDAR, SASW, Remi	Landfill failure due to inappropriate compaction, leachate and gas pressure generation and steepening slope
			Numerical simulation		
Influence of minerals on the elastic behaviour of cohesive soil (No. 2705)	Sarma & Sarma (South Africa)	Cohesive soil	N/A		Influence of prime mineral and its weathering process on elastic behaviour

Nationality is based on the first author

TX : Triaxial, VST : Vane Shear Test, UC : Unconfined Compression, DS : Direct Shear, TXC : Triaxial Compression, TXE : Triaxial Extension, BE : Bender Element test, CPT : Cone Penetration Test

Triaxial testing of asphalt

Essais triaxiaux de l'asphalte

Airey D.

University of Sydney, Australia

Prathapa R.

Roads and Maritime Services, New South Wales, Australia

ABSTRACT: The response of asphalt in conventional triaxial tests has received very little attention in the soil mechanics or pavement literature. As a result, the relative importance of the aggregate and the asphalt binder are not well understood, and in particular how they contribute towards the resistance to permanent deformation. This paper describes the results of a series of conventional drained and undrained triaxial tests on two types of asphalt, stone mastic asphalt (SMA) and dense asphaltic concrete (DAC). Tests without the asphalt binder have been conducted for each asphalt type. Results show that the DAC, which is well-graded and has a bitumen content of about 5%, behaves similarly to the aggregate without the bitumen at conventional soil mechanics rates of loading. However, significant strain rate effects are observed as the rate of loading is increased. For the SMA mixture, which has a higher bitumen content of about 6%, the asphalt appears to affect the frictional resistance of the mixture, and it is also far more significantly affected by strain rate. It is also noted that the asphalt binder has the effect of reducing the effects of strain localisation and allows uniform and barrelling type deformations despite the very dense aggregate.

RÉSUMÉ : La réponse de l'asphalte dans les essais à triaxiaux conventionnels a suscité peu d'intérêt dans la littérature de la mécanique des sols et des chaussées. Par conséquent l'importance relative de l'agrégat et du liant asphaltique n'est pas bien comprise, notamment comment ils participent à la résistance à la déformation permanente. Cet article décrit les résultats d'une série d'essais triaxiaux drainés et non drainés sur deux types d'asphalte, l'asphalte coulé gravillonné (SMA) et le béton asphaltique dense (DAC). Des essais sans liant asphaltique ont été effectués pour chaque type d'asphalte. Les résultats montrent que le DAC, qui a une granulométrie continue et une teneur en bitume d'environ 5 %, se comporte de manière similaire à un agrégat sans bitume pour les taux de chargement conventionnels en mécanique des sols. Cependant, on observe des effets significatifs de taux de contrainte à mesure que le taux de chargement augmente. Pour le mélange de SMA qui a une teneur plus élevée en bitume d'environ 6 %, l'asphalte semble affecter la résistance à la friction du mélange, qui est beaucoup affectée par le taux de contrainte. On note également que le liant asphaltique a l'effet de réduire les effets de la localisation de la contrainte et permet les déformations de type uniforme et en tonneau, malgré la très grande densité de l'agrégat.

KEYWORDS: aggregate, asphalt, triaxial tests.

1 INTRODUCTION

Analysis of asphalt materials has focused on their response under traffic loading, and in particular on the effects of temperature and stress level on the resilient stiffness and permanent deformation (Antes et al, 2003, Li et al, 2010). However, there has been much less interest in the behaviour of asphalt under slow rates of loading, and of the role of the aggregate skeleton (Muraya et al, 2009). The latter can be important when a pavement is subject to subsidence, for example caused by underground mining. In this situation the role of the aggregate controls the asphalt behaviour and the stiffness can be as much as 2 orders of magnitude lower than the resilient modulus determined from conventional asphalt testing. To capture this behaviour elastic-visco-plastic models of asphalt have been developed, but there is little data available to determine the parameters for these more sophisticated models.

The triaxial tests described in this paper are part of a study designed to provide the basic soil mechanics framework for the aggregate and to enable the effects of different aggregate gradations and bitumen contents to be more rigorously interpreted. The paper will describe a series of triaxial tests performed on two types of asphalt, stone mastic asphalt (SMA) and dense asphaltic concrete (DAC). Tests of the two materials without the bitumen have also been performed. The gradations

of the materials, their method of preparation and the results of the triaxial tests are presented.

1.1 Specimen preparation

Asphalt test specimens for these tests were provided by the Roads and Maritime Services (RMS) and were manufactured in the Fulton Hogan (FH) asphalt laboratory in Sydney. The majority of the aggregates in the asphalt mix were comprised of crushed basalt from the Bass Point quarry.

Dense graded asphalt mixes with 14mm nominal aggregate size conforming to RTA Roadworks specifications R116 (DAC) and R121 (SMA) were used in testing. The particle size distributions are outlined in Table 1, and the materials and their proportions used to achieve these gradations are shown in Table 2.

Samples of at least 200 kg of each SMA and DAC mix were taken from a plant production run and were delivered to the FH asphalt laboratory. Care was taken during handling to avoid any segregation of the mixes. In the laboratory the asphalt mixes were reheated and compacted in a shear box compactor to produce prismatic compacted specimens with dimensions of 450 mm x 150 mm x 185 mm. Approximately 10 cylindrical specimens with diameter of 70 mm and length of 140 mm were cored from each of the compacted rectangular prisms. The

cylindrical specimens were then subjected to conventional CIU and CID triaxial tests.

Table 1. Particle size distributions.

Sieve Size	DAC (%)	SMA (%)
19.0	100.00	100.00
13.2	99.00	90.24
9.5	83.38	50.06
6.7	66.74	30.98
4.75	56.81	24.14
2.36	39.97	20.78
1.18	27.44	17.92
0.60	20.74	15.79
0.30	11.79	13.04
0.15	7.69	11.53
0.075	5.92	10.27
Filler/Binder	1.15	1.55
Bitumen	5.17	6.64

Table 2. Mix proportions.

Aggregate	DAC %	SMA%
14mm Basalt	19.8	57.98
10mm Basalt	19.4	14.22
7mm Basalt	6.40	0.00
Basalt Dust	35.1	9.52
Benedict's Glass Sand	13	3.27
Hydrated Lime	1.14	5.24
Finely Ground Limestone	-	2.97
Cellulose Fibres	-	0.28
C450 Bitumen	5.20	6.50

Additional triaxial tests have been performed on aggregate specimens with identical gradings to the asphalt specimens, except for the absence of the bitumen. These specimens were 100 mm in diameter by 200 mm tall, and were prepared by tamping the moist aggregate mixture into a split mould located on the triaxial base platen.

The asphalt specimens had nominal air voids of 5.5% and very low permeability. To saturate the specimens elevated back pressures of between 600 kPa to 1000 kPa were used, but even so several days were required for the rate of water inflow to drop below 3 mm³/min at which stage the specimens were considered to be effectively saturated.

2 RESULTS AND DISCUSSION

The results of drained triaxial tests on the DAC mix are shown in Figures 1 and 2. These figures compare the response of asphalt specimens isotropically compressed to 200 kPa and 700 kPa, with a specimen comprised only of the aggregate compressed to 500 kPa.

Figure 1 shows that the differences between the two asphalt (DAC) specimens and the aggregate only specimen are primarily due to the different effective stress levels, and that the effects of the bitumen on the stress-strain response are relatively minor for these tests, which were conducted at a strain rate of

0.5%/hr. From the volume strain responses shown in Figure 2 it is evident that the asphalt specimens are more dilative. This is a consequence of the asphalt specimens being denser than the aggregate only specimen.

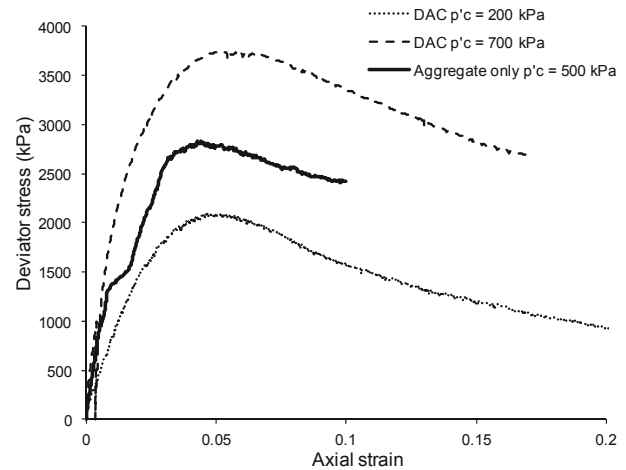


Figure 1. Deviator stress, strain responses from drained tests of DAC.

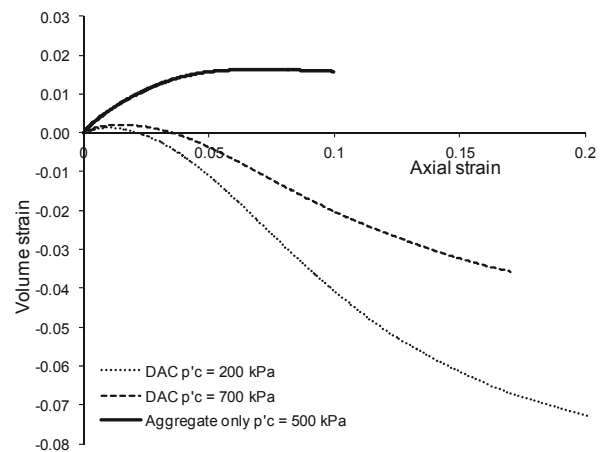


Figure 2. Volume strains from drained tests on DAC.

From a soil mechanics perspective the behaviour of the asphalt might be expected to depend on the void ratio, but for the asphalt specimens where the voids are mostly filled with bitumen a number of definitions of void ratio can be employed. The results presented in Figures 1 and 2 show the bitumen has little influence on the stress-strain response which suggests that the definition of void ratio should include the space filled by the bitumen as well as the water. For the asphalt specimens this results in a value of approximately 0.24 for the aggregate void ratio, e_{agg} . However, if the bitumen is included with the solids then a much lower void ratio of 0.08 is obtained. If the bitumen makes no contribution to the strength of the material then we might expect to observe a unique critical state line, using the aggregate void ratio, e_{agg} , which is unaffected by the presence of the bitumen. This is explored in Figures 3 and 4. Figure 3 shows the effective stress paths in deviator stress, q , versus mean effective stress, p' , from several asphalt specimens subjected to both drained and undrained tests, and Figure 4 shows the resulting changes in aggregate void ratio. It can be observed that the asphalt specimens behave as expected for dense granular materials, with significant dilation at low effective stresses and with the effects of dilation diminishing as the effective stress level increases. All specimens approach a unique critical state line described by $M = 1.68$, a friction angle of 41° . When the

variations of the aggregate void ratio are inspected it can be seen that these also provide support for a critical state line with gradient $\lambda = 0.056$.

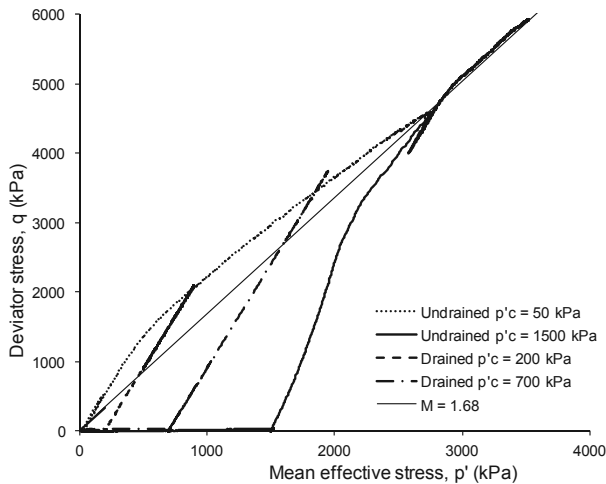


Figure 3. Effective stress paths for DAC specimens.

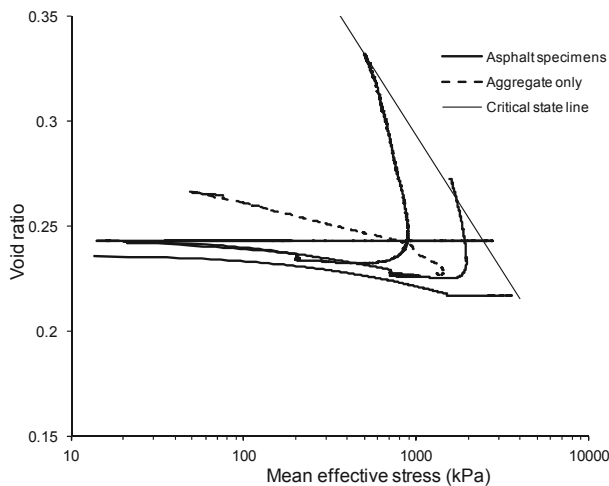


Figure 4. Variation of aggregate void ratio for DAC specimens.

When the test with aggregate only is plotted on Fig. 4 it appears that it does not reach the critical state line. However, the aggregate only specimen formed a pronounced shear plane and ruptured the membrane, and it is believed that if the test had continued the specimen would have expanded and approached the same ultimate locus as the asphalt specimens. It may also be noted that the asphalt binder resulted in more uniform deformations being observed in all the DAC specimens, which tended to bulge out despite their dense and dilative nature.

Because different confining stresses have been used in the drained tests for the asphalt specimens and the specimen without the bitumen, the test results cannot be directly compared. An alternative approach is to compare the stress ratio (q/p') versus axial strain responses as presented in Figure 5. This shows that the specimen without the binder at an effective confining stress of 500 kPa is behaving very similarly to the asphalt specimen at 700 kPa. As discussed previously, and shown in Figure 4, the asphalt specimens are slightly denser and this can explain, at least part of, the difference in the behaviour with stress level. The similarity of the shape of the responses also suggests that the ultimate frictional resistance of the aggregate is similar to that of the asphalt specimens, and as expected the frictional resistance of the asphalt is controlled by the aggregate. The results presented in Figures 1 to 5 all suggest that the binder has only a minor effect on the stress-strain

response of the DAC asphalt specimens. At the slow loading rates used in these tests the bitumen is simply acting as a viscous pore fluid, and it is viscosity is sufficiently low for it to flow out of the way of the aggregate to aggregate particle interactions.

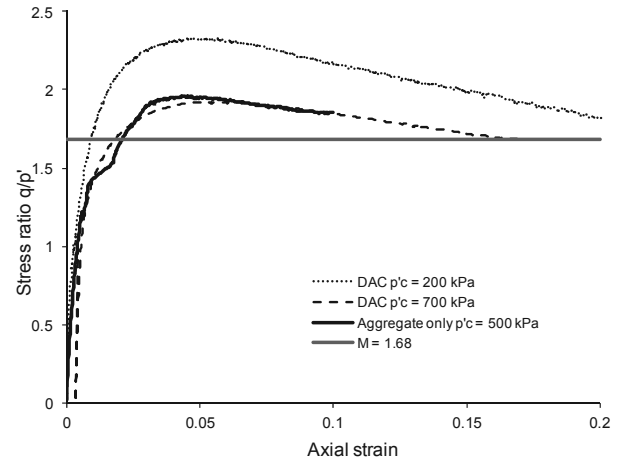


Figure 5. Stress ratio, strain responses in drained tests.

Tests have also been conducted on Stone Mastic Asphalt (SMA) specimens. SMA is used in pavements as it is reported to have better resistance to rutting, and this is believed to result from the greater contact between the larger particles of aggregate, which occurs because of the removal of some of the intermediate particle sizes in the grading.

Figure 6 shows a comparison between the stress-strain responses of the SMA and DAC specimens in drained tests. This shows that there are significant differences between the two asphalt materials. The SMA specimen has a peak deviator stress only half that of the DAC specimen even though both specimens have the same confining stress. This significant reduction is not the result of differences in the aggregate void ratio, which is essentially identical for the two materials, and neither can it be explained by differences in the frictional characteristics of the aggregate as the same aggregate is used for both types of asphalt. However, the SMA aggregate only specimen had a significantly higher void ratio, approximately 0.4, and this can explain why the SMA aggregate appears less stiff and more compressible than the DAC aggregate.

Figure 7 shows the volume strains measured during the drained tests shown in Figure 6. The DAC specimen is significantly more dilative than the SMA specimen even though the specimens are compacted with similar energies and with

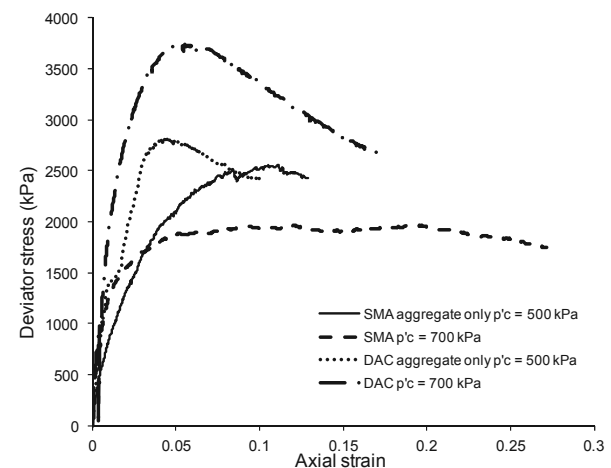


Figure 6. Comparison of SMA and DAC materials in drained tests.

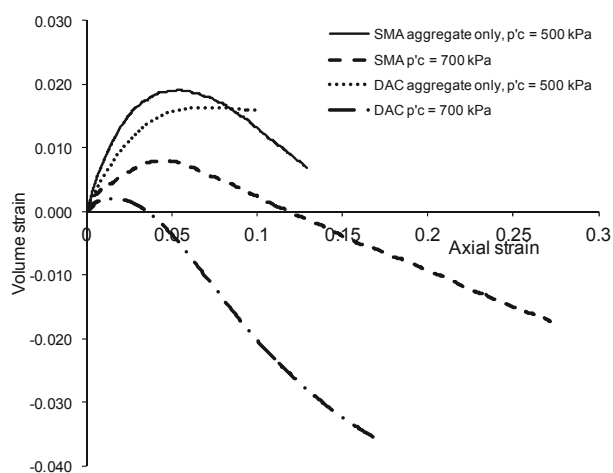


Figure 7. Comparison of volume changes for SMA and DAC.

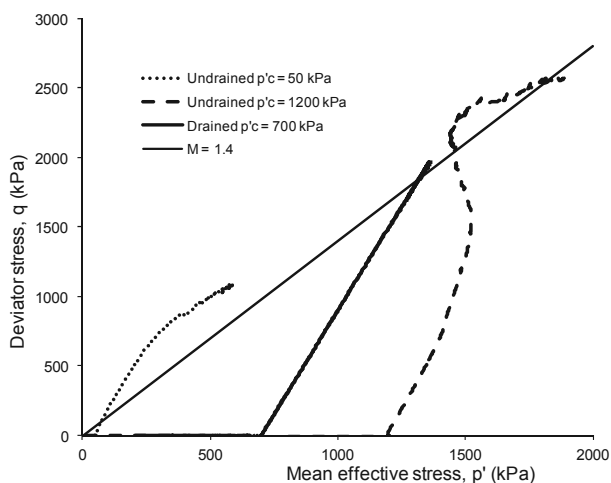


Figure 8. Effective stress paths of SMA specimens.

similar aggregate void ratios. Although the lower strength of the SMA specimen is consistent with the reduced dilation, this does not explain the very much lower mobilized frictional resistance of the SMA specimen, which can be seen from the plot of the effective stress paths of the drained test, and two undrained tests of SMA, shown in Figure 8. Figure 8 shows the mobilized stress ratio for the tests at the higher stress levels is 1.4, corresponding to a friction angle of 34.6° . This is significantly less than the friction angle of the aggregate which is expected to be 41° , similar to the DAC aggregate because the material is the same and the grading is not that different.

It can be seen that the SMA specimens mobilize a significantly lower resistance than the DAC specimens, even though they have mostly the same aggregate material with the same frictional characteristics. The differences between the two asphalt materials include: the gap grading of the SMA aggregate; the slightly higher bitumen content in the SMA; and the presence of a small amount of cellulose fibres in the SMA mix. The presence of fibres would be expected to provide some additional strength to the asphalt mix, and there was no evidence that they had a significant effect on the aggregate only mixture, so the fibres do not appear to be responsible for the difference in behaviour. It thus appears that in SMA the bitumen is interacting with the fine particles and acting to minimize the frictional interaction of the larger aggregate. In contrast in the well graded DAC there appears to be sufficient interaction between the larger aggregate particles so that the bitumen acts independently of the particle structure.

The results also suggest that SMA has a lower residual strength than DAC once the aggregate structure is disturbed at slow rates of loading. This is in direct contradiction to how the SMA product is supposed to perform at high rates of loading, which is to have better resistance to permanent deformation. This difference in behaviour at higher loading rates could be related to the much greater sensitivity of the SMA to changes in loading rate which has been observed in the triaxial tests. This is also consistent with the greater influence of the bitumen in the SMA tests than for the DAC.

3 CONCLUSION

Conventional soil mechanics triaxial tests have been conducted on saturated asphalt specimens. These have shown that the behaviour of the asphalt can be very sensitive to details of the aggregate grading. For dense asphaltic concrete, which has a well-graded aggregate and 5% bitumen, the behaviour in slow triaxial tests is controlled by the aggregate particles, and the bitumen appears to have little effect. For the stone mastic asphalt, which has a gap-graded aggregate and 6% bitumen, the bitumen has a significant effect on the stress-strain-strength behaviour.

The sensitivity of the asphalt to relatively minor changes in grading and bitumen content is a surprising result, and further tests are in progress to help understand the mechanisms responsible for this behaviour.

4 ACKNOWLEDGEMENTS

The authors are grateful to the Roads and Maritime Services (NSW) for supplying the asphalt specimens and materials used in this study.

5 REFERENCES

- Antes, PW, Van Dommelen, AE, Houben, LJM, Molenaar, AA and Parajuli, U (2003) Stress dependent behaviour of asphalt mixtures at high temperatures, *Proceedings of the Technical Sessions*, v 72, p 173-195, *Asphalt Paving Technology*.
- Li, P, Liu, J and Saboundjian, S. (2010) Materials and temperature effects on the resilient response of asphalt-treated Alaskan base course materials, *Journal of Materials in Civil Engineering*, 23, 2, p. 161-168.
- Muraya, PM, Molenaar AAA and van de Ven MFC (2012) Contribution of the bituminous mortar and stone skeleton to the resistance to permanent deformation in Asphalt mixtures, *Journal of Testing and Evaluation*, 37, 5, 1-7.
- Roads and Maritime Services(RMS) (2010), R121. QA Specifications, Stone Mastic Asphalt, Ver. 2.1 NSW, Australia.
- Roads and Maritime Services(RMS) (2012) R116. QA Specifications, Dense Graded Asphalt, Ver. 8.2 NSW, Australia.

Bounding surface plasticity model parameters for Bagdad soils

Paramètres du modèle de plasticité de surface de délimitation pour les sols de Bagdad

Al-Farouk O., Al-Damluji S., Al-Shakarchi Y.J., Albusoda B.S.
University of Baghdad

ABSTRACT: The attempt of this paper is to provide and determine essential properties and input parameters that the bounding surface plasticity model requires and utilizes for Bagdad clay in order to be used in non-linear soil behavior analyses under static and dynamic conditions. Site investigations were carried out and undisturbed samples of high quality were obtained from the capital city at depths of 3-3.5 m, 5-5.5m and 7-7.5 m. A series of advanced soil laboratory tests were then performed under K_0 -consolidated compression and extension conditions. The obtained soil laboratory test results were then used in a computer algorithm in order to evaluate soil properties and compute input parameters for the Iraqi clayey soil that are required for the bounding surface plasticity model.

RÉSUMÉ : La tentative de ce document est de fournir et de déterminer les propriétés essentielles ainsi que les paramètres requis et utilisés dans le modèle de la plasticité de surface de délimitation pour les sols et argiles de Bagdad afin d'être utilisé dans les analyses non linéaires de comportement des sols dans des conditions statiques et dynamiques.

Études et enquêtes sur le terrain ont été effectuées et des échantillons intacts de haute qualité à des profondeurs de (3 à 3,5 m), (5-5,5 m) et de (7 à 7,5m) ont été obtenus des sols de la capitale (Bagdad).

Une série de tests de pointe du sol sous des conditions de compression et d'extension K_0 -consolidés ont ensuite été effectuées. Les résultats obtenus en laboratoire des sols d'essai ont été ensuite utilisés dans un algorithme informatique afin d'évaluer les propriétés du sol, et de calculer les paramètres d'entrée pour le sol argileux irakienne qui sont nécessaires pour le modèle de plasticité de surface de délimitation.

1. ESTABLISHMENT OF MODEL PARAMETERS

1.1. Introduction

The bounding surface plasticity model, as developed by Dafalias and Herrmann (4.1), is a versatile constitutive relation that can account for the behavior of clayey soils under various loading conditions. It is currently well established as a reliable algorithm for static and dynamic applications on condition of adoption of appropriate input parameters.

1.2. Site investigation

A scheme was developed in order to obtain representative high quality undisturbed and disturbed soil samples from central Bagdad, the capital city of Iraq in order to obtain the required input parameters for the model.

1.3. Drilling and sampling

Drilling is done by the use of continuous flight augers, undisturbed samples are obtained by using Shelby tubes and an appreciable amount of disturbed soil (enough for conducting soil classification tests) is also extracted. The samples obtained were waxed, sealed and transported to the soil mechanics laboratory of the College of Engineering of the University of Baghdad in order to conduct further laboratory tests on them.

1.4. Testing program and results

As shown in Figure (1), A series of classification, engineering and ultrasonic tests are performed in this study. Classification tests are conducted first, in addition to physical and chemical tests. Physical tests comprise of specific gravity, Atterberg limits, and hydrometer tests while engineering tests that are carried out include standard

consolidation tests, cyclic consolidation tests and triaxial tests according to the following conditions:

- K_0 consolidated undrained triaxial compression tests (K_0 CUCT $\rightarrow$ OCR=1 and 2).
- K_0 consolidated undrained triaxial extension tests (K_0 CUET $\rightarrow$ OCR=1).
- Isotropically consolidated undrained triaxial compression tests (ICUCT $\rightarrow$ OCR = 1, 1.2 and 5).
- Isotropically consolidated undrained triaxial extension tests (ICUET $\rightarrow$ OCR = 1, 1.2 and 5).

Figure 2 shows the soil type according to the Casagrande chart, Table 1 summarizes the results of the ultrasonic tests while Figures 3 and 4 show the conventional and cyclic consolidation test results. Table 2 presents the results of these consolidation tests. Plates 1 and 2 show the testing setups for the triaxial tests, Figures 5 to 7 show the test results and Table 3 shows the angle of internal friction obtained from these triaxial tests.

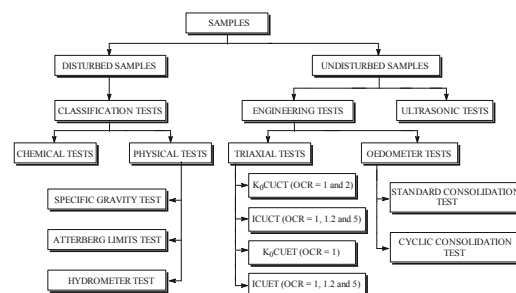


Figure (1) The testing program.

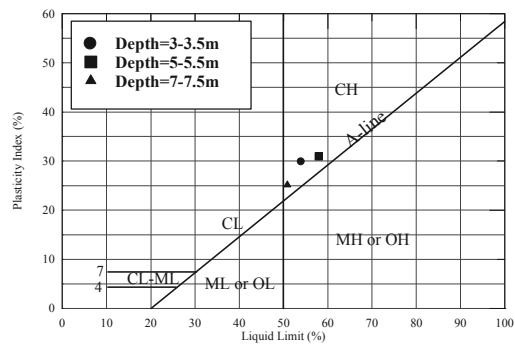


Figure 2. Soil type using Casagrande chart.

Table (1) Results of ultrasonic tests

Depth (m)	V_p (m/sec)	V_s (m/sec)	E (kPa)	G (kPa)	ν	K (kPa)
3-3.5	412	134	1.01×10^5	35104	0.44	2.85×10^5
5-5.5	1238	516	1.4×10^6	$\frac{51121}{2}$	0.40	2.26×10^6
7-7.5	966	998	1.28×10^6	$\frac{48360}{8}$	0.32	1.17×10^6

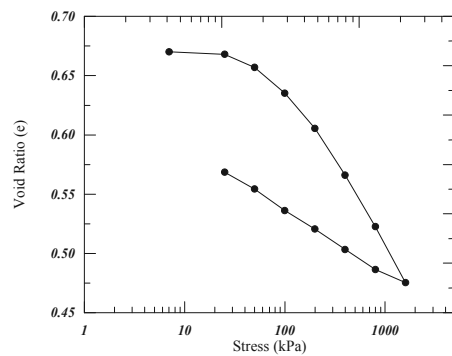


Figure 3. Results of a Standard Consolidation Test at Depth (5-5.5 m).

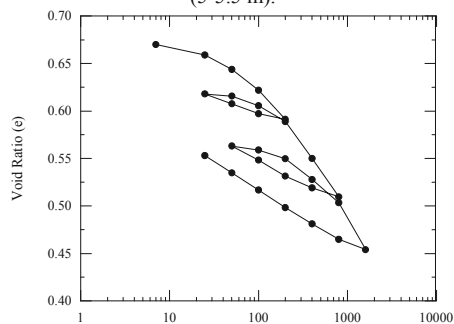


Figure 4. Results of a cyclic consolidation test at depth (5-5.5 m)

Table (2) Results of standard consolidation tests

Depth (m)	C_c	C_s	P_c	P_o	OCR
3-3.5	0.162	0.001	250	47.5	5.26
5-5.5	0.156	0.037	115	68.7	1.67
7-7.5	0.300	0.048	155	145.5	1.065

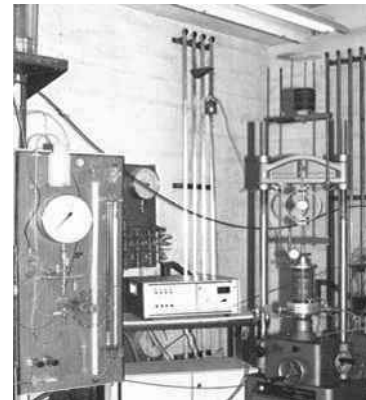


Plate .1. Triaxial setup for anisotropic consolidation.

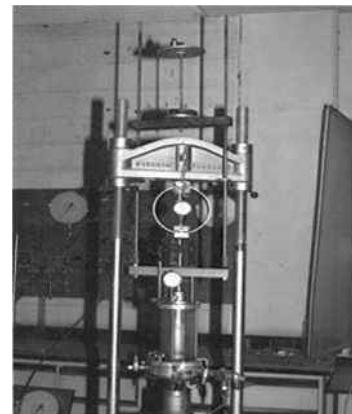


Plate 2. Complete triaxial setup for extension test.

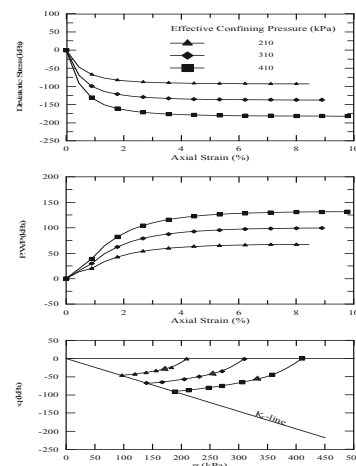


Figure .5. Results of isotropically consolidated undrained triaxial extension tests at depth (5-5.5 m).

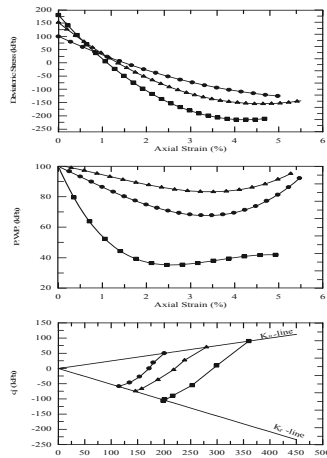
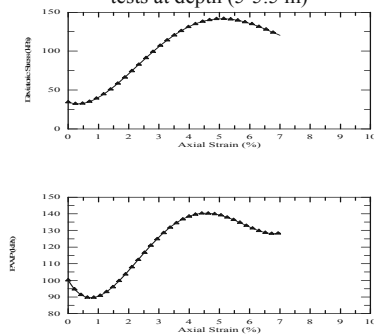
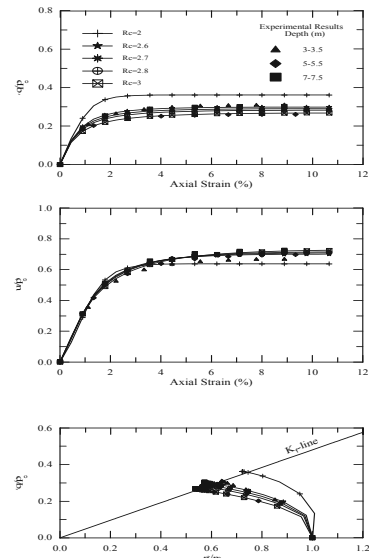
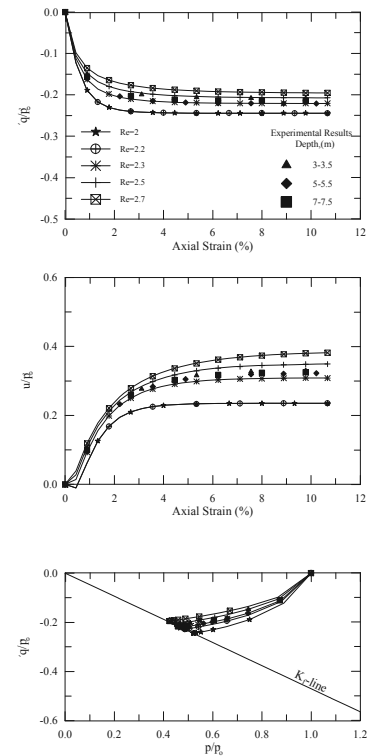

 Figure (6) Results of K_0 -consolidated undrained triaxial extension tests at depth (5-5.5 m)

 Figure (7) Results of a K_0 -consolidated triaxial compression test of overconsolidated soil ($OCR = 2$) at Depth (5-5.5 m)

 Table (3) Values of Angle of Internal Friction ϕ'

Depth (m)	Isotropically consolidated specimens		K_0 -consolidated
	Compression CIUCT	Extension CIUET	Compression CK_0 UCT
3-3.5	29°	27°	28°
5-5.5	29°	29°	28°
7-7.5	32°	28°	31°
ϕ'_{av}	30°	28°	29°

Figures 8 to 10 show the calibration outcomes when following the procedures outlined in reference (4.2).


 Figure (8) Calibration of parameter R_0 for Baghdad brown silty clay (BBSC) soil

 Fig.(9) Calibration of parameter R_0 for Baghdad brown silty clay (BBSC) soil

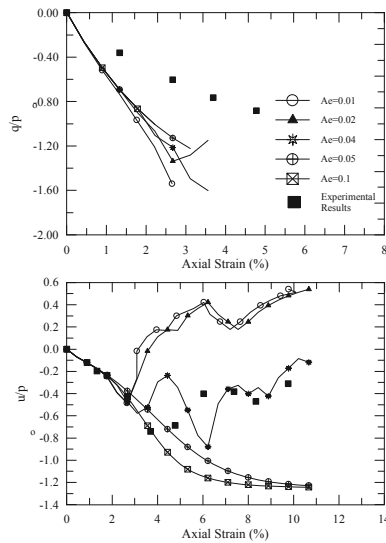


Figure (10) Calibration of parameter A_e for Baghdad brown silty clay (BBSC) soil

Table 4 outlines the full set of parameters obtained from the testing program on the extracted samples from the central Baghdad area soil after conducting the necessary calibration procedures outlined in reference (4.2).

Table (4) Complete set of obtained input parameters of Bounding Surface Plasticity Model for Baghdad brown silty clay (BBSC) soil

Parameter	Value	Parameter	Value
λ	0.064	R_e/R_c	0.85
κ	0.017	A_e/A_c	0.80
M_c	1.2	C	0.0
M_e/M_c	0.676	S_p	1.0
ν	0.4	m	0.02
R_c	2.7	h_e/h_c	0.5
A_c	0.05	h_c	2.0
T	-0.1	h_o	1.5

2. CONCLUSIONS

A set of Bounding Surface Plasticity Model parameters are established from high quality comprehensive site investigation and laboratory testing programs. The parameters presented in Table 4 are further successfully used in prediction/observation studies on foundation problems in the central Baghdad city area (4.3).

3. ACKNOWLEDGEMENTS

The authors wish to thank the Directorate of the National Center for Construction Laboratories (NCCL) of Iraq for supporting this research work.

4. REFERENCES

- Dafalias, Y.F., and Herrmann, L.R., (1982), "Bounding Surface Formulation of Soil Plasticity", Soil Mechanics -Transient and Cyclic Loads, G.N. Pande and O. C. Zienkiewicz, Eds., John Wiley and Sons, 253.
- Dafalias, Y.F., and Herrmann, L.R., (1986), "Bounding Surface Plasticity II: Application to Isotropic Cohesive Soils", ASCE Vol. 112, No. 12, December, pp.1263-1291.
- al-Damluji, O.F., al-Shakarchi, Y.J. and al-Busoda, B.S., "Static and Dynamic Non-Linear Behaviour by the Bounding Surface Plasticity Model for Iraqi Soil", Al-Handassa, Engineering Journal, College of Engineering, University of Baghdad, Volume 11, No. 4, Pp 641-654, December 2005.

The December 29th 2010 Xerolakka Municipal Solid Waste landfill failure

29 décembre 2010 : l'échec d'enfouissement Xerolakka

Athanasopoulos G., Vlachakis V.

Dept. of Civil Engineering, University of Patras, Greece

Zekkos D.

Dept. of Civil and Environmental Engineering, University of Michigan, Ann Arbor, USA

Spiliotopoulos G.

Patras Municipality, Greece

ABSTRACT: On December 29th 2010, a 30 m high slope failed at the Xerolakka Municipal Solid Waste landfill in Greece. The failure resulted in temporary interruption of the landfill disposal activities and closure of the landfill access road; it also received significant media attention. A reconnaissance of the landfill slope instability was performed a few hours after the failure. Subsequent data collection, field investigations and numerical analyses were performed to better characterize the causes of the instability. Data collection included review of available data regarding the landfill design and the waste material at the Xerolakka landfill. Field investigations included Lidar surveying to closely map the post-failure geometry, as well as shear wave velocity measurements that were used as a basis for characterization of the MSW material and comparison with data available in the literature. Numerical analyses included limit equilibrium as well as finite element analyses. The results of the investigation indicate that the failure was caused by a combination of factors, including, inappropriate waste disposal practices, inadequate compaction, leachate and gas pressure generation and increased steepening of the landfill slopes.

RÉSUMÉ: Le 29 décembre 2010, une pente de 30 m de haut a échoué à la mise en décharge des déchets Xerolakka solides municipaux en Grèce. L'échec a entraîné une interruption temporaire des activités de mise en décharge et la fermeture de la route d'accès à la zone d'enfouissement et d'élimination ont reçu une attention médiatique importante. Une reconnaissance de l'instabilité de la pente d'enfouissement a été réalisée quelques heures après l'échec. Après la collecte des données, enquêtes sur le terrain et des analyses numériques ont été réalisées afin de mieux caractériser les causes de l'instabilité. La collecte des données comprenait un examen des données disponibles concernant la conception de la décharge et les déchets à la décharge Xerolakka. Les enquêtes de terrain inclues Lidar arpentage de près cartographier la géométrie post-rupture, ainsi que les mesures de vitesse de cisaillement d'ondes qui ont été utilisées comme base pour la caractérisation de la matière MSW et comparaison avec les données disponibles dans la littérature. Des analyses numériques incluent équilibre limite ainsi que des analyses par éléments finis. Les résultats de l'enquête indiquent que la panne a été causée par une combinaison de facteurs, y compris, les mauvaises pratiques d'élimination des déchets, le compactage insuffisant, le lixiviat et la génération de la pression du gaz et l'augmentation accentuation des pistes d'enfouissement.

KEYWORDS: Municipal solid waste, landfill, slope failure, shear wave velocity

1 INTRODUCTION

To protect public health and the environment, Municipal Solid Waste (MSW) landfill slopes need to be stable. Unfortunately, numerous landfill slope failures have been documented in the literature (e.g. Eid et al. 2000, Hendron et al. 1999, Kavazanjian and Merry 2005, Huvaj-Sarihan and Stark 2008) and many more remain undocumented. Although such failures are undesirable, it is important to learn from them so that similar occurrences are avoided in the future.

This paper presents the field observations from a reconnaissance study performed within hours after the December 29th 2010 Xerolakka landfill slope failure, as well as subsequent field measurements and stability analyses that were executed to better understand the causes of the failure.

2 THE XEROLAKKA LANDFILL

The Xerolakka landfill is one of the nine MSW landfills in the Region of Western Greece, located 5 km east of the City of Patras. It is a canyon landfill at the foothills of the Panachaikon Mountain. It started receiving waste in September 1993 and presently receives 300 tn of waste daily (Sufalnet, 2006).

The site is located on a geologic sequence of Pleistocene and Pliocene claystone, marls and siltstones with lenses of

sandstones that are generally considered intact. The groundwater table fluctuates seasonally significantly.

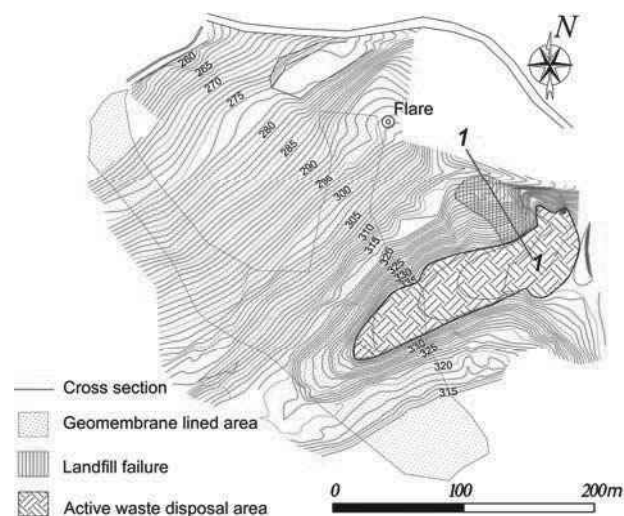


Figure 1. February 2011 topographic map of the Xerolakka canyon landfill.

A topographic map of the landfill is shown in Fig. 1. The first cell of the landfill was geomembrane-lined. Subsequently the use of geosynthetics was discontinued because of the presence of impermeable geologic formations (Seisakis and Roussos, 1994). Due to strong public opposition, new cells were not constructed, as anticipated in design. In the absence of an alternative waste management solution, the landfill continued to receive waste. Thus, a waste mound with increasing height and slope inclination was formed (shown in the southeast side of Fig. 1) which partially failed on December 29th 2010.

3 FIELD OBSERVATIONS

On December 29th 2010, early in the morning (around 08:00 am), a failure of one of the landfill slopes occurred in the active waste disposal area. The authors performed on-site reconnaissance at 14:00. The waste slide had plan dimensions of 50 m by 30 m and its crest was located at the top of the landfill (absolute elevation of +340 m) whereas its toe reached the access bench 27 m below. The volume of the slid waste mass is estimated to be equal to 12,000 m³. The waste slide debris covered one of the landfill benches that was used as access road to the active waste disposal area, thus disrupting landfill disposal operations. During the reconnaissance visit, the waste that covered the access road was already partially removed and pushed downhill. A view of the slide from the West is shown in Fig. 2 and a view of the slide from its toe after removal of the waste from the landfill access bench is shown in Fig. 3.



Figure 2. Waste slide view from the western side of the MSW landfill.



Figure 3. Waste slide view from the access bench located at the toe of the slide.

The waste slide is located adjacent to the graded canyon slopes with the Northeast portion of the slide exposing the native rock mass (also shown on the left side of Fig. 3). Precipitation on the steep canyon slopes in the vicinity of the

waste slide drains towards the waste mass due to the absence of surface water cutoff drainage ditches and percolates in the waste.

The uppermost layer of MSW in the active waste disposal area (i.e., the landfill crest) was not compacted and did not include any daily soil cover. The compaction of waste had reportedly ceased for at least a year prior to the failure and daily soil cover was not used for many months, possibly years. The absence of daily soil cover on the top waste layers can be seen at the right side of Fig. 2. In addition, the gas collection system was not operational.

The crest of the landfill was not graded properly to manage surface water runoff due to precipitation and in the vicinity of the failure slide mass, rainfall water was found to be ponding. Leachate was observed to pour from the toe of the waste slide whereas an interceptor trench that was built next to the landfill bench was also found to contain leachate. Media photos from earlier in the morning of the 29th of December indicate a large wet area in the vicinity of the failure, apparently from liquids that came out of the waste mass.

The December 29th 2010 failure occurred four days after a rainfall event. A weather station located in the Port of Patras at a distance of 4.5 km away from the landfill and at an absolute elevation of +6 m, recorded approximately 11 mm of precipitation for that event and a total of 16.5 mm in the five days prior to the failure. Ten days earlier, another event with a precipitation of 20 mm occurred. This amount of precipitation is lower than the corresponding amount of rainfall in the past two years; however, the geometry (height and inclination) of the landfill slopes had changed in the last year, adversely affecting its stability. The complete absence of surface water management system and daily soil cover, would have allowed for the rainfall water to easily percolate in the waste mass.

4 FIELD MEASUREMENTS

A high-resolution 3-D topographic map of the landfill area was generated by performing terrestrial LIDAR (Light Detection and Ranging) measurements, in addition to conventional geodetic survey. The measurements utilized land-based laser scanning technology and allowed a reliable definition of the failed waste mass. Field measurements of the in situ shear wave velocity (V_{s0}) were also performed. Shear wave velocity is a critical parameter that has been used to characterize the MSW (Zekkos, 2011). In this project, V_{s0} was used to characterize the MSW and assist in the selection of values for MSW material properties. Shear wave velocity profiles were also explicitly used for the performance of seismic stability analyses that are not described herein.

The small strain shear wave velocity of waste material was evaluated as a function of depth by applying the Spectral Analysis of Surface Waves (SASW) and Refraction Microtremor (ReMi) techniques. The application of these techniques is preferred in the case of landfills due to their non-intrusive nature (Matasovic et al., 2011). The V_{s0} vs. depth profile is shown in Fig. 4.

Fig. 4 compares the V_{s0} vs depth profiles measured at Xerolakka landfill with the data available in the literature. The mean and mean $\pm$ sigma V_{s0} curves are shown for MSW in three geographic regions, specifically southern California (Kavazanjian et al. 1996), northern California (Lin et al. 2004) and Michigan (Sahadewa et al. 2011). It is observed that the in situ data from Xerolakka are in the lower range of the literature V_{s0} data. This difference may be attributed to a number of factors including waste composition, but more importantly the absence of waste compaction and daily soil cover. It should be mentioned that, following the waste slide, the placement of waste (from Dec. 2011 to May 2012) was carried out in a single thick lift (~8 m), overlain by a soil cover ranging from 1 to 3 m.

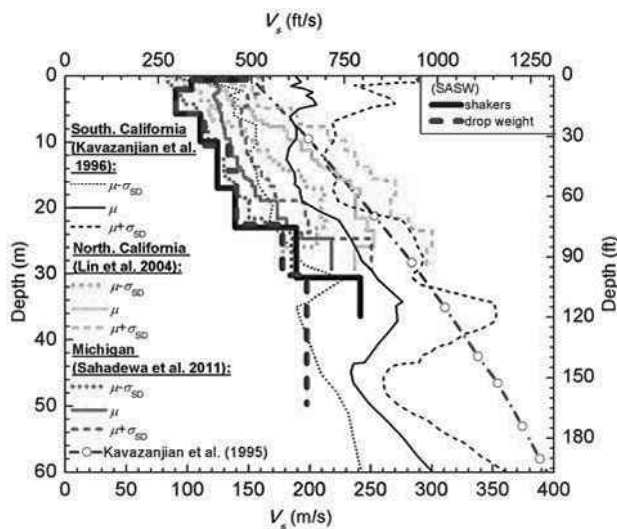


Figure 4. Comparison of V_{s0} vs depth profile at Xerolakka landfill with other published cases of landfills.

5 PROPERTIES OF MUNICIPAL SOLID WASTE

No site specific data was available on the MSW that was disposed of at the Xerolakka landfill. Thus, for the performance of the limit equilibrium and finite element stability analyses, the measured V_{s0} was used to guide the selection of MSW properties. For the performance of the analyses the unit weight, shear strength, deformation modulus (for the finite element analyses) and Poisson's ratio of MSW are required and were selected as follows:

Unit Weight: The selection of the MSW unit weight has an impact on the stability of the waste mass. On the basis of the available landfill information and the Zekkos et al. (2006) recommendations, for the 30 m thick waste mass, an average unit weight value of 12 kN/m^3 was used. This value is also consistent with the unit weight value used for the design of the landfill facility (Koronis 1995).

Shear Strength: The selection of appropriate shear strength parameters is critical in evaluating the stability of the waste mass. Bray et al. (2009) recommended a generic MSW shear strength envelope. The recommended strength envelope was the mean fit to a large dataset, however, various factors such as waste composition and unit weight may result in variations from this envelope. For example, the unit weight has an important impact on the shear resistance of MSW. As reported by Zekkos et al. (2010), for waste with the same waste composition, a reduction in unit weight by 2 kN/m^3 results in an approximate reduction in shear strength by 20%. Considering the absence of compaction and daily soil cover as well as the particularly low measured shear wave velocity of the MSW, the shear strength of the Xerolakka landfill MSW was reduced by 20% from the shear strength envelope recommended by Bray et al. (2009).

MSW elastic modulus and Poisson's ratio: The large-strain elastic modulus E_{ref} is an explicit input parameter in finite element analysis. The value of E_{ref} impacts the calculated displacements, but does not influence significantly the calculated factor of safety. In the present study, it was assumed that the modulus is equal to $1/10$ of the small-strain elastic modulus E_0 , which was calculated from the measured small strain shear modulus G_0 , whereas the Poisson's ratio value was assumed to be equal to 0.1, based on data available in the literature (Zekkos, 2005).

6 STABILITY ANALYSES

Stability analyses of the Xerolakka landfill slope failure were performed using both limit equilibrium (Geo-Slope 2007 –

SLOPE/W) and finite element (PLAXIS, 2004) analyses and the material properties described earlier. Each analysis methodology has its strengths and limitations. In finite element analyses, there is no requirement to predefine candidate failure surfaces; instead, the failure surface with the lowest factor of safety is identified using the phi-c reduction methodology (PLAXIS, 2004). Another known advantage of the FEM is its ability to calculate displacements in every prescribed stage of calculation as well as its ability to model progressive failure. In limit equilibrium methodology, the factor of safety for a large number of failure surfaces is calculated and the one with the lowest factor of safety is the critical one. For the calculation of the factor of safety, the Spencer method (Spencer, 1967) is used. Limit equilibrium methods do not account for the presence of strain softening materials, since no consideration of strains or displacements is made.

It is important to note that, in the case of Xerolakka landfill, it is very difficult to evaluate the actual pore pressure regime within the waste mass due to the unavailability of field data. Thus, stability analyses were performed for two cases: complete absence of leachate table (provided a leachate and gas collection system was operating properly) and for the case of a high leachate table resulting from the absence/non operative leachate and gas collection system. The leachate table used in the analyses was estimated on the basis of field observations, namely: 1) the presence of ponding water at the crest of the landfill (near the waste slide) and 2) observed seepage at the toe of the waste slide. The high leachate table is intended to account in a conventional manner for the presence, and possibly flow, of leachate and more importantly the generation of gas due to biodegradation. The amount of gas generated can be significant and for that reason, modern landfill facilities are equipped with a gas collection system that collects the gas and either combusts it using a flame or uses it to generate energy. There was no gas collection system in the active waste disposal area. Gas and leachate pressures would result in a reduction of the effective stress in the waste and a subsequent reduction in the factor of safety.

7 RESULTS OF ANALYSES AND DISCUSSION

Analyses were performed for the selected properties and the cross-section geometry at the location of the failure. The inclination of the slopes in the upper part of the landfill is as high as 1.2:1 (horizontal to vertical). In the case of absence of leachate table and gas pressure ("dry tomb" landfills), the results of analyses indicate a stable condition with a calculated factor of safety equal to 1.60, i.e., higher than the 1.50 typically required. Additional analyses were performed with the assumed leachate table, as shown in Fig. 5.

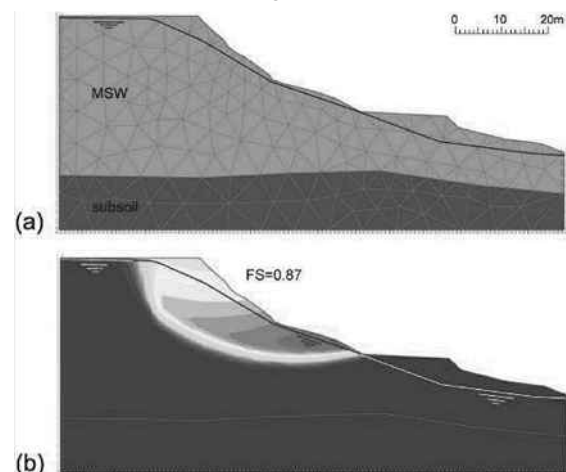


Figure 5. Finite element mesh (PLAXIS 8.6) of the critical failure surface with (a) soil stratigraphy and (b) critical failure surface for the estimated leachate table.

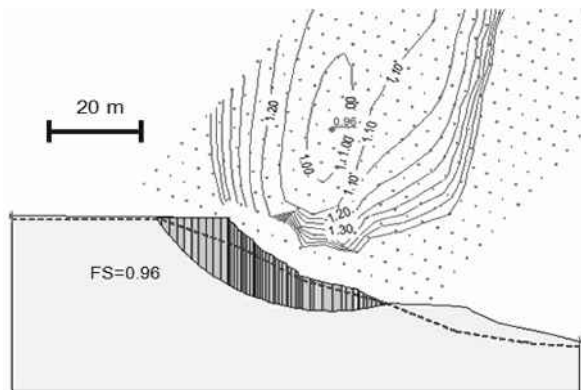


Figure 6. Limit equilibrium model (GeoStudio 2007 – SLOPE/W) with critical failure surface for the estimated leachate table.

For these conditions, the factor of safety based on finite element analyses is calculated equal to 0.87, indicative of unstable conditions.

Analyses using the limit equilibrium method (Geo-Slope 2007 – SLOPE/W) resulted in similar critical failure surfaces for dry conditions and for the assumed leachate table (shown in Fig. 6). The factor of safety is equal to 1.60 for dry conditions and 0.96 for the assumed water table.

The results of the above and additional analyses indicate that the reduction in the factor of safety due to the presence of leachate is significant and much greater than the impact of other uncertainties, such as the unit weight of waste material.

8 CONCLUSIONS

The waste slide that occurred on December 29th 2010 has height of 27 m and width of 30 m and involved a waste mass of 12,000 m³. The waste slide engaged MSW material only. On the basis of the reconnaissance studies, the field measurements and the stability analyses, the waste failure is attributed to poor landfill practices (absence of compaction and daily soil cover), the steep inclination of the waste mass and the increased percolation of rainfall water in the waste mass (and associated gas pressure generation) due to the absence of daily soil cover and surface water management system. The analyses also indicated that failure would not be incipient under dry conditions.

9 ACKNOWLEDGMENTS

The authors would like to thank Mr. D. Sardelianos, Civil Engineer at the Municipality of Patras for his assistance and support of the surveying measurements. Special thanks are due to Dr. P. Pelekis and A. Batilas, M.Sc. Civil Engineers for their participation and help in performing surface wave measurements and field data processing.

10 REFERENCES

- Bray, J.D., Zekkos, D., Kavazanjian, E., Athanasopoulos, G.A., and Riemer, M.F. 2009. Shear Strength of Municipal Solid Waste, *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 135 (6), 709–722, June 2009.
- Eid, H.T., Stark, T.D., Douglas, W.D. and Sherry, P.E. 2000. Municipal Solid waste slope failure I. Waste and foundation properties. *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 126 (5), 397–407.
- Geo-Slope International. 2007. "SLOPE/W, software for analyzing slope stability." Part of the GEO-STUDIO 2007 suite of applications. Calgary, Alberta.
- Hendron, D.M., Fernandes, G., Prommer, P.J., Giroud, J.P., and Orozco, L.F. (1999) "Investigation of the Cause of the 27 September 1997 Slope Failure at the Doña Juana Landfill," *Proc. Sardinia '99: Seventh International Waste Management and Landfill Symposium*, Cagliari, Italy, 545–554.
- Huvaj-Sarihan, N., and Stark, T. D. 2008. Back-analyses of landfill slope failures. In 6th International Conference on Case Histories in Geotechnical Engineering, Arlington, VA, August 11–16, 2008, Paper # 2.34 (in CD proceedings).
- Kavazanjian, E., Jr. and Merry, S.M. 2005. The July 10, 2000 Payatas Landfill Failure. In *Proc. Sardinia '05 - 10th International Waste Management and Landfill Symposium*, Environmental Sanitary Engineering Centre (CISA), University of Padua, Italy (in CD ROM).
- Kavazanjian, E., Jr., Matasovic, N., Stokoe, K.H.II, Bray, J.D. 1996. In situ shear wave velocity of solid waste from surface wave measurements. In *Environmental Geotechnics*, edited by M. Kamon, 1996 Balkema, 1, 97–102.
- Koronis S.A. 1995. *Implementation study of western stream cover, for the project: Design and construction of infrastructure for MSW landfill of the City of Patras*, Athens.
- Lin, Y.-C., Rosenblad, B., and Stokoe, K. H., II 2004. *Data report on shear wave velocity profiles determined by SASW method at: Altamont landfill, Redwood landfill, and Tri-Cities landfill*. Geotechnical Engineering Center, Civil and Environmental Engineering Department, The University of Texas at Austin, 29 October 2004, Geotechnical Engineering Report GR04-3.
- Matasovic N., El-Sherbiny, R. and Kavazanjian, E. 2011. Chapter 6: In-Situ Measurements of MSW Properties. In *Geotechnical Characterization, Field Measurements, and Laboratory Testing of Municipal Solid Waste*, ASCE, GSP 209, pp. 248, (ed. Zekkos, D.).
- PLAXIS 2004. *A Finite Element Code for Soil and Rock Analyses*. Brinkgreve, R. B. J., Broere, W. and Waterman, D. Delft University of Technology & PLAXIS b.v., Ver. 8.6, 2004, Plaxis b.v., Netherlands.
- Sahadewa, A., Zekkos, D., Lobbstaël, A., and Woods, R. D. 2011. Shear wave velocity of Municipal Solid Waste in Michigan Landfills. In *14th Pan-American Conference on Soil Mechanics and Geotechnical Engineering and 64th Canadian Geotechnical Conference*, Geo-Innovation Addressing Global Challenges, October 2–6, 2011, Toronto, Ontario, Canada (in cd-rom).
- Seisakis, G., and Roussos, G. 1994. *Geotechnical Investigation – Evaluation of geotechnical site conditions for the project: Design and construction of infrastructure for MSW landfill of the City of Patras – Stream cover* (Technical report to Koronis S.A.). Geotechniki, Laboratory of Soil Mechanics and Geotechnical Studies, December 2004, Athens.
- Spencer, E., 1967. A method of Analysis of the Stability of Embankments Assuming Parallel Inter-Slice Forces. *Geotechnique*, 17, 11–26.
- SUFALNET 2006. *Report on the Situation of Landfills in the Region of Western Greece*. Reported by Partner: TEDK of Achaia County.
- Zekkos, D., (ed.) 2011. *Geotechnical Characterization, Field Measurements, and Laboratory Testing of Municipal Solid Waste*. ASCE, GSP 209, 248pp.
- Zekkos, D., Athanasopoulos, G.A., Bray, J.D., Grizi, A., and Theodoratos, A., 2010. Large-scale direct shear testing of municipal solid waste. *Waste Management*, 30 (8–9), 1544–1555.
- Zekkos, D., Bray, J.D., Kavazanjian, E., Matasovic, N., Rathje, E.M., Riemer, M.F., and Stokoe, K.H. 2006. Unit Weight of Municipal Solid Waste. *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 132 (10), 1250–1261.
- Zekkos, D. 2005. *Evaluation of static and dynamic properties of municipal solid-waste*. Ph.D. thesis, Dept. of Civil and Environmental Engineering, Univ. of California, Berkeley, California.

Shear Strength and Deformation Modulus of Tailing Sands under High Pressures.

Résistance au cisaillement et module de déformation de sables de rejets sous hautes pressions.

Campaña J., Bard E.
ARCADIS Chile

Verdugo R.
CMGI Chile

ABSTRACT: This article discusses the results obtained for 4 different tailing sands from copper mines located in Chile and Peru, which were subjected to triaxial tests performed in drained and undrained conditions (CID and CIU), applying confining pressures up to 3 MPa. The evolution of the deformation moduli, static and cyclic shear strength under drained and undrained paths for a wide range of confining pressure, densities and fines content have been obtained. Also, the tests results have been interpreted, analyzed, and compared with those reported for natural sands in the international literature, highlighting the differences in the mechanical behavior resulting from the difference in genesis among these materials.

RÉSUMÉ : Le comportement mécanique de 4 sables de rejets différents, provenant de mines de cuivre situées au Chili et au Pérou, a été analysé à l'aide d'une série d'essais triaxiaux effectués en conditions drainées et non drainées (CID et CIU), en appliquant des pressions de confinement jusqu'à 3 MPa. Les essais, pratiqués sur des échantillons avec différentes densités initiales et contenus de particules fines, ont permis de suivre l'évolution du module de déformation et de la résistance au cisaillement statique et cyclique, drainée et non drainée, pour une large gamme de pression de confinement. Ces résultats ont été interprétés, analysés et comparés à ceux rapportés dans la littérature internationale pour des sables naturels, mettant en évidence les différences dans le comportement mécanique résultant de la genèse différente de ces matériaux.

KEYWORDS: tailing sands, high pressure, deformation modulus, copper tailing, triaxial, shear resistance.

1 INTRODUCTION.

Currently, both the important increase of production in the copper mining industry and the decrease of available space for the construction of tailing deposits have led to the design of dam walls having unprecedented heights (above 250 m). This situation requires that the geotechnical properties of the coarse sands used for the dam construction be determined adequately and rigorously. Also, the evolution of the deformation modulus and shear strength due to changes in the compaction density and in the fines content, among other characteristics, needs to be assessed to feed the numerical modeling of these structures, which is required to predict their behavior in the short and long term as well as when faced with extreme seismic events. In the technical literature, there is scarce information about the evolution of non-drained shear strength as well as about the cyclic shear stress and deformation modulus of tailing sands, particularly under high confining pressures.

Results of tests performed on 4 types of cycloned copper tailings sands, under high confining pressures, with a wide range of confining pressures, densities, and fines contents are presented. These tests include: grain size, index properties, compaction, static triaxial (drained and undrained), cyclic triaxial, small deformations and resonant column triaxial tests.

2 SANDS TESTED

The sands tested come from 4 copper mining processes (in operation and projects). The "Arenas Tórtolas" (S1) come from the Las Tórtolas Tailings Impoundment, from the Los Bronces Mine (Anglo American Chile); of the remaining samples (S2 to S4), two correspond to mines in Chile (2) and one in Peru (1). The main mineral that makes up the sands tested is quartz (45% to 60%), then sericite / sericite-muscovite (15% to 30%) and other minerals (Campaña, 2011). The index properties of the samples tested are summarized in Table 1. All the sands classify

as silty sands (SM), of low plasticity with a $PI < 7$ (Plastic Index).

Table 1. Index Properties of Tested Sands (Campaña, 2011)

USCS Classif.	% F	$e_{\max}$	$e_{\min}$	C_c	C_u	MP		SP	
						MDD (kN/m^3)	w_{opt} (%)	MDD (kN/m^3)	w_{opt} (%)
Sand S1 - Las Tórtolas ($G_s=2.73$)									
SM	15	1.100	0.460	1.4	3.3	16.78	14.0	15.89	18.0
SM	18	1.068	0.444	1.5	3.6	16.97	14.0	16.38	16.0
SM	21	1.084	0.414	1.0	4.0	17.36	12.5	16.38	16.0
Sand S2 ($G_s=2.67\text{-}2.75$. G_s average= 2.72)									
SM	15	1.310	0.647	1.2	1.7	16.68	10.0	15.30	14.0
SM(§)	15	1.068	0.517	-	-	17.07	12.8	-	-
SM(§)	18	0.992	0.537	-	-	16.88	13.8	-	-
SM	21	1.331	0.571	1.2	1.8	16.87	11.0	15.99	18.0
SM									
Sand S3 ($G_s=2.70$)									
SM	12	0.942	0.525	1.2	4.2	17.46	13.5	16.48	15.5
SM	18	0.956	0.484	3.1	12.9	18.25	11.0	16.77	14.5
Sand S4 ($G_s=2.69$)									
SM	24	0.915	0.406	2.7	165	18.84	8.0	17.36	8.5
SM	20	1.280	0.601	1.2	4.2	16.48	15.0	15.40	18.0
SM	30	1.280	0.592	3.1	12.9	17.85	11.5	16.87	14.0
% F: particles <0.074 mm				MDD: Maximum Dry Density					
Gs: Specific gravity				MP: Modified Proctor Test					
$e_{\max}$: Maximum void ratio				SP: Standar Proctor Test					
$e_{\min}$: Minimum void ratio				w_{opt} : Optimum water content					
Cu: Coefficient of uniformity				(§) Tested at low pressures, $\sigma_3' < 0.8\text{MPa}$					
Cc: Coefficient of curvature				USCS: Unified Soil Classif. System					

The preparation of the samples considered the following steps: 1) drying the complete sample; 2) sieving the entire sample; 3) separation of the fines (<0.074 mm) and coarse (>0.074 mm) fractions; 4) preparation of the sample with the fines content predefined by project. The probes were compacted in layers to an initial density representative of the expected in-situ compaction, with the tamping method and with moisture close to optimum ($\pm 2\%$, according to Standard Proctor).

3 UNDRAINED SHEAR STRENGTH (S_u)

Figure 1 presents the variation of S_u for the different sands as a function of the void ratio (e) and the fines content. It is important to note that the void ratio (e) indicated correspond at the end of consolidation phase.

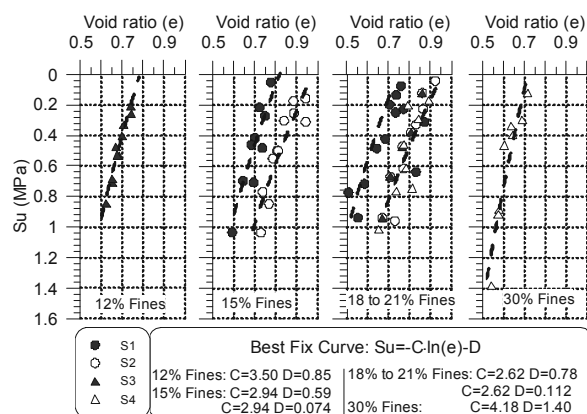


Figure 1. Summary of undrained shear strength (S_u) variation.

Figure 2a presents the variation of S_u as a function of the void ratio and Figure 2b presents the average tendency curves for all samples tested. A tendency of S_u to decrease as the fines content increases was observed. In addition, and as reference, in these figure 2b results obtained from the Toyoura Sand (fines=0%), tested by Verdugo (1992) in a wide range of relative density ($DR=7\%$ to 67%) have been included.

The variation of the undrained shear strength with the fines contents estimate a value of zero for S_u for average values of $e=0.72$ (30% and 18-21% of fines), $e=0.89$ (15% of fines) and $e=0.78$ (12% of fines). For the different sands tested, these void ratios are associated to relative densities lower than $DR<40\%$. In the Toyoura Sand, Verdugo (1992) recorded zero values of S_u for values of $DR<15\%$.

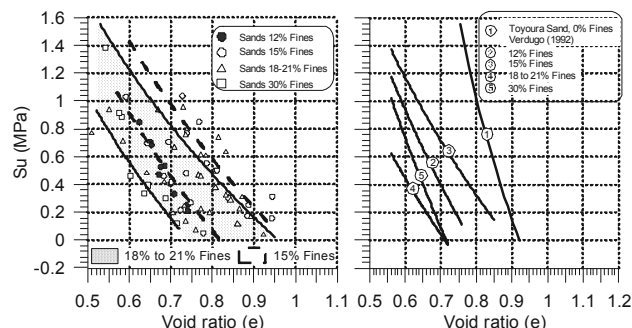


Figure 2. Variation of undrained shear strength (S_u) with the void ratio (e) and the fines content. Sand samples S1 to S4. (Campaña, 2011)

4 DRAINED SHEAR STRENGTH

The shear strength envelopes in the (q - p') plane for CID triaxial tests are presented in summary form on Figure 3. Values of residual strength obtained in the CIU tests (S_u) were also included. In spite of the wide range of fines content in the four samples tested, the shear strength envelopes obtained are similar. For $\sigma'_3 < 1$ MPa, it are characterized by internal friction

angles (ϕ) between $\phi=32^\circ$ and $\phi=36^\circ$; for $\sigma'_3 > 1$ MPa, the internal friction angles vary between $\phi=32^\circ$ to $\phi=34^\circ$. No strong reduction in the strength envelope at high pressures was observed, this behavior is in contrast with the one observed with the Sacramento Sand reported by Lee & Seed (1967). The little quantity of particles crushing that is produced in the tailings sands with the increase of confinement could be explained this unexpected behaviour. This evidence was observed systematically in the grain size tests performed before and after each triaxial test (Campaña et al., 2007).

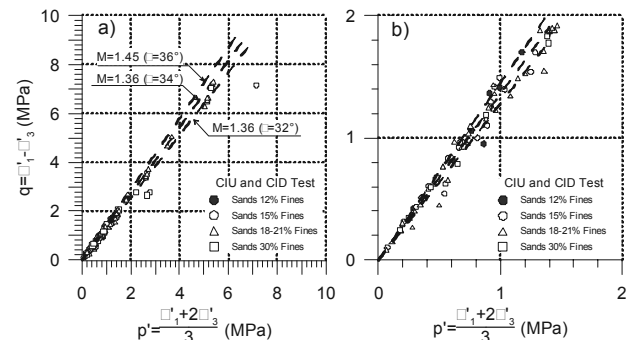


Figure 3. Drained and undrained residual strength curves.

a) all the tests, b) detail for $\sigma'_3 < 2$ MPa.

5 CRITICAL STATE OR PERFECT PLASTICITY

Although the ultimate state (steady state) has not clearly reach in all the tests, by plotting the final results in the graph (e - $\log p'$), it can be observed that the results allow to mark off with a reasonable precision the combination of density and confinement pressure that determine that the sands present a contracting or dilating behavior. By comparing the results of this study with the ones reported in the technical literature, the obtained behavior of tailing sands is in accordance with other natural sands of similar mineralogy. These results allow to extrapolate the behavior of tailings sands at higher confining pressures and estimate that these would not present an important decrease in the shear strength. Figure 4 presents all the tests, compared with several results of other sands.

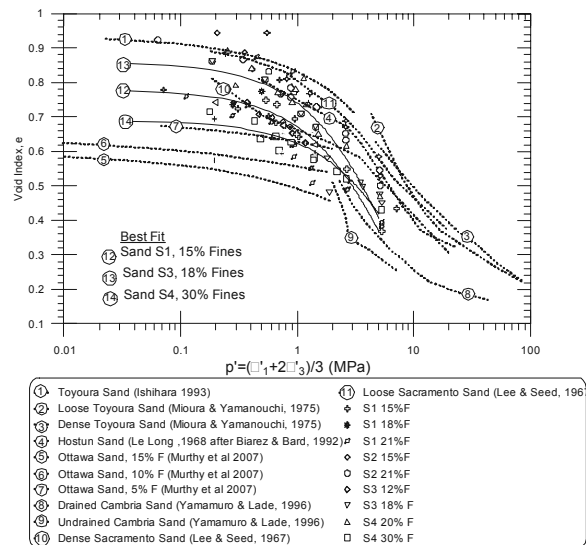


Figure 4. Results of triaxial tests in the e - $\log p'$ plane.

6 DEFORMATION MODULI, E_i ($\varepsilon=10^{-3}$)

To determine the variation of the initial tangent deformation moduli (E_i) at large deformations with the confining pressure, a hyperbolic adjustment (Duncan et. al, 1970) was performed, which was applied both for drained as well as for undrained triaxial tests. The variation of the deformation modulus (E_i) as

a function of the confining pressure (σ'_3) and the void ratio (e) may be defined by the following expression:

$$E_i = A \cdot \sigma'_3{}^n \cdot e^{-1} \quad (1)$$

where A , n are constants that depend on the type of soil (Biares et al., 1993; Bard, 1993).

Figures 5a and 5b present the values of E_i obtained through the hyperbolic adjustment of the stress-deformation curves, for the tests performed at different initial densities and different fines contents in drained and undrained conditions (large deformations).

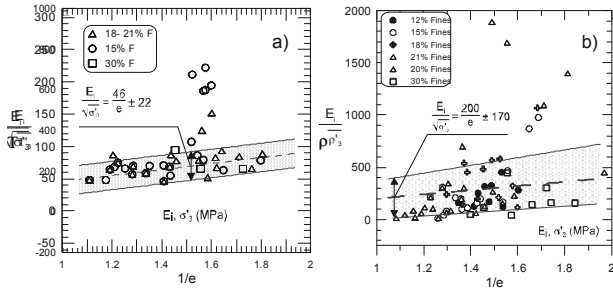


Figure 5. Normalized E_i variation. All the sands tested: a) drained case, b) undrained (Campaña, 2011).

In case of the drained tests, the resulting values of E_i are all practically contained in a narrow band, even when the range of fines content of the sample vary between 15% to 30%. In the case of the undrained tests, a significant dispersion was observed which may be explained by the variation of the pore pressure according the initial value of σ'_3 and density (e). The lower values of E_i correspond to 30% fines and the higher values correspond to fines between 15% and 21%.

7 SHEAR MODULUS G_{MAX} ($\gamma < 10^{-5}$)

According to Hardin & Drnevich (1972), the variation of the shear modulus at small deformations (G_{max}) as a function of the confining pressure (σ'_3) and the void ratio (e) may be expressed as indicated by equation (2), where C_0 and n are constants of the material. Plotting the results of the tests (triaxial at small deformations in Sands S1 and S2; resonant column for S3) in one graph to determine the values of C_0 and n of equation (2), it was observed that, for the range of pressures and densities tested, the tendency of expected variation for confining pressures $\sigma'_3 < 1$ MPa is reproduced, which does not occur for $\sigma'_3 > 1$ MPa (Figure 6).

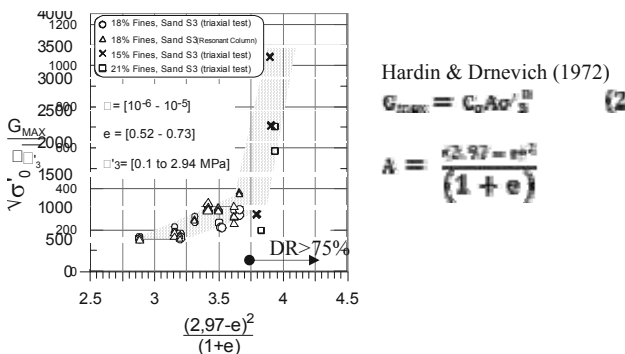


Figure 6. Results of resonating column and triaxial at small deformations tests (Campaña, 2011).

8 CYCLIC SHEAR RESISTANCE (CSR)

The cyclic triaxial tests were carried out on reconstituted samples with a nominal diameter of 5 cm (S1, S2 and S4) and 7 cm (S3) and nominal heights of 10 cm and 15 cm, respectively. In all cases, parameter B at the end of the saturation stage was greater than 0.95. Backpressure was

applied according to the magnitude of isotropic confining pressure: 0.3 MPa for Sands S1, S2, and S4 and up to 0.9 MPa for Sand S3. A uniform sinusoidal cyclic load was applied with a frequency of 0.1 Hz for Sand S4 and of 0.05 Hz for Sands S1, S2, and S3. Test results are summarized in several graphs, differentiated by the fines content, confining pressure, and density achieved at the end of the consolidation stage (see Campaña 2011). The results of this tests confirm the tendency reported by Verdugo (1983) for lower confining pressures ($\sigma'_3 \leq 0.5$ MPa), related to the decrease of the required CSR to produce liquefaction when the non-plastic fines content increases. However, this tendency disappears for confining pressures greater than 0.5 MPa where the CSR practically is a constant. To strengthen this important detection, all the results obtained have been plotted separately on Figure 7a for $\sigma'_3 \leq 0.5$ MPa, and on Figure 7b, for $\sigma'_3 \geq 1$ MPa. On Figure 6a, a great scattering in the CSR values can be observed for the four sands tested, while a general decreasing trend of CSR values with the increasing number of cycles can be perceived. The results corresponding to Sand S3 contribute significantly to the width of the resulting band. Contrary, on Figure 6b for $\sigma'_3 \geq 1$ MPa, the CSR values obtained range in a very narrow band, without important differences due to initial density or fines content or the cycloned tailings sands.

Another relevant aspect to point out from the results of the tests carried out in Sands S2 and S4 for confining pressures greater than 0.5 MPa, are the relatively constant CSR values obtained despite the confining pressure applied (factor $K_\sigma = 1$), the fines content, the relative density, and even the initial shear stress. Figure 8 presents a comparison of K_σ values obtained in this paper with those obtained by several authors.

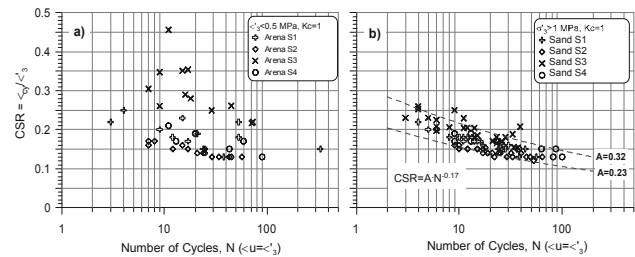


Figure 7. Summary all test results: a) for $\sigma'_3 \leq 0.5$ MPa, b) for $\sigma'_3 \geq 1$ MPa, $K_\sigma = 1.0$ (Campaña, 2011)

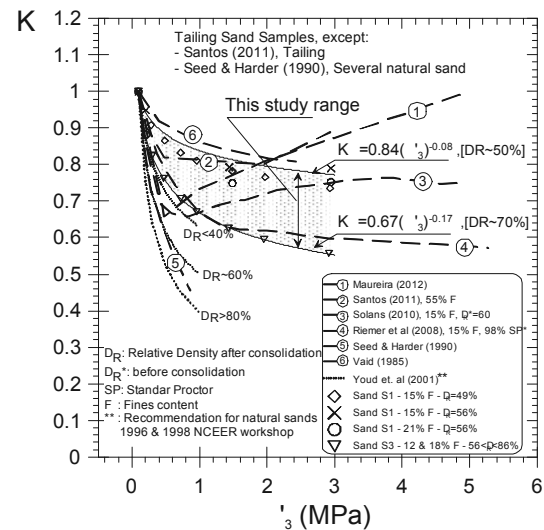


Figure 8. Variation of factor K_σ with regard to σ'_3

9 CONCLUSIONS

The interpretation and analysis of the results of the testing program show that, in the domain of confining pressures applied

and the relative densities tested, the shear strength envelopes obtained are very similar to each other. For average confining pressures (p') above 0.5 MPa, a little reduction in the envelope strength at high pressures, associated to a very reduced crushing of particles is observed. For this reason, the shear strength parameters are not modified significantly even for higher confining pressures.

In the same manner, the drained as well as the undrained shear strength envelope is not sensitive to the different fines contents tested, with no significant differences being detected for the range of fines tested, which varied between 12% and 30%. Notwithstanding the above, by considering the average values obtained for the undrained shear strength S_u , it is possible to observe a decreasing tendency of the S_u as the fines content is increased.

With the hyperbolic adjustment of the stress – deformation curves, the evolution of the initial tangent deformation modules (E_i) was determined. In drained tests, even when the range of fines content tested varied between 12% and 30%, a narrow band of normalized values of E_i with the confining pressure was obtained. On the other hand, for the undrained case, a larger dispersion of results was observed.

The values of G_{max} deduced from the triaxial tests at small deformations and resonant column tests, normalized with the confining pressure, show that for $\sigma'_3 < 0.10$ MPa, the results follow a similar tendency observed by Hardin & Drnevich (1972) in sands of natural origin. This tendency is not observed for $\sigma'_3 > 1$ MPa, where the values of G_{max} are larger than expected in the range of densities tested ($80 < DR(\%) < 90$).

For the fines content tested and confining pressures greater than 0.1 MPa, the cyclic stress ratio (CSR) are practically insensitive to the fines content. This result differs from what was observed in tests performed on tailings sands at low pressures, in which this content significantly affects the required CSR value (Verdugo, 1983). The effect of the high pressures upon the CSR value was practically null in three of the four cycloned sands tested (S1, S2 and S4), and a relative common value was obtained for all the range of confining pressures applied. In the S3 cycloned sand sample, the CSR value to induce liquefaction decreases as the confining pressure increases until 1.5 MPa. For greater confining pressures, the same CSR obtained with the other three samples is reached. Due to these results, for cycloned tailings sands, the consideration of typical factors K_σ reported in technical literature may lead to an underestimation of the CSR value.

The difference observed in the behavior of cycloned tailings sands with respect to natural sands could be explained due to the following aspects: the fines of tailing sands have a low plasticity index and correspond mainly to a “rock flour” composed by angular and hard particles (the mineralogical analysis indicated that more than 40% of sand tailings tested is quartz). Also, a very small amount of particle crushing was detected at the highest pressures tested, with fines increasing less than 3% in the grain size distributions post tests with respect to the initial grain size distribution (Campaña, 2007). According to this detection, it is possible to anticipate that tailing sands could have shear resistance (drained and cyclic) and shear moduli values greater than expected (compared with natural sand behaviour). A lesser detriment of the drained shear resistance due to the effects of both high confining pressures and fines content was observed.

10 ACKNOWLEDGEMENTS

The authors would like to express their gratitude to Anglo American Chile for the authorization to publish the results of the tests of the Las Tórtolas Sand (S1), ARCADIS Chile for the permission to access their data base, and to the Ing. Luis Valenzuela (MSc) for the review of this paper.

11 REFERENCES

- Bard, E. (1993): “Comportement des Matériaux Granulaires secs et a Liant Hydrocarboné”. These présentée pour l’obtention du Titre de Docteur. Ecole Centrale de Paris.
- Biarez, J., & Bard, E. (1993). “Behaviour of Granular Materials: The Moduli”, 3^{er} Congreso Chileno de Ing. Geotécnica, La Serena.
- Campaña, J. (2011). “Comportamiento de Arenas de Relave a Altas Presiones”. Tesis Magister en Cs. de la Ing., mención Ing. Geotécnica, U. de Chile.
- Campaña, J. and Bard, E. (2007). “Resistencia al Corte de Arenas de Relaves a Altas Presiones”. VI Congreso Chileno de Geotecnia.
- Campaña, J. & Bard, E. (2011). “Cyclic Behavior of Tailings Sands Under High Pressures”. 5^o International Conference on Earthquake Geotechnical Engineering, Santiago-Chile.
- Duncan, J.M. and Chang, C.Y. (1970). “Nonlinear Analysis of Stress and Strain in Soils”. Journal of the Soil Mechanics and Foundations Division, ASCE, 96 (SM5), 1629-1653.
- Hardin, B.O., and Drnevich, V.P. (1972). “Shear Modulus and Damping in Soils: Design Equations and Curves”. JSMFD, ASCE vol.98, SM7, pp-667-692.
- Ishihara, K., 1993. “Liquefaction and Flow Failure During Earthquakes”, Rankine Lecture. Geotechnique 43, N°3, 351-415.
- Lee, K., Seed, B., 1967. “Drained Strength Characteristics of Sands”. Journal of the Soil Mechanics and Foundation Division ASCE, SM6, pp. 117-141.
- Lee, K., Seed, B., 1967. “Undrained Strength Characteristics of Cohesionless Soils”. Journal of the Soil Mechanics and Foundation Division ASCE, SM6, pp. 117-141.
- Maureira, S., Verdugo, R., (2012). “Respuesta Cíclica de Arenas de Relaves Sometidas a Altas Presiones”. VII Congreso Chileno de Geotecnia, Concepción-Chile.
- Miura N., Yamanouchi, T., 1975. “Effect of Water on the Behavior of Quartz-Rich Sand Under High Stresses”. Soils and Foundation, Japanese Society of Soil Mechanics and Foundation Engineering, Vol 15, N°4, pag. 23-34.
- Murthy, T.G., Loukidis, D., Carraro, J.A.H., Prezzi, M. & Salgado, R., 2007. “Undrained Monotonic Response of Clean and Silty Sands”. Geotechnique 57, N°3, pag. 273-288.
- Riemer, M.F., Y. Moriwaki and J. Obermeyer (2008): “Effect of High Confining Stresses on Static and Cyclic Strengths of Mine Tailing Materials”. Geotechnical Earthquake Engineering and Soil Dynamics IV, Sacramento CA, May. 19-22.
- Solans, D., (2010). “Equipo Triaxial Monótono y Cíclico de Altas Presiones y su Aplicación en Arenas de Relaves”. Tesis Magister en Cs. de la Ing., mención Ing. Geotécnica, U. de Chile.
- Santos, E., (2011). Comportamiento Monótono y Cíclico No-Drenado de Arenas de Relaves Integrales. Tesis Magister en Cs. de la Ing., mención Ing. Geotécnica, U. de Chile.
- Vaid, Y.P. & Chern, J.C. (1985): “Cyclic and Monotonic Undrained Response of Saturated Sands”. Advances in the Art of Testing Soils under Cyclic Conditions, ASCE, New York, 120-147.
- Verdugo, R. (1983). “Influencia del Porcentaje de Finos en la Resistencia Cíclica de Arenas de Relaves”. Memoria para optar al título de Ingeniero Civil. Pontificia Universidad Católica de Chile. Departamento de Ingeniería Estructural.
- Verdugo, R. (1992). “Characterization of Sandy Soil Behavior Under Large Deformation”. Thesis presented in partial fulfillment of the requirements for the degree of Doctor of Engineering. University of Tokyo. Department of Civil Engineering.
- Verdugo, R. & Ishihara, K. (1996). “The Steady State of Sandy Soils”, Soils and Foundations, 36(2), 81-91.
- Verdugo, R. & Santos, E. (2009): “Liquefaction Resistance of Thickened Tailings of Cooper Mines”. Proceedings 17th Intern. Conf. on Soil Mechanics and Geotechnical Engineering, 295-298.
- Youd, T.L., et al (2011). Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils. Journal of Geotechnical and Geoenvironmental Engineering.
- Yamamuro, J., Lade, P., 1996. Drained Sand Behavior in Axisymmetric Tests at High Pressures. Journal of Geotechnical Engineering, February 1996, pag. 109-119.
- Yamamuro, J., Lade, P., 1996. Undrained Sand Behavior in Axisymmetric Tests at High Pressures. Journal of Geotechnical Engineering, February 1996, pag. 120-129.

A Comparison Between the Shear Strength Measured with Direct Shear and Triaxial Devices on Undisturbed and Remolded Soils

Une comparaison entre la résistance au cisaillement mesurée avec appareils de cisaillement direct et triaxiaux sur les sols non remaniés et remoulés

Castellanos B.A., Brandon T.L.
Virginia Polytechnic Institute and State University

ABSTRACT: A comparison is presented between the shear strength measured with direct shear and triaxial devices based on the results from tests conducted on undisturbed and remolded soils. A series of consolidated-undrained (CU) triaxial tests and consolidated-drained (CD) direct shear tests were performed on undisturbed samples from the New Orleans area. These tests were conducted on soils ranging from low plasticity silts to organic fat clays. The results from the undisturbed samples showed that the drained friction angles obtained from the CU triaxial tests were considerably higher than those obtained from the CD direct shear tests. On the other hand, results obtained from remolded test specimens originating from a variety of locations showed almost no difference in the effective stress shear strength. The difference in the results can be explained in part by the fact that natural soils can have a preferred particle orientation or anisotropic fabric based on the deposition or formation of the soil. In the case of the New Orleans soils, the undisturbed samples exhibited evidence of horizontal deposition. In remolded samples, the soil is more homogeneous and isotropic, so every plane in the soil may be expected to have similar shear strength.

RÉSUMÉ : En utilisant des résultats de tests effectués sur des sols non perturbés et remoulés, une comparaison entre la résistance au cisaillement mesurée par cisaillement direct et avec des appareils triaxiaux est présentée. Une série d'essais triaxiaux consolidés non drainés et d'essais de cisaillement direct consolidés drainés ont été effectués sur des sols non remaniés de la région de la Nouvelle-Orléans. Ces tests ont été effectués sur des sols allant de vases de plasticité faible à des argiles organiques de haute plasticité. Les résultats des échantillons non perturbés ont démontré que les angles de friction drainés obtenus à partir d'essais triaxiaux consolidés non drainés sont considérablement plus élevés que ceux obtenus à partir d'essais de cisaillement direct. D'autre part, les résultats obtenus à partir de spécimens remoulés originaires de lieux divers n'ont montré pratiquement aucune différence dans la résistance effective au cisaillement. La différence des résultats peut s'expliquer en partie par le fait que les sols naturels peuvent avoir une orientation préférée des particules ou un tissu anisotrope basé sur le dépôt ou la formation du sol. Dans le cas des sols de la Nouvelle-Orléans, les échantillons intacts ont présenté des signes de déposition horizontale. Dans les échantillons remoulés le sol est beaucoup plus homogène et isotrope, de sorte que l'on peut s'attendre à ce que chaque plan du sol ait la même résistance au cisaillement.

KEYWORDS: direct shear, triaxial, shear strength, fully softened, remolded, undisturbed, shear strength measurement.

1 INTRODUCTION

The triaxial and direct shear devices have been historically used successfully to measure the peak shear strength of soils. In geotechnical projects, these tests are often used interchangeably to determine effective stress or drained shear strength parameters without regard to the potential difference in the results. A series of consolidated-undrained (CU) triaxial tests and consolidated-drained (CD) direct shear tests were performed on undisturbed samples from the New Orleans area. These tests were conducted on soils ranging from low plasticity silts to organic fat clays. The bulk of these tests were conducted as part of the reconstruction of the flood protection system, which was severely damaged by Hurricane Katrina in 2005.

The city of New Orleans is located on alluvial soils that are part of the delta formed by the sediments of the Mississippi River. According to Dunbar and Britsch (2008), the surficial soils were formed during the Holocene and consists of fine grained soils extending to about 17 m to 25 m deep in most of the area, while exceeding 46 m deep at some locations. Dunbar and Britsch (2008) stated that this layer is characterized by stacked and generally horizontal layering created by the deposition mechanism.

Numerous CU triaxial compression tests and CD direct shear tests were performed on undisturbed test specimens trimmed from 125 mm diameter tube samples to characterize the effective stress shear strength parameters of the soils in the greater New Orleans area. These test results were assessed to

allow a comparison of the effective stress shear strength parameters for the undisturbed samples based on soil type, plasticity characteristics, etc. The tests conducted on the undisturbed tube samples were assessed based on the peak shear strength.

A series of direct shear tests and triaxial tests were also conducted on remolded soil specimens to determine the *fully softened* shear strength. The fully softened shear strength was defined by Skempton (1970) as the drained peak shear strength of a clay in its normally consolidated state. According to Skempton (1977), the fully softened shear strength can be measured on remolded normally consolidated specimens. Three different soils from various locations in the USA were tested to examine the difference in the fully softened strength parameters obtained from triaxial and direct shear test apparatuses.

2 PREVIOUS WORK

The direct shear and triaxial devices have been used for over 70 years to determine drained shear strength parameters of soils (Saada and Townsend 1981). These two devices have marked differences in the stress condition that is developed in the test specimen. Some of the biggest differences are: 1) strain boundary conditions, 2) failure plane orientations, and 3) principal stress orientation. In the triaxial device, the intermediate and minor principal stresses are equal and are normally specified at the beginning of the test. In the direct

shear device, the magnitudes of the intermediate and minor principal stresses are not known and are governed by the vertical stress applied and the strength properties of the soil being tested.

Progressive failure is the condition where the peak shear strength is not mobilized in every point of the failure plane at the same instance. This is caused by the non-uniform distribution of strain in the failure plane combined with the strain-softening characteristic of the soil. Some locations on the failure plane will mobilize the peak shear strength while others, having achieved more or less displacement, will mobilize a shear strength below the peak shear strength. For the direct shear device, investigators have reached different conclusions about the effect of stress concentrations in this device. Hvorslev (1960) measured horizontal displacements along the failure plane in a direct shear device and found that the displacements are not uniformly distributed, thereby causing progressive failure to occur. Alternatively, a finite element study of the direct shear box presented by Potts et al. (1987) showed that although stress concentration exist on the failure plane of the direct shear box, at the moment of failure the stresses on the failure plane are more or less uniform and the peak shear strength measured is not affected by progressive failure. For the triaxial device, stress concentration caused by the end restraints can influence the results. Research performed by Taylor (1941) showed that if the ratio of length to diameter is between 1.5 and 2.5, the effect of the stress concentration is negligible.

In the direct shear device, the orientation of the principal stresses on the failure plane varies during the shearing stage of the test and the final orientation is unknown. In the triaxial test, the major and minor principal stresses act on the horizontal and vertical planes and this orientation does not change during shear.

The orientation of the failure plane in the direct shear device is predetermined as being near the midpoint between the upper and lower halves of the shear box. In the triaxial device, the orientation of the failure plane is governed by the soil structure and the strength properties of the soil.

A literature review was undertaken to locate previous comparisons of the shear strength obtained using the triaxial and direct shear apparatuses. Skempton (1964) stated that the same effective stress shear strength parameters were obtained from tests conducted on eight specimens of Boulder Clay using the direct shear and triaxial devices. Casagrande and Poulos (1964) presented the results of CD direct shear and CD triaxial tests performed on compacted specimens of a lean clay which showed that about the same shear strength envelope was obtained with the two tests. Moon (1984) performed CD direct shear and CU triaxial tests on undisturbed samples of a fat clay and found differences of less than one degree for the effective stress friction angle and less than 8 kPa for the effective cohesion intercept obtained from these two tests using the maximum principal stress ratio as the failure criterion. Thomson and Kjartansson (1985) performed CD direct shear tests and CD and CU triaxial tests on undisturbed samples of a lean clay and a fat clay and found that the results plotted on the same failure envelope. Abdel-Ghaffar (1990) compiled results from the literature where direct shear and triaxial tests were performed on undisturbed samples of the same soil. He concluded that the direct shear and triaxial devices provide comparable values for the effective stress friction angle and cohesion intercept. Maccarini (1993) performed CD direct shear and CD triaxial tests on a residual soil from Rio de Janeiro. For these tests, the tests specimens were oriented so that the failure plane in both devices coincided with the direction of stratification of the soil. The stratification of the soil had a dip angle of 25°. Based on the results obtained, Maccarini concluded that similar values of effective stress cohesion and friction angle are obtained from both tests if the stratification is taken into account.

Based on the results presented above, it can be seen that the available information in the literature is divided on whether

these two devices will provide the same effective stress shear strength parameters. Investigators that have found agreement in the results did not state whether the soil tested had a preferred particle orientation or layering that could influence the results.

3 DATA COLLECTION AND RESULTS

3.1 Undisturbed test specimens

A subset of 63 CU triaxial test series and 146 CD direct shear test series was selected from the test results available from the New Orleans investigation. Each test series normally consisted of three individual tests conducted at different confining pressures. Only high quality tests were selected. The test results selected had to comply with the following requirements: 1) At least two test specimens were consolidated to stresses that were higher than the preconsolidation stress; 2) The end of primary consolidation was achieved during the consolidation stage of the test; 3) Index properties were available; 4) A peak deviator stress was reached for CU triaxial tests in less than 15% strain and a peak shear stress was reached for CD direct shear tests at a horizontal displacement less than 0.4 inches.

Since all the samples used were normally consolidated, the shear strength interpretation assumed that the effective stress cohesion intercept was zero. The least-squares method was used to obtain the corresponding effective stress friction angle.

Shown in Figure 1 are the values of the effective stress friction angles as a function of the Plasticity Index (PI) for the undisturbed New Orleans area test specimens for CD direct shear and CU triaxial tests. The trend lines shown on this figure are based on a statistical analysis performed by the authors. These lines are presented to better show the trend of the data and are not intended to be used as a correlation to predict friction angles. From this figure, it can be seen that the effective stress friction angle measured with the CU triaxial device is generally higher than that measured with the direct shear device. This difference was found to increase with increasing plasticity index of the soil. In general, the difference in friction angle ranged from about 2 degrees to 5 degrees.

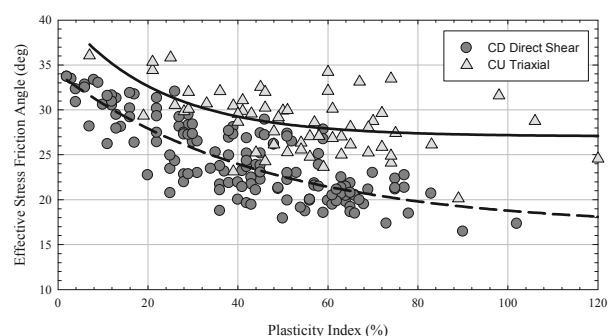


Figure 1. Relationship between effective stress friction angle and plasticity index for CD direct shear tests and CU triaxial tests on undisturbed samples.

3.2 Remolded test specimens

Three remolded clays were tested to allow a comparison of the fully softened shear strength measured in CD direct shear tests and CU triaxial tests. The index properties of these clays are shown in Table 1. NOVA clay was obtained from a site in Northern Virginia in Fairfax County. Vicksburg Buckshot Clay (VBC) was obtained from the stockpile maintained at the Engineering Research and Development Center of the U.S. Army Corps of Engineers. It has been the subject of many research projects (Ladd and Preston 1965; Mitchell et al. 1965; Al-Hussaini and Townsend 1974). Colorado Clay is a lean clay (CL) from Silverthorne, Colorado.

To prepare the remolded samples, the soils were first soaked in water for at least 48 hours, and then processed through a No.

40 (425 μm) sieve. At this point, the samples were at a water content much greater than the liquid limit and were therefore air-dried to reduce the water content. The soil was considered to be at the desired water content when 23 to 25 blows using a Casagrande liquid limit cup were required to close the groove cut in the soil as specified in ASTM D4318-10.

Table 1. Soil index properties.

Soil	Specific Gravity	LL	PL	PI	Clay-sized Fraction
Colorado Clay	2.78	42	22	20	23.7
NOVA Clay	2.80	66	28	38	16.8
VBC	2.79	78	26	52	68.9

To perform CD direct shear tests, the test specimens were formed directly in the direct shear box using a spatula or “piped” from a pastry bag. Care was taken to form the test specimens without entrapping air-bubbles. Prior to shearing, the samples were consolidated under K_0 conditions to vertical pressures ranging from 24 kPa to 288 kPa. The samples were then sheared slow enough to allow full dissipation of the pore pressures generated during shear using the criterion presented in ASTM D3080-11.

Triaxial test specimens were more difficult to form for tests at low effective consolidation stresses. At a water content equal to the liquid limit, the soil does not have the shear strength required to be formed into triaxial test specimens that can be installed in a triaxial cell. For this reason, a batch of soil was first consolidated in a batch consolidometer under K_0 conditions to a vertical pressure of 38 kPa. The batch consolidometer used in this investigation is shown in Figure 2. After the batch of soil reached the end of primary consolidation, the sample was extruded from the specimen container, and triaxial test specimens were trimmed. The batch consolidometer formed samples that had a diameter of 15 cm, which allowed five to six 3.6 cm diameter test specimens to be trimmed. The triaxial test specimens were then installed in a triaxial cell, back pressure saturated, and consolidated to an all-around pressure ranging from 48 kPa to 483 kPa prior to shearing. The samples were sheared slow enough to allow equilibration of the pore pressures according to the criterion presented by Head (1986).

The results from the triaxial and direct shear tests performed on remolded samples are presented in Figure 3. The envelopes were clearly non-linear, and a power function was fit to the test results. The power function used was consistent with the format described by Lade (2010) (see Eq. 1).

$$\tau = a \cdot p_a \left(\frac{\sigma'}{p_a} \right)^b \quad (1)$$

with:

- σ' = normal effective stress on the failure plane
- p_a = atmospheric pressure in the same units as σ' .
- a and b = curve fit parameters.

The results show that no significant differences were obtained in the failure envelope determined with the direct shear device and triaxial device for these remolded samples.

4 DISCUSSION

The difference in the results of the triaxial and direct shear tests on undisturbed and remolded samples can be explained, in part, by the difference in soil structure. The undisturbed tests specimens obtained from the greater New Orleans area were lacustrine and riverine alluvial deposits. These soils were deposited in horizontal layers, and could be expected to have a

preferential structure. Upon visual inspection, the New Orleans soils appear to be relatively homogenous. Distinct layering is often not discernible. Shown on the left side of Figure 5 is a section cut through an undisturbed New Orleans sample. The right side of Figure 5 shows the same sample after drying. The horizontal layers, which can represent planes of weakness, become evident after drying.

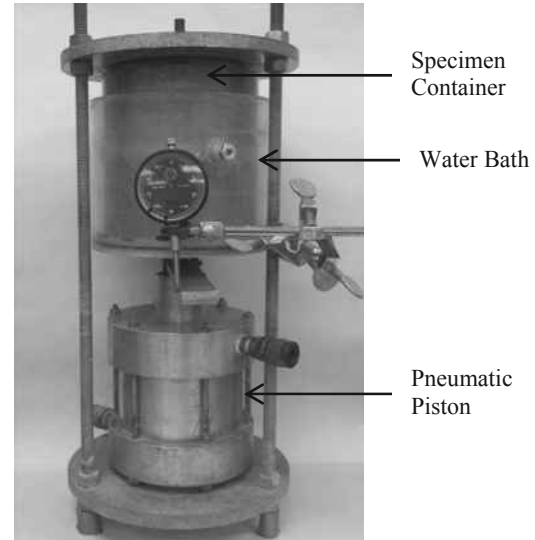


Figure 2. Batch consolidometer.

This horizontal layering can influence the shear strength measured using the direct shear device. In the direct shear device, the failure plane is horizontal. This forced failure plane can sometimes coincide with the natural layering or planes of weakness. In the triaxial device, the failure plane is not predetermined by the configuration of the device and horizontal planes of weakness would not be expected to control the measured shear strength. The difference in shear strength as a function of failure plane orientation has been documented by many investigators (Duncan and Seed 1966a; b; Filz et al. 1992).

Remolding soil destroys any previous layering or structure that might have been present, and the resulting sample is more or less homogenous. The influence of the failure plane orientation should be much less prevalent as compared to undisturbed samples. This suggests that fully softened shear strength (i.e. remolded normally consolidated peak shear strength) might be less dependent on the choice of the test apparatus than undisturbed shear strength.

5 CONCLUSIONS

The choice of using direct shear tests or triaxial tests to determine drained shear strength parameters can be important for natural soil deposits. Although these two different test methods can often provide similar results in some soil deposits, there is considerable evidence that direct shear tests provide much lower friction angles in riverine and lacustrine alluvial deposits than triaxial tests. A comparison of numerous CD direct shear and CU triaxial test results conducted on alluvial soils from the greater New Orleans area show that the friction angle determined from the direct shear apparatus is normally about 2 to 5 degrees lower than that determined using the triaxial apparatus. This can be attributed to the anisotropic shear strength characteristics of the alluvial soils.

The difference in the results of the two test devices is much less when remolded test specimens are used. The remolding process destroys the anisotropic fabric, and the shear strength parameters are not as dependent on the orientation of the failure plane. When fully softened shear strength parameters are desired, direct shear and triaxial test apparatuses appear to provide comparable results.

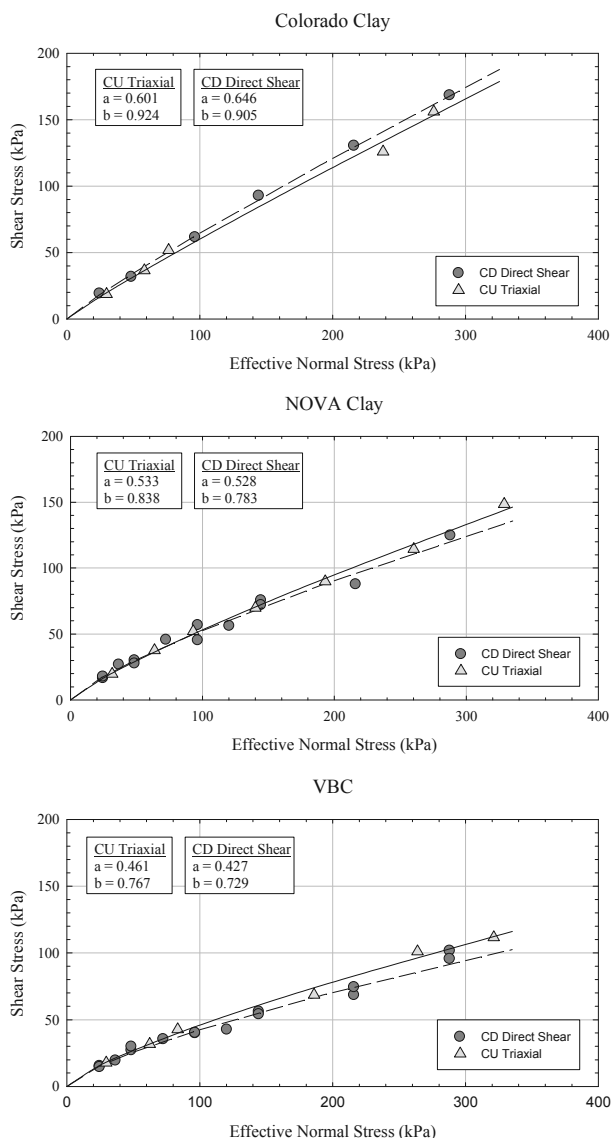


Figure 3. CD direct shear and CU triaxial test results on remolded test specimens.

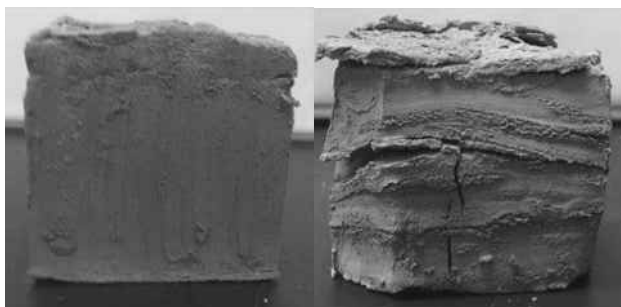


Figure 4. Undisturbed soil specimen before and after drying.

6 REFERENCES

- Abdel-Ghaffar, M. E. M. (1990). "The Meaning and Practical Significance of the Cohesion Intercept in Soil Mechanics." Thesis presented to University of Illinois, Urbana-Champaign, IL in partial fulfillment of the requirements for the degree of Doctor of Philosophy, 262.
- Al-Hussaini, M. M., and Townsend, F. C. (1974). *Investigation of Tensile Testing of Compacted Soils*. Vicksburg, MS, 76.
- ASTM Standard D3080. (2011). "Standard test method for direct shear test of soil under consolidated drained conditions." ASTM International, West Conshohocken, PA. 2011. DOI: 10.1520/D3080_D3080M-11. www.astm.org.
- ASTM Standard D4318. (2010). "Standard test method for liquid limit, plastic limit, and plasticity index of soils." ASTM International, West Conshohocken, PA. 2010. 1 10.1520/D4318-10. www.astm.org.
- Casagrande, A., and Poulos, S. J. (1964). "Fourth report on investigation of stress-deformation and strength characteristics of compacted clays." *Harvard Soil Mechanics Series No. 74*.
- Dunbar, J. B., and Britsch III, L. D. (2008). "Geology of the New Orleans area and the canal levee failures." *Journal of Geotechnical and Geoenvironmental Engineering*, 134(5), 566–582.
- Duncan, J. M., and Seed, H. B. (1966a). "Anisotropy and stress reorientation in clay." *Journal of the Soil Mechanics and Foundations Division*, 92(5), 21–50.
- Duncan, J. M., and Seed, H. B. (1966b). "Strength variation along failure surfaces in clay." *Journal of the Soil Mechanics and Foundations Division*, 92(6), 81–104.
- Filz, G. M., Brandon, T. L., and Duncan, J. M. (1992). "Back analysis of Olmsted Landslide Using Anisotropic Strengths." *Transportation Research Record 1343*, Transportation Research Board, National Research Council, National Academy Press, Washington, DC, 72–78.
- Head, K. H. (1986). *Manual of Soil Laboratory Testing. Vol. 3: Effective Stress Tests*. John Wiley & Sons, 1238.
- Hvorslev, M. J. (1960). "Physical components of the shear strength of saturated clays." *Research Conference on Shear Strength of Cohesive Soils*, Boulder, Colorado, 169–273.
- Ladd, C. C., and Preston, W. B. (1965). *On the Secondary Compression of Saturated Clays*. Vicksburg, MS, 116.
- Lade, P. V. (2010). "The mechanics of surficial failure in soil slopes." *Engineering Geology*, 114(1-2), 57–64.
- Maccarini, M. (1993). "A comparison of direct shear box tests with triaxial compression tests for a residual soil." *Geotechnical and Geological Engineering*, 11(2), 69–80.
- Mitchell, J. K., Shen, C., and Monismith, C. L. (1965). *Behavior of Stabilized Soils Under Repeated Loading. Report I. Background, Equipment, Preliminary Investigations, Repeated Compression and Flexure Tests on Cement-Treated Silty Clay*. Vicksburg, MS, 136.
- Moon, A. T. (1984). "Effective shear strength parameters for stiff fissured clays." *4th Australia-New Zealand Conference on Geomechanics*, 107–111.
- Potts, D. M., Dounias, G. T., and Vaughan, P. R. (1987). "Finite element analysis of the direct shear box test." *Géotechnique*, 37(1), 11–23.
- Saada, A. S., and Townsend, F. C. (1981). "State of the art: laboratory strength testing of soils." *Laboratory shear strength of soil. ASTM STP 740*, R. N. Yong and F. C. Townsend, eds., American Society for Testing and Materials, 7–77.
- Skempton, A. W. (1964). "Long-term stability of clay slopes." *Géotechnique*, 14(2), 77–102.
- Skempton, A. W. (1970). "First-time slides in over-consolidated clays." *Géotechnique*, 20(3), 320–324.
- Skempton, A. W. (1977). "Slope stability of cuttings in Brown London Clay." *Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering*, 3, 261–270.
- Taylor, D. W. (1941). *Cylindrical Compression Research Program on Stress-deformation and Strength Characteristics of Soils*. Cambridge, MA, 90.
- Thomson, S., and Kjartanson, B. H. (1985). "Study of delayed failure in a cut slope in stiff clay." *Canadian Geotechnical Journal*, 22(2), 286–297.

Experimental analysis of the mechanical properties of artificially cemented soils and their evolution in time

Analyse expérimentale des propriétés mécaniques des sols cimentés artificiellement et leur évolution dans les temps

Consoli N.C., Fonini A., Maghous S., Schnaid F.
Engineering School, Federal University of Rio Grande do Sul, Brazil

Viana da Fonseca A.
Faculty of Engineering, University of Porto, Portugal

ABSTRACT: In view of the new challenges in the field of foundation design induced by the necessity to resorting to reinforced soils, the evaluation of behavior parameters of soils and soil-binder mixes by means of laboratory tests has become an important task. In this context, experimental characterization of soil-cement mixtures in terms of elastic stiffness and its evolution in time has been carried out. Both shear and oedometric moduli at infinitesimal strains have been measured through the technique of wave propagation along cylindrical specimens of 70 mm and 140 mm in diameter. The analyzed specimens were molded with quartzitic sand and fast curing cement using four relative densities of sand and five cement contents. Several loading times (i.e., times when the specimens are subjected to wave propagation), ranging between the first and seventh curing day, were considered. The results have given evidence of a significant increase in the magnitude of the stiffness properties as the curing process continues. When compared to uncemented sand specimens with similar porosities, increases of about 360% to 6300% due to cement insertion are observed for the shear modulus, and of 900% to 6700% for the oedometric modulus. In addition, the analysis showed that the higher is the amount of added cement used in the mixture, the higher the ratio of initial modulus to final one.

RÉSUMÉ : Compte tenu des nouveaux défis induits, dans le domaine du dimensionnement de fondations, par la nécessité de recourir à des sols renforcés, l'évaluation des paramètres définissent le comportement des sols ainsi que des mélanges sol-liant au moyen de tests de laboratoire est devenue une tâche importante. Dans ce contexte, la caractérisation expérimentale des sols cimentés en termes de rigidité élastique et son évolution dans le temps a été réalisée. Les deux modules de cisaillement et oedométrique sous déformations infinitésimales ont été mesurées par la technique de propagation d'ondes le long d'échantillons cylindriques de 70 mm et 140 mm de diamètre. Les échantillons analysés ont été moulés utilisant du sable et du ciment à prise rapide en considérant quatre densités relatives de sable et cinq pourcentages de teneur en ciment. Plusieurs temps de chargement (c'est à dire, les instants où les échantillons sont soumis à la propagation d'ondes), compris entre le premier jour et le septième jours en termes de temps de prise, ont été considérés. Les résultats ont montré une augmentation significative de la rigidité au fur et à mesure que le processus de prise continue. Selon la teneur en ciment, des augmentations allant de 360% à 6300% sont observées pour le module de cisaillement, et de 900% à 6700% pour le module oedométrique. En outre, l'analyse a montré que le rapport entre module initial et le module final est d'autant plus élevé que la teneur en ciment du mélange est élevée.

KEYWORDS: ground improvement, artificially cemented soils, shear moduli, curing time period.

1 INTRODUCTION

Soil-cement is a geo-composite formed by highly compacted mixture of soil, Portland cement, and water. As the cement hydrates, the mixture gains strength, stiffness and improves the engineering properties of the raw soil. The major variables that control the properties and characteristics of soil-cement mixtures are the type of soil, the proportion of cement in the mix, the degree of compaction and curing time period. It is possible, simply by varying the cement content and/or porosity of mixture, to produce soil-cement that ranges from a basic modification of the compacted soil to fully hardened soil-cement that is strong, stiff and durable. The soil-cement technique has been used successfully in pavement base layers, slope protection for earth dams, as a base layer to shallow foundations and to prevent sand liquefaction (Consoli *et al.* 2012a).

The unconfined compression test has been used as the most convenient means to investigate the effect of different variables on the soil-cement strength and to carry out dosage methodologies. The first rational dosage methodology for sand-cement was developed by Consoli *et al.* (2009) considering the porosity/cement ratio (η/C_{iv}), defined by the porosity of the compacted mixture divided by the volumetric cement content

(volume of cement divided by total specimen volume), as an appropriate parameter to evaluate the unconfined compressive strength (q_u) of the sand-cement mixture. Consoli *et al.* (2011), has shown that compressive strength increases with curing time period in artificially cemented soils and that there is a unique function controlling strength with curing time period and that such relation is a function of porosity and cement content. However, nothing is known regarding stiffness behavior with time. So, present study aims at fulfilling a breach of knowledge at quantifying the influence of the curing time period, the amount of cement and the porosity on the initial shear and oedometric moduli of an artificially cemented sand, as well as searching for a unique relationship linking both moduli (G_o and M_o) with porosity/cement ratio (η/C_{iv}) and curing period (t). The main contribution of present work is showing the existence of a direct relationship between G_o , M_o and η/C_{iv} for all studied curing time periods and only scalars differ regarding the effect of curing time. So, for the sand-cement studied, unique relationships were reached linking G_o and M_o with η , C_{iv} and curing time (t).

2 EXPERIMENTAL PROGRAM

The experimental program has been carried out in two parts. First, the soil was characterized. Then a number of

bender/extender elements tests were carried out considering four porosities and five cement contents at four distinct curing time periods (1, 3, 5 and 7 days).

2.1. Materials

The Osorio sand used in the testing was obtained from the region of Porto Alegre, in Southern Brazil, being classified as non-plastic uniform fine sand. Specific gravity of the solids is 2.63. Mineralogical analysis showed that sand particles are predominantly quartz. The grain size is purely fine sand with a mean diameter of 0.16 mm, being the uniformity and curvature coefficients of 1.9 and 1.2, respectively. The minimum and maximum void ratios are 0.6 and 0.9, respectively.

Portland cement of high early strength (Type III) was used as the cementing agent. Its fast gain of strength and stiffness allowed the adoption of 1, 3, 5 and 7 days as the curing time periods. The specific gravity of the cement grains is 3.15.

Distilled water was used for these characterization tests and tap water for molding specimens for the compression tests.

2.2. Methods

Molding and curing of specimens, as well as bender element tests are detailed below.

2.2.1. Molding and curing of specimens

For the bender element tests, cylindrical specimens 70mm in diameter and 140mm high were used. A target dry unit weight for a given specimen was then established through the dry mass of soil-cement divided by the total volume of the specimen. In order to keep the dry unit weight of the specimens constant with increasing cement content, a small portion of the soil was replaced by cement. As the specific gravity of the cement grains (3.15) is greater than the specific gravity of the soil grains (2.63), for the calculation of porosity, a composite specific gravity based on the soil and cement percentages in the specimens was used.

After the soil, cement and water were weighed, the soil and cement were mixed until the mixture acquired a uniform consistency. The water was then added continuing the mixture process until a homogeneous paste was created. The amount of cement for each mixture was calculated based on the mass of dry soil and the moisture content. The specimen was then statically compacted in three layers inside a cylindrical split mold, which was lubricated, so that each layer reached the specified dry unit weight. The top of each layer was slightly scarified. After the molding process, the specimen was immediately extracted from the split mold and its weight, diameter and height measured with accuracies of about 0.01g and 0.1mm, respectively. The samples were then placed inside plastic bags to avoid significant variations of moisture content. They were cured in a humid room at $23^{\circ}\pm 2^{\circ}\text{C}$ and relative humidity above 95%. The samples were considered suitable for testing if they met the following tolerances: Dry unit weight (γ_d): degree of compaction between 99% and 101% (the degree of compaction being defined as the value obtained in the molding process divided by the target value of γ_d); Dimensions: diameter to within $\pm 0.5\text{mm}$ and height $\pm 1\text{mm}$.

The molding points were chosen considering relative densities of 10%, 33%, 66% and 90%, with the same moisture content (about 10%). Each point was molded with five different cement percentages: 1%, 2%, 3%, 5% and 7%. These percentages were chosen considering the Brazilian and international experience with soil-cement [e.g., Mitchell (1981), Consoli *et al.* (2010, 2012b)], both in experimental and practical work.

2.2.2. Bender/extender element tests

T-shaped pairs of bender/extender (BE) elements, installed on the top and bottom specimen platens, were used in present study

for emission and reception of shear “S” waves (2 to 20 kHz frequencies) and compression “P” waves (20 to 80 kHz frequencies), being directly related to shear and oedometric moduli measurement, respectively. The bender/extender elements penetrated the specimen by 3mm at each end. For present sand-cement mixtures, time domain method of identification of first arrivals was adopted.

Single sine-wave input pulses were used at pre-set frequencies of 1, 3, 5, 7, 9, 11, 13 kHz, which covered the range of resonant frequencies of the sample-BE system. The output signals were captured on an oscilloscope, directly transferred to the PC and plotted to a common time base. The first arrival of the shear wave was taken (on the basis of previous calibration) as the point at which the wave descended, with low noise higher frequency results being preferred in order to avoid near field effects.

3 RESULTS OF STIFFNESS MEASUREMENTS

Results of both initial shear (G_o) and initial oedometric (M_o) moduli versus relative density (DR) of Osorio sand are presented in Figure 1. Equations (1) and (2), fitted from experimental data, give the expressions of G_o and M_o with relative density of Osorio sand.

$$G_o^{sand} \text{ (MPa)} = 17.60 + 0.076DR \quad (1)$$

$$M_o^{sand} \text{ (MPa)} = 41.18 + 0.294DR \quad (2)$$

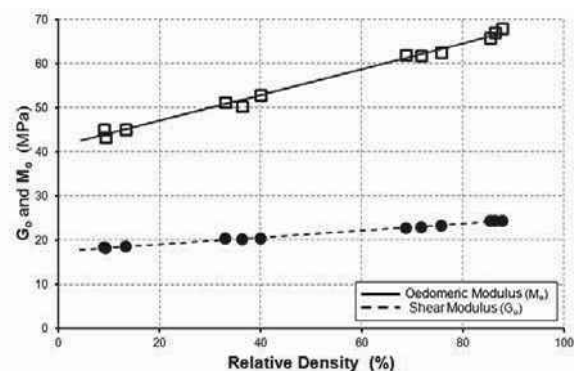


Figure 1. Shear (G_o) and oedometric (M_o) moduli versus relative density for Osorio sand.

Results of G_o and M_o on artificially cemented are also presented in Figures 2, 3, 4 and 5, respectively for, 1, 3, 5 and 7 curing days.

Figure 2 presents the variation of the shear (G_o) and oedometric (M_o) modulus with porosity/cement ratio (η/C_{iv}) considering all four studied relative densities (10, 33, 66 and 90%), five cement contents (1%, 2%, 3%, 5% and 7%) and 1 (one) day of curing. Equations (3) and (4) present the variation of G_o and M_o with η/C_{iv} , both with high coefficient of determination ($R^2=0.95$ and 0.93 , respectively for shear and oedometric modulus).

$$G_o^{1day} (MPa) = 6,694 \left[\frac{\eta}{C_{iv}} \right]^{-0.95} \quad (3)$$

$$M_o^{1day} (MPa) = 14,350 \left[\frac{\eta}{C_{iv}} \right]^{-0.81} \quad (4)$$

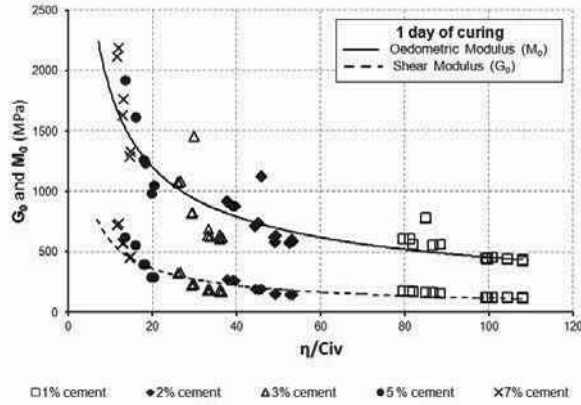
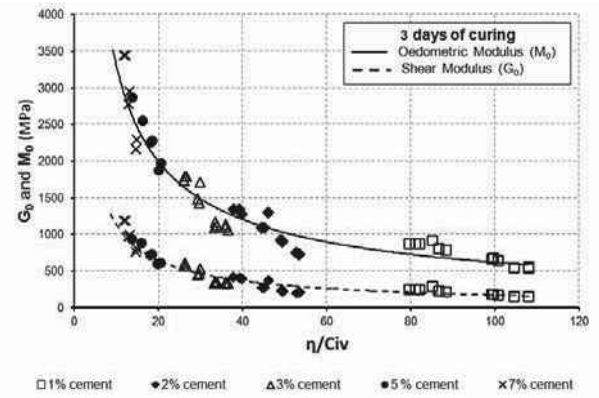

 Figure 2. Shear (G_o) and oedometric (M_o) moduli at one day of curing.

Figure 3 presents the variation of the shear (G_o) and oedometric (M_o) modulus with porosity/cement ratio (η/C_{iv}) considering all four studied relative densities (10, 33, 66 and 90%), five cement contents (1%, 2%, 3%, 5% and 7%) and 3 (three) days of curing. Equations (5) and (6) present the variation of G_o and M_o with η/C_{iv} , both with high coefficient of determination ($R^2=0.95$).

$$G_o^{3days} (MPa) = 11,500 \left[\frac{\eta}{C_{iv}} \right]^{-0.95} \quad (5)$$

$$M_o^{3days} (MPa) = 23,257 \left[\frac{\eta}{C_{iv}} \right]^{-0.81} \quad (6)$$

Figure 4 presents the variation of the shear (G_o) and oedometric (M_o) modulus with porosity/cement ratio (η/C_{iv}) considering all four studied relative densities (10, 33, 66 and 90%), the five cement contents (1%, 2%, 3%, 5% and 7%) and 5 (five) days of curing. Equations (7) and (8) present the variation of G_o and M_o with η/C_{iv} , both with high coefficient of determination ($R^2=0.95$).


 Figure 3. Shear (G_o) and oedometric (M_o) moduli at three days of curing.

$$G_o^{5days} (MPa) = 13,304 \left[\frac{\eta}{C_{iv}} \right]^{-0.95} \quad (7)$$

$$M_o^{5days} (MPa) = 26,377 \left[\frac{\eta}{C_{iv}} \right]^{-0.81} \quad (8)$$

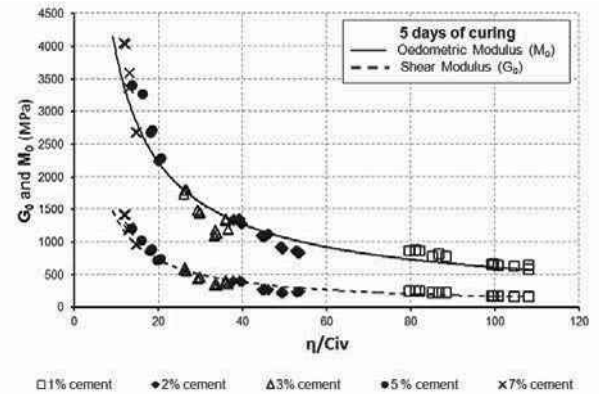

 Figure 4. Shear (G_o) and oedometric (M_o) moduli at five days of curing.

Figure 5 presents the variation of the shear (G_o) and oedometric (M_o) modulus with porosity/cement ratio (η/C_{iv}) considering all four studied relative densities (10, 33, 66 and 90%), the five cement contents (1%, 2%, 3%, 5% and 7%) and 7 (seven) days of curing. Equations (9) and (10) present the variation of G_o and M_o with η/C_{iv} , both with high coefficient of determination ($R^2=0.95$ and 0.93 , respectively for shear and oedometric modulus).

$$G_o^{7days} (MPa) = 15,595 \left[\frac{\eta}{C_{iv}} \right]^{-0.95} \quad (9)$$

$$M_o^{7\text{days}}(\text{MPa}) = 30,220 \left[\frac{\eta}{C_{iv}} \right]^{-0.81} \quad (10)$$

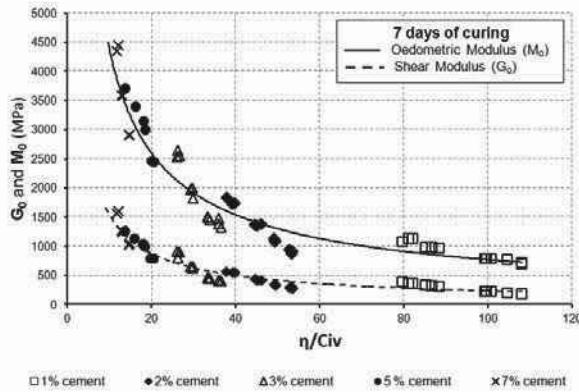


Figure 5. Shear (G_o) and oedometer (M_o) moduli at seven days of curing.

So, the use of the porosity of the compacted mixture divided by the volumetric cement content to assess both shear (G_o) and oedometer (M_o) moduli in the soil-cement mixtures studied herein is valid for all curing time periods studied. The results presented in this manuscript therefore suggest that using the porosity/cement ratio as represented by the porosity of the compacted mixture divided by the volumetric cement content, the engineer can choose the amount of cement and the porosity appropriate (within the studied range) to provide a mixture that meets the stiffness required by the project at the optimum cost. It can be seen from comparison of Eqs. (3), (5), (7) and (9) that G_o has a direct relationship with $(\eta/C_{iv})^{-0.95}$ for the all curing time periods and only a scalar differs regarding the effect of curing time. A similar situation is observed by comparing Eqs. (4), (6), (8) and (10), showing that M_o has a direct relationship with $(\eta/C_{iv})^{-0.81}$ for the all curing time periods and only a scalar differs regarding the effect of curing time. So, unique relationships can be achieved linking the G_o and M_o with η , C_{iv} and days of curing (t), as displayed in Fig. 6 and expressed mathematically by Eqs. (11) and (12), respectively for G_o and M_o . The coefficients of correlation are high [$R^2=0.98$ and 0.97 , respectively for shear and oedometer modulus].

$$G_o^t(\text{MPa}) = [2,022 + 7,250 \cdot \ln(t)] \left[\frac{\eta}{C_{iv}} \right]^{-0.95} \quad (11)$$

$$M_o^t(\text{MPa}) = [7,631 + 12,038 \cdot \ln(t)] \left[\frac{\eta}{C_{iv}} \right]^{-0.81} \quad (12)$$

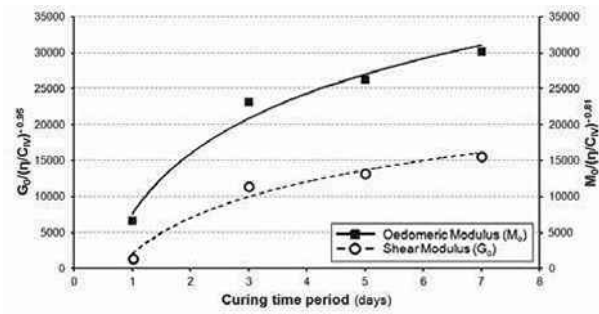


Figure 6. Increasing shear (G_o) and oedometer (M_o) moduli with curing time period.

4 CONCLUSIONS

From the data presented in this manuscript the following conclusions can be drawn:

- The porosity/cement ratio (η/C_{iv}) has been shown to be an appropriate index parameter to assess the shear (G_o) and oedometer (M_o) modulus in the sand-cement mixture studied herein for all curing time periods studied. Unique relationships were achieved linking the G_o and M_o with η , C_{iv} and time of curing (t);
- Based on the dosage equation established in present research for the studied sand-cement mixtures, there are several technical ways of reaching G_o or M_o target values for a given project and the best solution might change from situation to situation, depending on time period available, accessibility to equipment to reach a given porosity and cost of cement.

3. ACKNOWLEDGEMENTS

The authors wish to express their gratitude to Brazilian Research Council (CNPq) for the financial support to the research group.

4. REFERENCES

- Consoli N.C., Viana da Fonseca A., Cruz R.C. and Heineck K.S. 2009. Fundamental parameters for the stiffness and strength control of artificially cemented sand. *Journal of Geotechnical and Geoenvironmental Engineering* 135 (9), 1347-1353.
- Consoli N.C., Cruz R.C., Floss M.F. and Festugato L. 2010. Parameters controlling tensile and compressive strength of artificially cemented sand. *Journal of Geotechnical and Geoenvironmental Engineering* 136 (5), 759-763.
- Consoli N.C., Cruz R.C. and Floss M.F. 2011. Variables controlling strength of artificially cemented sand: Influence of curing time. *Journal of Materials in Civil Engineering* 23 (5), 692-696.
- Consoli N.C., Cruz R.C., Viana da Fonseca A. and Coop M.R. 2012a. Parameters controlling stiffness and strength of artificially cemented soils. *Géotechnique* 62 (2), 177-183.
- Consoli N.C., Viana da Fonseca A., Cruz R.C. and Heineck K.S. 2012b. Influence of cement-voids ratio on stress-dilatancy behavior of artificially cemented sand. *Journal of Geotechnical and Geoenvironmental Engineering* 138 (1), 100-109.
- Mitchell J.K. 1981. Soil improvement—State-of-the-art report. *Proc., 10th Int. Conf. Soil Mechanics and Foundation Engineering*, International Society of Soil Mechanics and Foundation Engineering, Stockholm, 4, 509-565.

Influence of diatom microfossils on soil compressibility

Influence des microfossiles de diatomées sur la compressibilité des sols

Díaz-Rodríguez J.A., González-Rodríguez R.

Department of Civil Engineering, National University of Mexico, MEXICO

ABSTRACT: There are several sites in the world where diatom microfossils have been detected in the soil deposits (e.g., Mexico City, Mexico and Osaka Bay, Japan). These soil deposits are characterized to have singular physical and mechanical properties that do not follow the well established empirical equations relating index properties with strength and deformation parameters. To evaluate the influence of diatom microfossils on the soil compressibility, this paper presents the experimental results of a series of odometer tests using artificially prepared mixtures of diatom microfossils and kaolin (D+K). Test results indicate that the presence of diatom microfossils substantially alters the index properties as well as compressibility.

RÉSUMÉ : Il ya plusieurs endroits dans le monde où les diatomées microfossiles ont été détectées dans les dépôts de sol (par exemple, la ville de Mexico, le Mexique et la baie d'Osaka, Japon). Ces dépôts de sols se caractérisent par des propriétés physiques et mécaniques singulières, qui ne suivent pas les équations empiriques bien établies reliant les propriétés d'index avec les paramètres de résistance et de déformation. Pour évaluer l'influence des microfossiles de diatomées sur la compressibilité du sol, cet article présente les résultats expérimentaux d'une série de tests d'œdomètre utilisant des mélanges préparés artificiellement de microfossiles de diatomées et de kaolin (D + K). Les résultats des tests indiquent que la présence de diatomées microfossiles modifie considérablement les propriétés de l'index ainsi que de compressibilité.

KEYWORDS: laboratory tests; compressibility; diatomite; kaolinite; mixtures

1 INTRODUCTION

There are several sites in the world where diatom microfossils have been detected in natural marine and lacustrine soil deposits (Shiwakoti et al., 2002; Díaz Rodríguez, 2003; Díaz-Rodríguez, et al., 1998; Terzaghi, et al., 1996).

Natural soils are mixtures of clay particles and coarse-grained soil constituents, therefore the engineering behavior of a particulate material, consisting of individuals of a number of different components (including pores or voids), is in general, not simply given by the sum of reactions of the single individuals. Much more important for the whole system is usually the mutual interactions and interference between these individuals (particles or grains), which will be influenced by their spatial arrangement (*i.e.* fabric). The soil fabric and its close relationship to soil behavior have been studied intensively by several researchers since many years ago. The level of participation by different types and sizes of particles within the soil matrix in the transfer of interparticle contact stresses dictates the stress-strain behavior.

Soil microstructure is one of the governing factors that are responsible for the mechanical behaviour of clays. The microstructure of soils implies the combined effects of fabric, chemical composition, mineralogical constitution, and interparticle forces. It is required a greater understanding of soil microstructure and the contribution of soil particles of different size to its mechanical response.

This paper addresses the analysis of soil compressibility of soil mixtures ranging from pure kaolinite to 60% diatomite and 40% kaolinite.

2 DIATOM MICROFOSSILS

Diatomite or diatomaceous earth is a porous and lightweight sedimentary rock resulting from accumulation and compaction of diatom remains over a geological time scale. It is a chalk-like

sedimentary rock that is easily crumbled into a fine white to off-white powder. Diatomite is relatively inert. The typical chemical composition of diatomite is approximately 90 percent silica, and the remainder consists of compounds such as aluminum and iron oxides. It has a high absorptive capacity, large surface area, low bulk and density. This powder has an abrasive feel, similar to pumice powder.

Diatoms skeletons or frustules are symmetric in shape and contain a large proportion of voids. Losic et al. (2007) indicate that a large percentage ($\approx 60 - 70\%$) of the diatom's frustules is essentially void space. Then the intraskeletal pore space of diatoms provides a chamber which can store water. Although the intricate frustules of diatoms have been well described, at the micro- and nano-scales of single cells and their interactions, interpretations based solely on physical principles have often been enlightening (Purcell, 1977; Vogel, 1983). Particle-surface interactions remain poorly understood and can produce unpredictable or inexplicable results (Feitosa and Mesquita, 1991).

3 ARTIFICIAL MIXTURES OF DIATOMITE AND KAOLIN

The experimental program followed in this investigation consisted in the elaboration of six mixtures: diatomite (D) was mixed with kaolin (K) in proportions of 0% diatomite (100K), 20% diatomite (20D + 80K), 30% diatomite (30D + 70K), 40% diatomite (40D + 60K), 50% diatomite (50D + 50K), 60% diatomite (60D + 40K). All mixtures were performed based on dry weight proportions.

3.1 Material used in mixtures

The kaolin and diatomite used in this investigation are products commercially available in Mexico. Kaolin is mainly composed of clay sized particles and soil particles smaller than

2 micrometers account for about 60%. The value of specific gravity of solid particles is 2.6. Diatomite is mainly silt sized; its silt sized particle account for more than 88% while clay size particles are about 3%.

3.2 Preparation of sample mixtures

Dry kaolinite powder and dry diatomite were thoroughly mixed in required proportion by weight. The homogenized mixtures were placed in the bowl of a blender and the necessary distilled water was added so that the mixture had the water content corresponding to its liquid limit. There was no any particle segregation when two different materials were mixed. The wet mixture was placed by spooning, in a cylindrical mold of 12.9 cm in diameter and 16 cm in height. The cylindrical mold served as consolidation cell, for that, two porous stones were introduced, one in the superior part and the other in the inferior one, both protected with filter paper, to avoid occluding the porous stones. The cylindrical mold with the mixture was placed in a consolidometer frame and a vertical stress of 130 kPa was applied during 28 days. Once the time of consolidation has elapsed, we extracted the soil sample, using a wire between separate the mold from the soil.

3.3 Index Tests

ASTM standards were followed in determining index properties, summarized in Table 1.

Atterberg limits increase with the increase in diatomite content, however, I_p decreased. Activity (i.e., the ratio between the plasticity index and clay size particles percentages) of the diatomite-kaolin mixtures increased with the increase in diatomite content (Table 1), this apparent increase is quite contradictory to the conventional perception, as mentioned by Shiwakoti et al. (2002). This fact suggests that the diatoms do not behave as silt size inert particles; on the contrary, they behave as very active clay particles.

Table 1. Index properties and physical properties of soil mixtures

Mixture D + K	w (%)	w _L (%)	w _P (%)	I _p (%)	γ _d (kN/m ³)	Activity
100K	43.00	56.40	28.90	27.50	11.06	0.42
20D + 80K	45.60	58.90	33.70	25.20	10.98	0.48
30D + 70K	49.00	63.75	40.25	23.50	10.70	0.51
40D + 60K	52.40	68.60	46.80	21.80	10.43	0.53
50D + 50K	55.15	72.05	51.75	20.30	9.95	0.59
60D + 40K	57.90	75.50	56.70	18.80	9.48	0.65

4. COMPRESSIBILITY OF SOIL MIXTURES

ASTM standards were followed in determining compressibility properties of the soil mixtures, and strain data are given in Table 2.

Table 2 Results of compression tests on diatomite-kaolin mixtures

Mixture D + K	Stress						
	25 Kpa	50 Kpa	100 Kpa	200 Kpa	400 Kpa	800 Kpa	1600 Kpa
100K	6.992	7.949	9.630	11.843	14.248	16.996	20.374
20D + 80K	1.047	1.492	2.142	3.205	4.634	7.441	11.189
30D + 70K	1.134	1.445	1.941	2.913	4.453	7.693	12.724
40D + 60K	1.354	1.752	2.398	3.449	5.264	9.114	15.602
50D + 50K	1.083	1.492	2.142	3.362	5.413	9.732	16.449
60D + 40K	1.268	1.654	2.268	3.425	5.402	9.689	16.823

A series of six tests was carried out on 63.3 mm diameter and 25.4 mm high specimens through the incremental load (IL). The equipment used was similar to that described by Head (1982). The specimens were separated from the porous stones with a single thickness of Whatman No. 50 filter paper. The applied pressures ranged from 25 kPa to 1,600 kPa. Each test involved 7 load increments that were applied daily. The effect of the stress increment ratio on the compression curve is not considered herein. Calibrations were made of each apparatus to reduce the effects of compliance in the measurement of sample displacement.

The stress-strain relationship ($\epsilon_v - \sigma'_v$ plots) from incremental one-dimensional consolidation tests, are presented in Figure 1. It can be seen from the figure that the stress-strain curves show different characteristic that depend on the diatomite content.

- The compression curve for 100% kaolinite is shown as reference curve
- The mixture 20D+80K shows a marked decrease of deformation.
- The mixtures of diatomite content of 30, 40, 50, and 60% show different behavior. As diatomite content increase the vertical strain begins to increase.

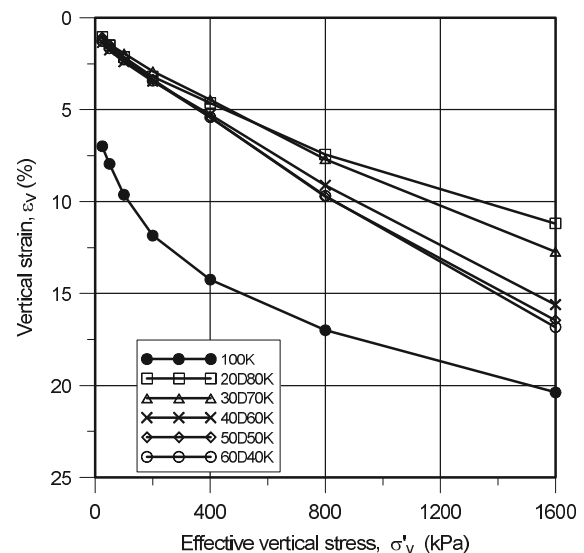


Figure 1 Stress-strain behavior on diatomite-kaolin mixtures

The variation of strain in diatomite content is shown in Figure 2, each curve is identified with the vertical stress applied. It can be seen from the figure that each curve shows different characteristic that depend on the vertical stress.

- For small values of the vertical stress (25 to 100 KPa) there is a plateau between 20 to 60% of diatomite content.
- From a vertical stress of 200 KPa the plateau tends to disappear and the shape of the strain pattern denotes a change in the behavior for high stress levels.

The stress-strain relationship ($\epsilon_v - \log \sigma'_v$ plots) for different mixtures, are presented in Figure 3. It can be seen from the figure that there is a sudden change in the pattern when diatomite is added to the mixture. Curve 100K represents the general $\epsilon_v - \log \sigma'_v$ relationship for random type of structure typical of a remolded clay. Other compression curves represent incipient structured soils.

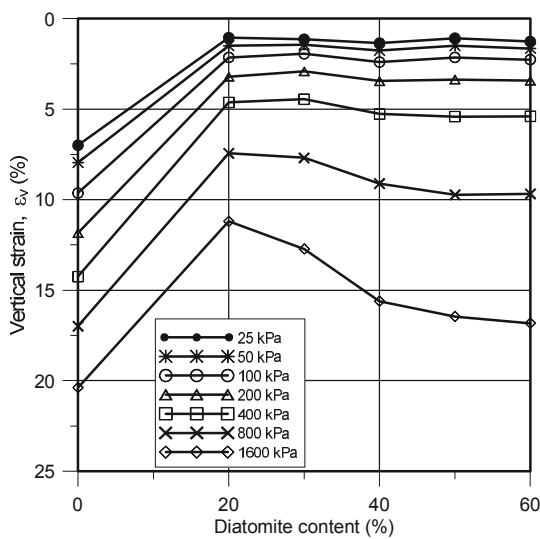


Figure 2 Variation on strain with diatomite content

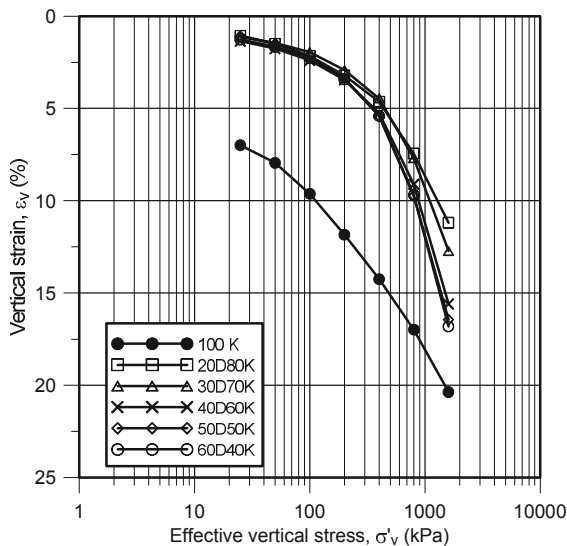


Figure 3 Comparison between stress-strain curves obtained with different diatomite content

5. DISCUSSION

The compressibility of fine-grained soils containing a high percentage of clay minerals depends not only on the mechanical properties of its constituents, but also on physico-chemical factors.

The difficulty in dealing with soil mixtures is manifested not only by the complexity of their behavior, but also in the absence of index parameters to characterize and compare soil mixtures. Consequently, a soil with a different composition (e.g., diatomite-kaolin with varying diatomite content) must be treated as a different material every time there is a change in the soil composition, and laboratory tests must be performed wherever there is a significant variation in the soil composition.

Therefore part of their compressibility is determined by the mechanical properties of the soil particles and the balance by the physico-chemical interaction of their constituents.

5.1 Index properties

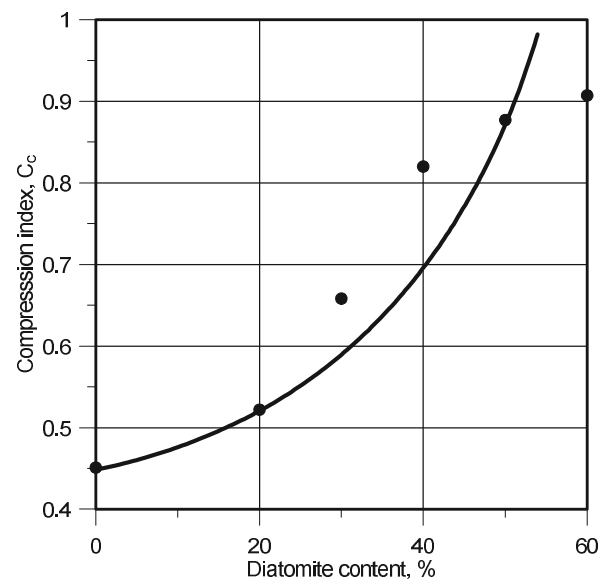
According to Tanaka and Locat (1999) the increase in activity is believed to be an example of the influence of diatom microstructure on the test results.

5.2 Compressibility

The influence of diatom microfossils on soil compressibility is complex and depends on several factors (e.g., diatomite content and stress level). The results of this experimental program show that the influence of diatoms microfossils in the engineering behavior of soils cannot be ignored.

There are several very interesting phenomena associated with the flow over cavities, like that presented by the frustules of diatoms. Models developed to examine shear flow over pores of different shapes, radii, and depths have shown that pores morphology is important in determining the streamlines of flow (Pozrikidis, 1994). The kinematics structure of the flow is discussed with reference to eddy formation and three-dimensional flow reversal. This associate phenomenon suggests that water inside diatom frustules could not be mechanically inert and affect the pore water pressure generated in the interstices between diatoms and clay particles. It therefore, influences the effective stress and the change the stiffness.

Relationship between diatomite content and C_c are shown in Figure 4 for the six mixtures. The results show that the compressibility of kaolin increases sharply with the addition of diatomite. Therefore, the C_c value for mixtures with diatomite is larger than pure kaolinite.


 Figure 4 Relationship between diatomite content and C_c

6. CONCLUSIONS

Diatoms are outstanding examples of natural micro- and nano-structured materials that control the mechanical, hydraulic, and physico-chemical properties of soils. It is concluded that diatomaceous soils exhibit the following characteristics:

1. Activity increase with the increase in diatomite content.
2. The stress-strain curves show different characteristics depending on the diatomite content.
3. The strain-diatomite content curves show different characteristics depending on the stress level.
4. The presence of a significant amount of microfossils can significantly influence the mechanical behavior of soils, particularly the compressibility.

5. The compressibility for mixtures with diatomite is larger than pure kaolin
6. The compressibility of fine-grained soils containing a high percentage of clay minerals depends not only on the mechanical properties of its constituents, but also on physico-chemical factors.
7. However, our knowledge about the interaction between diatoms, clay particles, and water is in its infancy. Hence, it is necessary to pursue in greater detail the study of topics such as hydrodynamic effects and chemical reactions of diatom frustules.

7. ACKNOWLEDGEMENTS

This study has been supported by the Dirección General de Apoyo al Personal Académico, (DGAPA) UNAM. Editing of the manuscript was completed with the superb assistance of Guadalupe Salinas. The writers are grateful to Prof. Yudhbir Yudhbir for comments during preparation of this paper.

8. REFERENCES

- Díaz Rodríguez, J. A. (2003). "Characterization and engineering properties of Mexico City lacustrine soils" in *Characterization and Engineering Properties of Natural Soils*, Balkema Publishers, Vol. 1, 725-755.
- Díaz-Rodríguez, J. A., Lozano-Santa Cruz, R., Davila-Alcocer, V. M., Vallejo, E. & Girón, P. (1998). "Physical, chemical, and mineralogical properties of Mexico City sediments: a geotechnical perspective." *Canadian Geotechnical Journal* 35(4): 600-610.
- Feitosa, M. I. M. & Mesquita, O. N. (1991). Wall-drag effect on diffusion of colloidal particles near surfaces: A photon correlation study. *Physical Review A*, 44(10): 6677-6685.
- Head, K. H. (1982). *Manual of Soil Laboratory Testing*. Vol. 2. Second Edition, John Wiley & Sons Inc.
- Locat, J., Tremblay, H., Leroueil, S., Tanaka, H. & Oka, F. (1996). Japan and Québec clays: their nature and related environmental issues. In *Proceedings of the 2nd International Congress on Environmental Geotechnics*, Osaka, Japan: 127-132.
- Pozrikidis, C. (1994). Shear flow over a plane wall with an axisymmetric cavity or a circular orifice of finite thickness. *Phys. Fluid Journal*, vol. 6: 68-79.
- Purcell, E. M. (1977). Life at low Reynolds number. *Am. J. Physics*, vol. 45: 3-11
- Shiwakoti, D. R., Tanaka, H., Tanaka, M. & Locat, J. (2002). Influence of diatom microfossils on engineering properties of soils. *Soils and Foundations*, 42(3): 1-17.
- Terzaghi, K., Peck, R. B., and Mesri, G. (1996). *Soil mechanics in engineering practice*. 3rd Ed., Wiley, New York.
- Tanaka, H. & Locat, J. (1999). A microstructural investigation of Osaka Bay clay: the impact of microfossils on its mechanical behavior. *Canadian Geotechnical Journal*, 36: 493-508.
- Vogel, S. (1983). *Life in moving fluids*. Princeton University Press. Princeton, N. J.

Strength properties of densely compacted cement-mixed gravelly soil

Propriétés de résistance des graves cimentées fortement compactées

Ezaoui A.

CETE de Lyon, DLA, France

Tatsuoka F., Furusawa S., Hirao K., Kataoka T.

Tokyo University of Science, Civil Engineering Department, Japan

ABSTRACT: A series of drained triaxial compression tests (TC) were performed on specimens prepared in the laboratory and rotary core samples retrieved from the field of well-compacted cement-mixed gravelly soils (CMG). The effects of the following factors on the compressive strength q_{max} were evaluated: the degree of compaction D_c and cement content (when compacted at the optimum water content); the curing time; the grading characteristics of gravelly soil; and the specimen volume. The followings were found from the data for a wide variety of CMG having $c/g = 2.0 \sim 6.0$ % compacted by using two energy levels (1.0Ec & 4.5Ec). The effects of D_c are equally important as the cement content. An empirical equation is proposed using two independent variables, the soil skeleton porosity n_s , which controls the initial compressive strength, and the cement void ratio C_r , which controls the increasing manner of q_{max} with curing time. The effects of sieving method to obtain smaller particle materials for small TC specimens from an original gravelly soil having larger particles are not significant. However, significant effects of specimen volume (i.e., 72 mm x 72 mm in cross-section times 150 mm high versus 300 mm in diameter times 580 mm high) is highlighted.

RÉSUMÉ : Une campagne d'essais triaxiaux en compression a été réalisée sur des échantillons de grave, cimentée et compactée (GC), préparés en laboratoire mais également sur des échantillons carottés sur site. Les effets de différents facteurs sur les résistances en compression q_{max} ont été évalués : le degré de compaction D_c (teneur en eau de l'optimum Proctor), la teneur en ciment, le temps de cure; la distribution granulométrique et le volume de l'échantillon. Les tendances suivantes ont pu être relevées à partir de données expérimentales obtenues sur une grande variété de GC présentant différents teneurs en ciment $c/g = 2.0 \sim 6.0$ % et compactées suivant deux modalités: normale et modifiée (1.0Ec & 4.5Ec). Il est apparu que les effets de la compaction ou de la teneur en ciment sur la résistance en compression étaient tout aussi importants. Une formulation empirique est proposée afin d'évaluer la résistance en compression à partir de deux variables indépendantes : la porosité du squelette n_s , qui contrôle la résistance initiale en compression, et la proportion du volume des vides occupée par le ciment C_r , qui contrôle l'augmentation de q_{max} avec le temps de cure. Les effets d'une réduction granulométrique ne sont pas significatifs. Cependant, les effets du volume de l'échantillon sont mis en évidence.

KEYWORDS: cement-mixed gravel, curing, triaxial compression, compaction, size effect

1 INTRODUCTION

Ground improvement by cement-mixing has been successfully used in many construction projects. This technology includes mixing-in-place of soft clay without compaction (i.e., the deep mixing method), under-water placing of cement-mixed soil slurry without compaction and highly compacted cement-mixed gravel for high roller-compacted concrete (RCC) dams. More recently, cement-mixed gravelly soil (CMG) compacted by energy lower than the one for RCC is used to construct bridge abutments for high-speed trains in Japan. Such use of CMG for deformation-sensitive structures as above has been motivated by a high cost-effectiveness. To develop the design and construction procedures, the stress-strain properties of CMG have been studied by many researchers (Lohani et al. 2004, Kongsuprasert & Tatsuoka 2005; Kongsukprasert et al. 2005, 2007; Tatsuoka et al. 2008, Ezaoui et al. 2010). Yet, the whole picture of effects of dry density, water content at compaction and cement content ratio on the strength and stiffness are not well understood (e.g., Horpibulsuk et al. 2003; Consoli et al. 2007).

One of other practically important issues is the use of well-graded gravelly soil in the field with the maximum diameter exceeding 35 mm, which is too large to be tested by ordinary laboratory stress-strain tests, such as triaxial compression (TC) tests. For this reason, it is usually prepare materials for specimens for TC tests, for example, by sieving out large particles from a given original gravelly soil. So, it becomes necessary to evaluate the effects of grading characteristics (i.e., the particle size and the shape of grading curve) and the volume of specimen.

The present study aims at finding major factors that control the strength and stiffness of CMG for its proper and cost-effective use. In particular, to evaluate the effects of compacted degree of

compaction, cement content, curing time, particle grading characteristics and specimen volume were evaluated, a comprehensive series of drained TC tests were performed.

2 EFFECTS OF SEVERAL FACTORS ON STRENGTH

2.1 Specimens prepared in the laboratory

The first TC series performed in the present study used Sieved Chiba Gravel (SCG in Fig. 1, crushed quarry sandstone comprising sub-angular particles). The TC specimens were rectangular prismatic (72 mm x 72 mm in cross-section times 150 mm high), for which a maximum particle size of 10 mm was selected. Fig. 2a shows compaction curves for standard and modified Proctor energy levels (1.0Ec = 550 kJ/m³ and 4.5Ec) and cement/gravel ratio in weight $c/g = 2.5$ % and 4.0 % of SCG. $\rho_{d,max} = 2.12$ g/cm³ and $w_{opt} = 9.3$ % were obtained for 1Ec and $\rho_{d,max} = 2.21$ g/cm³ and $w_{opt} = 7.4$ % for 4.5Ec. Effects of cement-mixing on the compaction curves are negligible (Ezaoui et al., 2010). The TC specimens were produced by tamping moist cement-mixed gravelly soil immediately after adding water for respective optimum water contents w_{opt} to a degree of compaction $D_c = 95$ %, where $D_c = \rho_{d, test} / \rho_{d, max} \times 100$ % for 1.0Ec or 4.5Ec (Fig. 2a). The weight and height of each of the five sub-layers were carefully controlled. In the test results shown hereafter, vertical (axial) and horizontal (lateral) strains, ε_v and ε_h , measured locally with of a pair of vertical local deformation transducers (LDTs) arranged on two opposite lateral faces of specimen and horizontal LDTs arranged on the other two opposite lateral faces are presented.

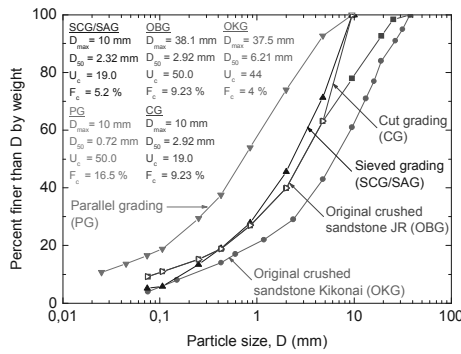


Figure 1. Grading curves and characteristics of the tested cement-mixed gravels

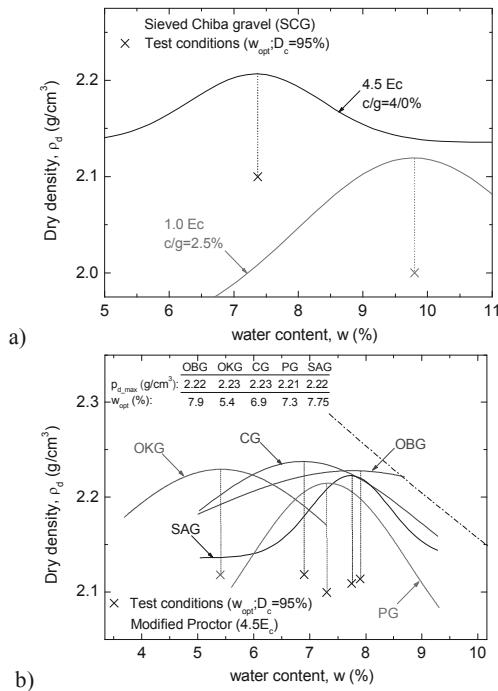


Figure 2. Compaction curves: a) SCG for 1.0Ec & 4.5Ec and $c/g = 2.5\%$ & 4.0% ; and b) five cement-mixed gravelly soils presented in Fig. 1 for 4.5Ec and $c/g = 4.0\%$ (except for PG and OKG)

Fig. 3 presents the development of the compressive strength q_{max} at confining pressure of 20 kPa with the period of initial curing under the atmospheric pressure at a constant water content (the same as prepared) of four kinds of SCG specimen prepared under different conditions (1.0Ec or 4.5Ec and $c/g = 2.5$ or 4.0%). The following trends can be noted. Firstly, the q_{max} value increases considerably with time, which should be due to the cement hydration process. The increase until a curing period of 14 days is rather proportional to the “initial” value at 7 days. Secondly, the q_{max} value is largely different among the four kinds of specimen (up to a factor of 100 %). Thirdly, the effects of compacted dry density ρ_d on the q_{max} value when $c/g = 2.5\%$ are significant. An increase more than 100 % results from an increase in ρ_d of only about 5 % associated with an increase in the compaction level from 1Ec to 4.5Ec. On the other hand, when the compaction level is 1.0Ec, the q_{max} value increases by a factor of only about 40 % with an increase in c/g from 2.5 % to 4.0 % (i.e., an increase of about 60 %).

E_c and c/g are the parameters commonly used in practice, because they are easily measured and controlled. However, they are not the basic parameters that control the strength and deformation characteristics of CMG. This feature can be easily seen from the following inherent drawbacks (Watanabe et al. 2003, Kongsukprasert et al. 2005). Firstly, with materials having different specific density ρ_s , for the same c/g value and the same soil void ratio, the volume of cement per volume increases with

an increase in ρ_s , despite that ρ_s has no direct effect on the stress-strain properties. Secondly, the effect of compaction level for the same c/g value (and same ρ_s) has two components: 1) a better interlocking among soil particles with a decrease in the soil void ratio; and 2) a larger amount of cement in a less volume of the total void of soil particle skeleton. Based on this consideration, two independent parameters are postulated: the soil skeleton porosity n_s (representing the structure of the skeleton of gravelly soil particles only); and the cement void ratio C_r (representing the fraction of the void of the soil skeleton occupied by cement):

$$n_s = \frac{V - V_s}{V} \quad (1)$$

$$C_r = \frac{V_c}{V_v} = \frac{V_c}{V - V_s} \quad (2)$$

where V is the total volume; V_s is the volume of gravelly soil particles; V_v is the volume of the void of the skeleton of gravelly soil; and V_c is the volume of cement.

Ezaoui et al. (2011) also proposed the following hyperbolic function for the q_{max} value that increases with time and is a function of these two parameters independently:

$$q_{max}(t_c) = q_0(n_s) \left[1 + \frac{a(C_r)t_c}{b + t_c} \right] \quad (3)$$

where t_c is the curing period; q_0 is the initial compressive strength (when $t_c = 0$) that decreases with n_s ; a is the parameter showing the cementation effect that increases with C_r ; and b is the constant parameter that depends on cement type. The functions $a(C_r)$ and $q_0(n_s)$ are obtained based on the data presented in Fig. 3 together with those from CD TC tests on rotary core samples retrieved from the field, as shown below.

Three solid lines presented in Fig. 4 denote the iso-strength lines for constant $q_{max} = q_1, q_2$ and q_3 at a specified t_c according to Eq. (3) with known values of q_0, a and b . The dash-dot curves are the corresponding $c/g =$ constant curves. From such a plot as shown in Fig. 4, the most suitable (i.e., the most cost-effective) combination of (C_r, n_s) of a given type of CMG that achieves a given required compressive strength can be chosen referring to the cost for cement and compaction work.

2.2 U tests on specimens made using a material use in the field

The use of CMG is now spreading in Japan, particularly to construct bridge abutments for high speed train lines. Very recently, a geosynthetic-reinforced soil (GRS) integral bridge was constructed. The backfill immediately behind the facing is well-compacted cement-mixed gravelly soil. The grading curve of this backfill material (i.e., crushed gravel from a quarry, denoted as the Original Kikonai Gravel, OKG) is presented in Fig. 1. The compaction curve for 4.5Ec is presented in Fig. 2b. Before the construction, to determine the cement-mixing proportion, a series of U tests were performed on specimens mixed at $c/g = 2.0, 4.0$ and 6.0% , compacted to $D_c = 95\%$ at $w = w_{opt}$ (4.5Ec) and cured for 7 and 28 days. To accommodate the maximum particle size of 37.5 mm, large specimens (150 mm in diameter and 300 mm in height) have been used.

To apply Eq. (3) to the data from these U tests, $b = 19.62$ was used, which was obtained by analyzing a data set from a series of CD TC tests on cement-mixed SCG cured for a period of 1 ~ 180 days (Ezaoui et al., 2010, 2011). The values of $a(C_r)$ and $q_0(n_s)$ for the three c/g values and the two curing periods were determined from the reported values of $q_{max}(t_c)$. They are plotted against their corresponding n_s and C_r values in Figs. 6a and b (round symbols). In so doing, to eliminate possible effects of specimen size and grading characteristics (discussed in the next section), these q_{max} values had been corrected to become the same values as those obtained for the smaller specimen (i.e., the data presented in Fig. 3) under the same test conditions (i.e., a curing period of 7 days; $D_c = 95\%$; and $c/g = 4.0\%$). Then, in Figs. 6a and b, only relative variations of q_0 and the parameter a due to variations in the n_s and C_r values among those U test

specimens are highlighted. In these figures, the data points of SCG (square symbols) obtained from the data presented in Fig. 3 are also presented. It may be seen that these two sets of data, which are for the specimens compacted to the same D_c (i.e., 95 %) at the respective optimum water contents, are consistent with each other. Linear functions can be fitted to the $q_0 - n_s$ and $a - C_r$ relations with a small variance less than 10 %.

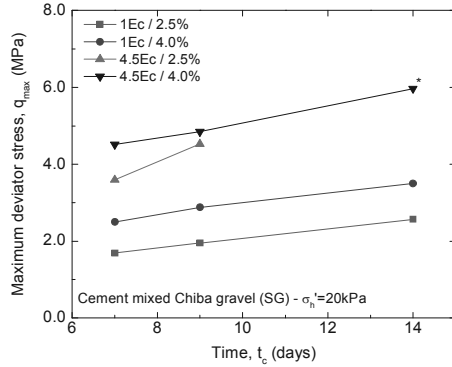


Figure 3. Development of compressive strength q_{max} with time t_c for different cement contents c/g and different compaction energy levels E_c (laboratory specimen of Sieved Chiba Gravel, SCG)

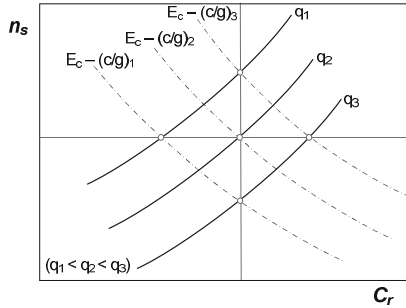


Figure 4. Illustration of the effect of skeleton porosity n_s and cement void ratio C_r on the compressive strength $q_{max} = q_{1,2,3}$

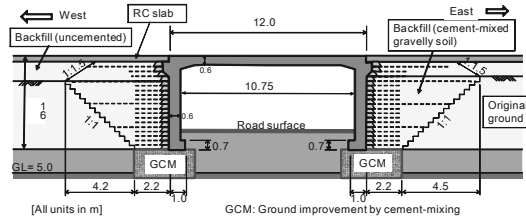


Figure 5. General structure (the width=11.7 m) of the first prototype GRS integral bridge, for the new bullet train line at Kikonai, the south end of Hokkaido (by the courtesy of the Japan Railway Construction and Technology Agency), constructed in 2011 (Tatsuoka et al., 2012)

2.3 TC tests on core samples retrieved from the field

It is examined below whether the empirical relations shown in Figs. 6a and b can be applied to the data from CD TC tests on rotary core samples retrieved from the CMG backfill of the GRS integral bridge (Fig. 5). The backfill was compacted to $D_c = 100$ % (4.5Ec), higher than the laboratory-prepared specimens analyzed above, and at $c/g = 3.0$ %. The core samples were 80 mm in diameter and 160 mm or less in height. The q_{max} values evaluated by CD TC tests at confining pressure of 20 kPa are plotted against t_c (from the time of compaction in the field until respective TC tests) in Fig. 7. The data exhibit a large scatter, which is inevitable with such core samples from the field (e.g., Taheri et al., 2012). Yet, the trend that q_{max} increases with t_c is obvious. It may also be seen that, even at a relatively low c/g (= 3.0 %), the strength tends to exceed 10 MPa after $t_c =$ three months, which is a nearly half of the strength of ordinary concrete. This property should be due to very high compaction (i.e., $D_c = 100$ % (4.5Ec) with $\rho_d \sim 2.23$ g/cm³) achieved by field compaction. It is to be noted that very high stiffness values were

measured with the two core samples cured for about 90 days presented in Fig. 7. That is, the small-strain Young modulus (at the isotropic stress-state) was about 35 GPa, which is the same order of magnitude as ordinary concrete. With these core specimens, $n_s = 20.2$ % and $C_r = 9.3$ % were obtained from $\rho_d = 2.23$ g/cm³ and $c/g = 3.0$ %. Based on the respective linear relations presented in Figs. 6a and b, the parameters q_0 and a for Eq. (3) were evaluated by two methods:

Method 1): By substituting $n_s = 20.2$ % into the linear relation in Fig. 6a, $q_0 = 1.61$ MPa is obtained (i.e., a data point of triangular symbol 1), located on the linear relation). Then, by assuming the same coefficient $b = 19.62$ as the laboratory-prepared specimens for Eq. (3) and using this q_0 value, an average relation is best-fitted to the data presented in Fig. 7. Then, $a = 6.7$ % is obtained for $C_r = 9.3$ % (i.e., a data point of triangular symbol 1) in Fig. 6b).

Method 2): By substituting $C_r = 9.3$ % into the linear relation shown in Fig. 6b, $a = 7.7$ % is obtained (i.e., a data point of triangle symbol 2), located on the linear relation). Then, by assuming $b = 19.62$ as above for Eq. (3) and using this a value, an average relation is best-fitted to the data presented in Fig. 7 (which is eventually the same as the one obtained by Method 1)). Then, $q_0 = 1.43$ MPa is obtained for $n_s = 20.2$ % (i.e., a data point of triangular symbol 2) in Fig. 6a).

As seen from Figs. 6a and b, the results obtained by two methods are consistent. The results shown above indicate that Eq. (3) is relevant for various kinds of CMGs with different grading characteristics, different cement contents ($c/g = 2.0 \sim 6.0$ %) and a relatively wide range of compaction level ($\rho_d = 2.0 \sim 2.23$ g/cm³).

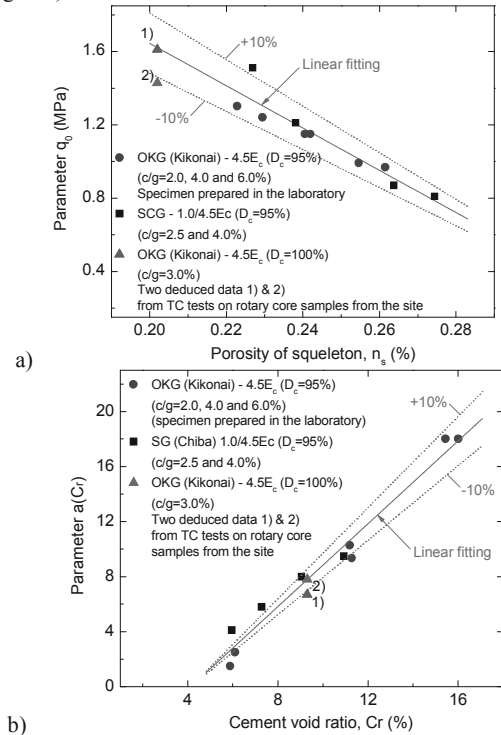


Figure 6. a) Variation of parameter q_0 , Eq. (1), with skeleton porosity n_s and; b) variation of parameter C_r , Eq. (1) with cement void ratio C_r

3 EFFECTS OF SPECIMEN AND PARTICLE SIZES

The compressive strength at given confining pressure is also controlled by factors other than those analyzed in the section above, including: grading characteristics, such as the mean diameter d_{50} and the uniformity coefficient, fines content etc. (factor G); and the size of specimen (factor S) (Ezaoui et al., 2011). Two relevant parameters, among others, for factor S are: i) the particle/specimen size ratio (represented by the ratio of the specimen diameter D to d_{50}); and; ii) the specimen volume V .

Factors G and S may be different between the full-scale backfill and the one used in laboratory stress-strain tests.

To evaluate factors S and G, a series of drained TC tests were performed using small and large CMG specimens of the same gravelly soil but having four different grading curves, PG, SAG, CG and OBG shown in Fig. 1, produced from an original gravelly soil (OBG), a sub-angular crushed sandstone as Chiba gravel but from another quarry. Among these sieved materials, SAG was produced just by removing particles larger than 10 mm. The fact that SAG is easiest to produce is a strong advantage for the use in tests performed in practice. On the hand, PG is extremely time-consuming to produce, as a large amount of fine particles should be produced. So, the use of this material in practice cannot be recommended. The production of CG is intermediate. The compaction curves of these sieved materials are noticeably different due to different grading curves (Fig. 2b). The small specimens have the same size as those of which the data are presented in the preceding sections. The large specimens are 300 mm in diameter and 580 mm in height. All the specimens were prepared under the same conditions: i.e., $c/g = 4.0\%$ and $D_c = 95\%$ at the respective optimum water contents by $4.5E_c$. As the $\rho_{d,max}$ values of these sieved materials are nearly the same (Fig. 2b), the eventually obtained compacted dry densities are very similar.

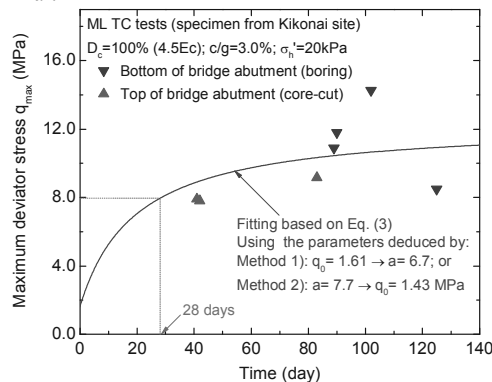


Figure 7. Compressive strength q_{max} versus total curing time for rotary core samples from the field (Kikonai GRS integral bridge) and fitting curve based on Eq. (3) with a set of deduced parameters (Fig. 6a and b)

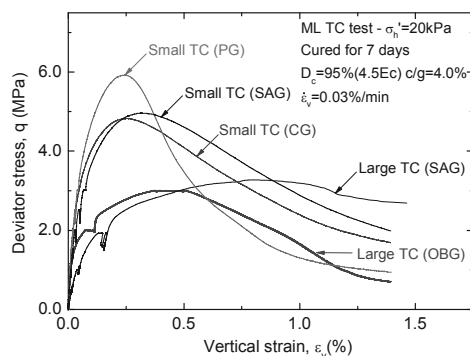


Figure 8. Deviator stress (q) – vertical strain (ϵ_v) relations from CD TC tests at $\sigma_h = 20$ kPa on small and large specimens having different types of cement-mixed gravel presented in Fig. 1

Fig. 8 shows the stress-strain curves at $t_c = 7$ days of the small and large specimens of SAG, CG and OBG. The effects of specimen size (factor S) are significant, while the effects of grading characteristics (factor G) are not so. It was found that there is no trend in the relationship between q_{max} and D/d_{50} (also with D/d_{max}). On the other hand, a well-defined trend exists in the relationship between q_{max} and the ratio of the specimen volume to the small specimen volume $V/V_{small TC}$ (Fig. 9): i.e., q_{max} decreases nearly 40 % with an increase in the specimen volume by a factor of nearly 50. This fact indicates that Eq. (3) should be modified to take into account this size effect. More research is necessary to examine whether this linear relation can be extended

to volumes that exceeds the largest value examined in this study (such as field full-scale structures).

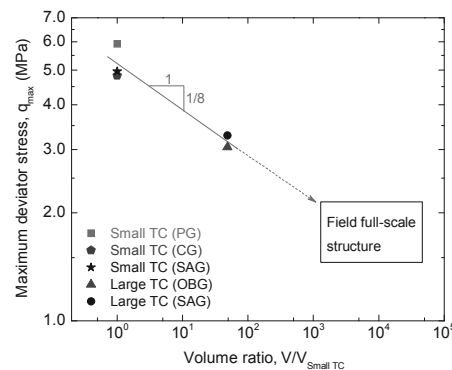


Figure 9. Compressive strength q_{max} ($D_c = 95\%$ at $4.5E_c$; $c/g = 4.0\%$; & $t_c = 7$ days) plotted against normalized volume by the volume of the small specimen $V_{small TC}$

4 CONCLUSIONS

The following conclusions can be derived:

1. High compaction is very effective to obtain high strength of cement-mixed gravelly soil (CMG).
2. The porosity of the skeleton of gravelly soil only, n_s ; and the fraction of the void of soil skeleton occupied by cement, C_r , are two major independent parameters for the strength of CMG. An empirical equation is proposed to predict the compressive strength of a given CMG based on a given initial compressive strength that is a function of n_s with an increase with curing time following a function of C_r .
3. Within the limit of test conditions in the present study, for the same degree of compaction at the respective optimum water contents with the same cement content, the effects of grading characteristics on the strength are not significant, while the effects of specimen volume are significant.

5 ACKNOWLEDGEMENTS

The core samples from Kikonai were provided by the Japan Railway Construction, Transport and Technology Agency and the study was financially supported by Japan Society for the Promotion of Science.

6 REFERENCES

- Consoli, N.C., Foppa, D., Festugato, L. and Heineck, K.S. (2007): "Key parameters for strength control of artificially cemented soils", *J. Geotech. Geoenviron. Eng.*, 133(2): 197–205.
- Ezaoui, Tatsuoka, F., Sano, Y., Iguchi, Y., Maeda, Y., Sasaki, Y. and Duttine, A. (2010): Ageing effects on the yielding characteristics of cement-mixed granular materials, *Soils and Foundations*, 50(5), 705–724.
- Horpiulsuk, S., Katkan, W., Sirilerdattana, W. and Rachan, R. (2006): "Strength Development in Cement stabilized low plasticity and Coarse grained soils: Laboratory and Field Study", *Soils and Foundation*, 46(3), 351–366.
- Kongsukprasert, L. and Tatsuoka, F. (2005): "Ageing and viscous effects on the deformation and strength characteristics of cement-mixed gravelly soil in triaxial compression", *Soils and Foundations*, 45(6), 55–74.
- Taheri, A., Y. Sasaki, Y., Tatsuoka, F. and Watanabe, K. (2012): "Strength and deformation characteristics of cemented-mixed gravelly soil in multiple-step triaxial compression", *Soils and Foundations*, 52(1).
- Tatsuoka, F., Di Benedetto, H., Kongkitkul, W., Kongsukprasert, L., Nishi, T. and Sano, Y. (2008) Modeling of ageing effects on the elastoviscoplastic behavior of geomaterial, *Soils and Foundations*, 48(2), 155–174.
- Tatsuoka, F., Munoz, H., Kuroda, T., Nishikiori, H., Soma, R., Kiyota, T., Tateyama, M. and Watanabe, K. (2012): Stability of existing bridges improved by structural integration and nailing, *Soils and Foundations*, 52(2).

Tensile Strength of Lightly Cemented Sand through Indentation Tests

Résistance à la traction de sable légèrement cimenté par des tests d'indentation

Ge L.

Department of Civil Engineering, National Taiwan University

Yang K.-H.

Department of Construction Engineering, National Taiwan University of Science and Technology

ABSTRACT: Compared to the compressive or shear strength of soil, its tensile strength is generally assumed to be zero, or insignificant, in geotechnical engineering practice because of its relatively small value and lack of a satisfying laboratory technique. The tensile strength of soil is, however, an important parameter in the design of geosystems, where tensile cracks contribute to progressive erosion or landslides in excavation, slopes, dams, highway embankments, riverbanks, hydraulic barriers, and other earth structures. In this paper, a recent development of the indentation test is presented. The theoretical framework is re-evaluated followed by a series of test results on lightly cemented sand.

RÉSUMÉ : Par rapport à la résistance à la compression ou de cisaillement du sol, sa résistance à la traction est généralement considérée comme nulle ou insignifiante dans la pratique de la géotechnique en raison de sa valeur relativement faible et le manque d'une technique de laboratoire satisfaisante. La résistance à la traction de sol est, cependant, un paramètre important dans la conception des géosystèmes, où des fissures de traction contribuent à l'érosion progressive des glissements de terrain dans l'excavation, les pistes, les barrages routiers, des remblais, des rives, des barrières hydrauliques, et d'autres structures terrestres. Dans cet article, un développement récent de l'essai d'indentation est présenté. Le cadre théorique est réévalué, suivi d'une série de résultats d'essais sur le sable légèrement cimenté.

KEYWORDS: tensile strength, indentation, limit analysis.

1 INTRODUCTION

Determination of tensile strength of soil can be categorized to direct and indirect methods. In the direct method, the soil specimen is directly pulled apart and assumed to split in the middle (e.g. Das and Dass 1995, Tang and Graham 2000, Nahlawi et al. 2004, Tamrakar et al. 2007, Zeh and Witt 2007). Its tensile strength is computed as the measured maximum pull-up force divided by the cross sectional area. However, it is normally quite challenging to have the soil specimen clamped or glued in the split mold. In the indirect method, tensile failure is induced by compressive load. A well-known Brazilian test is of such kind. The indentation test, also called unconfined penetration tests (Fang and Chen 1972, Fang and Fernandez 1981, and Kim et al. 2012) is an indirect method for tensile strength determination. It uses a pair of cylindrical metal indenters, or punches, to compress a cylindrical soil specimen as shown in Figure 1. The tensile strength is computed through the equation developed from the limit analysis (e.g. Chen 1975). The test gives an applied compressive axial load and indenter displacement curve. The maximum load is identified for further data reduction to computer the tensile strength. The limitation of this test method is that a certain level of brittleness of the specimen is required so that a split tension failure would occur.

2 BACKGROUND

As described in Chen (1975), an ideal failure mechanism developed in the soil specimen can be presented in Figure 1. By equating the external work with the internal work, an upper bound solution can be obtained as follows.

$$\frac{p^u}{\pi a^2} = \frac{1 - \sin \phi}{2 \sin \alpha \cos(\alpha + \phi)} \frac{q_u}{2} + \tan(\alpha + \phi) \left(\frac{bH}{a^2} - \cot \alpha \right) q_t \quad (1)$$

p^u is the upper bound axial compressive load that causes the split tension failure. a is the radius of the indenter. b is the radius of the soil specimen. ϕ is the internal frictional angle of the soil. α is the developed angle underneath the indenter when failure occurs. q_u is the unconfined compressive strength while q_t is the tensile strength to be determined.

The upper bound solution has a minimum value when $\delta p^u / \delta \alpha = 0$, where

$$\frac{p^u}{\pi a^2} = \left[\frac{bH}{a^2} \tan(2\alpha + \phi) - 1 \right] q_t \quad (2)$$

By equating (1) and (2), the equation below can be obtained,

$$\cot \alpha = \tan \phi + \sec \phi \left[1 + \frac{\frac{bH \cos \phi}{a^2}}{\frac{q_u}{q_t} \left(\frac{1 - \sin \phi}{2} \right) - \sin \phi} \right]^{1/2} \quad (3)$$

Substituting (3) into (1) and let p is the maximum compressive load applied from the test, which is less than the upper bound load, p^u . Therefore,

$$p \leq p^u = \pi \left[bH \tan(2\alpha + \phi) - a^2 \right] q_t \quad (4)$$

$$q_t = \frac{p}{\pi (bHK - a^2)} \quad (5)$$

where $K = \tan(2\alpha + \phi)$

The value of K is challenging to determine in laboratory testing as it is a function of the internal frictional angle of the soil, the unconfined compressive-tensile strength ratio, as well as the size of the indenter. The recommended values of K based

upon a given assumed materials properties, such as the brittleness ratio, $q_u/q_t = 10$, $\phi = 30^\circ$, and $b/H = 1$ were suggested for practical use (Fang and Fernandez, 1981). However, they may not be suitable for other material setting. In this study, we carried out unconfined compression tests and direct shear tests to acquire information on q_u and ϕ . Afterwards, from our indentation test results, we can iteratively compute both values of the K and the tensile strength q_t .

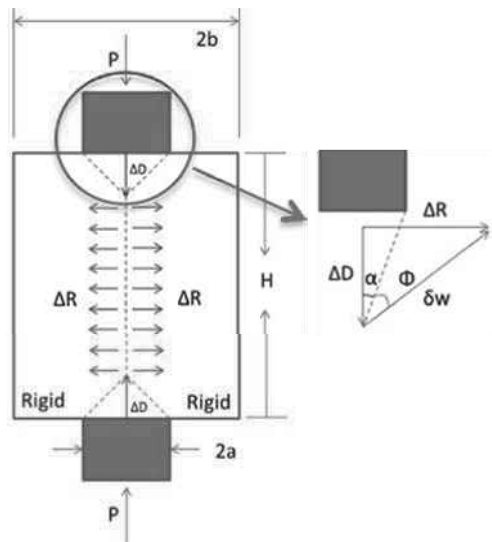


Figure 1. Limit analysis of indentation test

3 TESTING PROGRAM

To assess the theoretical framework for indentation test described above, nine different test configurations were presented in this paper. The diameter of each specimen, $2b$, was kept as 100 mm. The height of each specimen, H , was chosen as 50, 100, and 150 mm respectively. The ratio of the indenter diameter to specimen diameter, $2a/2b$, was selected as 0.225, 0.281, and 0.338 respectively. Replication for one test configuration indicated that the results of the indentation tests are repeatable within 5% variation. Therefore, only the results for each test configuration were shown here.

The specimens were compacted silica sand-Kaolinite mixture. As we would like to have a lightly cemented sandy specimen, which is brittle enough for the indentation tests, the combination of 15% dry weight of Kaolinite plus 85% dry weight of sand were selected after several trials. The water was then added to reach moisture content of 8.1% followed by sealing the soil mixture in a bag resting for 24 hours. After compaction, the dry density was about 1.62 g/cm³. The specimen was mounted in the testing apparatus shown in Figure 2. An indentation rate of 0.5 mm/min was applied to each test. The indenter penetration and the corresponding compressive load were recorded for data reduction. An unconfined compression test on a specimen with 100 mm in diameter and 200 mm in height was conducted as well as direct shear tests under normal stress of 40, 80, and 160 kPa.

4 RESULTS AND DISCUSSION

Figures 3-5 show the load-displacement curves for specimen height 50 mm, 100 mm and 150 mm respectively. In each figure, three curves represent the results of different sizes of indenter. All nine curves show softening after peak load, indicating split tension failure mode occurred. Before computing the tensile strength, unconfined compression tests were conducted with a specimen height-to-diameter ratio 2

followed the ASTM standards. The q_u was found to be around 16336.5 N/m². A internal frictional angle $\phi = 30.8^\circ$ was determined through a series of direct shear tests.

With the information of peak axial force p , q_u , and ϕ , tensile strength, q_t and the corresponding K value can be determined through the iterations using equations 3 and 5. Table 1 shows the iterated q_t and K associated with each test configuration.

From the test results show in Table 1, it is recognized that the tensile strength of the lightly cemented sand is specimen size dependent. For a given specimen height, the higher the indenter-to-specimen diameter ratio, a/b , the smaller the tensile strength is. The developed cone angle α is also found specimen and indenter size dependent. For a given specimen height, the α angle is in proportional to indenter size. For a given indenter size, higher specimen had smaller developed angle α . The coefficient K is also found dependent on test configuration. The trend for K to specimen height and indenter size is similar to the trend for the α angle. The computed K values fall between 1.5 and 3.5 depending on test configuration as displayed in Figure 6. This is not consistent with the recommended values, where 1.0 to 1.2 for soil was presented in Fang and Chen (1972). The major difference between Fang and Chen's work and the current study lies in the brittleness ratio, q_u/q_r . In Fang and Cheng's pioneer work, the brittleness ratio was assumed to be 10 for compacted soil. In the current study, the brittleness ratio was determined to be around 16 to 20, as shown in Figure 7. Looking back on equation 5, the tensile strength is dependent on K value, which is dependent on the developed cone angle α . Also, the angle α is a function of brittleness ratio as shown in equation 3. As a result, using an iterated value for K seems more attractive as the brittleness for soil could have a wide range.

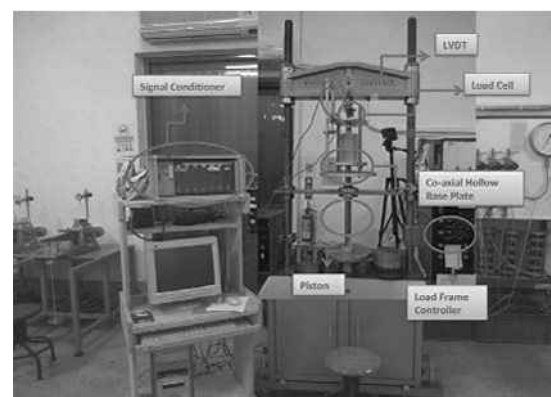


Figure 2. Indentation tensile strength testing apparatus

Table 1. Summary of test results

H (mm)	$2a/2b$	α ($^{\circ}$)	p (N)	K	q_i (N/m ^{2})
50	0.225	15.9	15.7	1.93	1061.1
50	0.281	18.9	18.6	2.55	955.3
50	0.338	21.4	22.1	3.42	849.1
100	0.225	12.6	23.5	1.49	1020.2
100	0.281	14.9	27.7	1.77	1013.2
100	0.338	18.1	29.7	2.36	819.9
150	0.225	11.1	29.5	1.33	951.8
150	0.281	13.1	34.5	1.55	961.0
150	0.338	15.7	37.6	1.90	853.8

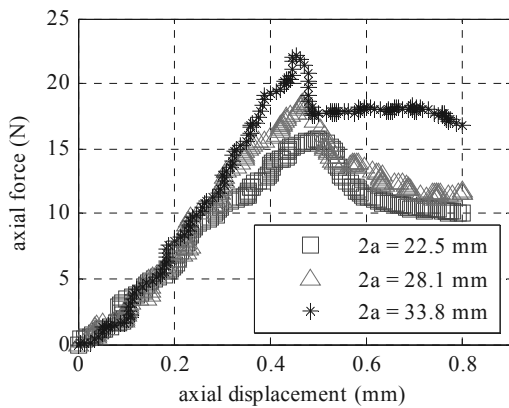


Figure 3. Indenter load-displacement curve for 50 mm-high specimens

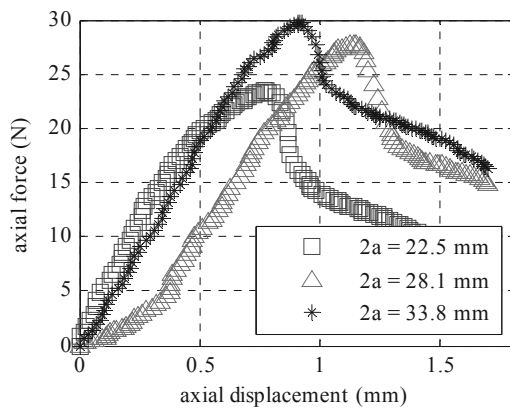


Figure 4. Indenter load-displacement curve for 100 mm-high specimens

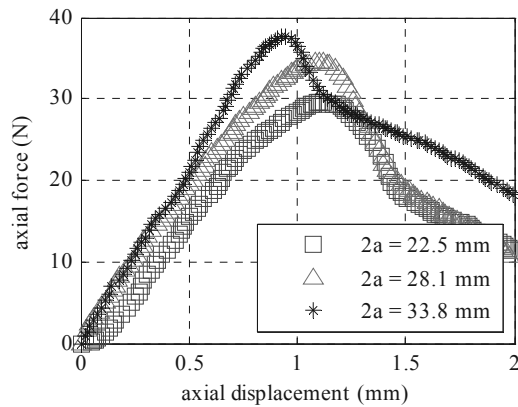


Figure 5. Indenter load-displacement curve for 150 mm-high specimens

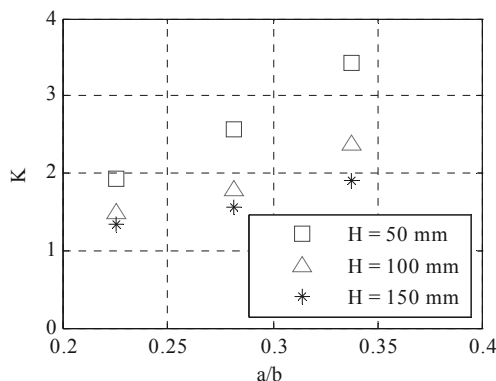


Figure 6. The coefficient K vs. the indenter-to-specimen diameter ratio

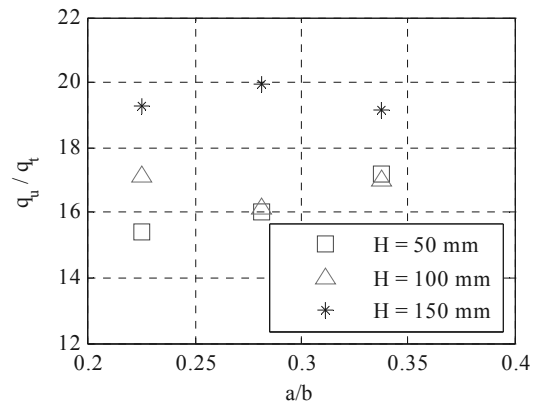


Figure 7. The brittleness vs. the indenter-to-specimen diameter ratio

5 CONCLUSIONS

An indirect method for determining tensile strength of lightly cemented sand was re-visited. The method is based upon an upper bound solution to a split tension failure in limit analysis. In order to assess the analytical solution, three different sizes of indenters were used along with three different specimen heights. As the coefficient K is a function of the developed α angle and the brittleness ratio, evaluating K through a numerical iteration is recommended.

6 ACKNOWLEDGEMENTS

The financial support from the National Science Council of Taiwan under the contract of 101-2218-E-002-005- is greatly appreciated.

7 REFERENCES

- Das, B.M. and Dass, R.N. 1996. Lightly cemented sand in tension and compression. *Geotechnical and Geological Engineering* 13, 169-177.
- Tang, G. X., and Graham, J. 2000. A method for testing tensile strength in unsaturated soils. *Geotechnical Testing Journal* 23(3), 377-382.
- Nahlawi, H., Chakrabarti, S., and Kodikara, J. 2004. A direct tensile strength testing method for unsaturated geomaterials. *Geotechnical Testing Journal* 27(4), 356-361.
- Tamrakar, S.B., Mitachi, T. and Toyosawa, Y. 2007. Measurement of soil tensile strength and factors affecting its measurements. *Soils and Foundations* 47(5), 911-918.
- Zeh R.M. and Witt, K.J. 2007. The tensile strength of compacted clays as affected by suction and soil structure. In: *Experimental Unsaturated Soil Mechanics*, Edt. T. Schanz, Springer-Verlag, Berlin, Heidelberg, 219-226.
- Chen, W.F. 2007. *Limit Analysis and Soil Plasticity* (electronic resource). J. Ross Publication, Fort Lauderdale, FL. Originally published in 1975, Elsevier, Amsterdam.
- Fang, H.Y. and Chen, W.F. 1972. New method for determination of tensile strength of soils. *Highway Research Record*, 354, 62-68.
- Fang, H.Y., and Fernandez, F. 1981, Determination of tensile strength of soils by unconfined-penetration test. *ASTM STP* 740, 130-144.
- Kim, T.-H, Kim, T.-H, Kang, G.-C, and Ge, L. 2012. Factors influencing crack-induced tensile strength of compacted soil. *Journal of Materials in Civil Engineering* 24(3), 315-320.

Mechanisms During Formation of Ice Lenses and Suction in Freezing Soils

Les mécanismes de la formation des lentilles de glace et de succion au cours de la congélation du sol

Herzog F., Boley C.
Universität der Bundeswehr, Munich, Germany

ABSTRACT: In the course of frost penetration in the soil the formation of ice lenses may result under certain boundary conditions. Due to the development of suction, water is drawn from unfrozen parts to the frost line. In this paper an apparatus for conducting experiments is presented which serves to determine the ice lens building process as well as the suction. Within the scope of the test program different dependencies are analyzed for understanding the mechanisms of suction development and for making predictions on the formation of ice lenses possible. The test results show the connection between suction development and ice lens formation. It also shows that even without external water supply reallocation processes take place.

RÉSUMÉ : La pénétration du givre dans le sol contribue, sous certaines conditions limites, à la formation de lentilles de glace. A cause du développement du processus de succion, l'eau des parties non gelées migre vers le front de gel. Cet article présente un centre d'essai, qui permet, d'une part, d'effectuer les tests sur la formation de lentilles de glace et, d'autre part, d'analyser le processus de succion. Dans le cadre du programme expérimental, plusieurs dépendances ont été analysées afin de mieux comprendre les mécanismes de formation de succion et d'améliorer les prédictions sur la formation de lentilles de glace. Les résultats de ses expériences montrent un lien entre le développement du processus de succion et la formation des lentilles de gel, et que, sans apport supplémentaire d'eau en dehors de l'échantillon, le processus de migration peut avoir lieu.

KEYWORDS: frozen soil, ice lenses, suction, frost heave

1 INTRODUCTION

In the course of artificial and natural ground freezing, when the frost penetrates into the ground, in many cases, frost heaves occur. If they are not well controlled (Kellner et al 2006) they may lead to damages to buildings that are close by. Thus, the objective is to predict frost heaves and preferably to minimize them.

Depending on the existing boundary conditions, the heaves are caused for two different reasons. Generally, there is an increase in volume of 9 % caused by the transformation of water into ice. Moreover, ice lenses may develop for fine-grained soils. During the frost penetration into the ground water migrates from the unfrozen areas of the soil towards the frost line. There, the water is accumulated in layers of pure ice that splits the ground. The reason for the water migration is a suction which can be measured when the water migration is impeded.

The fact that the fundamental processes may change the thawed soil properties makes the prediction of the expected frost heaves and their potential minimization even more substantial.

In the past, many experiments were conducted to investigate different influences on the formation of ice lenses (Konrad and Morgenstern 1980, 1982). Until now, the assessment of the frost susceptibility of a soil was based on the particle size distribution and the plastic soil properties but did not take into account the influences of the mineralogy or the ion concentration of the pore water. Nor had the analysis of the suction as cause of the water migration to the ice lens been paid particular attention. However, in recent research the influence of overburden pressure has been investigated (Kellner 2008) with its influence on pore water development due to freezing.

Nevertheless, the underlying mechanisms of suction and their influencing factors are not yet finally resolved. For this reason freezing tests are run at our institute to investigate the

development of ice lenses and determine the suction. Our institute has developed a model of osmotic pressure (Zou and Boley 2008) to describe the mechanisms. Based on these tests, the model can be verified and improved. The freezing tests are presented in the following chapters.

2 EXPERIMENTAL APPARATUS AND PROCEDURE

Figure 1 shows the test apparatus to run freezing tests investigating the development of ice lenses as well as determining the suction.

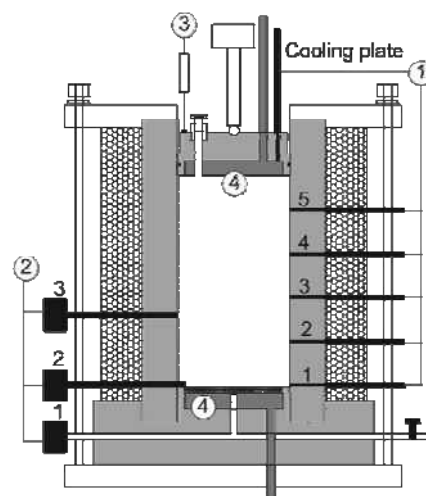


Figure 1. Schematic of freezing cell: (1) temperature sensors 1-5 and temperature sensor at upper cooling plate, (2) pore water pressure transducer, (3) displacement transducer, (4) cooling plate

Test program investigates the influence of the applied surcharge, material type and the ion concentration of the pore water on the suction and development of ice lenses. The test apparatus essentially consists of an acrylic glass cylinder with an inner diameter of 80 mm. The two cooling plates (4) allow setting up a defined temperature gradient in the specimen. The upper sealing and loading is realized through a piston. The ice lens experiments are conducted in open-system freezing, meaning that the specimen is connected to a storage vessel and hence it has free access to water. To investigate the suction, the water access is disabled so the system changes into a closed-system freezing test. The temperature sensors (1) are distributed over the height and record the time depending temperature development. To determine the suction in the closed-system, pore water pressure transducers (2) are used. The frost heave is measured with a displacement transducer (3).

In advance, the test material is mixed with deionized water so that the swelling and ion exchange processes have taken place prior to commencement of the experiments. Subsequently, the ion composition of the water is determined. The material is filled into the cylinder in pasty condition and is consolidated in the test apparatus by applying a defined surcharge. For the closed-system freezing tests, the valve for the water supply is shut. The step-freezing method is used to freeze the specimen. Therefore, the temperature of the upper cooling plate is lowered to a defined value meanwhile the temperature of the lower cooling plate is still held constant. Thus, the frost migrates top down, starting fast and slowing down over time until thermal steady state is reached. The cylinder is lubricated with a high vacuum silicon grease to reduce the friction between the specimen and the acrylic glass. At the end of the test, the specimen is pushed out of the cylinder to saw them into slices. This allows determining the distribution of the water content over the specimen height.

3 MATERIALS TESTED AND TESTING PROGRAMM

The development of ice lenses and suction during freezing strongly depend on the respective material. Different materials are chosen to investigate the influence of grain-size distribution and the mineralogy: kaolin (K), bentonite (TS), silt (S), limestone powder (KK) and quartz powder (Q) were chosen as fine grained materials. Also, tests with fine sand (Fs) were conducted. Figure 2 shows the grain size distribution and further soil properties are given in Table 1.

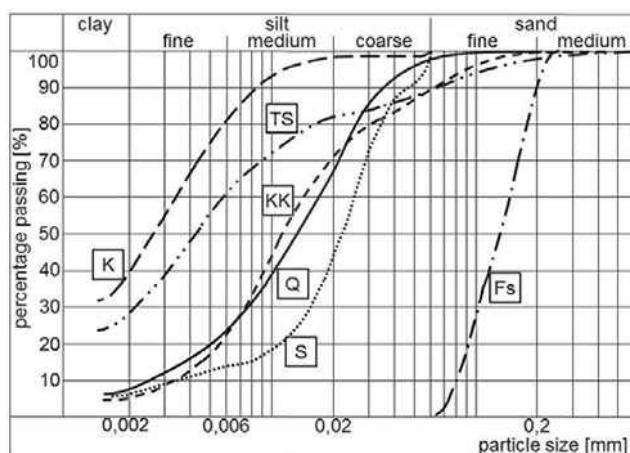


Figure 2. Grain-size distribution curve of the tested materials

Within the scope of the test program, different dependencies shall be analyzed. Primarily, the influence of material on the development of ice lenses as well as on the suction is analyzed. Influences of the applied surcharge will be investigated by applying loads of 20, 50, 100, 300 and 600 kN/m². Besides the

normal consolidated specimens with the loads as stated, overconsolidated specimens were tested as well. Further, the tests are conducted with 2 different temperature gradients between the cooling plates. The influences of the ion concentration in the pore water on the suction and the development of ice lenses are analyzed by comparing the results of tests with natural ion concentration in the pore water and those with particularly enriched ion concentration in the water. The natural ion concentration results from the exchange between dried material and the deionized water.

Table 1. Soil properties of the tested materials

	<i>K</i>	<i>TS</i>	<i>S</i>	<i>KK</i>	<i>Q</i>	<i>Fs</i>
Liquid limit w_L [%]	55,6	134,7	22,7	-	-	-
Plastic limit w_P [%]	24,6	37,1	21,5	-	-	-
Hydraulic conductivity k [m/s]	$1,5 \cdot 10^{-9}$	$1,6 \cdot 10^{-10}$	$1,1 \cdot 10^{-8}$	$4,0 \cdot 10^{-8}$	$2,9 \cdot 10^{-8}$	$1,5 \cdot 10^{-6}$
(surcharge 100 kN/m ²)						
Grain density ρ_s [g/cm ³]	2,63	2,73	2,70	2,73	2,66	2,65
Cation exchange capacity Γ [meq/100g]	4,7	88,9	6,1	2,4	0,9	0,3
Specific surface area A_s [m ² /g]	10,1	44,4	6,3	2,5	1,1	0,02

4 TEST RESULTS

In the following, the results of the experiments with varying materials in combination with different surcharges will be presented. The specimens have a natural ion concentration in the pore water and are frosted with a temperature gradient of 1.9 °C/cm.

4.1 Frost penetration

Figure 3 shows the temperature distribution over the specimen at different times exemplarily for one test.

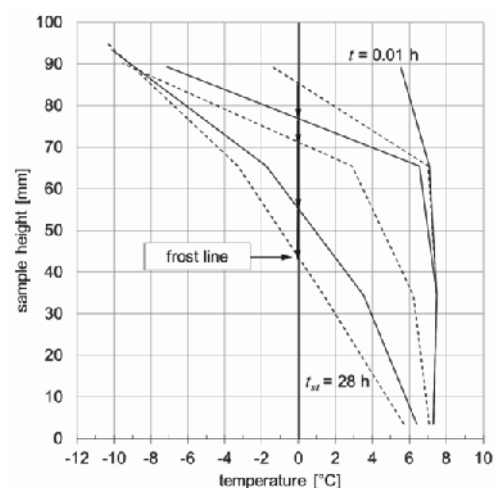


Figure 3. Temperature profile within the specimen (kaolin) at different times, from right to left: 0.01 h, 0.07 h, 0.15 h, 0.83 h, 4.17 h, 28 h

The temperature of the upper cooling plate drops rapidly to the set temperature meanwhile the temperature of the lower cooling plate is kept constant. The temperatures at the other sensors are decreasing time delayed until the thermal steady state is established after the time t_{st} . Based on the diagram the position of the frost line and their temporal shift can be identified. The frost line is the transition between the frozen upper sections and the unfrozen part. The position stays constant by reaching the time t_{st} . The fast frost penetration at the beginning is linked to a sharp temperature gradient. Only within the steady state the requested temperature gradient appears.

4.2 Ice lens formation in open-system freezing

Figure 4 shows the different ice lens formation in open system freezing for selected materials. The biggest ice lenses develop in the transition zone between frozen and unfrozen areas in the thermal steady state.

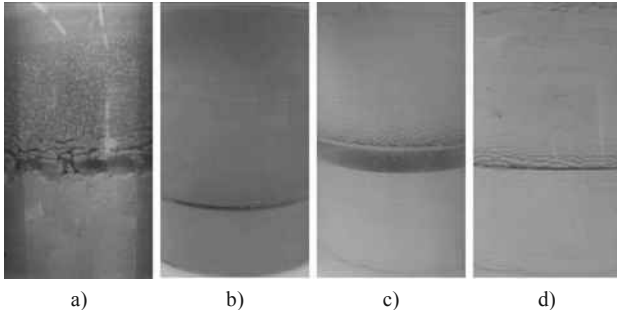


Figure 4. Specimens at the end of open system freezing tests: a) TS (100 kN/m²), b) S (20 kN/m²), c) K (20 kN/m²), d) K (100 kN/m²)

Under the chosen test conditions, the greatest changes can be observed for the bentonite (a) due to its intense capacity to draw water to the frost line and also due to the initially high water content. Water migrates from the storage vessel into the specimen as a result of frost penetration. Furthermore, the existing water in the specimen is reallocated initially resulting in fine distributed ice lenses (dark areas) and with a slowing frost penetration in a development of bigger ice lenses. In contrast, the Kaolin (c) tests show only little structural changes. In the upper part only little water enrichment is noticeable and it is smoother distributed throughout the height. Comparing the results for silt (b) and kaolin (c) with a surcharge of 20 kN/m² it is apparent that the final ice lens can grow thicker for kaolin than for silt. The comparison of ice lens formation in kaolin with a surcharge of 20 kN/m² (c) and 100 kN/m² (d) clearly shows that a higher surcharge is restricting the formation. In tests with quartz powder and fine sand no water migration into the specimen was observed. Tests with limestone powder have shown little water intake and reallocation but it is too less to be visible on the picture.

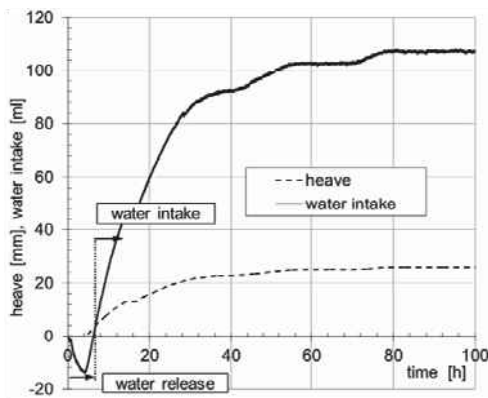


Figure 5. Changes in water content in the specimen and displacement with time for kaolin (20 kN/m²)

Correlating with the development of ice lenses are the changes of water content in the specimen and the displacement (heaves). Figure 5 shows the results of a test with kaolin with a surcharge of 20 kN/m². Initially, water is expelled until it is water drawn in the specimen and a noticeable heave builds up. This effect was also observed in investigations of Konrad and Morgenstern (Konrad and Morgenstern 1982). In the course of the experiment, the kaolin specimen absorbed approximately 122 ml of water. In contrast, the water content for a limestone powder specimen under the same test conditions is at the end lower than at the beginning of the test. This is due to the fact

that the absorbed amount of water is not compensating for the water that was initially expelled.

4.3 Processes in open-system freezing

The test series aims at investigating the suction as principle for the water flow and the development of ice lenses. For a test with kaolin, Figure 6 shows the development of suction and heave in the course of frost penetration. Simultaneously with the beginning of the freezing process and movement of the frost line into the specimen suction, builds up. Its maximum of 0.88 bar is already reached after approximately 8.5 hrs. The thermal steady state is reached after 16 hrs. The course of the heave follows the temperatures. The initially fast frost penetration is reflected in a strong increase of heave. With the slowing of frost penetration the heave curve flattens. After the thermal steady state has been reached the further increase of the heave is only marginal.

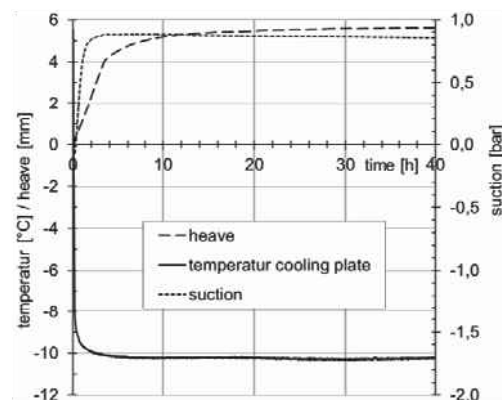


Figure 6. Development of suction and heave for kaolin due to drop of temperature of the cooling plate (surcharge 20 kN/m², overconsolidated, closed system)

Even in closed-system freezing reallocation processes emerge within the specimen due to the suction that develops at the frost line. Under the assumption that water is incompressible and no air is in the system, the water can technically not flow towards the frost front. If the adjacent unfrozen soil could be consolidated further under the influence of suction, it is possible that water can be drawn to the frost line from these unfrozen areas. The amount of water transported in this way is limited by the predefined water content of the specimen and therefore not necessarily visible. However, sawing the specimen allows determination of the water content distribution over the specimen.

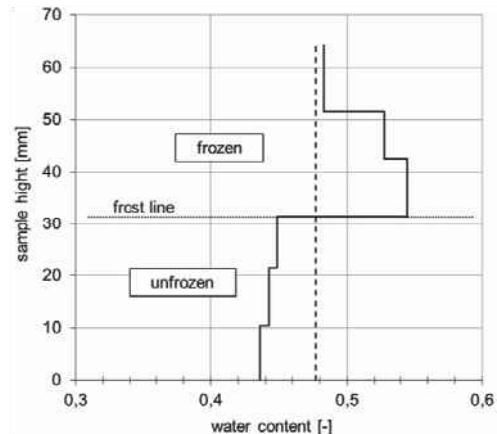


Figure 7. Water content distribution for a test with kaolin with an applied surcharge of 300 kN/m² (closed system)

Figure 7 shows the water distribution for a test with kaolin (300 kN/m²). Shown as the dashed vertical line is the initial

water content. The horizontal line describes the border between the frozen and unfrozen areas at the final state of the test. With the onset of freezing the suction increases and the water is drawn into the area of the frost line. The slower the frost penetrates, the more water can reach the frost line. This leads to an increasing water content from the top down and small ice lenses become visible. In the closed system, the consolidating of the unfrozen area leads to a reduction of the initial water content. This water redistribution is also the reason for the slight heave increase after reaching the steady state.

Analyzing suction in a closed system is limited by laws of physics. If the water pressure falls below the atmospheric pressure or increases the suction, the boiling temperature decreases and the water changes its physical state from liquid to vapor even at lower temperatures. This can occur in free pore water as well as in the water of the pressure transducers. Since gas is able to expand in vacuum, the pore water pressure will change if a gas bubble builds up in the system. In order to delay this process, the water used for the tests is conditioned in a vacuum to release dissolved gases in advance. Nevertheless, suctions greater than 0.9 bar are difficult to reach. After reaching the maximum suction and the potential development of gas bubbles the measured suction drops. The water reallocation that occurs in closed systems is supported by the gas forming, because additional water can flow to the frost front when the gas expands.

The magnitude of the measured suction in closed-system freezing varies significantly between the different materials. In tests with fine sand, no suction could be measured. Accordingly no water intake was observed in tests with an open system. The results for quartz powder only show a low suction. Therefore, the consistency of the results will be verified in further tests. In general the highest suction values were measured for kaolin with absolute values around 0.9 bar and bentonite with 0.8 to 0.9 bar. Also silt reached high suction values of 0.4 to 0.9 bar. Lower values were measured in the tests with limestone powder in the range of 0.2 to 0.6 bar.

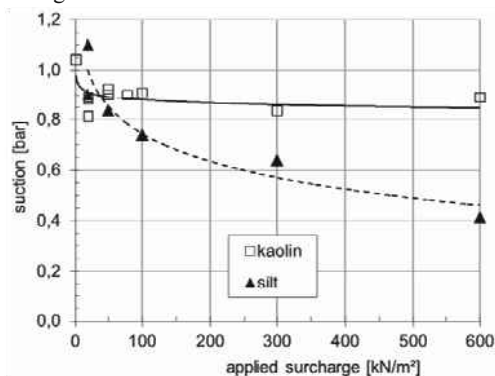


Figure 8. Dependency of the maximum suction on the applied surcharge

Figure 8 shows a comparison of the suction values for kaolin and silt with the same surcharge. A clear correlation between the maximum suction and the surcharge can be deduced from the results of silt. Hence, the higher surcharge is not only hindering the intake of water into the specimen in the open system, but also influencing the suction as principle of the water flow. Constantly high ultimate suction values greater than 0.8 bar were measured in the tests with kaolin under different surcharges. Compared to the results of silt, kaolin shows a stronger suction for surcharges greater than 50 kN/m². This also explains the less intense water reallocation of silt in a closed system and as well as the lower amount of water intake in the open system compared to the same tests done with kaolin. For kaolin under the same test conditions, the ultimate suction values show no correlation with the surcharge. The extremely high suction values suggest that the greatest possible suction could not be reached in these experiments due to the changing

aggregate state of the water that is finally limiting the measurement. Consequently, the theoretical possible suction for kaolin is greater than 1 bar, as long as the water has not passed from liquid into a gaseous state.

5 CONCLUSIONS

An experimental apparatus was presented to run tests determining the suction in closed systems and analyzing ice lenses development in open systems. It was demonstrated, that in materials allowing a development of ice lenses as well as an intake of water into the specimen in open-system freezing, a corresponding suction could be measured in the closed system. The test data is used to verify and extend the theoretical model. Besides the dependency of suction on the applied surcharge also the correlation between suction and ion concentration in the pore water and in particular, the material specific properties such as the specific surface area and the surface charge shall be taken into account. Accordingly, further soil properties need to be determined as well as the ion concentration in the free pore water. In the main the magnitude of the cation exchange capacity corresponds to the magnitude of the measured suction. Transferring the results from closed system to open system and to ice lenses development the determining factor for the water migration is suction. However, the hydraulic permeability is crucial for the amount of water that is moved. Bentonite shows a pronounced suction but in comparison to kaolin or silt under the same surcharges only little water can be moved in a certain time due to the lower coefficient of hydraulic conductivity.

6 ACKNOWLEDGEMENTS

The authors would like to thank the Deutsche Forschungsgemeinschaft for supporting this research project.

7 REFERENCES

- Kellner, C., Vogt, N., Orth, W., Konrad, J.-M. (2006): Ground freezing: an efficient method to control the settlements of buildings, In: *Proceedings of the International Conference on simulation of Construction Processes*. Editor Triantafyllidis, Bochum
- Kellner, C. 2008: Frosthebungsverhalten von Böden infolge tief liegender Vereisungskörper, Schriftenreihe Lehrstuhl und Prüfam für Grundbau, Bodenmechanik, Felsmechanik und Tunnelbau der Technischen Universität München, Heft 42
- Konrad, J.-M. and Morgenstern, N.R. 1980: A mechanistic theory of ice lens formation in fine-grained soils, *Canadian Geotechnical Journal* (17), 473-486
- Konrad, J.-M. and Morgenstern, N.R. 1982: Effects of applied pressure on freezing soils, *Canadian Geotechnical Journal* (10), 494-505
- Zou, Y. and Boley, C. 2008: Der Gefriersog bei der eindimensionalen Frosteindringung, *Grundlagen und Anwendungen der Geomechanik GKK 08 - Geomechanik Kolloquium Karlsruhe*, Veröffentlichungen des Instituts für Bodenmechanik und Felsmechanik der Universität Friderician in Karlsruhe (179) Teil 2, 231-242

Comportement en petites déformations d'un sol traité à la chaux

Small strain behavior of a lime-treated soil

Hibouche A., Taibi S.

Laboratoire LOMC, CNRS UMR 6294, Université du Havre, 76600 Le Havre, France

Fleureau J.-M.

Laboratoire MSS-Mat, CNRS UMR 8579, Ecole Centrale Paris, 92295 Châtenay-Malabry, France

Herrier G.

Lhoist Recherche et Développement s.a., 1400 Nivelles, Belgique

RÉSUMÉ : L'étude des sols en petites déformations ($\varepsilon \sim 10^{-5}$) nécessite des mesures locales du champ de déformation. Dans le cas des sols traités aux liants, leur comportement en petites déformations dépend de l'état initial, du degré de saturation et de l'âge du matériau traité. L'article présente une étude expérimentale sur des matériaux naturels traités aux liants et compactés. Le dispositif expérimental est composé d'une cellule triaxiale équipée de deux types de capteurs de proximité pour mesurer les déformations locales: des jauges de déformations et des capteurs à Effet Hall. Une étude comparative de ces deux systèmes de mesure est réalisée. Les modules élastiques déduits des cycles dans le domaine des déformations inférieures à 10^{-4} ont été comparés avec ceux déduits des mesures des vitesses d'ondes élastiques. Des relations entre les modules, les niveaux de déformation et l'âge du matériau traité sont ainsi établies. Les domaines d'application concernent les ouvrages de terrassements d'infrastructure en sols traités à la chaux.

ABSTRACT: The study of soils in small strains ($\varepsilon \sim 10^{-5}$) requires local measurements of the strain field. In the case of soils treated with binders, their behavior depends on the initial state, the degree of saturation and the hardening time. The paper presents an experimental study on natural materials treated with binders and compacted. The experimental device consists of a triaxial cell equipped with two types of proximity sensors to measure local strains: strain gauges and Hall Effect sensors. A comparative study of the two systems is performed. The elastic moduli derived from cycles in the strain field less than 10^{-4} were compared with those deduced from the elastic wave speed. Relations between the modules, the strain levels and the hardening time are thus established. Applications include lime-treated infrastructure earthwork.

Mots-clés: Sols traités, chaux, petites déformations, non saturation, triaxial, effet Hall

Keywords: Treated soils, lime, small strains, unsaturation, triaxial, Hall effect

1 INTRODUCTION

La technique du traitement des sols à la chaux est connue, principalement dans les domaines routiers et ferroviaires, pour améliorer la maniabilité des sols, faciliter leur mise en oeuvre et leur conférer, après compactage, des performances mécaniques répondant aux exigences des ouvrages. La compréhension du comportement des matériaux en petites déformations est nécessaire du point de vue pratique. En effet, les déformations de service dans la plupart des ouvrages restent généralement très faibles (typiquement inférieures à 10^{-4} dans les routes et 10^{-3} dans les fondations). La littérature souffre d'un manque de données expérimentales sur la rigidité des sols dans ce domaine, en particulier des sols non saturés (Brull 1980, Wu et al. 1989, Quin et al. 1991, Kheirbek-Saoud 1994, Gomes Correia et al. 1987). Aussi, une analyse plus rationnelle basée sur le comportement mécanique des sols, en particulier en ce qui concerne les changements dans la succion matricielle (ou pression capillaire) est nécessaire (Fleureau et al., 2001, 2002, Taibi et al. 2008).

Différentes techniques de mesures des petites déformations ont été développées depuis une trentaine d'années. On distingue essentiellement deux techniques (Dufour-Laridan 2001): les méthodes quasi-statiques et les méthodes dynamiques. Concernant les techniques quasi-statiques, différents types de capteurs ont été développés. Jardine et al. (1984) ont développé un montage de précision en compression se basant sur des inclinomètres « electrolevel ». Cuccovilo & Coop 1997, cités par Cabarkapa et al., 1999 proposent l'utilisation de LVDT (Linear Variable Differential Transformer), qui sont des capteurs inductifs linéaires de déplacement miniatures submersibles pour mesurer les déformations axiales et radiales

locales jusqu'à 10^{-6} . Plusieurs auteurs choisissent d'utiliser les capteurs de proximité pour mesurer la déformation des échantillons. Ces capteurs de proximité fonctionnent sur le principe d'induire un courant dans une cible conductrice par un champ électromagnétique et de mesurer le champ électromagnétique induit par ce courant. La mesure se fait sans contact (Santucci de Magistris et al., 1999, Hoque et al., 1997).

Clayton & Khattrush (1986), ont développé un capteur de déplacement basé sur l'effet Hall pour mesurer des petites déformations, de l'ordre de $2 \cdot 10^{-5}$, directement sur les échantillons. Les LDT (Local Deformation Transducer), développés par Goto et al. (1991), utilisent comme composant de mesure des jauges de déformations locales de l'ordre de 10^{-6} . Araújo et Gomes Correia (2009) ont adapté ces capteurs LDT pour la mesure des déformations axiales et diamétrales sur des chemins triaxiaux isotropes et déviatoriques.

Contrairement aux méthodes quasi-statiques, les méthodes dynamiques mettent en jeu des variations de vitesse de déformation suffisamment élevées pour que les forces d'inertie soient non-négligeables dans la relation fondamentale de la dynamique. Elles font intervenir des propagations d'ondes stationnaires ou non: Géophones (Modoni et al., 1999), bender elements (Dyvik & Madhus, 1985; Kuwano et al., 1999; Fioravante, 2000), colonne résonante (Cascante & Santamarina 1997, Tatsuoka et al., 1997, Tatsuoka et al., 1999).

Dans cet article, les résultats d'une étude expérimentale de la mesure des modules élastiques en petites déformations sur un limon traité à la chaux compacté sont présentés. Le but de cette étude est d'évaluer la possibilité d'utiliser un capteur de proximité composée d'une jauge de déformation collée à même l'échantillon. Les résultats sont comparés avec ceux obtenus à l'aide d'un capteur à effet Hall.

2 MATÉRIAUX ET MÉTHODE

2.1. Matériaux

Il s'agit d'un limon traité à la chaux provenant d'un remblai expérimental compacté in situ.

La chaux utilisée est une chaux vive calcique CL 90-Q fournie par Lhoist, conforme à la norme EN 459-1.

Les caractéristiques physico-chimiques du matériau sont résumées dans le tableau 1.

2.2. Méthodes

Pour la mesure du comportement du matériau en petites déformations, deux techniques expérimentales ont été testées:

La première technique est basée sur l'utilisation de capteurs à effet Hall, composés d'une plaque dans laquelle on fait circuler un courant électrique tout en mesurant la différence de potentiel sur une ligne transversale à la circulation du courant (Figure 1). Si un champ magnétique normal à la plaque est appliqué, les électrons du courant seront déviés vers l'un ou l'autre des points dont on mesure le potentiel, créant ainsi une accumulation de charges en ce point et donc une tension mesurable (Dufour-Laridan 2001).

La seconde technique est basée sur l'utilisation de jauges de déformation. Il s'agit de composants électroniques ayant une résistance variable en fonction de leur elongation dans la direction de mesure. L'originalité de cette technique est que les jauges sont collées à même le matériau. Le problème posé habituellement avec les géomatériaux dont la nature granulaire ne permet souvent pas de collage direct a été résolu en réalisant à la surface du matériau une interface très souple dans la zone de mesure sur laquelle vient se coller la jauge. L'interface est composée d'un mastic étalé dans une rainure préalablement taillée dans le matériau. La figure 2 présente un schéma de principe de cette technique.

Tableau 1. Caractéristiques physico-chimiques du limon naturel et traité à la chaux

Limon naturel	
Passants à 80 µm (%)	98
Valeur de bleu de méthylène (g/100g)	2.50
Indice de plasticité (%)	7 à 8
Teneur en eau à l'échantillonnage (%)	17.9
Limon traité à 2.5% de chaux	
Teneur en eau finale (%)	19.5
Densité sèche après compactage (g/cm ³)	1.68
Taux de compactage (% $\gamma_{d\text{OPN}}$)	97

3 COMPARAISON DES DEUX TECHNIQUES

Des essais ont été réalisés sur un échantillon prélevé depuis un remblai expérimental constitué de limon traité à la chaux compacté et âgé de 180 jours (Tableau 1). L'échantillon de 80 mm de diamètre et 160 mm de hauteur, est équipé des deux capteurs de proximité pour la mesure locale de la déformation axiale : deux capteurs à effet Hall et deux jauges de déformation diamétralement opposées. Pris séparément, les deux capteurs à effet Hall et les deux jauges donnent des réponses quasi identiques. En revanche, on constate une différence de réponse entre les capteurs à effet Hall et les jauges de déformation.

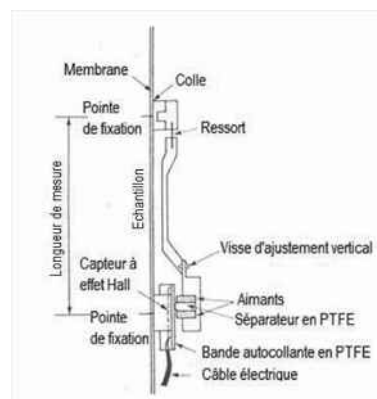


Figure 1. Principe de fixation d'un capteur à effet Hall sur l'échantillon

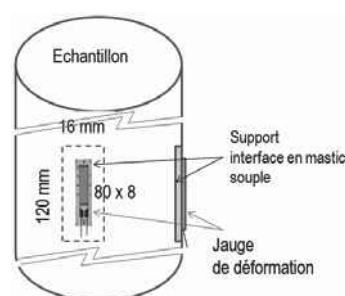


Figure 2. Principe de fixation d'une jauge de déformation sur l'échantillon

Les résultats comparatifs entre la jauge et le capteur à effet Hall sont représentés dans les plans [temps ; déplacement axial] et [déformation axiale ; déviateur de contrainte], et ce, pour deux contraintes de confinement différentes (Figure 3).

On constate pour les deux contraintes de confinement que le capteur à effet Hall mesure des amplitudes de déformation plus importantes que la jauge. Par ailleurs, on remarque que la réponse de la jauge est quasi instantanée lors des cycles de décharge-recharge, alors que le capteur à effet Hall affiche un retard dans sa réponse matérialisé par des pics légèrement arrondis dans le plan [temps ; déformation axiale], contrairement aux pics vifs des jauges. L'interprétation de ces résultats dans le plan [déformation axiale ; module d'Young] (Figure 4) montre que les modules élastiques donnés par la jauge sont nettement supérieurs (environ trois fois) à ceux donnés par le capteur à effet Hall. Par ailleurs, la mesure des vitesses de propagation des ondes sonores sous confinement nul donne un module du même ordre de grandeur que celui donné par la jauge.

Ce constat permet de remettre en cause le bon fonctionnement du capteur à effet Hall. En effet, la fixation de ce capteur est basée sur l'introduction d'épingles métalliques dans un matériau compacté et rigidifié après 180 jours par le traitement à la chaux. Cette opération est délicate et génère généralement des trous de diamètre légèrement supérieur à celui des épingles, ce qui se traduit par un léger jeu des points de fixation qui peut avoir des conséquences importantes sur la réponse du capteur.

Dans ce qui suit, nous avons sélectionné et retenu le capteur basé sur les jauges de déformation et qui semble plus fiable et conforme aux mesures des vitesses des ondes sonores.

4 RÉSULTATS

Ces résultats concernent des essais triaxiaux en petites déformations à teneur en eau constante sur des échantillons non saturés. Les contraintes de confinement varient de 0 à 300 kPa. Pour chaque contrainte de confinement, des chargements cycliques sous des déviateurs de plus en plus importants avec

mesure locale des déformations axiales sont effectués. Le tableau 2 résume les contraintes appliquées et les chemins déviatoires suivis. La teneur en eau initiale moyenne du matériau est de 20%.

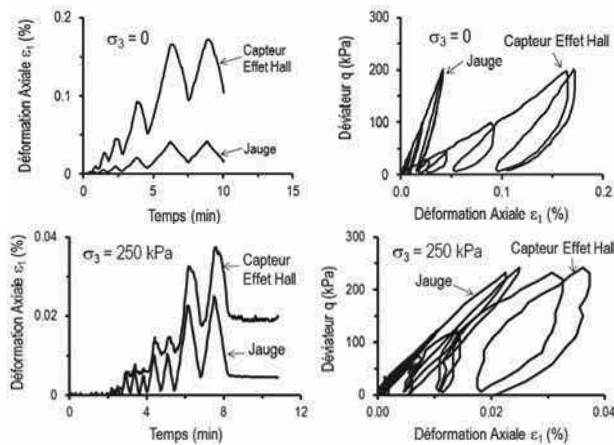


Figure 3. Comparaison des réponses des deux capteurs lors de chargement en petites déformations pour deux contraintes de confinement.

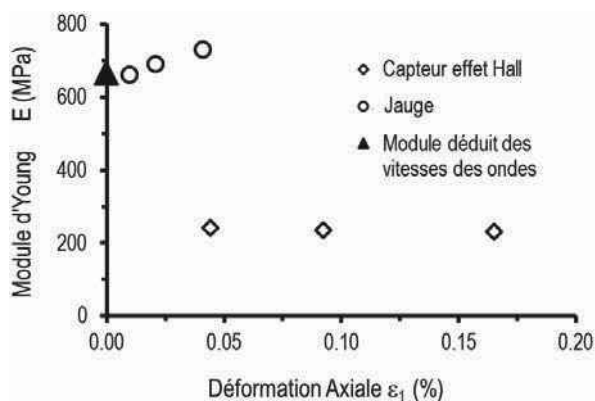


Figure 4. Variation du module élastique en fonction de la déformation axiale et comparaison avec les mesures déduites des vitesses des ondes sonores, dans le cas d'une contrainte de confinement nulle.

Tableau 2- Chemins déviatoires suivis

Contrainte de confinement totale σ_3 (kPa)	Cycles sous déviateur q (kPa) pour chaque valeur de σ_3
0 → 300 par increment de 50 kPa	Cycle 1- 0 → 50 → 0 (2 cycles) Cycle 2- 0 → 100 → 0 (2 cycles) Cycle 3- 0 → 200 → 0 (2 cycles)

Les figures 5 et 6 présentent les chemins de chargement déviatoire des matériaux âgés respectivement de 28 et 180 jours. Les résultats concernent à titre d'exemple les essais réalisés sous une contrainte de confinement égale à 150 kPa.

Pour chacun des cycles réalisés à un niveau de déviateur donné, le module élastique est mesuré en considérant la pente moyenne du cycle déchargement-rechargement.

Pour interpréter ces essais, les figures 7 et 8 présentent pour les deux échéances, la variation du module élastique en fonction respectivement de la contrainte totale de confinement et de la déformation axiale pour différentes valeurs de confinement et différents déviateurs. Sur ces courbes, la valeur du module initial déduite des mesures des vitesses de propagation des ondes est représentée sur l'axe des ordonnées. On remarque qu'il y a un bon accord avec les valeurs déduites de l'essai

triaxial. Par ailleurs, pour les niveaux des déviateurs testés, où les déformations ε_1 résultants n'excèdent pas 0,045% (Figure 8), le module baisse légèrement avec le niveau de déformation pour une contrainte de confinement donnée sauf dans le cas où la contrainte de confinement est nulle, on remarque que le module reste quasi-constant et égal à la valeur déduite des vitesses de propagation des ondes. Ces observations sont valables dans le plan [q,E] (Figure 9). En revanche, le module augmente globalement avec la contrainte de confinement, évoluant de 0,68 GPa pour une contrainte nulle à 1,34 GPa pour un confinement de 300 kPa.

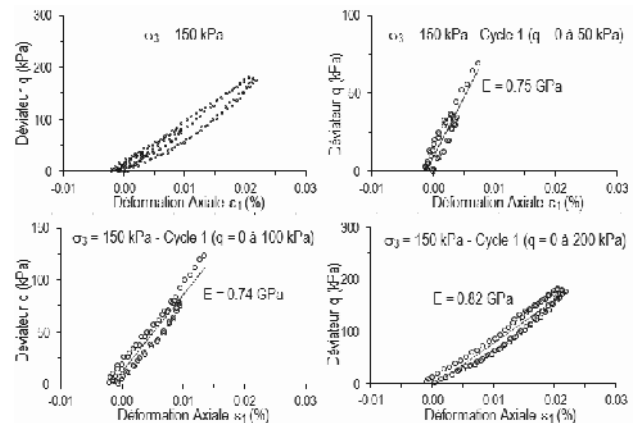


Figure 5. Evolution du module en fonction du déviateur de contrainte à 28 jours d'âge, pour une contrainte de confinement de 150 kPa

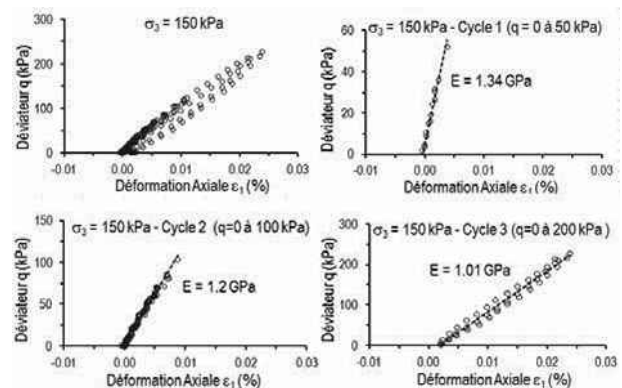


Figure 6. Evolution du module en fonction du déviateur de contrainte à 180 jours d'âge, pour une contrainte de confinement de 150 kPa.

5 CONCLUSION

La mesure des modules élastiques en petites déformations dans les sols traités reste une opération complexe et délicate, liée aux conditions aux limites géométriques de fixation des capteurs sur un sol rigidifié.

Les résultats présentés ont consisté d'abord en une comparaison de trois techniques de mesure différentes, qui a mis en évidence les limites des capteurs à effet Hall lorsqu'il s'agit de les fixer sur un sol traité et donc rigidifié.

La technique de mesure mise au point, basée sur les jauges de déformation collées à même l'échantillon via une interface élastique, semble cohérente avec les mesures indirectes basées sur les vitesses de propagation des ondes sonores.

Les résultats obtenus sur un limon traité à la chaux ont montré que le module augmente avec la contrainte de confinement et l'âge du matériau. En revanche, il varie très peu avec le niveau de déformation dans le domaine de contraintes testées.

6 REMERCIEMENTS

Les auteurs remercient le FEDER (Fonds européen de développement régional) et le groupe Lhoist pour leur soutien et leurs encouragements.

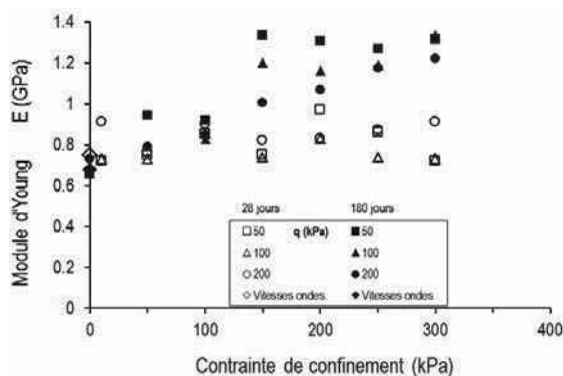


Figure 7. Evolution du module en fonction de la contrainte totale de confinement pour différentes valeurs du déviateur de contrainte.

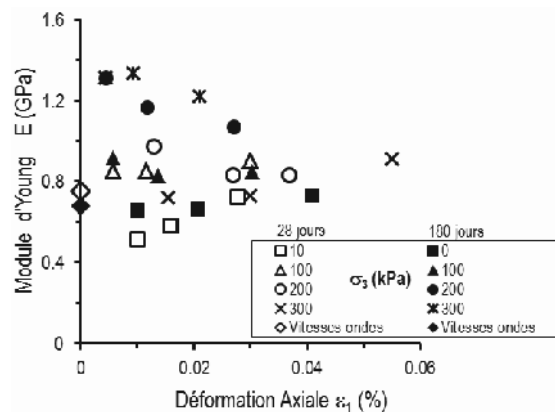


Figure 8. Evolution du module en fonction de la déformation axiale pour différentes valeurs de contrainte de confinement.

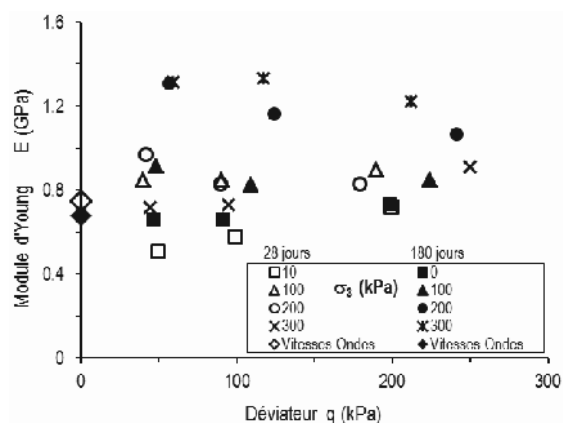


Figure 9. Evolution du module en fonction du déviateur de contrainte pour différentes valeurs de contrainte de confinement.

7 RÉFÉRENCES

Araújo N. and Gomes Correia A. 2009. Precision triaxial equipment for the evaluation of the elastic behaviour of soils. *Proceedings of the 8th International Conference on the Bearing Capacity of Roads and Airfields*. Taylor & Francis Group. 125-134.

Brull A. 1980. Caractéristiques mécaniques des sols de fondation de chaussées en fonction de leur état d'humidité et de compacité.

Compte-rendus du Colloque International sur le Compactage, Paris, Presses des Ponts & Chaussées, (1), 113-118

Cabarkapa Z., Cuccovillo T. and Gunn M. 1999. Some aspects of pre-failure behavior of unsaturated soil. *Pre-failure Deformation Characteristics of Geomaterials*, 159-165.

Cascante G. and Santamaria C. 1997. Low Strain Measurements Using Random Noise Excitation. *Geotechnical Testing Journal* 20 (1), 29-39.

Clayton C. and Khatriush S.A. 1986. A new device for measuring local axial strains on triaxial specimens. *Géotechnique* 36 (4), 593-597.

Dufour-Laridan Emmanuel, 2001. Propriétés mécaniques des sols en petites déformations ; étude expérimentale d'un sable silteux. *Thèse de doctorat, Ecole Centrale Paris, France*

Dyvik R. and Madhus C. 1985. Laboratory measurements of Gmax using bender elements. *Advances in the engineering, NGI publication* (161), 117-137.

Fioravante V. 2000. Anisotropy of small strain stiffness of Ticino and Kenya sands from seismic wave propagation measured in triaxial testing. *Soils and Foundations* 40 (4), 129-142.

Fleureau J.M., Dufour-Laridan E. and Gomes Correia A. 2001. Influence of compaction and loading conditions on the dynamic properties of a silty sand. *Proceedings of the 4th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA, Edited by S. Prakash, Paper 1.30*

Fleureau J.M., Hadiwardoyo S., Dufour-Laridan E., Langlois V. and Gomes Correia A. 2002. Influence of suction and loading conditions on the dynamic properties of a silty sand. *Proceedings of the 3rd International Conference on Unsaturated Soils, Recife (Brasil), Edited by J. Juca, T. de Campos and F. Marinho, Lisse, Balkema, Vol. 2, 463-472.*

Gomes Correia A., Livet M. and Guillin J. 1987. The influence of hydric state on the mechanical behaviour of a sand under cyclic loading. *Bulletin de Liaison des Laboratoires des Ponts et Chaussées, Paris*, 152

Goto S. Tatsuoka F., Shibuya S., Kim Y.S. and Sato T. 1991. A simple gauge for local small strain measurements in laboratory. *Soils and Foundations* 31 (1), 169-180.

Hoque E., Sato T. and Tatsuoka F. 1997. Performance evaluation of LDTs for use in triaxial tests. *Geotechnical Testing Journal* 20 (2), 149-167.

Jardine R.S., Symes M.J. and Burland J.B. 1984. The measurement of Soil stiffness in the triaxial apparatus. *Géotechnique* 34 (3), 323-340.

Kheirbek-Saoud S. 1994. Comportement mécanique du ballast et de la couche de fondation d'une voie ferrée. *PhD Thesis, Ecole Centrale Paris, France*

Kuwano R., Connolly T.M. and Kuwano J. 1999. Shear stiffness anisotropy measured by multi-directional bender element transducers. *Pre-failure Deformation Characteristics of Geomaterials*, Rotterdam : Balkema, 205-212.

Modoni G., Flora A., Mancuso C., Anh Dan L. Q., Koseki J., Balakrishnaier K. and Tatsuoka F. 1999. A simple experimental procedure for the complete characterization of small strain stiffness of gravels. *Pre-failure Deformation Characteristics of Geomaterials*, Rotterdam : Balkema, 123-130.

Quin X., Gray D.H. and Woods R.D. 1991. Resonant column tests on partially saturated sands. *Geotechnical Testing Journal, ASCE*, 14(3), 266-275

Santucci de Magistris F., Koseki J., Amaya M., Hamaya S., Sato T. and Tatsuoka F. 1999. A triaxial testing system to evaluate stress-strain behavior of soils for a wide range of strain and strain rate. *Geotechnical Testing Journal* 22, 44-60.

Taibi S., Fleureau, J.M., Hadiwardoyo and Kheirbek-Saoud, S. 2008. Small and large strain behaviour of an unsaturated compacted silt. *European Journal of Environmental and Civil Engineering*. 12(3) 203-228

Tatsuoka F., Jardine R.J., Lo Presti D., Di Benedetto H. and Kodaka T. 1997. Characterising the pre-failure deformation properties of geomaterials. *XIV ICSMFE in Hamburg*, 1-36.

Tatsuoka F., Modoni G., Jiang G.L., Anh Dan L.Q., Flora A., Marsushita M. and Koseki J. 1999. Stress-strain behavior at small strains of unbounded granular materials and its measurements in laboratory. *Modelling and advanced testing for unbonded granular materials in Lisboa*, 1-34.

Wu S., Gray D.H. and Richart F.E. 1989. Capillary effects on dynamic modulus of sands and silts. *Journal of the Geotechnical Engineering Division, ASCE*, 110(9), 1188-1203.

A Key Parameter for Strength Control of Lightweight Cemented Clays

Un paramètre clé pour le contrôle des forces de légères argiles cimentées

Horpibulsuk S., Suddeepong A., Chinkulkijniwat A.

School of Civil Engineering, Suranaree University of Technology, Nakhon Ratchasima, Thailand

ABSTRACT: Lightweight cemented clays have wide applications in the infrastructure rehabilitation and in the construction of new facilities. The void/cement ratio, V/C , which is defined as the ratio of the void volume of clay to the cement volume, is proved as the prime parameter governing the strength and compression characteristics of lightweight cemented clays. The fabric (arrangement of clay particles, clusters and pore spaces) reflected from both air foam content and water content is taken into consideration by the void volume while the inter-particle forces (levels of cementation bond) are governed by the input of cement (cement volume). Three types of clay namely, kaolin, Bangkok clay and bentonite as representatives of non- to high swelling clays were used in this study. A strength equation in terms of V/C at a particular curing time is introduced using Abram's law as a basis.

RÉSUMÉ : Les légères argiles cimentées ont de larges applications dans la réhabilitation des infrastructures et de la construction de nouvelles installations. Le rapport de vide/ciment, V/C , qui est défini comme le rapport du volume de vide de l'argile au volume de ciment, comme le prouve le paramètre premier régit la force et les caractéristiques de compression des argiles légères cimentées. Le tissu (arrangement de particules d'argile, des grappes et des espaces de pores) à la fois réfléchi par le contenu de mousse à air et la teneur en eau est prise en compte par le volume de vide, tandis que les forces inter-particulaires (niveaux de liaison cimentation) sont régis par l'entrée de ciment (ciment volume). Trois types d'argile, à savoir le kaolin, l'argile de Bangkok et la bentonite en tant que représentants d'une quantité d'argiles gonflantes ont été utilisés dans cette étude. Une équation de la force en termes de V/C à un temps de durcissement particulier est introduite par la loi d'Abram comme base.

KEYWORDS: compressibility, lightweight cemented clay strength, void/cement ratio.

1 INTRODUCTION

When infrastructures such as road embankments and bridge foundations are constructed on soft soil deposits, several geotechnical engineering problems are encountered. These deposits tend to consolidate and undergo large vertical settlement and lateral deformation during and after construction due to incumbent loads. The use of lightweight materials with unit weight of 8 to 12 kn/m^3 and moderate to high strength as a backfill material to reduce the weight of the structure on the soft clay is an effective means. Lightweight materials have wide applications in the infrastructure rehabilitation and in the construction of new facilities. They can be used as a backfill for quay walls and bridge abutments to reduce the earth pressure behind the wall, as a fill for construction of embankments on soft soil to reduce overburden pressure, as a method of reducing pressure on the tunnel lining.

For soft clay admixed with cement, the clay-water/cement ratio, w_c/C was proved as the prime parameter governing engineering properties (Miura et al., 2001; Horpibulsuk and Miura, 2001 and Horpibulsuk et al., 2005). Horpibulsuk et al. (2003; 2011a, b and 2012) successfully employed this parameter to develop a generalized strength equation based on Abrams' law (Abrams, 1918). The equation is useful for laboratory mix design. This parameter was also successfully used to predict the strength development in cement stabilized coarse-grained soils on the wet side of optimum water content wherein the degree of saturation is higher than 80% (Horpibulsuk et al. 2006 and Chinkulkijniwat and Horpibulsuk, 2012). Consoli et al. (2007) extended the clay-water/cement ratio hypothesis to analyze the strength development in

compacted (unsaturated) cement-stabilized sand. They proposed a key parameter taking the role of air bubble in pore space (void) on the strength development into account. The parameter was designated as void/cement ratio, V/C and was defined as the ratio of absolute volume of void (water and air) to absolute volume of cement of the compacted sand.

Even though the cement stabilized soil have very low water content due to compaction effect, it is composed of water, air, soil solid and cement, which is the same as lightweight cement clay. The modified clay-water/cement ratio (void/cement ratio, V/C), is thus considered to describe the engineering properties of lightweight cemented clays which possess very high water contents in this study. Three types of clay namely, kaolin, Bangkok clay and bentonite as representatives of non- to high swelling clays were used for this objective. Finally, the strength equation is proposed based on the parameter V/C .

2 MATERIALS AND METHODS

Bangkok clay was collected from Bangkok Noi district, Bangkok, Thailand at a 3 meter depth. Kaolin and bentonite were obtained from a commercial company. Bangkok clay was composed of 2% sand, 39% silt and 55% clay. The natural water content was 78% and the specific gravity was 2.64. The liquid and plastic limits were 73% and 31%, respectively. Based on the Unified Soil Classification System (USCS), the clay was classified as inorganic clay of high plasticity (CH). Groundwater was had a depth of about 1.0 m from surface. The undrained shear strength was 12 kPa. Kaolin was composed of 0% sand, 22% silt and 78% clay. The specific gravity was 2.65. The liquid and plastic limits were 46% and 36%, respectively. The clay was classified as inorganic clay of low plasticity (CL)

based on the USCS. Bentonite was composed of 0% sand, 50% silt and 50% clay. The specific gravity was 2.63. The liquid and plastic limits were 106% and 60%, respectively. It was classified as inorganic clay of high plasticity (CH).

The clay paste was passed through 2-mm sieve for removal of shell pieces and other bigger size particles, if present. The water content was adjusted to (2-5) times liquid limit. This intentional increase in water content is to simulate the clay slurry with high flow ability for pumping into the construction sites. Even with very high water content up to 5 times liquid limit, all tested clays still have viscosity with low magnitude, indicating that the sorting of the grain size does not occur. The clays were mixed with air foam to attain air contents, A_c , between 10 and 100% by volume of the clay-water-air mixture. This mixture was then thoroughly mixed with cement for 10 min. The cement content, C , was varied from 150 to 400 kg/m³ of the mixture. To verify the V/C as a prime parameter, the cement content and air content were varied to attain the V/C values of 30 and 10. Such a uniform paste was transferred to oedometer rings as well as to cylindrical containers of 50 mm diameter and 100 mm height, taking care to prevent any air entrapment. After 24 hours, the cylindrical samples were dismantled. All the cylindrical samples and oedometer samples were wrapped in vinyl bags and they were stored in a humidity room of constant temperature (20±2°C) until lapse of different curing times as planned. Oedometer tests were carried out after 14 days of curing. Unconfined compression (UC) tests were run on samples after 7 and 14 days of curing. The rate of vertical displacement in UC tests was 1 mm/min. Both tests were performed according to the American Society of Testing and Materials (ASTM) standards.

3 VOID/CEMENT RATIO, V/C

In cement admixed clay, the clay-water/cement ratio hypothesis (Horpibulsuk and Miura, 2001; Horpibulsuk et al., 2005; and Miura et al., 2001) is stated as follows:

"For given cement admixed clay, age and curing conditions, the strength is determined exclusively by the ratio of clay-water content to the cement content in the mix. Strength is independent of clay-water content and cement content in the mix."

As an analogy, the parameter that can be identified for lightweight cemented clays is void/cement ratio, V/C , which is the volume of void to the volume of cement in the mix. The parameter can be simply determined using four phase diagram of soil, water, air and cement (Horpibulsuk et al., 2012b). To obtain the same value of V/C for a particular clay water content, it is possible to vary the amount of air foam or cement or both as the case might be. In order to examine up to what extent the applicability of V/C is valid, the air foam content is varied over a wide range ($A_c = 10$ -50% of clay volume) in this study.

4 TEST RESULTS

The role of V/C on the compressibility is shown in Figures 1 and 2 for lightweight cemented kaolin and bentonite samples with the same V/C values but with different combinations of cement content and air content. The samples were made up from six conditions of air content namely, 0, 10, 20, 30, 40 and 50%. Figure 1 shows the compressibility of lightweight cemented kaolin at water content of 88%. Figure 2 shows the compressibility of lightweight cemented bentonite at water content of 280%. They show the $(e, \log \sigma'_v)$ and $(\epsilon_v, \log \sigma'_v)$ relations of the samples at V/C values of 30 and 10 after 14 days of curing. The resistance to compression prevails up to a certain stress level beyond which the sample experiences increase in compression. This stress level is identified as yield stress (Horpibulsuk et al., 2005). It does not represent pre-consolidation pressure because the cemented clay was not being

subjected to any stress history. The $(\epsilon_v, \log \sigma'_v)$ relationship is plotted so as to take care of the effect of the difference in void ratio for the vertical stresses less than the yield stress. For a certain water content, the yield stress and the deformation behavior in pre-yield stress of all samples with identical V/C values are practically the same. This implies that V/C is a prime parameter governing the compressibility in pre-yield state. The yield stress increases as the V/C value decreases. The samples with higher air content are stable at higher void ratios. Beyond the yield stress, drastic compression occurs as vertical pressure increases due the breakup of cementation bond (Horpibulsuk et al., 2004a, Horpibulsuk et al., 2010; Liu and Carter, 1999, 2000 and 2002; and Suebsuk et al., 2010 and 2011).

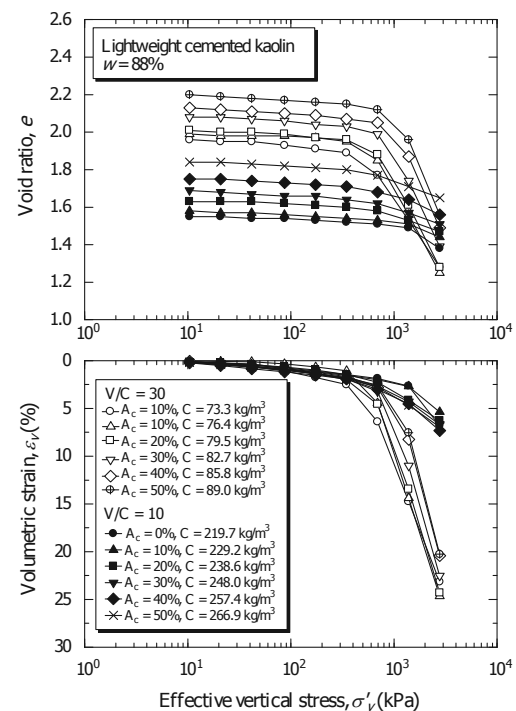


Figure 1. Compressibility of air-cement-admixed kaolin at $w = 88\%$.

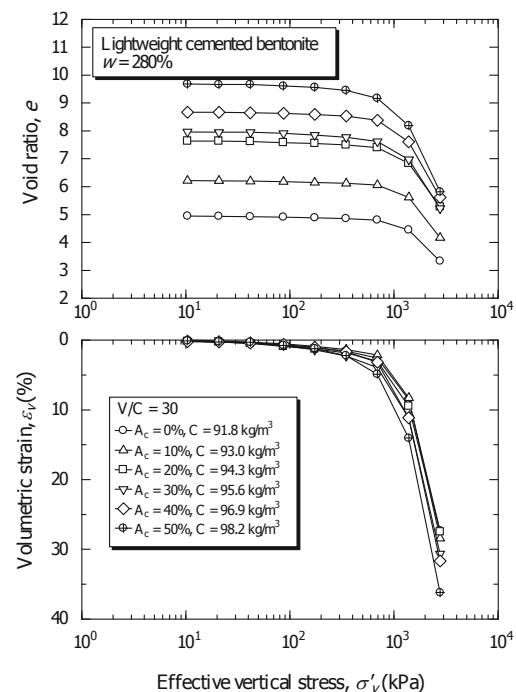


Figure 2. Compressibility of air-cement-admixed bentonite at $w = 280\%$.

Figures 3 to 5 show the stress-strain relationships in unconfined compression tests of samples with different air contents and cement contents but with the same V/C values of 30 and 10, at 14 days of curing. Figure 3 is for the lightweight cemented kaolin at water content of 88%. Figure 4 is for the lightweight cemented Bangkok clay at water contents of 136%. Figure 5 is for the lightweight cemented bentonite at water contents of 170%. It is noted that as V/C decreases, the cementation bond strength increases and hence strength. The lightweight cemented samples with the same void/cement exhibit the similar stress-strain behavior. To conclude, the V/C controls compressive strength and compression characteristic in pre-yield state for a particular water content, while the unit weight does not, which is different from natural clays. The fabric (arrangement of clay particles, clusters and pore spaces) reflected from both air foam content and water content is taken into consideration by the void volume while the inter-particle forces (levels of cementation bond) are governed by the input of cement (cement volume).

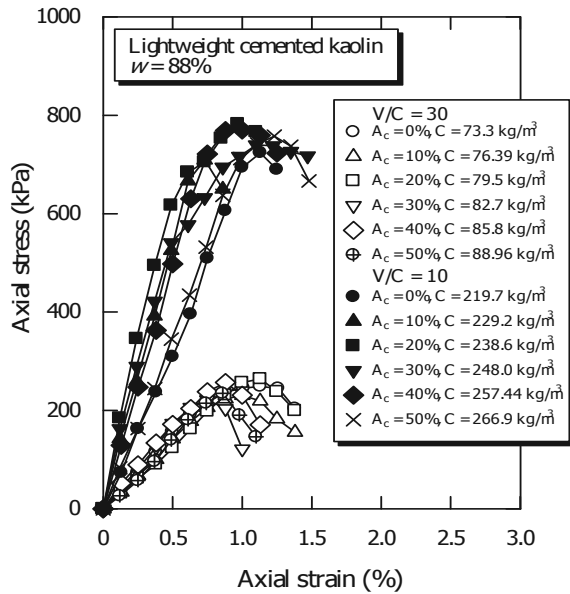


Figure 3. Stress-strain relationship of air-cement-admixed kaolin at $w = 88\%$.

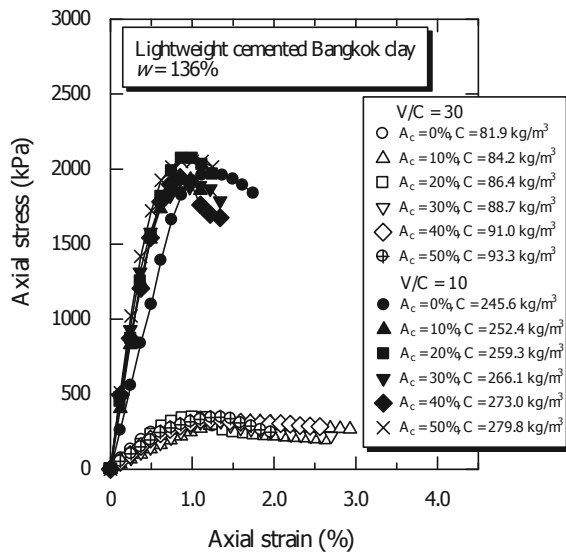


Figure 4. Stress-strain relationship of air-cement-admixed Bangkok clay at $w = 136\%$.

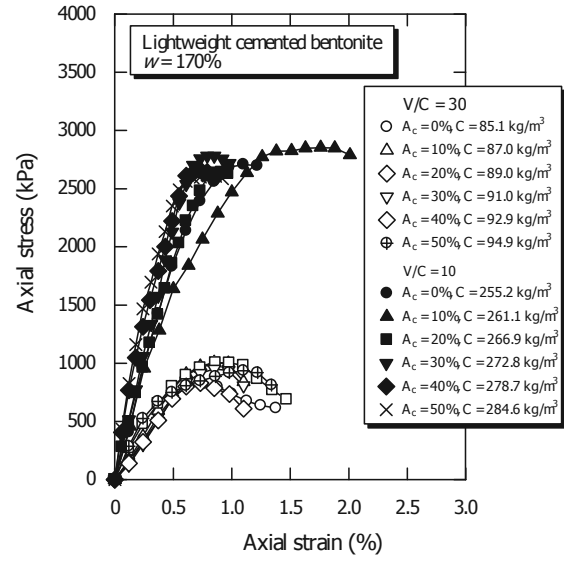


Figure 5. Stress-strain relationship of air-cement-admixed bentonite at $w = 170\%$.

Figures 1 to 5 show the advantage of using air form to produce the lightweight cemented clay. The strength reduction is observed when the void increase by air form due to the increase in contact area per cementation bond. However, the role of air content on the strength development is less significant than that of cement and water. This is because the hydration is strongly depended on the water/cement ratio in the mix. In other words, by using the air form, the unit weight is significantly reduced with less strength reduction.

Because the V/C is the prime parameter governing the engineering properties in elastic range (at low effective confining stress), it is possible to develop a relationship between strength and V/C for a particular curing time. Figure 6 shows the relationship between strength and V/C at 7 days of curing of the lightweight cemented Bangkok clay as an example to guarantee the applicability of the V/C . The unique relationship between strength and V/C can be found for a given initial water content at different cement contents and air contents. Based on the experimental observations ($5 < V/C < 40$ and 7 days of curing), it is possible to advance the following identity:

$$\left\{ \frac{V_1}{C_1} \right\} = \left\{ \frac{V_2}{C_2} \right\} = \text{Constant} \quad (1)$$

Once the void/cement ratio is fixed in the field, if the air content (void volume) is changed to achieve the required unit weight, the cement content can be estimated from Eq.(1) to attain the same strength and compressibility characteristics. For a mix design purpose, the relationship between strength and V/C at a certain water content is advanced on the basis of Abrams' law (1918):

$$q_u = \frac{A}{(V/C)^B} \quad (2)$$

where q_u is the unconfined compressive strength, V/C is the void/cement ratio, and A and B are constants. This equation when $A_c = 0$ yields the same equation proposed by Horpibulsuk et al. (2011a, b and 2012a). The A -value is dependent upon the clay type, curing time and air content. As the water content increases, the A -value decreases. The B -value is practically constant and equal to 1.26 to 1.29, which is the typical values

for cemented non- to low-swelling clays (Horpibulsuk et al., 2011b). It was suggested to take the B -value as 1.27 for the cemented non- to low-swelling clays (Horpibulsuk et al., 2011a, b and 2012a). To employ Eq. (2) for assessing the strength of any lightweight cemented clay at different void/cement ratios (air content and cement content), the parameters A and B must be predetermined. This task can be achieved by a back-calculation of at least two trial strength data.

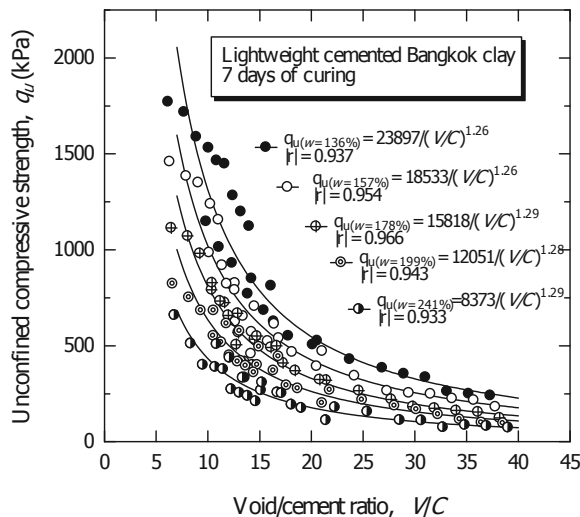


Figure 6. Analysis of strength development in lightweight cemented Bangkok clay using V/C .

5 CONCLUSIONS

Based on this study, it is brought out that the void/cement ratio is the prime parameter for analysis of strength and deformation behavior of lightweight cemented clays with non- to high swelling potential. This parameter takes into account the influence of both clay fabric reflected by the air volume and the level of cementation. The conclusion can be drawn as follows.

1. For a given soft clay at a particular water content, the cementation bond strength increases as void/cement ratio, V/C decreases. Consequently, the yield stress in K_0 -consolidation and compressive strength increases with the decrement of V/C . The stress-strain response and compression characteristics in pre-yield state are practically the same as long as the V/C value is identical.

2. Based on the void/cement ratio and Abram's law, a relationship between strength, void/cement ratio for a particular water content and curing time (Eq.2) is proposed. The relationship is useful in estimating the laboratory strength wherein air content and cement content vary over a wide range by a few trial tests. It also facilitates the determination of proper quantity of cement to be admixed for different air contents to attain the target strength. The formulation of the proposed relationship is on sound principle and developed from distinct clays (non- to high swelling clays). The A and B values can be determined by a back-analysis of at least two trial strength data. It is thus possibly applicable for various clays.

6 ACKNOWLEDGEMENTS

This work was supported by the Higher Education Research Promotion and National Research University Project of Thailand, Office of Higher Education Commission. The financial support and facilities that were provided by the Suranaree University of Technology were appreciated.

7 REFERENCES

- Gibson R.E. and Henkel D.J. 1954. Influence of duration of tests at constant rate of strain on measured "drained" strength. *Géotechnique* 4 (1), 6-15.
- Darcy H. 1856. *Les fontaines publiques de la ville de Dijon*. Dalmont, Paris.
- Abrams D.A. 1918. Design of Concrete Mixtures. In: Structural Materials Research Laboratory, Lewis Institute, Chicago, Bulletin 1, 20p.
- Chinkulkijniwat A. and Horpibulsuk S. 2012. Field strength development of repaired pavement using the recycling technique. *Quarterly Journal of Engineering Geology and Hydrogeology* 45 (2), 221-229.
- Consoli N.C., Foppa D., Festugato L. and Heineck K.S. 2007. Key parameters for strength control of artificially cemented soils. *Journal of Geotechnical and Geoenvironmental Engineering ASCE* 133 (2), 197-205.
- Horpibulsuk S. and Miura N. 2001. A new approach for studying behavior of cement stabilized clays. *Proc. 15th International Conference on Soil Mechanics and Geotechnical Engineering (ISSMGE)*, Istanbul, Turkey, Vol. 3, 1759-1762.
- Horpibulsuk S., Miura N. and Nagaraj T.S. 2003. Assessment of strength development in cement-admixed high water content clays with Abrams' law as a basis. *Géotechnique* 53 (4), 439-444.
- Horpibulsuk S., Bergado D.T. and Lorenzo G.A. 2004. Compressibility of cement admixed clays at high water content. *Geotechnique* 54 (2), 151-154.
- Horpibulsuk S., Miura N. and Nagaraj T.S. 2005. Clay-water/cement ratio identity of cement admixed soft clay. *Journal of Geotechnical and Geoenvironmental Engineering ASCE* 131 (2), 187-192.
- Horpibulsuk S., Katkan W., Sirilerdwattana W. and Rachan R. 2006. Strength development in cement stabilized low plasticity and coarse grained soils: Laboratory and field study. *Soils and Foundations* 46 (3), 351-36.
- Horpibulsuk S., Liu M.D., Liyanapathirana D.S. and Suebsuk J. 2010. Behavior of cemented clay simulated via the theoretical framework of the Structured Cam Clay model. *Computers and Geotechnics* 37, 1-9.
- Horpibulsuk S., Rachan R. and Suddeepong A. 2011a. Assessment of strength development in blended cement admixed Bangkok clay. *Construction and Building Materials* 25 (4), 1521-1531.
- Horpibulsuk S., Rachan R., Suddeepong A. and Chinkulkijniwat, A., 2011b. Strength development in cement admixed Bangkok clay: laboratory and field investigations. *Soils and Foundations* 51 (2), 239-251.
- Horpibulsuk S., Phojan W., Chinkulkijniwat A., and Liu M.D. 2012a. Strength development in blended cement admixed saline clay. *Applied Clay Science* 55, 44-52.
- Horpibulsuk, S., Suddeepong, A., Chinkulkijniwat, A., and Liu, M.D. 2012b. Strength and compressibility of lightweight cemented clays. *Applied Clay Science* 69, 11-21.
- Liu M.D. and Carter J.P. 1999. Virgin compression of structured soils. *Géotechnique* 49 (1), 43-57.
- Liu M.D. and Carter J.P. 2000. Modelling the destructuring of soils during virgin compression. *Géotechnique* 50 (4), 479-483.
- Liu M.D. and Carter J.P. 2002. Structured Cam Clay Model. *Canadian Geotechnical Journal* 39 (6), 1313-1332.
- Miura N., Horpibulsuk S. and Nagaraj T.S. 2001. Engineering behavior of cement stabilized clay at high water content. *Soils and Foundations* 41 (5), 33-45.
- Suebsuk J., Horpibulsuk S. and Liu M.D. 2010. Modified Structured Cam Clay: A constitutive model for destructured, naturally structured and artificially structured clays. *Computers and Geotechnics* 37, 956-968.
- Suebsuk J., Horpibulsuk S. and Liu M.D. 2011. A critical state model for overconsolidated structured clays. *Computers and Geotechnics* 38, 648-658.

Some notes concerning the dry density testing standards

Quelques remarques concernant les descriptions relatives aux essais de densité sèche

Imre E.^{1,2}, Lőrincz J.², Gerendai E.¹, Szalkai R.¹

¹*Ybl Miklós Faculty of Arch. and Civil Eng., Szent István University*

²*Department of Geotechnics, Budapest University of Technology and Economics*

Lins Y., Schanz T.

Ruhr University, Bochum, Germany

ABSTRACT: The results of two doctoral programs concerning the dry density of sands are as follows: (i) the ratio of the minimum and the maximum dry density of sands is about constant, (ii) the dry density of the soil fractions slightly increases with the maximum grain diameter d_{max} being the diameter range doubled with increasing grain diameter. These results are used through a statistical analysis to show that the e_{max} test of the German DIN may be biased. The possible reason for the bias is arching due to the too small ratio of the diameter and the height of the mold. As a by-product of the research, the earlier finding that the maximum density of the specimens may be obtained indifferently using a Modified Proctor procedure or a vibrating table is extended to the DIN (implying that the density methods of the calibration chamber sands are basically equivalent to the DIN).

RÉSUMÉ : Deux thèses de doctorat ayant pour objet l'étude de la compacité des sables ont récemment formulé les importantes conclusions suivantes : (1) le rapport de la compacité minimale et maximale des sables est plus ou moins constant ; (2) l'étendue des fractions de diamètre croît avec le diamètre des grains, par conséquent la compacité des fractions croît également. En partant de ces conclusions, on a procédé à la comparaison (partiellement statistique) des valeurs de compacité minimales et maximales de cinq groupes donnés. Les résultats ont montré qu'une partie des normes actuelles conduisent à des erreurs déterminées dans la valeur de compacité minimale et ne conduisent pas à des erreurs déterminées dans la valeur de compacité maximale. L'explication en est, probablement, l'apparition de l'effet de voûte aux pots et vases de largeur différente, de hauteur constante..

KEYWORDS: sand, dry density, arching, Proctor, statistical test, coefficient of variation

1 INTRODUCTION

Two doctoral programs on the dry density of sands - where the soils used were indeed carefully chosen, artificial mixtures of natural grains (Lőrincz 1986, and Kabai, 1968) - ended with the two basic observations.

Kabai (1968 to 1974) found that the ratio of the minimum and maximum dry density was basically constant. Its value started to decrease as the soil became slightly plastic. He stated this on the basis of his experimental studies made on continuous, artificial mixtures of natural soil grains of the Danube river sand.

Lőrincz (1986) made a theoretical and experimental study on the grading entropy. He observed that the dry density of the sand fractions are increasing with grain diameter since the diameter range of the fractions are doubled with increasing grain diameter. (This result is paradox if the fractions are modelled by equal spheres.) He demonstrated this fact by his minimum dry density measurements.

The data bases of the foregoing two doctoral programs was compared first to the Calibration Chamber sand database, published by Lunne (1992) and Mayne and Kulhawy (1992) and, a significant difference was found on the databases (Imre et al, 2011) which was attributed to the dependence on the applied testing standard for density and to the geological origin.

Therefore, two additional data sets (one composed from Danube sands and one composed from Bochum sand) were measured in the University of Bochum. These were statistically analysed and it was found that the result of the minimum density measurement of the German Standard DIN 18126 (and some other standards not using the Proctor mold for the minimum dry density) may be biased and, as a result, the result of the minimum dry density test may be dependent on the grain diameter. The geological origin of the soils was indifferent.

2 MATERIALS AND METHODS

2.1 Variables

The results of the density tests can be expressed in terms of the void ratio e , the dry density ρ_d , the solid volume ratio s or its inverse, the so called specific volume v . The basic definitions are as follows:

$$s = \frac{V_s}{V} = \frac{1}{v} = \frac{1}{1+e} \quad (1)$$

$$e = \frac{V_v}{V_s} = \frac{1-s}{s} \quad (2)$$

where V is the total soil volume, V_s is the volume of solids and V_v is the volume of voids, $\rho_d = s\rho_s$.

2.2 Density tests

The minimum dry density testing methods are related to a funnel pouring device, differing in the mold size. The Proctor mold is used by Kabai (1978) and Lőrincz (1986). The diameter of the mold is 0,625 times the height for the DIN18126, 0,769 times the height for the ASTM D4254 and 0,879 for the Proctor mold (Fig 1).

Concerning the maximum dry density tests, the similarity of the Modified Proctor procedure and the Vibrating Table procedure is known (Poulos and Hed, 1974). Kabai used an 8 mm thick steel plate was applied on the top of the sample, using the results of Leussink and Kutzner (1962), to prevent the local loosening of the sample.

2.3 Soils

The paper deals with 4 sets of density tests. The means ($\bar{x}$) and the standard deviations (σ) of the variables s_{min} , s_{max} and s_{min} / s_{max} and e_{min} , e_{max} and e_{min} / e_{max} were determined.

In the program of Kabai (1968 to 1974, Group I) 33 grading curve series (artificial mixtures made of natural soil grains of the Danube sands) were defined in terms of the coefficient of uniformity C_u and the grainsize parameters d_{max} , d_{10} . The grading curves can be seen in Imre et al (2009). The minimum and maximum dry density were determined. The results can be seen in Table 1.

The calibration chamber sand database (Group II, Lunne et al (1992), Mayne and Kulhawy (1992)) consists of 25 sands. It can be assumed that the minimum and maximum dry density tests were made by various methods, including the ASTM Standards (i.e. D 4253 Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table and D 4254 Standard Test Methods for Minimum Index Density and Unit Weight of Soils) and the German Standard (i.e. DIN 18126 for the maximum and minimum dry density). The results are shown in the Table 2.

In the ongoing testing program two additional data sets were produced with the German DIN (Groups III and IV) using 24 Danube sands 182 German sands, respectively. In Group III some Danube sands were used, the composition can be seen in Imre et al (2011). in Group IV some German commercial sands were used. the composition can be seen in Szalkai (2012). The results are shown in the Tables 3 and 4. The artificial mixtures were made of soil grains with 7 fractions of a diameter range of $d_{min} = 0.06$ mm and $d_{max} = 8$ mm.

Concerning the data of Lőrincz (1986), some artificial mixtures of mold sand grains) were defined in terms of grading entropy for various purposes, to elaborate some particle migration criteria and filtering law, to test the existing filtering laws, some grading curves can be seen e.g. in Imre et al (2009). Only the minimum dry density was determined using the Proctor mold. Some fractions results can be seen in Figure 2 showing that dry density of the sand fractions are increasing with grain diameter.

2.4 Statistical tests

The means ($\bar{x}$) and the standard deviations (σ) of the variables s_{min} , s_{max} and s_{min} / s_{max} for any two groups of data were compared using the standard parametric statistical tests (see e.g. in Rétháti, 1988).

The F -test was used to evaluate the null hypothesis that two data sets with normal distribution have the same variance. The Welch test and t tests were used to test the null hypothesis that two data sets with normal distribution have the same means with the data having possibly unequal and equal variances, respectively.

The Pearson correlation R is obtained by dividing the covariance of the two variables by the product of their standard deviations. The Pearson correlation is +1 in the case of a perfect positive (increasing) linear relationship (correlation) and, -1 in the case of a perfect decreasing (negative) linear relationship since the following relationship is met:

$$R(x, y) = 1 \leftrightarrow y = ax + b \quad (3)$$

The Pearson correlation is some value between -1 and 1 in all other cases, indicating the degree of linear dependence between the variables. As the Pearson correlation R approaches zero there is less of a linear relationship. The closer the coefficient is to either -1 or 1, the stronger the linear correlation between the variables.

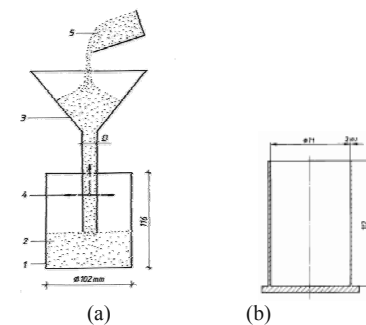


Figure 1. The molds (a) Proctor ($d=100$ mm, $h=112$ mm) (b) DIN ($d=71$ mm, $h=112$ mm). Note the same height and different width.

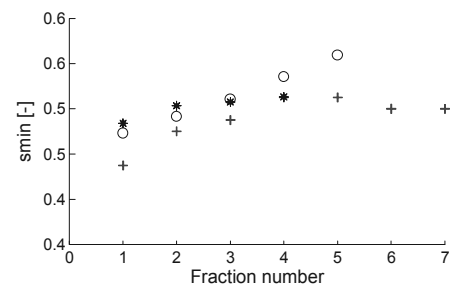


Figure 2. Fraction measurement of Lőrincz (1976), indicated by open circles, note the bias from fraction 3 (0.25 to 0.5 mm) in the new data.

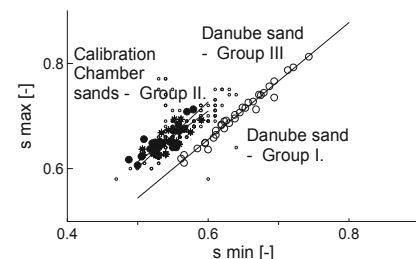


Figure 3. All data (Groups I to IV), $s_{min} - s_{max}$

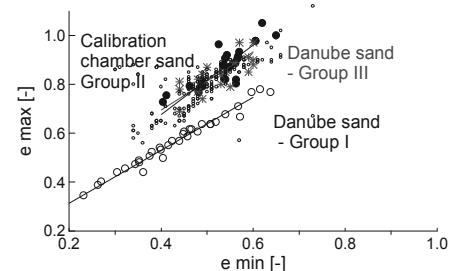


Figure 4. All data (Groups I to IV), $e_{min} - e_{max}$

If the variables are independent, Pearson's correlation coefficient is 0, but the converse is not true, the correlation coefficient detects only linear dependencies. The parameters of the best fit linear function are as follows (E denotes expected value which can be approximated by the mean value):

$$a = \sigma(y) / \sigma(x), \quad b = E(y) - aE(x) \quad [4]$$

3 INDIVIDUAL GROUP RESULTS

The results of the density tests for groups I to IV are summarized in Figures 3 to 6 where the small open circles are related to the Group IV. The results are presented in Tables 1 to 4 in statistical viewpoint.

3.1 Danube Sands data - Group I

In the work of Kabai (1972) the minimum and maximum dry densities are presented for Danube sand mixtures. From these, the void ratios e and the solid volume ratios s were computed and their ratios were determined (Table 1). The mean of the s_{min} and s_{max} values are 0.640 and 0.700, with mean coefficients of variation of 0.070 and 0.072, respectively. The mean of the s ratio (s_{min} / s_{max}) is equal to 0.915 with a coefficient of variation $CV=0.0116$, indicating that the s ratio is practically constant. The mean of the e_{max} and e_{min} values are 0.569 and 0.436, with coefficients of variation of 0.192 and 0.234, respectively. The mean of the e_{min} / e_{max} is equal to 0.760 with a coefficient of variation $CV=0.060$.

3.2 The Calibration Chamber sand data – Group II

The mean of the s_{min} and s_{max} values are 0.536 and 0.652, with the coefficients of variation of 0.045 and 0.039, resp. (Table 2). The mean of the s ratio (s_{min} / s_{max}) is equal to 0.822 with a coefficient of variation $CV=0.024$. The mean e_{max} and e_{min} values are 0.870 and 0.536, with coefficients of variation of 0.097 and 0.110, respectively. The mean of the e_{min} / e_{max} is equal 0.617 with a coefficient of variation of $CV=0.061$. The mean of the e_{max} and e_{min} values are 0.810 and 0.496, with coefficients of variation of 0.114 and 0.159, respectively. The mean of the e_{min} / e_{max} is equal to 0.610 with a coefficient of variation $CV=0.082$.

3.3 Danube Sands data – Group III

The minimum and maximum dry densities for 24 Danube sand mixtures are shown in Table 3. The mean of the s_{min} and s_{max} values are 0.554 and 0.670, with mean coefficients of variation of 0.052 and 0.053, respectively. The mean of the s ratio (s_{min} / s_{max}) is equal to 0.827 with a coefficient of variation $CV=0.025$, indicating that the s ratio is practically constant.

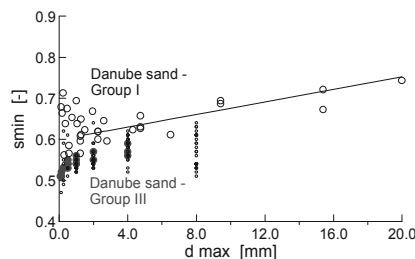


Figure 5. Groups I, III to IV data, s_{min} and maximum diameter. Note that the small open circles are 'below' the linear trend of Group I

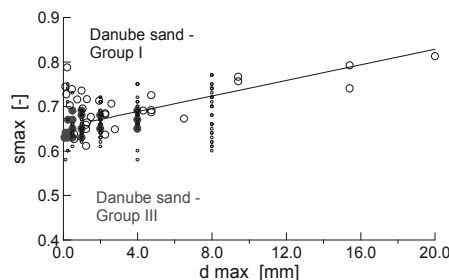


Figure 6. Groups I, III to IV data, s_{max} and maximum diameter. Note that the small open circles are 'centered' the linear trend of Group I

3.4 Bochum Sand data – Group IV

The minimum and maximum dry density results of the newly measured 182 Bochum sand mixtures are shown in Table 4.

The mean of the s_{min} and s_{max} values are 0.557 and 0.671, with mean coefficients of variation of 0.054 and 0.053, respectively. The mean of the s ratio (s_{min} / s_{max}) is equal to 0.831 with a coefficient of variation $CV=0.044$, indicating that the s ratio is practically constant.

The mean of the e_{max} and e_{min} values are 0.801 and 0.495, with coefficients of variation of 0.119 and 0.161, respectively. The mean of the e_{min} / e_{max} is equal to 0.619 with a coefficient of variation $CV=0.125$.

Table 1. Results of the statistical evaluation – Group

	s_{min}	s_{max}	$ratio_s$	e_{max}	e_{min}	$ratio_e$
X	0.640	0.700	0.915	0.569	0.436	0.760
σ	0.045	0.050	0.011	0.109	0.102	0.045
CV	0.070	0.072	0.012	0.192	0.234	0.060

Table 2. Results of the statistical evaluation – Group II

	s_{min}	s_{max}	$ratio_s$	e_{max}	e_{min}	$ratio_e$
X	0.536	0.652	0.822	0.870	0.536	0.617
σ	0.024	0.026	0.020	0.084	0.059	0.038
CV	0.045	0.039	0.024	0.097	0.110	0.061

Table 3. Results of the statistical evaluation - Group III

	s_{min}	s_{max}	$ratio_s$	e_{max}	e_{min}	$ratio_e$
X	0.554	0.670	0.827	0.810	0.496	0.610
σ	0.029	0.036	0.021	0.092	0.079	0.050
CV	0.052	0.053	0.025	0.114	0.159	0.082

Table 4. Results of the statistical evaluation - Group IV

	s_{min}	s_{max}	$ratio_s$	e_{max}	e_{min}	$ratio_e$
X	0,557	0,671	0,831	0,801	0,495	0,619
σ	0,030	0,036	0,037	0,095	0,080	0,077
CV	0,054	0,054	0,044	0,119	0,161	0,125

Table 5. Results of the statistical tests - s_{min}

	I	II	III	IV
I	+	-	-	-
II	-	+	+	-
III	-	+	+	-
IV	-	-	-	+

Table 6. Results of the statistical tests - s_{max}

	I	II	III	IV
I	+	-	+	+
II	-	+	+	+
III	+	+	+	
IV	+	+		+

 Table 7. Results of the statistical tests – ratio s_{min}/s_{max}

	I	II	III	IV
I	+	-	-	-
II	-	+	+	+
III	-	+	+	
IV	-	+		+

4 DISCUSSION, CONCLUSIONS

4.1 The statistical features of the data bases

The ratio of the minimum and the maximum dry density is statistically constant for sands since the coefficient of the variation is very small. The coefficient of variation of the e ratio is equal to 0.061 for the calibration chamber sands; 0.060, 0.082 and 0.125 for the Groups sands I and III to IV, respectively. The coefficient of variation of the s ratio is equal to 0.024 for the calibration chamber sands; 0.0116, 0.025 and 0.044 for the Groups sands I and III to IV, respectively.

This result implies linear correlations between the minimum and the maximum dry density data. The Pearson coefficient of correlation at the $s_{min} - s_{max}$ and $e_{min} - e_{max}$ was equal to 0.99, 0.99 for the Danube sands I indicating that both the $s_{min} - s_{max}$ and $e_{min} - e_{max}$ relations are linear. The Pearson coefficient of correlation at the $s_{min} - s_{max}$ and $e_{min} - e_{max}$ was equal to 0.84, 0.85; 0.89 and 0.88 and; 0.64, 0.65; respectively for Groups II, III and IV – being smaller, still indicating linear correlation.

The smallest coefficient of variation values (and, the largest correlation values) of Group I sand data (produced by using the Proctor mold for maximum void ratio) can be attributed to the facts that the quality of testing on was good and the tested soils were homogeneous (having continuous distribution only).

4.2 The information concerning the dry density ratio

The results of the statistical tests indicate that the ratio s_{min}/s_{max} is about equal about to 0.76 for Group I where the Proctor mold is used for the minimum dry density test and the ratio s_{min}/s_{max} is about equal about to 0.62 for the remainder Groups where a different mold is used (i.e. the two newly tests sand databases – being tested with the German Standard – and the Calibration Chamber sands).

The result of the statistical tests (Tables 5 to 7) – show that the minimum dry density data are significantly different for Group I and the remainder groups but the maximum dry density results are similar.

The significant difference in the minimum dry density testing methods can be attributed to the fact that the molds have different sizes (Fig 1). This point needs some further (e.g. micromechanical) research.

4.3 The dependence of the dry density on the diameter

In accordance to the expectations, the measured s_{min} and s_{max} values slightly increase with the maximum grain diameter d_{max} in the tested sand databases (Figs 5 to 6).

However, an increasing bias with grain diameter – possibly due to arching – is found in the minimum density results. As it can be seen on Figure 2, the density increase is not possible to be reproduced by the DIN 18126 for fractions being larger than 0,5 mm, instead of this, even a decrease is experienced.

4.4 Conclusions

The different dimension of the mold of the various minimum dry density tests has some impact on the test results as follows. The smaller width and same height may cause some kind of arching which leads to smaller minimum dry density values and, this effect is the function of the grain diameter.

As a by-product of the research, an additional comment can be made. The result of the statistical analysis indicates that the equivalence of the Modified Proctor procedure and the Vibrating Table procedure (Poulos and Hed, 1974) can be extended to the DIN maximum dry density testing method.

Some further research is suggested, including the separation of the continuous and gap-graded mixtures and, on the investigation of the micromechanical features of the minimum dry density test.

5 ACKNOWLEDGEMENTS

The suggestion Tom Lunne, the help of Tóth Ádám és Tóth Szabina, the support of the National Research Fund Jedlik Ányos NKFP B1 2006 08 and the Norwegian research fund HU-0121 are greatly acknowledged.

6 REFERENCES

- Imre, E; Lőrincz, J; Trang, Q.P; Fityus, S. Pusztai, J; Telekes, G; Schanz, T. (2009) Some dry density transfer function for sands. Invited paper. KSCE Journal of Civil Engineering 13(4):257-272. DOI 10.1007/s12205-009-0257-7
- Imre E, Fityus S, Keszeyné E, Schanz T (2011) A Comment on the Ratio of the Maximum and Minimum Dry Density for Sands. Geotechnical Engineering 42(4) pp. 77-82.
- Imre E, Gerendai, E; Lins, Y; Schanz T (2012) A Comment on the dry density standards. Geotechnical Testing (submitted).
- Szalkai, R. (2012) The result of the minimum –maximum dry density test on 182 soil samples. Student research project (manuscript).
- Lőrincz, J (1986). Grading entropy of soils Doctoral Thesis, Technical Sciences, TU of Budapest.
- Kabai, I. (1968). The compactibility of sands and sandy gravels. Acta Technica Acad. Sci. Hung., 113-124.
- Kabai, I. (1972). Relationship between the grading curve and the compactibility. University doctoral thesis TU of Budapest, Hungary (in Hungarian).
- Kabai, I. (1974). The effect of grading on the compactibility of coarse grained soils. Periodica Polytechnica.18 (4) 255-275.
- Leussink, L.H. and Kutzner, Ch (1962) Laboratoriumversuch zur Feststellung der dichtesten Lagerung körniger Erdstoffe. Veröff. Des Inst. F. Bodenmech. Techn. Hochschule Frid. In Karlsruhe.
- Lunne, T; Robertson, P.K.; Powell, J.J.M. (1992). Cone Penetration testing. Blackie Academic & Professional.
- Mayne P.W. and Kulhawy F. H. (1992). Calibration Chamber database and boundary effects corrections for CPT data. Proc. On Calibration Chamber Testing, New York, 1991, 257-54, Elsevier.
- Rétháti, L. (1988). Probabilistic solutions in geotechnics Elsevier, Akadémiai K. Amsterdam.
- Poulos, S.J., Hed, A (1973) Density Measurements in a Hydraulic Fill. Evaluation of Relative Density and its Role in Geotechnical Projects Involving Cohesionless Soils. ASTM STP523. p 402-424.

Interpretation of stress-dependent mechanical behaviour of rockfill materials

Interprétation de stress-dépendante et comportement mécanique de matériaux enrochement

Jannati Aghdam R.

Senior Geotechnical Engineer, Tehran, Iran

Soroush A.

Amirkabir University of Technology, Tehran, Iran, soroush@aut.ac.ir

ABSTRACT: This paper studies thoroughly the mechanical behavior of thirty types of rockfill materials subjected to triaxial compression shearing, each with three different confining stresses. The materials' characteristics, including mineralogy, gradations and shape of particles; and also the tests' results have been collected from the literature. The Hyperbolic Model (Duncan and Chang 1970) is employed as a framework for interpreting the mechanical behavior of the materials. Features of the behavior of the rockfill materials, as compared with that of soils, are highlighted through the exponent parameter (n) of the Hyperbolic Model. It is shown that unlike for soils, the exponent number, n , is not constant for a given rockfill material, and that the n value depends on the confining stress level; for the materials under high pressures, n can even takes a negative value, which is a sign of particle breakage of the materials. Two correlations for estimating initial elasticity modulus (E_i) and internal friction angle (ϕ) of these materials are suggested.

RÉSUMÉ : Cet article étudie soigneusement le comportement mécanique de trente types de matériaux enrochement soumis à un cisaillement triaxial chacun avec trois différentes contraintes de confinement. Les caractéristiques du matériau : minéralogie, gradations et forme des particules et aussi les résultats du test ont été collectés à partir de la littérature. Le modèle hyperbolique (Duncan et Chang 1970) est utilisé en tant que cadre pour l'interprétation du comportement mécanique des matériaux. Les caractéristiques du comportement des matériaux en enrochement, par rapport à celles des sols, sont mises en évidence par le paramètre d'exposant (n) du modèle hyperbolique. Il est montré que, contrairement aux sols, le nombre d'exposants, n , n'est pas constant pour un matériau donné en enrochement et, en ce que la valeur de n dépend du niveau de contrainte de confinement ; pour les matériaux à hautes pressions, n peut même prendre une valeur négative, ce qui est un signe de rupture des particules de matières. Deux corrélations pour estimer le module d'élasticité initial (E_i) et l'angle (ϕ) de frottement interne de ces matériaux sont proposées.

KEYWORDS: Rockfill Materials, Triaxial Compression Shearing, Hyperbolic Model, Initial Elasticity Modulus.

1 INTRODUCTION

Shear strength and deformation characteristics of rockfill materials depend generally on different parameters, such as mineralogy, grain size distribution, size of particles, stress level, and particle breakage (if any). The importance of particle breakage goes back to its capability of changing gradations of granular materials.

This paper presents the results of a comprehensive study on the mechanical behaviour of thirty rockfill materials under medium and large scale triaxial testing. Data about the materials and the tests are gathered from the literature. The Hyperbolic Model (HM) is employed as an analytical and behavioural framework for this study. The important parameters of the HM for the rockfill materials are determined and compared with similar parameters for typical loose and dense sands. Variations of deformation and strength parameters (E_i and ϕ) of the materials with confining stress (σ_3) are studied. On the basis of this study, two relationships for estimating E_i and ϕ of the rockfill materials are proposed.

2 PROPERTIES OF ROCKFILL MATERIALS

Thirty types of rockfill materials, on which conventional triaxial compression tests had been carried out, are used in this study. The material characteristics, including mineralogy, uniformity coefficient, shapes of particles and etc. for three types of these materials are presented in Table 1. Because of conciseness the presenting of the all of materials characteristics has been neglected.

3 HYPERBOLIC MODEL AND ITS APPLICATION

3.1. Theory of the model

The Hyperbolic Model (Duncan & Chang 1970) considers the behavior of a soil specimen under compressive triaxial testing as a hyperbola. According to the model, the gradient of the tangent to the stress-strain relationship ($q:\epsilon_a$), namely as tangential deformation modulus (E_t), is defined as follows:

$$E_t = \left\{ 1 - \frac{R_f (1 - \sin \phi) (\sigma_1 - \sigma_3)^2}{2C \cos \phi + 2\sigma_3 \sin \phi} \right\}^2 K P_a \left(\frac{\sigma_3}{P_a} \right)^n \quad (1)$$

Where ϕ = internal friction angle; C = cohesion; K = modulus number; n = exponent number, R_f = failure ratio; and P_a = atmospheric pressure.

The above parameters for a given material can be obtained by carrying out, usually, three triaxial tests on the soil's specimens.

Parameters K , n , and R_f are usually determined from the triaxial tests results and on the basis of a nonlinear stress- strain behavior, which is assumed as a hyperbola (Konder 1963). The Hyperbola equation is as follows:

$$\frac{\epsilon_a}{(\sigma_1 - \sigma_3)} = a + b \epsilon_a \quad (2)$$

Where σ_1 = maximum principal stress; σ_3 = minimum principal stress; and ϵ_a = axial strain in triaxial compressive testing. Parameters a and b =the reverses of initial elasticity modulus (E_i) and ultimate deviatoric stress $(\sigma_1 - \sigma_3)_{ult}$, respectively. These parameters can be determined by drawing a

fitting line to the tests results, as shown in Figure 1. R_f is defined as follows:

$$R_f = \frac{(\sigma_1 - \sigma_3)_f}{(\sigma_1 - \sigma_3)_{ult}} \leq 1.0 \quad (3)$$

Where $(\sigma_1 - \sigma_3)_f$ = the deviatoric stress at failure.

The following relationship for the initial modulus of elasticity for soils is suggested (Janbu 1963):

$$E_i = kP_a \left(\frac{\sigma_3}{P_a} \right)^n \quad (4)$$

Where, n represents the exponential effect of σ_3 on E_i . K and n can be determined by drawing a line fitted to data of the triaxial tests, as presented in Figure 2.

Table 1. Mechanical properties of three rockfill materials

Rockfill Type	Australia Railway Ballast	Roodbar Dam Material	Yamchi Dam Material
Mineralogy	Latite basalt	Lime stone	Andesite
D_{min} (mm)	20	0.15	0.072
D_{max} (mm)	65	50.8	74.3
C_u	1.5	23	65.4
σ_3 (kPa)	90	500	200
	120	700	400
	240	900	700
φ°	54.6		
	52	30.6	38.7
	45.8		
B_g^* (%)	10	11 12 13.5	NIA**
Shape	Highly Angular	Angular/ sub Angular	Rounded

** NIA: No Information Available

* B_g : Marsal's breakage index

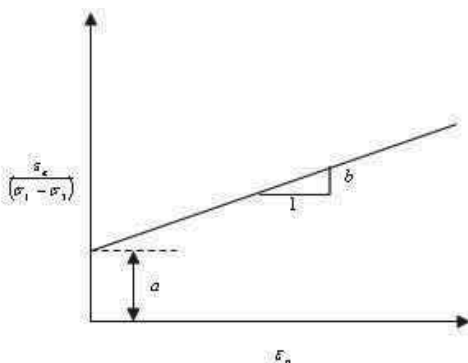


Figure 1. Determining constants a and b from a triaxial test results

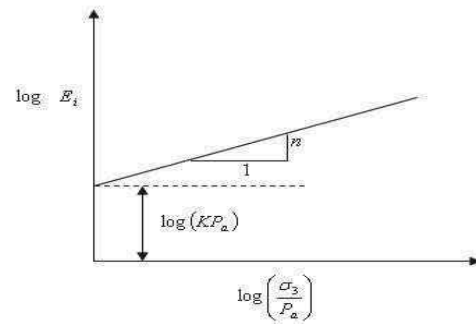


Figure 2. Determining constants n and K from three triaxial tests results

3.2. Application of the model

In this section, the mechanical behavior of the thirty rockfill materials, which the characteristics of three types of them are presented in Table 1, are investigated analytically in the framework of the Hyperbolic Model. Values of parameters n and K for the materials introduced in Table 1, for every two consecutive triaxial tests, and furthermore, for three triaxial tests, are extracted and presented in Table 2. Therefore, for each of the materials three values of n are calculated; one from the first and second triaxial tests results, one from the second and third triaxial tests results, and one from first, second and third triaxial tests. We intentionally calculated the first two values for n to highlight the effect of particle breakage on E_i (through n) with every increase of σ_3 . This is different from the similar procedure of determining n for soils, where usually a unique n value can be extracted from results of triaxial tests with three consecutive confining stresses (σ_3) on a given material. The details of determining n and K from the triaxial tests results are given in Duncan and Chang 1970.

Table 2. Values of n and K parameters for the rockfill materials introduced in Table 1

Rockfill Type	$(\sigma_3)_1, (\sigma_3)_2$ (kPa)	n	K
Australia Railway Ballast	90, 120	-7.45	3500
	120, 240	-0.95	1070
	90, 120, 240	-3.32	2100
Roodbar Dam Material	500, 700	-6.53	18×10^8
	700, 900	-1.15	52000
	500, 700, 900	-3.26	10×10^5
Yamchi Dam Material	200, 400	2.7	1180
	400, 700	0	5×10^4
	200, 400, 700	1.5	3000

Table 3 compares the average values of n for the relatively rounded and relatively angular rockfill materials of this study separately, with typical values for loose and dense sands. As expected, the average n for the rockfill materials are far less than that of the typical dense sand; moreover, n reduces with increasing of the materials' angularity. It should be mentioned that highly angular, angular, angular/sub angular and sub angular materials are assumed as relatively angular materials; while, rounded, rounded/sub rounded and sub rounded ones are assumed as relatively rounded materials.

Table 3. Average values of n and K for relatively rounded and relatively angular rockfill materials, compared with typical values for loose and dense sands

Type of Materials	Relatively Angular	Relatively Rounded	Loose Sand	Dense Sand
n	0.085	0.29	0.65	0.54
K	2004	700	300	2000

The main factor responsible for the comparatively lower (compared with sands) average values of n for the rockfill materials (especially the angular ones) is particles breakage which happens during both compression and shearing of the materials.

According to Eq. 4, n represents the exponential effect of σ_3 on E_i . As particle breakage in rockfill materials is far more than in sands, the average n value of rockfill materials is much less than that of sands. Materials with higher degrees of angularity suffer more particle breakage and therefore, they have lower values of n .

The modulus number (K in Eq. 4) for the studied rockfill materials takes values ranging widely from 53 to 36×10^9 . The very high values of K correspond to negative values of n .

4 VARIATION OF E_i AND ϕ WITH CONFINING PRESSURES (σ_3)

Figure 3 shows the variations of E_i with σ_3 while shown in Figure 4 is the variations of ϕ with σ_3 for one of the highly angular and one of the rounded materials which are studied in this paper. As Figure 3 shows, because of massive particle breakage, the increasing of σ_3 has resulted in the reduction of E_i for the highly angular rockfill material, while E_i has increased with a gentle slope with the increase of σ_3 for the rounded rockfill material. Moreover, the rate of reduction of ϕ with the increase of confining pressure is much more for the highly angular material than for the rounded material, as presented in Figure 4.

The above trends of variations were obvious more or less in the behavior of the rest of the materials. Our studies on the behavior of the materials showed that the following factors have major effects on the rate of particle breakage:

- Shape of particles: angular particles undergo more breakage.
- Mineralogy: particles of weaker materials break more.
- Size of particles: coarse particles are more vulnerable to breakage.
- Coefficient of Uniformity (C_u): A lower C_u , causes a higher rate of particle breakage.

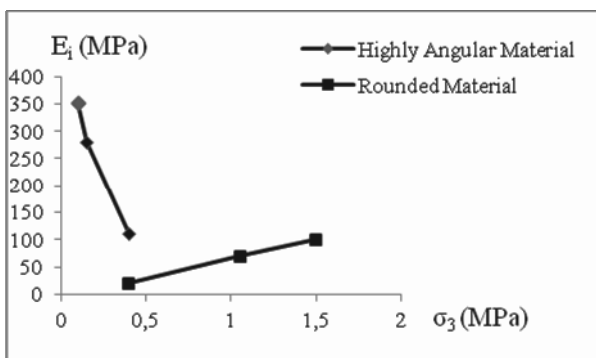


Figure 3. Variations of E_i with σ_3 for typical rounded and highly angular rockfill materials

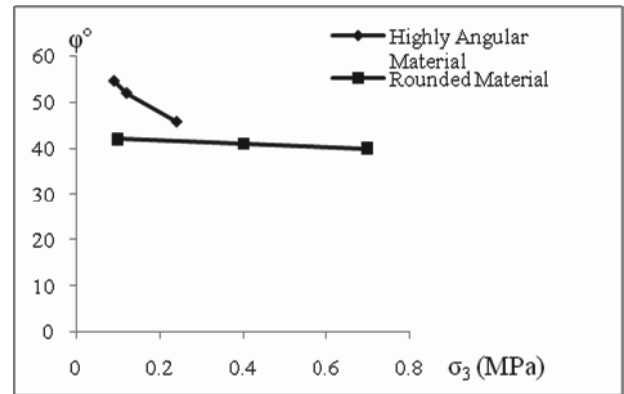


Figure 4. Variations of ϕ with σ_3 for typical rounded and highly angular rockfill materials

It should be mentioned that according to Equation 4, the particle breakage phenomenon affects the tangential deformation modulus (E_t) of rockfill materials by both " n " and " ϕ ".

The above conclusions strongly suggest that the variations of deformation modulus of rockfill materials with confining pressure should be taken into account in deformation analyses of rockfill structures.

5 CORRELATION BETWEEN E_i AND ϕ WITH σ_3

This study on the triaxial testing results of the rockfill materials led to two correlations between E_i and ϕ with σ_3 . They are as follows:

$$\frac{\Delta\phi}{\phi_0} = -\alpha \log\left(1 + \frac{\Delta\sigma_3}{\sigma_{30}}\right) \quad (5)$$

where $\Delta\phi$ = reduction of internal friction angle; ϕ_0 = internal friction angle corresponding to σ_{30} ; σ_{30} = initial confining stress, which is usually the minimum confining stress in triaxial testing; $\Delta\sigma_3$ = confining pressure increase; and α = a coefficient depending on shape of particles, coefficient of uniformity (C_u), and confining pressure increment ratio ($\Delta\sigma_3/\sigma_{30}$).

Considering the studied rockfill materials, α ranges between 0.051 and 0.59 for the relatively angular materials and between 0.046 and 0.42 for the relatively rounded materials.

It implies that α for the rounded materials are generally less than that for the angular ones; this represents the lower particle breakage potential for the rounded materials.

The relationship for variation of E_i with σ_3 is suggested, as follows:

$$\frac{\Delta E_i}{E_{i0}} = \beta \left(\frac{\Delta\sigma_3}{\sigma_{30}}\right) \quad (6)$$

where ΔE_i = changes in initial elasticity modulus; E_{i0} = initial elasticity modulus corresponding to σ_{30} ; and β = a coefficient depending on particle shapes, uniformity coefficient (C_u), and $\Delta\sigma_3/\sigma_{30}$ in triaxial compression shearing.

For the relatively angular materials, β was calculated as $-2.65 \leq \beta \leq 3.71$ and for the relatively rounded materials as $-1.14 \leq \beta \leq 5.50$. It is observed that the range of positive values of β , which implies the increase of E_i with σ_3 , for the relatively angular materials are smaller than the similar range for the rounded materials (3.71 versus 5.50). For the range of negative values of β , which implies the decrease of E_i with σ_3 , the trend is opposite (-2.65 versus -1.14). The above observation is logical concerning comparatively higher particle breakage and its reductive effect on E_i in the relatively angular materials.

6 CONCLUSIONS

This paper studied thoroughly the mechanical behavior of thirty rockfill materials subjected to triaxial compression shearing with three different confining stresses. The Hyperbolic Model (Duncan and Chang 1970) was employed as the behavioral model for this study. Features of the mechanical behavior of rockfill materials, as compared with soils, were highlighted through the exponent parameter (n) of the Hyperbolic Model. Unlike for soils, n is not constant for rockfill materials, and depends on confining stress levels; n can even takes a negative value, which is a sign of particle breakage in the materials.

The main focus in this paper was on the effect of confining pressure on the stiffness (initial deformation modulus and tangential deformation modulus) of the materials. It was shown that rockfill materials undergo particle breakage to some extents and therefore, they may behave comparatively softer under higher confining stresses. The extent of particle breakage depends on gradation characteristics, (especially coefficient of uniformity), particle shape (angular or rounded), wetting condition, and confining stress.

Two correlations for estimating initial elasticity modulus (E_i) and internal friction angle (ϕ) of the studied rockfill materials, based on particles shape, confining pressure (σ_3), and coefficient of uniformity (C_u) were suggested. Investigations on the variations of internal friction angle (ϕ) with confining stress (σ_3) also showed that ϕ decreases with increasing of σ_3 in all types of rockfill materials. The extent of reduction in ϕ depends on the extent of particle breakage.

7 REFERENCES

- Duncan J.M. and Chang Ch.Y. 1970. Nonlinear analysis of stress and strain in soils. *Journal of soil mechanics and foundations division*, 1629-1653.
- Konder R.L. 1963. Hyperbolic stress-strain response: cohesive soils. *Journal of the soil mechanics and foundations division, ASCE*, 89(1), 115-143.
- Janbu N. 1963. Soil compressibility as determined by oedometer and triaxial tests. *European Conference on Soil Mechanics and Foundations Engineering*, Wiesbaden, Germany, Vol. 1, 19-25.
- Indraratna B. Ionescu D. and Christie H. D. 1998. Shear behavior of railway ballast based on large - scale triaxial tests. *J. SMFE, ASCE*, 124(5), 439-449.
- BHRC 1994-2006. Testing of rockfill materials with large - scale triaxial equipment. TP, *Geotechnical Department*, Tehran, Iran.
- Tofigh Rayhani M.H. 2000. Studying the mechanical behavior of rockfill and earth dam faces using large scale triaxial and direct shear test. *M.Sc. Thesis, Tarbiat Moallem University*, Tehran, Iran.
- Varadarajan A. Sharma K. G. Venkatachalam K. and Gupata A. K. 2003. Testing and modeling two rockfill materials. *Journal of Geotechnical and Geoenvironmental Engineering*, 206-218.
- Moshanir Power Engineering Consultants. 1996. *Review on additional laboratory tests on materials of Masjed Soleyman Dam*. Tehran, Iran.
- Brauns, J. 1993. *Laboratory tests report on materials of Masjed Soleyman Dam*. Division of Earth Dam and Landfill Technology. University of Karlsruhe.
- Frasconi A. Hegg U. and Rossi P. 1982. Large-scale laboratory tests for the mechanical characterization of granular materials for embankment dams. *Proc. 14th Congres des Grands Barrages*, Rio de Janeiro, 727-751.
- Charles J.A. and Watts K.S. 1980. The influence of confining pressure on the shear strength of compacted rockfill. *Geotechnique* 30, No.4, 353-367.

Effects of Freeze-Thaw History on Deformation-Strength Properties and Permeability of Fine-Grained Soil

Effets de l'historique du gel-dégel sur les propriétés de résistance à la déformation et de perméabilité des sols à grains fins

Kawaguchi T., Nakamura D., Yamashita S., Yamasaki S.
Kitami Institute of Technology, Kitami, Japan

Ishikawa T.
Faculty of Engineering, Hokkaido University, Sapporo, Japan

ABSTRACT: In cold and snowy areas, surface soils on slopes often move downward especially in early spring. Its moving caused by repeated freeze-thaw cycles. If we properly evaluate slope stabilities and carry out effective slope protection works, we should understand variations of strengths, stiffnesses, permeabilities, and so on in the cycles. By the way, it is known that repeated freeze-thaw cycles make ice lenses within surface soil. They spread parallel to ground surface so they are arranged parallel to shear stress on the slope. In this study we carried out frost heave tests that obtained changes of the void ratio and permeability in the cycle of freezing and thawing, direct box shear tests that obtained strength-deformation characteristics in the cycles, and the bender element tests that also obtained velocities of shear waves with vertical and horizontal propagation in the cycles. Then we conclude that the formation of ice lenses made parallel to shear stress is important for mechanical properties of surface soils.

RÉSUMÉ : Dans les régions froides et enneigées, les sols de surface sur les pentes se déplacent souvent vers le bas, en particulier au début du printemps. L'évaluation correcte de la stabilité des pentes et la réalisation de travaux de protection des pentes efficaces devraient nous permettre de comprendre les variations de forces, rigidités, perméabilités, et ainsi de suite au cours des cycles. Dans cette étude, nous avons effectué des tests de soulèvement par le gel qui nous ont permis d'obtenir les variations de l'indice de vide et de la perméabilité dans le cycle de gel et de dégel, des essais de cisaillement direct à la boîte qui nous ont donné les propriétés de résistance à la déformation au cours des cycles et des tests par élément fléchissant qui nous ont également donné les vitesses des ondes de cisaillement avec la propagation verticale et horizontale au cours des cycles. Nous avons alors conclu que la formation de lentilles de glace parallèles à la contrainte de cisaillement joue un rôle important dans les propriétés mécaniques des sols de surface.

KEYWORDS: frost heave, freezing and thawing test, permeability, strength-deformation characteristics, elastic shear modulus

1 INTRODUCTION

Surface soil of natural and artificial slopes in cold and snowy areas, such as Hokkaido (the northernmost island of Japan), moves downwards in early spring due to the cycle of freezing and thawing (e.g. Ueno et al. 2010). A mechanism of this phenomenon is conceptually shown in Figure 1. In winter, ice lenses formed around freezing fronts within the soil on a slope, then ice lenses grow, causing frost heave. In spring, ice lenses thaw with increasing water contents of soil as temperature rise, then soil begins to move or flow downwards by gravity. Slopes often fail by snowmelt and rainwater. To evaluate slope stability and select effective slope protection work, we should obtain variations of strength, stiffness and permeability in the cycle of freezing and thawing. As shown in Figure1, most of directions of ice lenses are parallel to shear stress. However, there are few studies focusing the directionality of ice lens.

In this study, we pursued changes of the void ratio and permeability in the cycle of freezing and thawing, covered soil samples with various densities. To use soil samples containing ice lenses, we developed a new method of the direct box shear test, enabled to measure deformation-strength properties, and a new method of the freezing and thawing test to measure velocities of shear waves with vertical and horizontal propagation, enabled to measure elastic shear moduli and analyze anisotropic structures.

2 TEST APPARATUSES AND SPECIMENS

Three types of test apparatus were used in this study. The first is a frost heave test apparatus according to JGS 0172-2009: Test

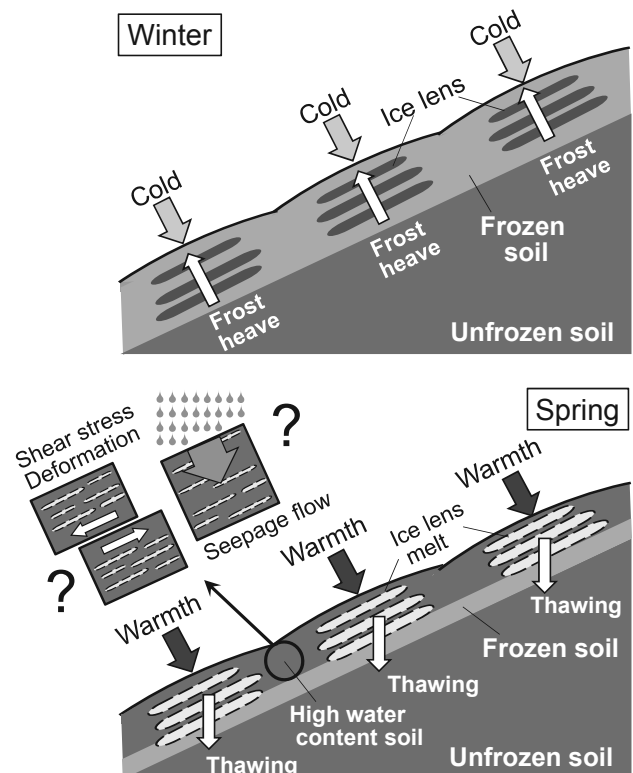


Figure 1. Mechanism of slope movement and slope failure in spring

method for frost susceptibility of soils, which uses specimens in a disk form 10 cm in diameter and 5 cm in thickness and is capable of falling head permeability tests using a burette before and after a freeze-thaw cycle (designated hereinafter as the frost heave test apparatus capable of permeability tests). The second is a frost heave test apparatus equipped with bender elements (BEs) for measuring the velocity of shear waves propagating in the specimen before and after a freeze-thaw cycle (designated hereinafter as the frost heave test apparatus equipped with BEs). As shown schematically in Figure 2a, two pairs of BEs are provided, in the upper and lower plates and in the mold, respectively. The BE pair in the cooling plates measures the velocity $(V_s)_{vh}$ of the shear wave that oscillates horizontally and propagates vertically, while the BE pair in the mold measures the velocity $(V_s)_{hh}$ of the shear wave that oscillates and propagates horizontally (Kawaguchi et al. 2001, Yamashita and Suzuki 2001). The BE pair in the mold was removed in freeze-thaw tests to avoid any damage by freezing, and mounted again after thawing for BE tests. The third apparatus is a direct box shear test apparatus that permits freeze-thaw test in the shear box (designated hereinafter as the temperature-controllable direct box shear apparatus). The shear box is schematically represented in Figure 2b. The specimen is a disk 6 cm in diameter and 4 cm in thickness. A coolant is circulated in the piston and pedestal for temperature control. The shear box, or the circumferential surface of the specimen, is thermally insulated by a two-centimeter-thick acrylic resin layer. A rubber sheet, 0.3 mm in thickness, is placed between the upper and lower halves of the shear box during freeze-thaw tests to prevent water leakage, and is removed for shear tests to leave a 0.2mm clearance between the box halves.

Figure 3 shows grain size distribution curves for the two frost-susceptible fine-grain soils used in this study. One is weathered volcanic ash obtained at Kitami City, Hokkaido (sample V) which was used in the tests using the two frost heave test apparatus. The test specimens were prepared by compacting the volcanic ash sample conditioned to be slightly drier than with the optimum water content. The other, used in the temperature-controllable direct box shear tester, is a mixture of clay commercially available in dry powder and silt at a ratio of 1:1 by weight, which was made to a slurry at twice the liquid limit and then consolidated one-dimensionally to a vertical stress $\sigma_v = 100$ kPa (sample CL, $w_L = 38\%$, $I_p = 19$).

3 RESULTS AND DISCUSSION

3.1 Void ratio and coefficient of permeability

Falling head permeability tests were performed before and after the freeze-thaw tests using the frost heave test apparatus in order to study effects of freeze-thaw cycles on the void ratio e and the coefficient of permeability k . Sixteen specimens were prepared using a rammer and a mold at three levels of compaction energy: 126, 284, and 550 kJ/m³ (Nakamura et al. 2011). The vertical stress was $\sigma_v = 10$ kPa for all the tests. Six of the specimens underwent three freeze-thaw cycles and k was measured before and after each cycle. The frost heave test method specified in the JGS 0172-2009 standard was used. In the thawing tests, the specimen was dewatered through the top and bottom surfaces held at 5°C, and then saturated again.

Figure 4 shows the rate of frost heave U_h (mm/h) and frost heave ratio ζ (%) of the specimen as functions of void ratio e at the beginning of freezing. A greater e means more pore water and lower tensile strength, which might suggest easier ice lens formation. Actually, however, both U_h and ζ are lower at higher e at the beginning of freezing, presumably because of inhibition of continuity of the unfreezable water needed for frost heave (Nakamura et al. 2011). The results of the three consecutive freeze-thaw cycles, indicated by the points connected with lines,

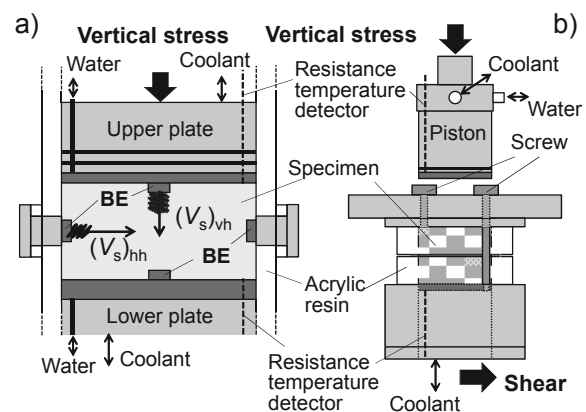


Figure 2. Test apparatus diagrams (a): frost heave apparatus equipped with BEs, b): temperature-controllable direct box shear apparatus)

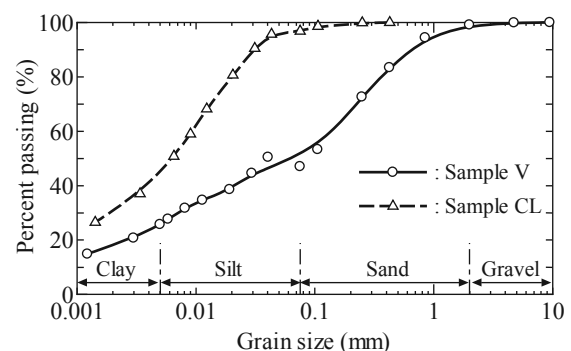


Figure 3. Grain size distribution of samples used in the tests

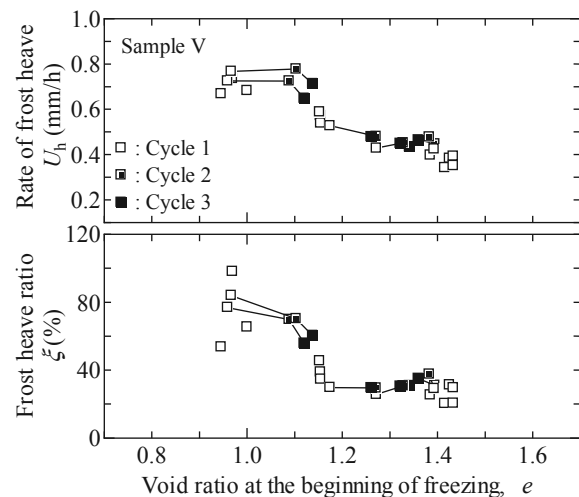


Figure 4. U_h and ζ as functions of e at the beginning of freezing

show that the repeated cycles lead to convergence to fairly constant values of U_h and ζ .

Figure 5 represents changes in the void ratio e and coefficient of permeability k of the specimen through the freeze-thaw cycles. The data for each specimen are connected by lines. As observed in Figure 4, the freeze-thaw cycles decrease greater initial e values and increase smaller initial e values, eventually leading to convergence to a relatively limited range of e values between 1.1 and 1.3. Ono et al.(2003) reported that freeze-thaw history decreases e of clay at a normally consolidated state and increases e of clay with a larger over consolidation ratio. This observation is in agreement with the present results assuming that higher compaction energies on the specimen result in over consolidation in terms of the specimen's mechanical

Table 1. Conditions of direct box shear tests and changes in void ratio

Test	Freeze -thaw cycle	σ_v (kPa)	Void ratio, e	
			After consol. (before freezing)	Before shear (after thawing)
DBS1	Yes	12	0.97	1.00
DBS2			0.99	1.02
DBS3			1.01	1.05
DBS4	No	75	1.01	
DBS5	Yes		0.97	0.91
DBS6			0.96	0.92
DBS7	No		0.95	

characteristics. The coefficient of permeability k changes along with the changes in e , thus the change in k through the freeze-thaw cycles is considered to be strongly dependent on the changes in e . Similar to the above, k converges on about 5×10^{-8} m/s. These observations explain why k has been reported to be increased by freeze-thaw cycles by some investigators and to be decreased by others (Chamberlain et al. 1990, Starke 1989).

3.2 Deformation and strength properties

Table 1 summarizes the conditions and results of the tests (DBS 1-7) performed with the temperature-controllable direct shear box apparatus. Two levels of the vertical stress σ_v , 12 and 75 kPa, were used for loading and shear. Freezing and thawing tests were replicated several times under identical conditions. The temperatures of the top and bottom surfaces of the specimen were set to 4°C and 0°C, respectively and lowered at a rate of 0.2°C/h for freezing and increased at a rate of 0.4°C/h for thawing until the top surface reached 20°C. The constant-pressure direct box shear tests were performed at a rate of 0.02 mm/min with a 0.2 mm gap, or clearance, between the upper and lower halves of the shear box. Table 1 shows that freeze-thaw cycles increase e of the specimens with high over consolidation ratios (OCRs) (DBS 1-3) and decrease e of those with lower OCRs (DBS 5 and 6).

Figure 6 illustrates the shear stress τ and vertical displacement ΔH as functions of horizontal displacement δ in the constant-pressure direct box shear tests. The specimens that have not undergone a freeze-thaw cycle clearly show higher stiffness at the beginning of shear, and generally higher maximum shear stress τ_{max} . Both increase and decrease of the shear strength after freeze-thaw cycles have also been reported (Aoyama et al. 1985, Ogata et al. 1985). Triaxial test results have also shown that freeze-thaw cycles make high-OCR cohesive soil to some extent similar to normally consolidated soil whose shear strength decreases with increase in e and, in contrast, make normally consolidated soil to some extent similar to over consolidated soil whose strength increases with decrease in e (Ono et al. 2003). This study shows a similar tendency for the dilatancy characteristics. An exception is the test DBS 7, where τ_{max} is relatively high in spite of e higher than in DBS 5 or 6. This is presumably due to structurally weak zones in the specimen formed by melting of ice lenses as the shearing direction is almost parallel to the surface where ice lenses formed in the present experiments. This mechanism suggests that zones with very low shear strength may appear in the soil elements in a slope in snowmelt season. Similarly, variation in deformation-strength characteristics after freeze-thaw cycles can be explained by the existence of structurally weak zones in the specimens.

3.3 Elastic shear modulus

Effects of freeze-thaw cycles on elastic shear modulus G and its anisotropy were studied using the frost heave test apparatus with BE on six specimens compacted to target densities by impacting a piston installed in the mold. BE tests were

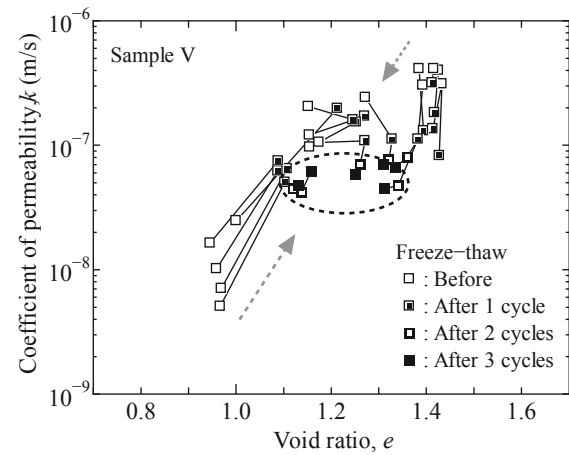
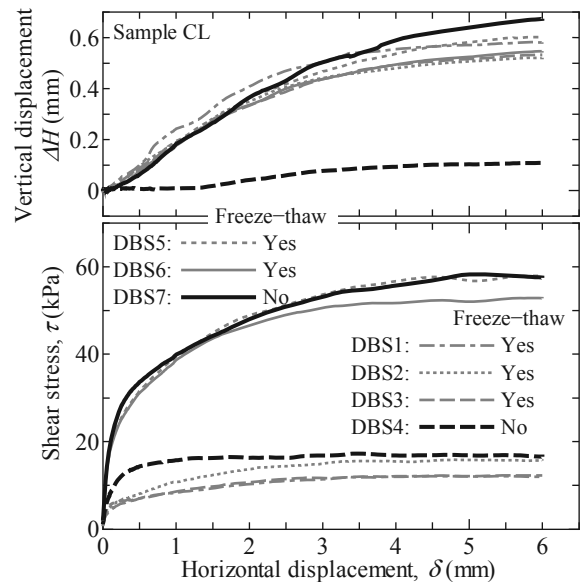
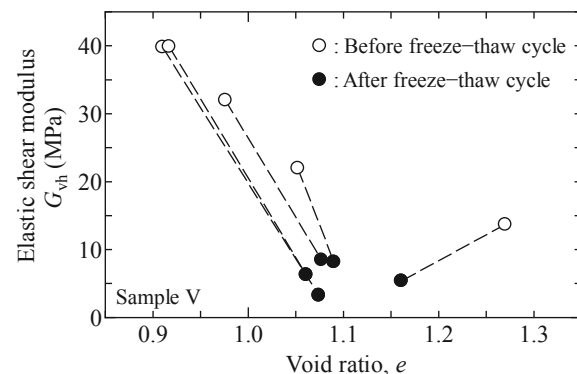

 Figure 5. Changes in k and e by freeze-thaw cycles


Figure 6. Difference in direct shear test results by freeze-thaw cycle


 Figure 7. Changes in G_{vh} and e by freeze-thaw cycles

conducted under $\sigma_v = 10$ kPa, and G_{vh} and G_{hh} of four specimens before and after freeze-thaw cycles were measured. The other two specimens underwent stepwise stress cycles (10 → freeze-thaw → 10 → 20 → 40 kPa or 10 → 20 → 40 → freeze-thaw → 40 → 20 → 10 kPa), during which G_{vh} and G_{hh} were measured.

Figure 7 shows changes in e and G_{vh} in freeze-thaw cycles at $\sigma_v = 10$ kPa for five specimens. The void ratios clearly converge on the same range as in Figure 5; G_{vh} sharply decreases to lower

than 10 MPa irrespective of the magnitude of e before freezing or increase/decrease in e after freeze-thaw cycles.

Figure 8 illustrates changes in G_{vh} and G_{hh} after freeze-thaw cycles. The decrease in G_{hh} is not so remarkable as that in G_{vh} , probably because the latter results from the velocity of the shear wave that traverses structurally weak zones formed by disappearance (melting) of ice lenses. The structural damages due to formation and disappearance of ice lenses are thus believed to greatly influence the deformation-strength properties of soil, in agreement with the results shown in Figure 6. The fact that G_{vh} is higher than G_{hh} before freezing may be due to σ_v acting on the specimen to a greater extent than the horizontal stress σ_h (Yamashita and Suzuki 2001, Roesler 1979).

Figure 9 represents changes in G_{vh} and e under stepwise stress cycles in the two tests mentioned above. The decrease in G_{vh} by freeze-thaw cycles under $\sigma_v=40$ kPa is smaller than in the other test because of a very small change in e during the cycle. While freeze-thaw cycles do not greatly influence the effect of σ_v on changes in G_{vh} , they lower G_{vh} to a considerably greater extent than do σ_v changes, and G_{vh} cannot be restored by loading or unloading in any of the tests. This suggests that structural damages in soil due to formation and disappearance of ice lenses are not readily restored.

The results described above indicate that changes in the permeability of soil exposed to freeze-thaw cycles can be explained in terms of changes in the void ratio e , but its deformation-strength property is not fully described in terms of changes in e because of effects of structural damages in the soil resulting from formation and disappearance of ice lenses.

4 CONCLUSIONS

- i) U_h and ζ of compacted fine grain soil are lower for higher e before freezing, and converge to fairly constant values after freeze-thaw cycles.
- ii) Changes in k follow those in e due to freeze-thaw cycles.
- iii) The initial stiffness and τ_{max} in the constant pressure box shear tests were greater before freeze-thaw cycles irrespective of increase or decrease in e due to freeze-thaw cycles.
- iv) Decrease in G_{vh} is considerable and greater than that in G_{hh} irrespective of e before freezing or increase/decrease in e due to freeze-thaw cycles.
- v) Results of the box shear tests and BE tests suggest that structural damages resulting from formation and disappearance of ice lenses have marked influence on the deformation-strength properties of soil.
- vi) Results of BE tests with different vertical stresses before and after freeze-thaw cycles suggest that structural damages resulting from formation and disappearance of ice lenses cannot be readily restored.

5 REFERENCES

- Ueno, K., Rui, D., Nakamura, D., Ito, Y., Yamashita, S. and Suzuki, T. 2010. Behavior of vegetation protection slopes during freezing and thawing, *Japanese Geotechnical Journal*, 5(3): 413-424 (in Japanese).
- Kawaguchi, T., Mitachi, T. and Shibuya, S. 2001. Evaluation of shear wave travel time in laboratory bender element test, *Proc. of the 15th ICSMGE*, 1: 155-158.
- Yamashita, S. and Suzuki, T. 2001. Small strain stiffness on anisotropic consolidated state of sands by bender elements and cyclic loading tests, *Proc. of the 15th ICSMGE*, 1: 325-328.
- Nakamura, D., Suzuki, T., Goto, T., Kim, H., Ito, Y. and Yamashita, S. 2011. Changes in the permeability coefficient and the void ratio of compacted soil by the effect of freeze-thaw cycles, *Journal of JSCE, C*, 67(2): 264-275 (in Japanese).

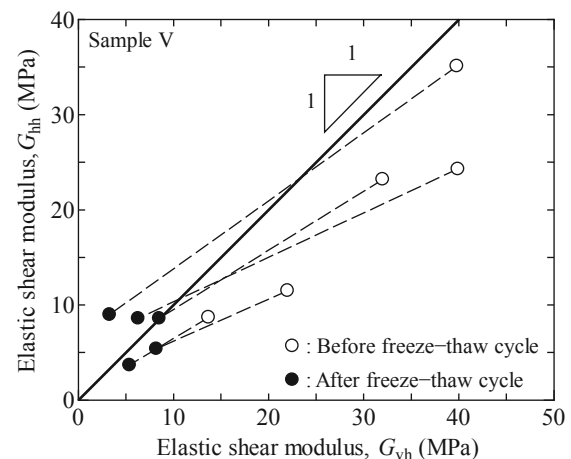


Figure 8. Changes in G_{vh} and G_{hh} by freeze-thaw cycles

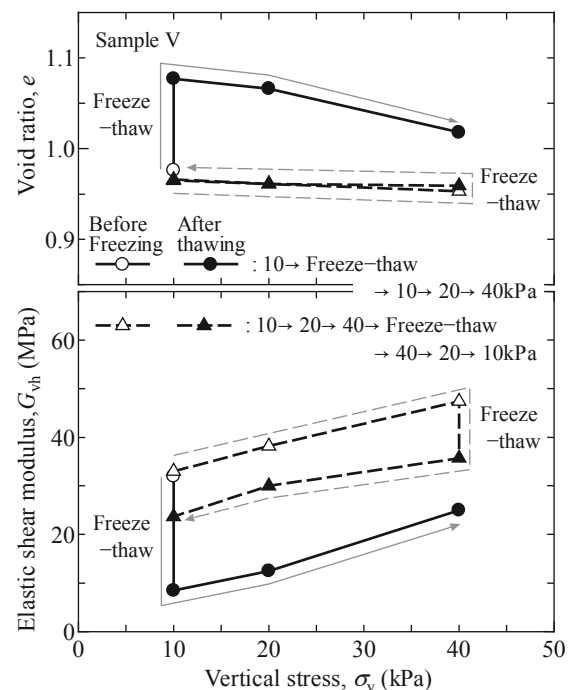


Figure 9. Difference in direct shear test results by freeze-thaw cycle

- Ono, T., Kodama, D. and Kato, T. 2003. The properties of normally and overconsolidated clay after freezing and thawing history, *Journal of JSCE*, 743(III-64): 47-57 (in Japanese).
- Chamberlain, E., Iskandar, I. and Hunsicker, S. E. 1990. Effect of freeze-thaw cycles on the permeability and microstructure of soils, *Cold Region Research and Engineering Laboratory*, 90(1): 145-155.
- Starke, J. O. 1989. Effect of freeze-thaw weather conditions on compacted clay liners, *Proc. of the 12th Annual Waste Conference*: 412-420.
- Aoyama, K., Ogawa, S. and Fukuda, M. 1985. Temperature dependencies of mechanical properties of soils subjected to freezing and thawing, *Proc. of the 4th International Symposium on Ground Freezing*: 217-222.
- Ogata, N., Kataoka, T. and Komiya, A. 1985. Effect of freezing-thawing on the mechanical properties of soil, *Proc. of the 4th International Symposium on Ground Freezing*: 201-207.
- Roesler, S.K. 1979. Anisotropic shear modulus due to stress anisotropy, *Journal of the Geotechnical Engineering Div., ASCE*, 105(GT7): 871-880.

Characterization of geomechanical and hydraulic properties of non-wettable sands

Caractérisation des propriétés géomécaniques et hydrauliques des sables non mouillants

Kim D., Yang H.-J., Yun T.S.

Department of Civil and Environmental Engineering, Yonsei University, Yonsei-ro 50, Seoul, 120-749, Republic of Korea

Kim B., Kato S.

Department of Civil Engineering, Kobe University, Rokkodaichou 1-1, Nada-gu, Kobe, 657-8501, Japan

Park S.-W.

Department of Civil and Environmental Engineering, Dankook University, Suji-gu, Yongin, 448-701, Republic of Korea

ABSTRACT: Wildfire, diagenesis, and organic contamination often induce the non-wettability in soils which in turn dominate physico-mechanical behaviours and control the contact angle and capillary pressure. This study presents the characterization of hydraulic and geomechanical properties of non-wettable sands using artificially synthesized hydrophobic sands. The critical hydrostatic pressure that initiates the fluid intrusion into dry hydrophobic sands is measured to estimate permeation boundary. Hydraulic conductivity values of both hydrophilic and hydrophobic sands under fully saturated condition are examined to evaluate the drag force effect. For geomechanical study, a series of constant water content compression tests are performed to assess the mechanical behavior on a reconstituted specimen of unsaturated non-wettable sands. The stress-strain relationships according to the degree of saturation under confining pressure condition are observed so as to derive the suction stresses. Results highlight that the surface modification at nano-scale determine the spatial configuration of water phase in pore space and its impact on fluid flow and strength with varying degree of saturation prevails.

RÉSUMÉ : Les incendies, la diagenèse et la contamination organique induisent souvent une non-mouillabilité des sols qui, à son tour, domine leurs propriétés physico-mécaniques et contrôlent l'angle de contact et la pression capillaire. Cette étude présente la caractérisation des propriétés hydrauliques et géomécaniques de sables non mouillants en utilisant des sables hydrophobes synthétisés artificiellement. La pression hydrostatique critique qui déclenche l'intrusion de liquide dans les sables hydrophobes secs est mesurée pour estimer la limite de perméation. Les valeurs de conductivité hydraulique pour des sables hydrophiles et hydrophobes en conditions totalement saturées sont examinées afin d'évaluer l'effet de la force de trainée. Pour l'étude géomécanique, une série d'essais de compression à teneur en eau constante ont été réalisés pour évaluer le comportement mécanique d'éprouvettes reconstituées de sable non saturé et non mouillant. Les relations contrainte-déformation en fonction du degré de saturation sous condition de confinement sont étudiées afin de dériver les contraintes de succion. Les résultats soulignent que la modification de surface à l'échelle nanométrique détermine la configuration spatiale de la phase aqueuse dans l'espace poreux et son impact sur l'écoulement du fluide et sur la résistance avec la variation du degré de saturation.

KEYWORDS: non-wettable sands, water repellency, friction angle, critical pressure, hydraulic conductivity

MOTS-CLÉS : sables non mouillants, hydrophobie, angle de frottement, pression critique, conductivité hydraulique

1 INTRODUCTION

Soils in nature often become hydrophobic (non-wettable) features, due to organic pollutants, natural hazards such as wildfire and environmental pollution accidents such as oil spill, whereas the accumulated geotechnical knowledge tends to be somewhat limited to hydrophilic (wettable) soils, particularly in the field of unsaturated soil mechanics.

Rodriguez et al. (1997) defined that the cause of the soil particles property which draw water is the high surface free energy. On the contrary, surface of hydrophobic soil particle excludes water and acts as a diffusion barrier which disturb substances diffuse by water (Goebel et al., 2007). Water layer formed on the hydrophilic soil particle is 10 times thicker than the water layer of the hydrophobic soil particle (Derjaguin and Churaev, 1986). Such differences in surface properties of soils result in clear distinctions even in macro scale (Frattolillo et al., 2005; Nguyen et al., 1999). Therefore, the surface wettability at particle scale controls the macro scale manifestation in soils.

Previous studies have been carried out so as to examine the relationship between the suction and the shear strength using the direct shear test for natural soils (e.g., Donald, 1956; Escario, 1980; Escario and Saez, 1986; Gan et al., 1988; Kim et al., 2010a), exhibiting that the suction developed at inter-particle contact causes the unique evolution of shear strength in natural soils. The degree of saturation for both hydrophilic and

hydrophobic sands imposes the different evolution of thermal and electrical conduction as well as the friction angle (Byun et al., 2011; Kim et al., 2010b), attributed to the surface wettability.

This paper presents the series of experimentation to capture the surface wettability effect with the scope of hydraulic and geomechanical behaviors of both hydrophilic and hydrophobic soils. Moreover, it is examined on how the existence of the meniscus water affects the shear behavior using a direct shear apparatus.

2 EXPERIMENTAL STUDY

2.1 Materials

The granular materials used in this study were Jumunjin sands (specific gravity $G_s=2.59$, maximum porosity $n_{max}=0.465$, minimum porosity $n_{min}=0.375$). This materials are relatively uniformized (coefficient of uniformity $C_u=1.16$) with the mean particle size $D_{50}=0.5\text{mm}$. Sands without any treatment were tested for wettable (hydrophilic) samples whereas the non-wettable samples were sands chemically treated by silylation process (Zycosoil manufactured by Zydex industries, diluted ratio=1:100 with water). Cleaned sands were fully submerged within the reactive solution and the reaction was allowed for 72 hours at room temperature condition. The reaction formed

Alkylsiloxane on the particle surface with molecular level. Then, the mixture of sands and solution was oven-dried at 80°C.

2.2 Optical observation

Water distribution in hydrophilic and hydrophobic sand was observed by an optical microscope. Water exists mainly at the inter-particle contacts whereas hydrophobic sand induces the formation of water blobs on the surface attributed to the non-wettability (Figure 1). In order to obtain the contact angle, the glass slide was simultaneously treated with silane. Water droplets were dropped both on the non-treated glass slide and treated glass slide to observe the contact angle (θ) respectively (Figures 1-c and 1-d). The contact angle of hydrophobic surface is 85° which is more than six times higher than that of hydrophilic sand.

Large contact angle indicates that the soil has large interfacial tension between soil and water. Therefore meaning the soil surface has high hydrophobicity.

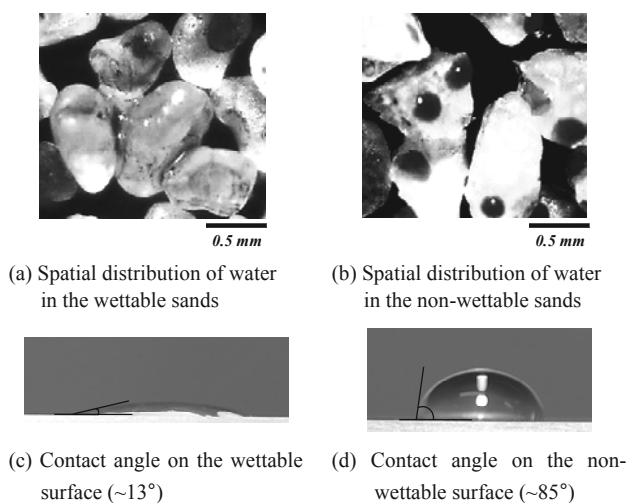


Figure 1. Optical observations of water droplet formed on the wettable and non-wettable samples in the micro scale.

2.3 Hydraulic properties

2.3.1 Critical hydrostatic pressure test

Water needs to overcome the capillary pressure in hydrophobic sand so that the critical pressure exist over which water begins permeating into pore space. The experiment was conducted by using various particle sizes of non-wettable sands (Ottawa 20-30, Jumunjin sands and Ottawa F110). Dry samples seat in a cylinder with diameter 7 cm, height 15 cm. The water pressure is increased gradationally by 1 mm interval, and the height is measured when the water starts to permeate.

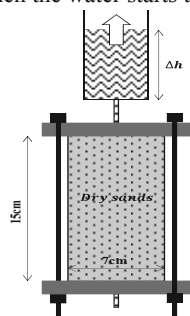


Figure 2. Experimental configuration of the critical hydrostatic pressures of non-wettable sands.

Figure 2 show the experimental configuration. The test was repeated three times for each specimen.

2.3.2 Hydraulic conductivity test

Although it is evident that the hydrophobic sand tends to repel water phase at unsaturated condition, caused by the modified surface wettability, the assessment of hydraulic conductivity at fully saturated condition is still required. Both hydrophilic and hydrophobic sands are subjected to constant head testing to obtain hydraulic conductivity values. The hydrophobic sands are forced for thoroughly mixing with water to attain 100% degree of saturation. The 147cm of head is maintained to the cell with the diameter of 15 cm and height 50 cm. The drained water was collected for 3 minutes and measured the weight

2.4 Compression test

A series of the direct shear tests were carried out under a constant pressure condition. The soil sample used was Jumunjin sand. The circular specimen is 60 mm in diameter and 20 mm in height, and has the relative density of about 75% for the maximum dry density. In case of a natural dried condition, the sand specimens were prepared by the air pluviation method. The vertical stresses of 20, 50, 80 kPa in the consolidation process were loaded. On the other hand, the sand specimens according to the degree of saturation under the unsaturated condition were prepared by the static compaction method. The vertical stress of 50 kPa in the consolidation process was loaded. The opening of 0.2 mm between the lower and upper shear boxes was set. The shearing rates of the natural dried condition and the unsaturated condition were 3.3×10^{-4} mm/sec and 3.3×10^{-5} mm/sec, respectively.

3 RESULT AND DISSCUSSION

3.1 Hydraulic properties

The critical pressure sharply increases as the particle size decreases (e.g., the corresponding size of pore throat should decrease) in Figure 3. The denial of water permeation acts as a hydraulic barrier for a given critical pressure while it may cause the surface erosion. Once the water begins permeating, the preferential flow is predominant by forming fingering. The evolution of critical pressure follows the reciprocal relationship between capillary pressure and pore radius defined in Young's equation.

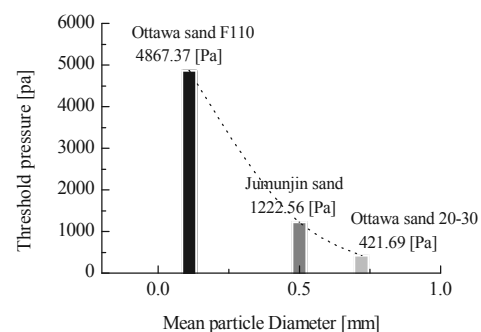


Figure 3. Critical hydrostatic pressures of non-wettable sands

The estimated hydraulic conductivity values increases with porosity whereas there is no noticeable difference between two specimens. It may be attributed that the surface modification by organic materials may not reduce the drag force in hydrophobic sands. It is noted that the feasibility of 100% saturation for hydrophobic sand is quite low so that the gathered hydraulic conductivity delineates the upper bound.

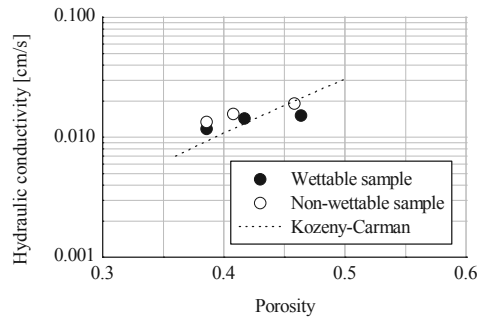


Figure 4. Hydraulic conductivity varied with porosity of wettable and non-wettable sands.

The dotted line in Figure 3 is made based on the Kozeny-Carman equation (Bear, 1972; Carman, 1956)

$$k = \frac{\rho g}{\mu} \cdot \frac{n^3}{(1-n)^2} \quad (1)$$

where ρ is the density of water, g is gravitational acceleration, and μ is the viscosity. The estimated value obtained by Kozeny-Carman equation show similar range and aspect to the values from the experiment, confirming validity of the experiment.

Wettable samples tend to adsorb water on the surface due to relatively strong capillary effect. On the other hand, non-wettable samples which have relatively low surface free energy, have characteristics that exclude water. So that the water flow at the interface between the particles is affected by the repellency of capillary pressure. It allows predicting faster fluid flow in non-wettable sands than the fluid flow in wettable Jumunjin sand. Results show that the hydraulic conductivity values of both soils run with nominal difference, which highlights that the wettability effect at particle scale seems negligible for fluid flow at pore scale.

3.2 Geomechanical properties

From the results of a series of the direct shear tests using the jumunjin sand under the natural dried condition, the cohesion and the angle of internal friction for the hydrophilic and hydrophobic sands were $c=0$, $\phi=36.1^\circ$, and $c=0$, $\phi=27.9^\circ$, respectively as shown in Figure 5. The difference in friction angle is attributed to the surface modification grafted by organic component. The existence of silane on particle surface serves as lubricant to reduce the friction angle.

Figure 6 shows the comparison of the shear strength due to the suction between the hydrophilic and hydrophobic sands. The formation of meniscus inducing the suction pressure increases the shear strength of hydrophilic sand while the hydrophobic sand imposes less defined meniscus at the inter-particle which causes the quasi-constant value of strength regardless of saturation. It emphasizes the role of suction in hydrophilic sands on the geomechanical properties. This tendency can be obviously observed through the comparison of the maximum shear stresses between the hydrophilic and hydrophobic sands with varying degree of saturation in Figure 7.

In case of the hydrophobic sand, the maximum shear stress at varying degree of saturation follows the value at dry condition without significant deviation. On the other hand, the maximum shear stress of hydrophilic sand is captured at $\sim 50\%$ degree of saturation. Note that the previous research on the evolution of maximum shear stiffness ($G_{\max}$) is attained at $\sim 5\sim 10\%$ degree of saturation (Byun et al., 2011). Although the negative capillary pressure should increase with decreasing degree of saturation, the different strain regime between the

small-strain stiffness and large-strain strength may result in the different peak saturation.

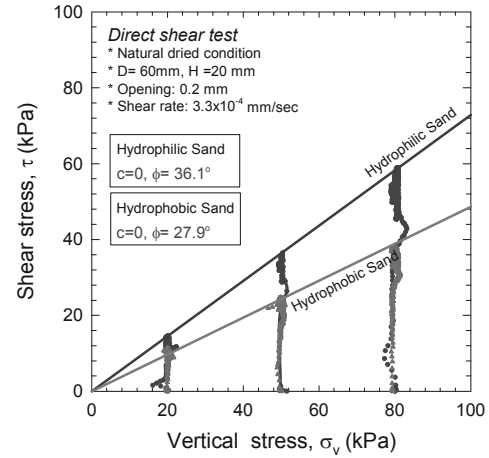


Figure 5. The relationship between vertical stress and shear stress under the natural dried condition.

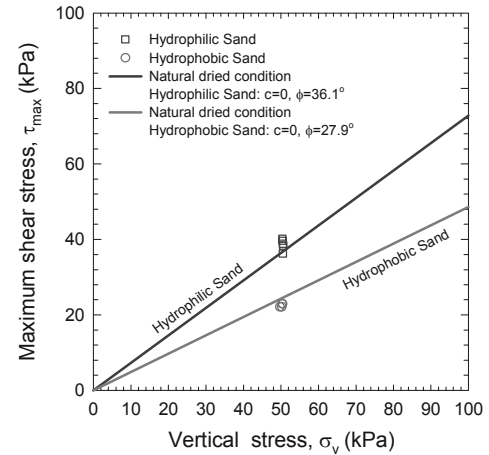


Figure 6. Comparison of the shear strength due to the suction between the hydrophilic and hydrophobic sands.

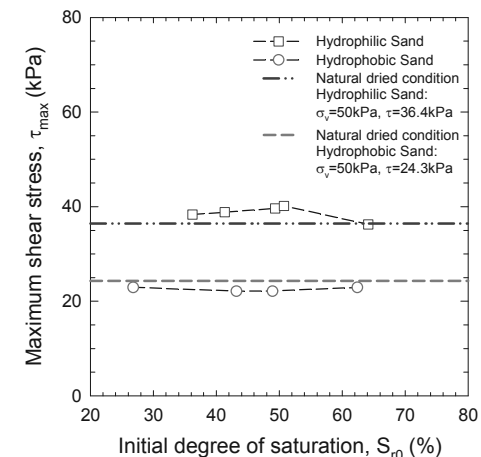


Figure 7. Variation of the maximum shear stresses between the hydrophilic and hydrophobic sands according to the initial degree of saturation.

4 CONCLUSIONS

Soils in geotechnical engineering have been regarded as the wettable condition while the non-wettable soils prevail in nature.

The water repellency and corresponding hydraulic and geomechanical aspects needs to be adopted in engineering practice to correctly estimate the overall behaviors. In this study, wettable and non-wettable specimens were artificially synthesized to preliminarily evaluate their basic properties. The following observations can be made:

1. The critical hydraulic pressure above which water begins permeating follows the Young's formula. The non-wettable sand exhibits the distinct water repellency and the critical pressure increases with decreasing pore size.
2. The hydraulic conductivity values of both hydrophilic and hydrophobic sands appear alike at fully saturated condition. Although the hydraulic conductivity with varying degree of saturation requires further study, the surface wettability may not affect the overall fluid flow behavior at macro-scale.
3. From a series of the direct shear tests using the hydrophilic and hydrophobic sands, the surface modification at sub-micrometer scale clearly reduce the shear strength of hydrophobic sand at dry condition. The effect of degree of saturation is minimized for hydrophobic sand due to the less defined meniscus at inter-particle contacts. Yet, the shear strength of hydrophilic sand evolves with varying degree of saturation, exhibiting the maximum value at ~ 50%.

5 ACKNOWLEDGEMENTS

This research is supported by the basic science research program (No. 2012-0008233) and the Public welfare & Safety research program (No. 2012M3A2A1050975) through the National Research Foundation of Korea(NRF) funded by the Ministry of Education, Science and Technology.

6 REFERENCES

- Bear, J. 1972. Dynamics of fluids in porous media. *New York: American Elsevier Publishing Company, Inc.*
- Byun, Y.-H., Tran, M.K., Yun, T.S., and Lee, J.-S., 2011. Strength and stiffness characteristics of unsaturated hydrophobic granular media, *Geotechnical Testing Journal*, 35(1), 1-8.
- Carman, P. C. 1956. Flow of gases through porous media. *Butterworths Scientific Publications London.*
- Derjaguin, B. and N. Churaev. 1986. Properties of water layers adjacent to interfaces. *Fluid interfacial phenomena*. 663-738.
- Donald, I.B., 1956. Shear strength measurements in unsaturated non-cohesive soils with negative pore pressures. *Proc. 2nd Australian and New Zealand Conference on Soil Mechanics and Foundation Engineering*. 200-205.
- Escario, V., 1980. Suction controlled penetration and shear tests. *Proc. 4th International Conference on Expansive Soils*, Denver, Co, 2, 781-797.
- Escario, V., Saez, J., 1986. The shear strength of partly saturated soils. *Geotechnique* 36(3), 453-456.
- Frattolillo, A., G. Giovinco, M. Mascolo and A. Vitale. 2005. Effects of hydrophobic treatment on thermophysical properties of lightweight mortars, *Experimental thermal and fluid science*, 29(6), 733-741.
- Gan, J.K.M., Fredlund, D.G., Rahardjo, H., 1988. Determination of the shear strength parameters of an unsaturated soil using the direct shear test. *Canadian Geotechnical Journal* 25(8), 500-510.
- Goebel, M. O., S. K. Woche, J. Bachmann, A. Lamparter and W. R. Fischer. 2007. Significance of wettability-induced changes in microscopic water distribution for soil organic matter decomposition. *Soil Science Society of America Journal*. 71(5), 1593-1599.
- Kim, B.S., Shibuya, S., Park, S.W., Kato, S., 2010. Application of suction stress for estimating unsaturated shear strength of soils using direct shear testing under low confining pressure. *Canadian Geotechnical Journal* 47(9), 955-970.
- Kim, D.H., Kim, Y.J., Lee, J.-S., Yun, T.S., 2010. Thermal and electrical response of unsaturated hydrophilic and hydrophobic granular materials. *Geotechnical Testing Journal* 34(5), 562-570.
- Nguyen, B.-L., Johannes, J., and Slob, E. C., 1999. Effects of Wettability on Dielectric Properties of Porous Media. *SPE Annual Technical Conference and Exhibition*, Oct 3-6 1999, Houston, TX, SPE, Richardson, TX.
- Rodriguez, E. and G. Newaz. 1988. The effect of sand surface treatment on sand-filled composites. *Polymer Composites*. 9(1), 93-102.

The strength change characteristics of weathering soil due to repeat freezing-thawing and drying-wetting

Les caractéristiques de changement de résistance du sol aux intempéries suite aux répétitions de la congélation-décongélation et du séchage-amortissement

Kim Y.S., Seong J.H.

Korea Infrastructure Safety and Technology Corporation, Republic of Korea

Kim S.S.

The Land and Housing Institute, Republic of Korea

ABSTRACT: Most researches are concentrated on sedimentary soil, and focused on the mechanical behavior of weathered soil from the viewpoint of sedimentary soil. Weathered soil shows great difference in engineering properties as the progression of weathering including the properties of parent rock. In this study, repeated freezing and thawing, drying and wetting test is conducted to recreate the effect of moisture and temperature, which might be considered as important for occurrence and efficacy on weathering among various factors in nature. In addition, to evaluate the strength changed by weathering degree, uniaxial test is conducted on sample with repeated freezing/thawing and drying/wetting. The specific gravity of the sample with the drying and wetting repeated decreased bigger than that with the freezing and thawing performed. In general, as the repetition number of freezing-thawing and drying-wetting increase, the compression stress of each sample decreased by a similar ratio. Especially, the strength change in the beginning by the repetition was significant. The uniaxial compression strength by the change of weathering degree decreases similarly for all the samples, but the axial deformation rate increases as the repetition numbers of the freezing and thawing and the drying and wetting increase.

RÉSUMÉ : La plupart des recherches se concentrent sur les sols sédimentaires et mettent l'accent sur le comportement mécanique des sols altérés du point de vue du sol sédimentaire. Le sol soumis aux intempéries montre une grande différence dans les propriétés mécaniques comme la progression de l'altération dont les propriétés de la roche mère. Dans cette étude, des tests de répétitions de congélation et de décongélation, de séchage et de mouillage sont effectués pour recréer l'effet de l'humidité et de la température, ce qui peut être considéré comme important pour l'apparition et l'efficacité sur l'altération des différents facteurs dans la nature. En outre, pour évaluer la résistance aux intempéries changée par degrés, l'essai uniaxial est effectué sur un échantillon répétés de congélation/ décongélation et séchage/humidification. Le poids spécifique de l'échantillon avec l'humidification et de séchage répétés diminution plus importante que celle de la congélation et la décongélation effectuée. En général, comme le nombre de répétitions de gel-dégel et séchage-mouillage augmente, la contrainte de compression de chaque échantillon a diminué dans une proportion similaire. Surtout, le changement de la force au début de la répétition est important. La résistance à la compression uniaxiale par le changement de degré intempéries diminue de même pour tous les échantillons, mais le taux de déformation axiale augmente à mesure que le nombre de répétition de la congélation et la décongélation et le séchage et de mouillage augmentation.

KEYWORDS: strength change characteristics, weathering soil, freezing/thawing, drying/wetting

1 INTRODUCTION

For weathering soil different from general deposit soil, its engineering properties sharply differ depending on the progress of weathering along with the characteristics of parent rock, which makes the study on this field difficult prior to an engineering research. In order to grasp the engineering properties of weathering soil, it is not enough to make a study from the dynamic view but a comprehensive investigation should be made from geological mineralogy, physics, chemistry, and dynamics. Through this, it is necessary to present how the properties of the soil from these views are expressed as a dynamic property and used in an engineering aspect.

Therefore, this study is to clarify the change of characteristics of granitic weathering soil due to the changes of temperature and water set by examining the existing studies on the effect of weathering factors on granitic weathering soil and considering the frequency and effect among the factors existing in the natural world. And for this, we are to promote the weathering artificially to express the weathering feature and weathering degree caused by weathering progress exponentially and consider correlations with the changes of soil's engineering characteristics so that a useful result may be drawn to accurately clarify the characteristics of granitic weathering soil in the future.

2 WEATHERING CHANGE TEST

For the weathering occurring in the real natural environment, various factors work together composite and the time scale of weathering is big that the effect of specific weathering factors on weathering and weathering acceleration are required in the study on weathering of soil. Thus, a test that controls the weathering speed and factors artificially by creating an extreme environment is suggested and it is necessary to identify the changes of soil's characteristics due to weathering in a quantitative aspect.

The most important thing in the weathering change test is to draw out the relation between testing environment and real environment as well as the relation between the weathering generated in the testing process and that in the real environment.

In this study, repetition tests of freezing and thawing as well as drying and wetting are conducted in order to reproduce the effect of temperature changes and water which are considered important in the aspect of occurrence frequency and effect among the various weathering factors existing in the natural world.

2.1. Repeat Freezing and Thawing

In the past, strength decrease due to freezing and thawing is thought to be caused by frozen gravitational water or absorption

water in the soil particles with a direct factor, but Aoyama and Fukuda(1990) proved that it can be generated in the closed system without water supply from under the ice layer.

In this study, Italian regulations that set the maximum and minimum temperatures based on real weather, are considered proper and applied to the test. In order to determine a proper temperature range suitable for the domestic weather conditions, the freezing and thawing temperatures are set based on the comparison of 10 years' weather data on Seoul, Icheon, Cheongju, Jeonju, Daegu, Milyang, Yeongju, Yeongwol, Wonju, Jecheon, and Taebaek along with its resulting temperature range of -21.1 to +26.5°C.

Therefore, the freezing and thawing temperatures to reproduce real weather conditions are set at -20°C and +20°C respectively, and the temperature duration at 4 hours to carry out a repetition test up to 200 times. For the water supply method, a freezing-induced method by maintaining comparative humidity rather than a flooding method is used, which is to reproduce a general effect occurring under the temperature condition below 0 not under special conditions such as rainfall or underground water.

2.2. Repeat Drying and Wetting

To reproduce the weathering caused by weakened binding power between the particles that compose soil when repeating the flooding and drying process in pure water excluding the effect of temperature, a drying and wetting repetition test was conducted.

For fresh rock or high consolidation rock, deterioration due to drying and wetting repetition does not occur significantly, however weakening of binding power between the particles and chemical weathering are promoted by the water permeated into a gap in the earth and sand state. In other words, chemical effects like reprecipitation of decomposed ions such as dissolution, filtration, deformation and salification change the characteristics of soil.

A method to copy rainfall is yet to be established that a total flooding method is applied to make a continuous contact between water and soil particles in this study. This is because only the surface of the soil particles would be affected if a periodic spraying method is applied.

3 STRENGTH CHANGE CHARACTERISTIC

3.1. Target Sample

The weathering soil sample to identify the strength change is classified into igneous rock, metamorphic rock, and sedimentary rock, and the soil underground has been collected after removing about 1.0m surface layer to exclude the effect of vegetation.

Table 1 shows the test codes by strength test for weathering degree change cycle.

3.2. Physical Characteristic Change

Weathering soil is a type of soil with a very high crushability compared to other types of deposit soil. Fig. 1 and Fig. 2 are representative results performed to know the effect of particle crushing caused by weathering progress.

As shown in the figures above, the grain-size distribution curves move to the left when repeating the freezing and thawing and the drying and wetting compared to that of the test beginning state, which means soil particles become finer and signifies that the distribution effect of weathering soil is very big because of the weathering caused by temperature and water. The previous studies shows that the granulation of quartz and feldspar by the particle change take the most cases of the granulation.

The increase in fine-grained soil due to the 190 time repetition of freezing and thawing and the 180 time repetition of drying and wetting are compared with each other and shown in Table 1. The report of Kim(2002) also pointed out that the shape and feature of the surface among basic structural characteristics have a huge impact on the physical and dynamic features of soil, and among many factors for the change of particle shape, the initial grain diameter, mineral composition of particles, and features for weathering work as important factors and the characteristics change according to the weathering progress.

Table 1. Classification of strength test and test codes

Artificial Weathering Conditions	Estimated Parent Rock	Sample Type	Classification Code	Cycle Times	Direct shear Test Condition	Uniaxial Compression Test Condition
Freezing Thawing (F)	Igneous rock (G)	Seoul	FGS	0	- initial condition 0.95Ydmax	- initial condition 0.95Ydmax -deformation Speed 0.1%/min -test code (U)
		Yuseong	FGY	10		
		Jeonju	FGJ	25		
	Sedimentary rock (S)	Cheongdo	FSC	40	-vertical Stress (kg/cm2)	
		Goryeong	FSG	60		
		Gwangju	FSK	80		
	Metamorphic Rock (M)	Sabuk	FMS	100	0.35 0.69 1.04 1.39	
		Mungyeong	FMM	120		
				150		
Drying Weting (D)	Igneous rock (G)	Seoul	DGS	190	-shear speed 0.5mm/min	
		Yuseong	DGY	0		
		Jeonju	DGJ	10		
	Sedimentary rock (S)	Cheongdo	DSC	40	-test code (S)	
		Goryeong	DSG	60		
		Gwangju	DSK	100		
	Metamorphic rock (M)	Sabuk	DMS	150		
		Mungyeong	DMM	180		

After checking the change of water content by weight during drying through a preparatory experiment, it turned out to have deteriorated below the optimum level in about 24 hours, so the repetition test of drying and wetting in this study has been performed with 24-hour cycle.

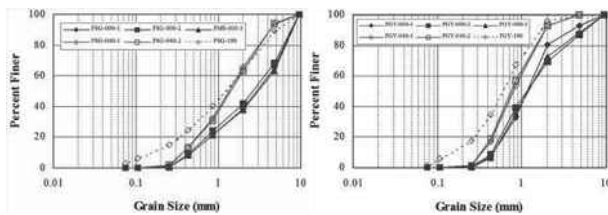


Figure 1. Grain-size distribution change caused by freezing and thawing repetition

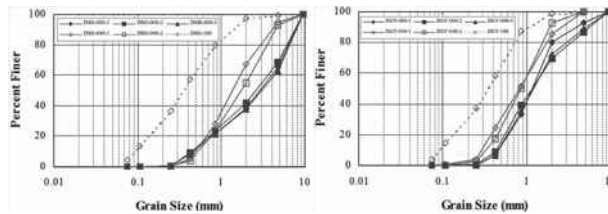


Figure 2. Grain-size distribution change caused by drying and wetting repetition

Table 2. Increase of fine grain by the change of weathering degree (unit : %)

Division	Seoul	Jeonju	Cheongdo	Goryeong	Gwangju	Sabuk
Freezing/ thawing	1.6	1.7	2.5	2.8	2.5	3.5
Drying/ wetting	3.2	5.9	3.3	4.6	1.6	0.8

The previous studies suggest that among primary minerals in weathering soil, especially feldspar and mica change selectively as weathering proceeds that it is possible to apply them as an index for weathering degree when distinguishing these components and measuring their specific gravities(Matsuo, 1979). This study is intended to measure the outward specific gravities and understand the changes according to the weathering progress.

First of all, after testing the samples with the freezing and thawing and the drying and wetting repeated 0, 40, and 190(180 for drying and wetting) times respectively according to KSF 2308, it turned out that the specific gravity soil particles decreases as the number of repetitions increases.

Figure 3 shows the change of specific gravity by the freezing and thawing repetition, signifying the specific gravity change appears high in the beginning of the repetition. Comparing the specific gravities in the beginning by parent rocks, it shows that metamorphic rock has the biggest one followed by sedimentary rock, and igneous rock has the lowest one, but for the decreasing specific gravity from the result of the weathering change test, it shows that sedimentary rock and metamorphic rock have a similar change while igneous rock has a comparatively smaller change.

Next, the change of specific gravity by the drying and wetting repetition are divided by parent rocks and shown in Figure 4, which indicates that the change of specific gravity decreases as the number of repetitions increases just like the freezing and thawing repetition, and the change in the beginning of the repetition appears great.

However, when comparing the decreasing specific gravities from the result of the weathering change test, igneous rock takes the highest one followed by metamorphic rock, and sedimentary rock takes the lowest one, which is different from the freezing and thawing case. Therefore, with the effect factors of weathering that causes the change of specific gravity, it is judged that the weathering by water has a more dominant effect on the change of specific gravity than that by temperature change.

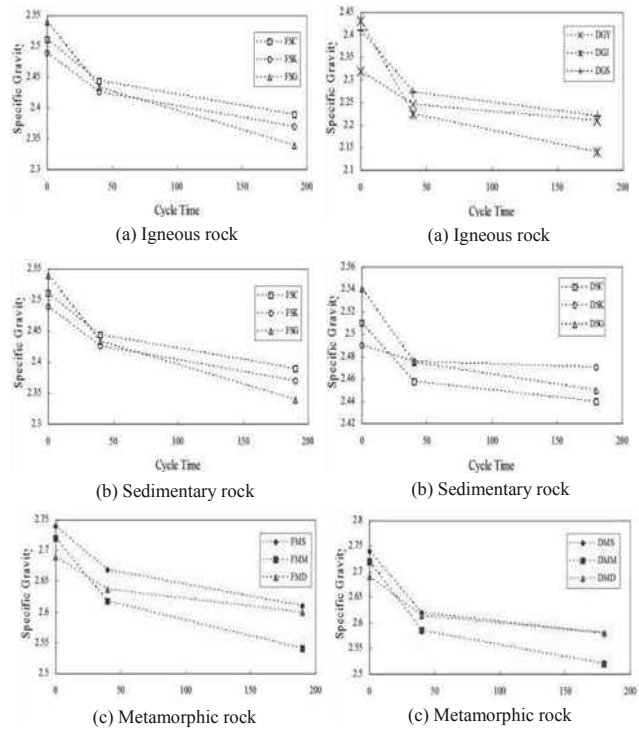


Figure 3. Change of Specific Gravity by Freezing and Thawing Repetition

Figure 4. Change of Specific Gravity by Drying and Wetting Repetition

Nevertheless, the real specific gravity of weathering soil changes very little, but feldspar and mica are changed selectively by the weathering that it is acknowledged that a comprehensive change trend should be traced after measuring the specific gravities of feldspar and mica.

3.3. Strength Characteristic Change

Figure 5 and Figure 6 are the result of an uniaxial compression test after thawing and drying so as to know the progress of strength changes due to the changes in the numbers of the freezing and thawing and the drying and wetting. In general, as the repetition number of freezing and thawing increased, the compression stress of each sample decreased by a similar ratio. The initial stress of each sample appeared different while the compression stress during the 190 time repetitions has a little difference.

In other words, the strength change in the beginning by the freezing and thawing is significant, which is similar to the study result that Yong et al.(1985) reported the strength decrease by the first repetition is the biggest. Compared with igneous rock, sedimentary rock and metamorphic rock shows a bigger strength decrease by the freezing and thawing repetition.

For the drying and wetting repetition, the strength by the increase of repetition number continued to decrease with a bigger change in the beginning than the overall decreasing tendency. The uniaxial compression strength by the change of weathering degree decreases similarly for all the samples, but the axial deformation rate increases as the repetition numbers of the freezing and thawing and the drying and wetting increase.

In other words, the softening of deformation rate appears in the initial result of the compression test, while the hardening of deformation rate becomes vivid as the repetition numbers of the freezing and thawing and the drying and wetting.

The reason why the strength decreases as the repetition number of the freezing and thawing increases, is not that the strength rovers completely due to thawing but it turns to the state with a certain degree of strength decrease and back to the

freezing state weakening its strength gradually. In addition, the strength decrease by the drying and wetting repetition is caused by the weakened binding power between soil particles.

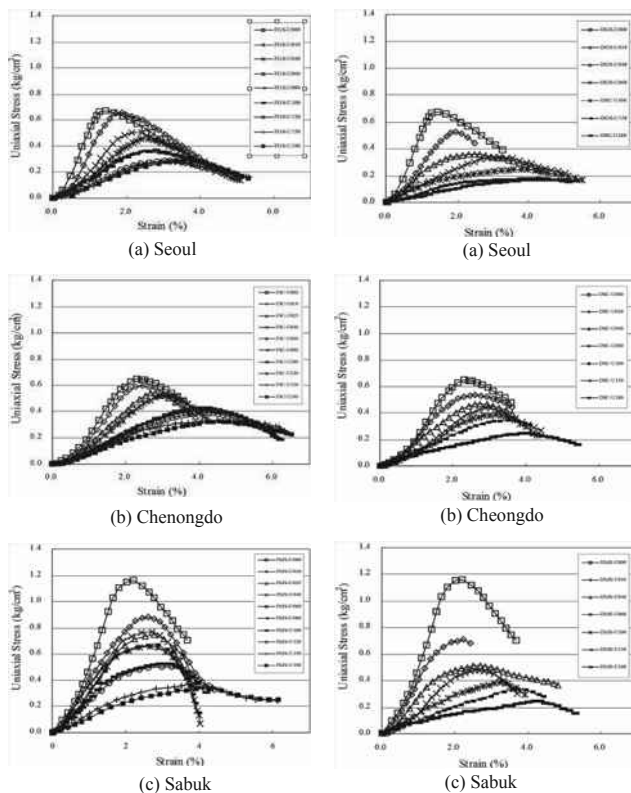


Figure 5. Uniaxial Compression Test Result of Freezing and Thawing Test Conditions

The soil particles in the strength recovery and expansion area of the soil in a dense state are oriented to show the maximum contact area perpendicular to the maximum main stress direction, so the expansion motion in a dense state is bigger than that in a loose state and with a bigger initial tangent modulus.

Negative pressure in the freezing and thawing boundary is the function of the gap size and on this point, the decrease of gap water pressure increased the effective stress. At this point, if a freezing starts, an ice nucleation is formed and a Cluster in between soil particles and water is discharged from the Cluster that the compression of the soil in the cluster is promoted by the increased effective stress and dehydration. Once the thawing starts, the cluster turns into a over consolidated state than the original soil through the process of re-absorption as a thawing water, leading to a collapse and weakening(Choi, 1996). From this, we can learn that some structural change may have occurred in the soil particles due to the freezing and thawing, and that it is similar to the study result by Kim(2002) that for the solid affected by the freezing and thawing, soil structure and physical and chemical properties change without increases of water content by weight.

4 CONCLUSIONS

For the characteristics of weathering soil, there should always be limits to a minute access because of the particularity of motion due to various weathering environment, weathering degree and effect factors. Especially, the domestic and overseas studies are focused on deposit soil and tend to analyze dynamic motion of weathering soil from the aspect of deposit soil that it is difficult to judge whether these results correspond to the real

motion of the weathering soil. Therefore, this study is to clarify the characteristic change of igneous weathering soil by the changes of temperature and water.

The strength change characteristics are identified by promoting the weathering artificially through the freezing and thawing and the drying and wetting processes.

After examining the effect of particle crushing by the progress of weathering, it is identified that the particles become finer through the freezing and thawing and the drying and wetting processes. It is understood that the granulation by water and temperature is caused by the granulation of quartz and feldspar due to the change of grain sizes.

By measuring the outward specific gravity and examining the change by weathering progress, it is understood that the specific gravity of soil particles decreases as the repetition numbers of the freezing and thawing and the drying and wetting increase. The change of specific gravity appeared big in the beginning of the repetition and decreasing as its number increases. The specific gravity of the sample with the drying and wetting repeated decreased bigger than that with the freezing and thawing performed.

And when judging the effect factors of the weathering that causes the change of specific gravity, it is judged that the weathering by water rather than that by temperature has more dominant effect on the specific gravity change.

In general, as the repetition number of freezing and thawing increased, the compression stress of each sample decreased by a similar ratio. Especially, the strength change in the beginning by the freezing and thawing was significant.

However, the initial stress of each sample appeared different while the compression stress during the 190 time repetitions has a little difference. For the drying and wetting repetition, the strength by the increase of repetition number continued to decrease with a bigger change in the beginning than the overall decreasing tendency. The uniaxial compression strength by the change of weathering degree decreases similarly for all the samples, but the axial deformation rate increases as the repetition numbers of the freezing and thawing and the drying and wetting increase.

It is very difficult to draw out the relation between testing environment and real environment as well as the relation between the weathering generated in the testing process and that in the real environment, and to identify this fact from the mere study result, so it is judged that a stored data and analysis for various soil qualities would enable a quantitative distinction in the weathering stage.

5 REFERENCES

- Aoyama, K., and Fucuda, M. 1990. Stress-Deformation characteristics of a soil after freezing and thawing. *Proceedings of the ninth Danube-European conference on soil Mechanics and Foundation Engineering*. Budapest. October. 2-5. pp. 393-403
- Matsuo, S., /Nisida, K. and Sasaki, S. 1979. Physical properties of weathered Granite soil particles and their effect on permeability. *soils and foundations*. Japanese Society of soil Mechanics and foundation engineering. vol. 19, No.1, pp.13-22
- S.Y. Choi. 1996. A study on the characteristics of the physical and engineering properties of weathered soil of freezing and thawing. Master Thesis, Kang-Won National Univ.
- Yong, R. N., Boonsinsuk, P., and Yin, C. W. P. 1985. Alteration of soil behavior after cycle freezing and thawing. *The 4th International Symposium on Ground Freezing*. August. pp.187-195
- Y.S. Kim. 2002. Development of the method to determine the weathering degree of decomposed granites soil for the evaluation of the engineering properties. Ph.D Thesis. Chung-Ang Univ.

Soil structure in gravel-mixed sand specimen and its influence on mechanical behavior

Structure du sol des échantillons de sable avec gravier et son influence sur le comportement mécanique

Kodaka T., Cui Y., Mori S., Kanematsu Y.
Meijo University

Lee K.-T.
CTI Engineering

ABSTRACT: Soil parameters are very important to conducting stability inspections of earth structures, e.g., river levees. However, the results of triaxial tests using reconstituted sand specimens change easily depending on the specimen preparation method. In the present paper, a series of conventional undrained triaxial tests is performed using specimens with the same void ratio, but different initial water contents. Even though the specimens are fully saturated before the consolidation process, the shear behavior varies remarkably with the different initial water contents. It is first assumed that the initial suction in the soil sample during the specimen preparation may produce various soil structures. Next, a numerical simulation by the SYS Cam-clay model is carried out to confirm this assumption. It is seen that the numerical simulation can reproduce the various types of experimental shear behaviors of the gravel-mixed sand derived from the different soil structures.

RÉSUMÉ : Les paramètres du sol sont très importants pour effectuer le contrôle de stabilité des structures en terre (ex : digue). Cependant, les résultats des essais triaxiaux utilisant des échantillons de sable reconstitués changent facilement en fonction de la méthode de préparation de ces échantillons. Dans cet article, une série d'essais triaxiaux non drainés conventionnels sont réalisés en utilisant des échantillons de même indice des vides mais avec une différente teneur initiale en eau. Bien que les échantillons aient été complètement saturés avant la phase de consolidation, le comportement des cisaillements change en fonction de la teneur en eau initiale. On peut supposer que la succion initiale lors de la préparation de l'échantillon peut induire des structures de sol différentes. Ensuite, une simulation numérique par modèle de SYS Cam-Clay est réalisée pour confirmer cette hypothèse. La simulation numérique peut reproduire les différents comportements en cisaillement des échantillons de sable avec gravier dérivés de différentes structures de sols.

KEYWORDS: soil structure, gravel-mixed sand, strength coefficient, triaxial test, numerical simulation, river levee.

1 INTRODUCTION

Soil parameters, e.g., strength coefficients c and ϕ , are very important to the design and the inspection of earth structures. However, practicing engineers do not necessarily note the reliability of the soil parameters. In the stability inspections of river levees consisting of gravel-mixed soils, the required soil parameters have to be obtained through laboratory triaxial tests using specimens reconstituted with mechanically stabilized soil and without large-sized gravel. In such cases, the test results change easily depending on the specimen preparation methods, even when the specimens are reconstituted to have the same common void ratio. In practice, however, there are no rules regarding the specimen preparation methods. The first aim of the present paper is to show the difference in the results obtained by triaxial tests using specimens of reconstituted gravel-mixed sand. Dry and wet soil samples, with various water contents, were tamped in steel molds to the prescribed degree of compaction. Specimens with the same void ratio, but different initial water contents, are prepared. These specimens are then fully saturated to conduct conventional triaxial tests under undrained conditions. Ishihara (1993) showed that each type of undrained shear behavior of the Toyoura sand specimens, made by various specimen preparation methods, is different from the others. In his study, the void ratios of all the specimens are different due to the initial structure of the sand particles. However, since the specimens in this study are reconstituted using well-graded natural gravel-mixed sand, it is possible for the initial void ratios of all the specimens to be the same. This point is quite different from the results of Ishihara's experiment.

A series of the triaxial tests is performed using the specimens, with three degrees of compaction and five initial water contents for each degree of compaction. The shear behavior, including both stress-strain and dilatancy relations, is seen to vary remarkably with the different initial water contents. The authors assume that the soil structure in the specimen causes this difference in shear behavior. Namely, the initial suction in the soil sample during the specimen preparation may produce the various soil structures. The second aim of this paper is to confirm the assumption using a numerical simulation by the SYS Cam-clay model (Asaoka et al. 2002).

2 TEST PROCEDURE

Figure 1 shows the grain size distributions of the gravel-mixed sand samples. The natural soil was taken from a real river levee. Mechanically stabilized soil is used in this study, which involved the removing of over 9.5mm gravels from the natural soil.

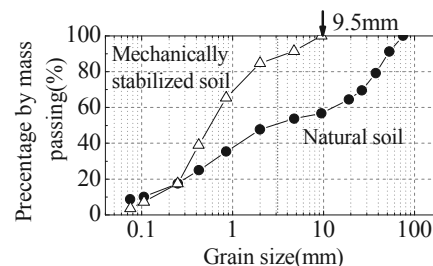


Figure 1. Grain size distributions of gravel-mixed sand used for this study.

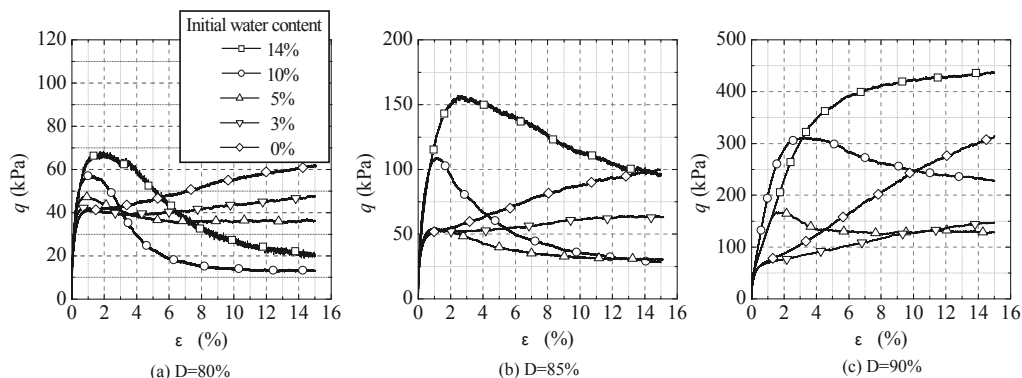


Figure 2. Deviator stress – axial strain relations of gravel-mixed sand with various initial water contents.

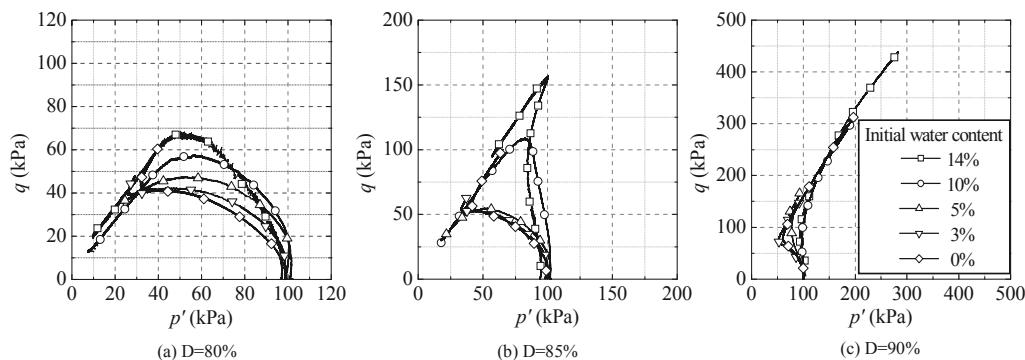


Figure 3. Effective stress paths of gravel-mixed sand with various initial water contents.

The specimens are 5cm in diameter and 10cm in height. The maximum dry density and the optimum water content of the mechanically stabilized soil were 2.0g/cm^3 and 10.5%, respectively. Three degrees of compaction are adopted, i.e., $D = 80, 85$ and 90%, in which the dry densities are 1.6, 1.7 and 1.8g/cm^3 . The test sample was mixed well by adding water to be the prescribed initial water contents. Then, the moisture-controlled sample was compacted to reconstitute it to the prescribed degree of compaction. The compaction was carried out by dividing the sample into five layers to make homogeneous specimens. In this study, ‘initial water content’ denotes the water content of the sample during the specimen preparation. All the specimens in this test were fully saturated, i.e., a B value of over 0.95, after setting them in a triaxial chamber by the double-vacuum method. Therefore, the effects of suction in the specimens are negligible in the evaluation of the test results.

In all the test cases, an isotropic consolidation was performed by applying an effective confining pressure of 100kPa, and then an undrained shear was conducted using the loading speed of 0.1%/min.

3 TEST RESULTS

Figures 2 and 3 illustrate the stress-strain relations and the effective stress paths, respectively. For each degree of compaction, strain-softening behavior can be observed in the cases of high initial water content. On the other hand, only the strain-hardening behavior can be seen in the cases of low initial water content. In particular, this tendency is very clear in the case of the degree of compaction of $D=85\%$. The specimens with 10% and 14% initial water contents show remarkable elastic characteristics in the early loading stage, i.e., large shear rigidity and $p'=\text{constant}$ behavior. Then, significant strain softening, accompanied by plastic volumetric compression, occurs.

Photos 1 and 2 show views of the reconstituted specimens with initial water contents of 10% and 0%, respectively. An overview of the specimen and a surface image observed by a

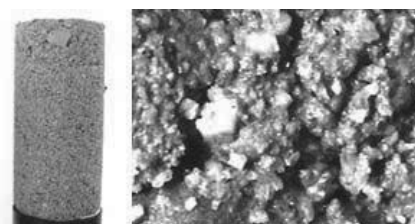


Photo 1. Overview of the specimen (left) and a surface image observed by a microscope (right) for initial water content 10%.

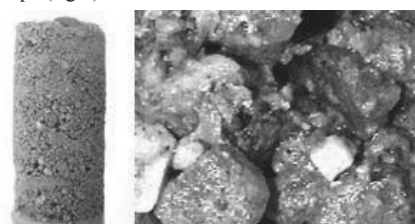


Photo 2 Overview of the specimen (left) and a surface image observed by a microscope (right) for initial water content 0%.

microscope are exhibited in each case. The degree of compaction for both specimens is $D=90\%$. From the overviews, the specimen with the 10% initial water content looks homogeneous. On the other hand, the specimen with the 0% initial water content looks uneven. In other words, there are two areas which containing rich coarse particles or rich fine particles. From the enlarged surface images, many fine particles sticking to coarse particles can be observed in the specimen with the 10% initial water content. Since relatively high suction existed in the specimen with 10% initial water content during the specimen preparation, the fine particles must be uniformly distributed around the coarse particles. This distribution may cause a highly structured soil skeleton. In the case of the initial water content of 0%, since dry particles without suction were used for the specimen preparation, the classification of coarse and fine particles may have occurred during the compaction.

The specimens with the 10% and 14% initial water contents, and a high soil structure, exhibit relatively rigid elastic behavior at the early stage of loading and show brittle behavior at the following stage. Plastic behavior can be observed from the early stage for the specimens with the 0% and 3% initial water contents without a high soil structure.

4. SIMULATION BY SYS CAM-CLAY MODEL

It can be assumed that the difference in the test results due to the initial water contents is caused by the soil structures that form in the specimens during the specimen preparation. In this study, a numerical simulation is performed to confirm this assumption using the superloading and subloading Cam-clay model named SYS Cam-clay model (Asaoka et al. 2002), which can describe the effects of the soil structure. The SYS Cam-clay model incorporates the concepts of the soil structure, overconsolidation and anisotropy into the modified Cam-clay model. In the SYS Cam-clay model, the soil structure is assumed to deteriorate with an increasing the plastic shear strain.

Figure 4 illustrates the simulation results with the values of the parameters for the soil structure used in each case. In this analysis, the initial values for the degree of soil structure $1/R_0^*$, initial overconsolidation ratio $1/R_0$ and soil structure degradation parameter a are defined as variables in order to explain the various complicated types of behavior of the structured soil. From the triaxial test results, it is assumed in this study that higher soil structures are generated in the specimens with higher initial water contents. Therefore, a higher the initial value for the degree of soil structure $1/R_0^*$ is set in the case of a higher initial water content. Furthermore, a smaller a value is adopted in that case, since the high soil structure may not be easily deteriorated due to shearing. Thus, parameter a expresses the rate of degradation of the soil structure and the larger value for a describes faster degradation of the soil structure. The other parameters used in this study, i.e., elasto-plastic parameters, evolution law parameters and initial conditions, are in common and are listed in Table 1. Since $1/R_0^*$ and $1/R_0$ are dependent on each other, when $1/R_0^*$ is set first, $1/R_0$ is automatically determined from the values of an initial specific volume v_0 and

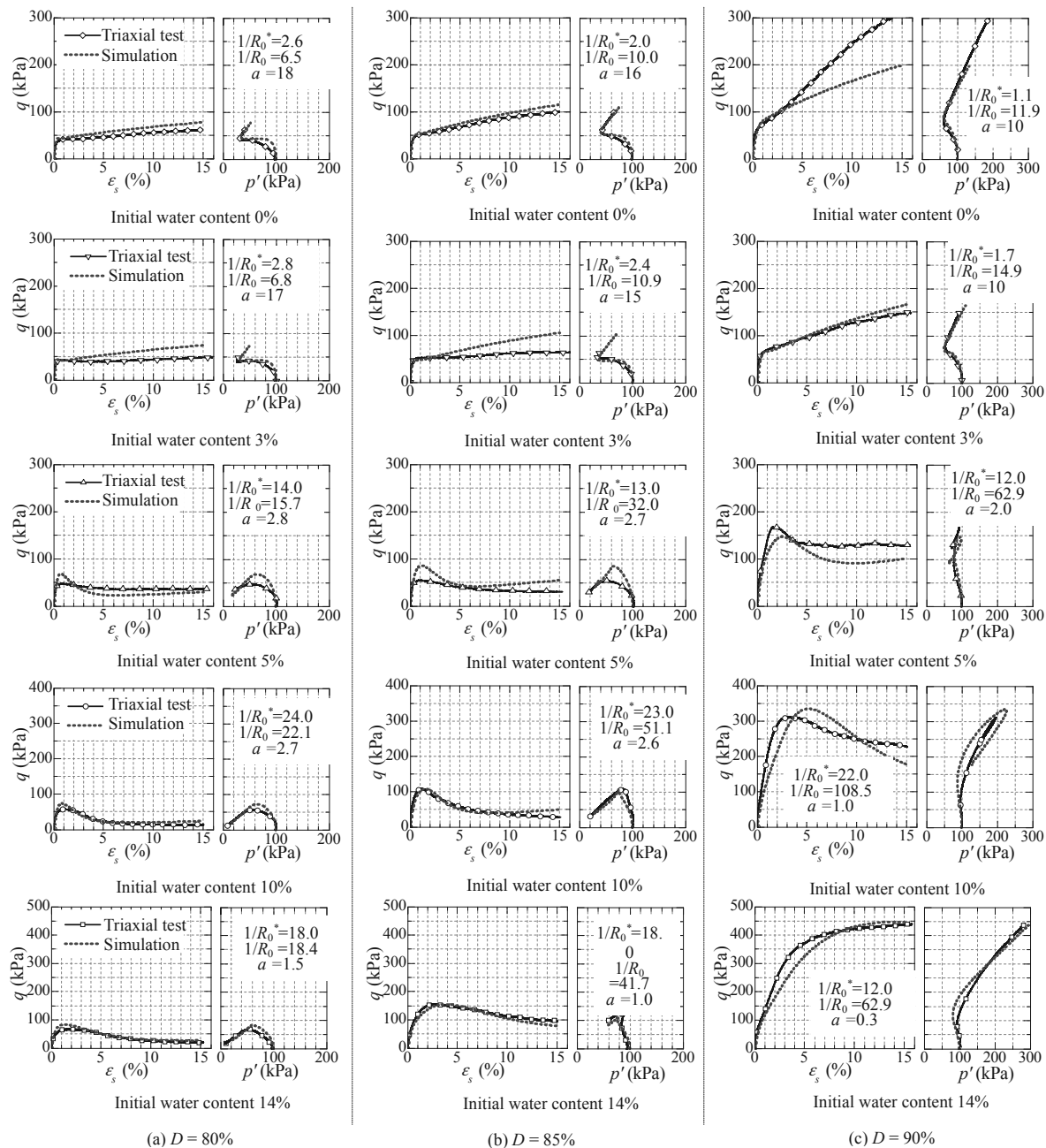


Figure 4. Numerically simulated stress- strain relations and effective stress paths with various initial water contents.

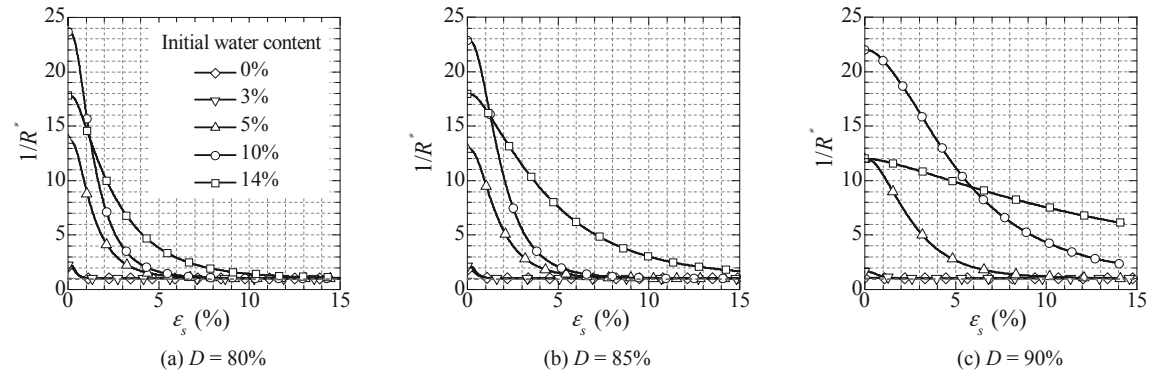


Figure 5. Degradation of degree of soil structure with increasing shear strain.

Table 1. Soil parameters used in the numerical simulation.

Elasto-plastic parameters	
Compression index $\tilde{\lambda}$	0.100
Swelling index $\tilde{\kappa}$	0.011
Critical state constant M	1.46
Intersect of NCL N (v at $q=0$, $p'=98.1$ kPa)	1.570
Poisson's ratio ν	0.300
Evolution parameters	
Degradation parameter of soil structure a ($b=c=1.0$)	see Fig.5
Degradation parameter of overconsolidated state m	0.030
Evolution parameter of β b_r	1.00
Limit of rotation m_b	0.001
Initial conditions	
Initial specific ratio $v_0=1+e_0$	1.637 ($D=80\%$) 1.536 ($D=85\%$) 1.457 ($D=90\%$)
Initial degree of structure $1/R_0^*$	see Fig.5
Initial overconsolidation ratio $1/R_0$	see Fig.5
Initial anisotropy $\zeta = \sqrt{2/3(\beta_0 \cdot \beta_0)}$	0.01
Initial mean effective stress p_0'	9.80 (kPa)

an intercept of normal consolidation line N . Although soil structure degradation parameters a , b and c are material coefficients, $b=c=1.0$ is adopted in this analysis for simplicity. The values for initial specific volumes v_0 are determined as the representative values from the specimens with a 10% initial water content. Compression index $\tilde{\lambda}$ is obtained from the oedometer tests, while critical state constant M and swelling index $\tilde{\kappa}$ are determined from the typical triaxial test results.

In each simulation, the initial effective stress p_0' is 9.8 kPa and the isotropic consolidation is 100 kPa; then the following undrained shearing is numerically performed.

In general, the simulation explains the triaxial test results well. Namely, basic parameters are used in common for all cases. Only the parameters for the soil structure are changed in order to reasonably explain the effects of the initial water content and the degree of compaction on the shear behavior. Therefore, the assumption that the initial water content of the specimen leads to generation of a high soil structure is considered to be valid.

Figure 5 illustrates the history of $1/R^*$, which describes how the soil structure deteriorates with an increasing the shear strain ε_s . In all the analytical cases, the degrees of soil structure $1/R^*$ gradually decrease and come closer to 1.0. Here, $1/R^*=1.0$

means the disappearance of the soil structure. Since $1/R_0^*$ and a are assumed in this analysis, the $1/R^*$ and ε_s relations are kinds of installed functions, as shown in Figure 5. Using these relations for the soil structure, the triaxial test results can be simulated well.

5. SUMMARY

When performing triaxial tests using specimens consisting of reconstructed gravel-mixed sand, the initial water content of the sample during the specimen preparation greatly influences the results of the tests. Various degrees of soil structures were generated with various values of sample suction in the specimen preparation. Although the specimens were fully saturated before the tests, the soil structure generated by the compaction under unsaturated conditions was probably maintained during the triaxial tests.

The numerical simulation conducted in this study with the SYS Cam-clay model, which can describe the effects of the soil structure on the mechanical characteristics of soils, has verified the reason for the difference in the shear behavior of the gravel-mixed sand due to the difference in the initial water contents. By increasing the initial water content until the optimum water content is reached, a higher soil structure seems to be generated in the specimen. On the other hand, the specimen reconstituted by the lower initial water content sample has a lower soil structure and easily loses it. By incorporating the above-mentioned concept, the complicated test results for the relation between the soil structure and the initial water content can be explained well.

In practice, when the triaxial tests are performed using reconstituted sand specimens, a dry deposition method is usually adopted for the specimen preparation for convenience. However, it is difficult to generate the soil structure in the specimen with dry sand. Therefore, the triaxial test results using reconstituted specimens, made by the dry deposition method, may not express the real behavior of earth structures.

6 ACKNOWLEDGEMENTS

The authors would like to express their sincere thanks to Dr. Noda, a professor at Nagoya University, for his useful advice on the numerical simulation performed in this study.

7 REFERENCES

- Ishihara K. 1993. Liquefaction and Flow Failure during Earthquakes, *Géotechnique*, 43 (3), 351-451.
- Asaoka A., Noda T., Yamada E., Kaneda K. and Nakano M. 2002. An elasto-plastic description of two distinct volume change mechanisms of soils, *Soils and Foundations*, 42 (5), 47-57

The expansive properties of Poland's clay subsoil

Propriétés de l'argile expansive de substrat de la Pologne

Kumor M. K.

Geotechnical Department, University of Technology and Life Sciences, Bydgoszcz, Poland

ABSTRACT: The paper presents the expansive properties of Neogen clays of the Poznań series that form the foundation stratum of Northern Poland. The phases of swelling and shrinkage of expansive clays were analysed as a function of humidity changes. Functional relationships were determined. Expansive clays of Northern Poland are classified as soils of a very high degree of expansiveness due to the contractibility range ($LL - SL$) = 82.1% > 50 %. In the natural condition, they are half-compact or, at most, rigid-flexible. The examination results of volumetric changes in clays in shrinkage and swelling phases make it possible to determine the range of dislocations of the foundation-expansive substratum contact zone at a free swelling or shrinkage.

RÉSUMÉ : L'article présente les propriétés d'expansion des argiles du Néogène de la série Poznań qui constituent la couche de fondation du nord de la Pologne. Les phases d'expansion et de contraction de ces argiles ont été analysées comme une fonction des changements d'humidité. Des relations fonctionnelles ont été définies. Ces argiles expansives du nord de la Pologne sont classées comme des sols possédant un grand degré d'expansivité en raison de l'étendue de leur contractilité ($LL - SL$) = 82,1 % > 50 %. À l'état naturel ils sont semi-compacts ou tout au plus rigide-flexibles. L'examen des résultats des variations volumétriques des argiles pendant les phases d'expansion et de contraction permet de déterminer l'éventail des dislocations de la zone de contact fondation/sous-couche expansive lorsqu'expansion ou contraction sont libres de toute contrainte.

KEYWORDS: expansive clays of Poland, expansive parameters, swelling, shrinkage.

1. INTRODUCTION

The notion of soil expansiveness in geotechnics is usually related to a definition of swelling, Chen (1988). As is well known, soil expansiveness encompasses the more general phenomena of swelling and shrinkage, Przysański (1991).

Shrinkage – is a process of reducing soil volume as a consequence of pore water loss; it is a characteristic property of cohesive soils with significant content of clay-like fraction.

The **swelling** of cohesive soils is a process which is opposite to shrinkage.

From a practical point of view, expansive soils are soils that show an increase in the initial volume in contact with water and shrink as a result of drying. A characteristic feature here is differentiated phases of shrinkage and swelling that always accompany changes in humidity.

In the literature relating to classification of expansive soils, e.g. Chen (1988), Seed et al. (1962), Sorooshan (1974), van der Merwe (1964) and others, attention is chiefly paid to the swelling process. In the classifications, the following indicative features are used in the first place: liquid limit – LL , contractility limit – SL , plasticity index – $I_p = (LL - PL)$, soil humidity index – w_o , specific surface – S .

In practice, there are few clay expansiveness classifications that introduce shrinkage parameters as classification criteria, e.g. Holtz (1959), Ranganathan and Satyanarayana (1965), Niedzielski (1993). In Polish soils the expansiveness classification developed by Niedzielski, the so called contractility range ($LL - SL$) (%) was introduced to characterise shrinkage. Four expansiveness stages were distinguished on the basis of the range:

- very high ($LL - SL$) > 50 %,
- high $35 < (LL - SL) < 50$,
- medium $20 < (LL - SL) < 35$,
- low ($LL - SL$) < 20.

From an engineering point of view, the occurrence of the substratum shrinkage phase after the swelling phase is the most

dangerous for constructions. Shrinkage brings about a post-consolidation settlement of expansive clays which in Poland's geotechnical conditions are the principal cause of nearly all construction failures, see Figure 1.

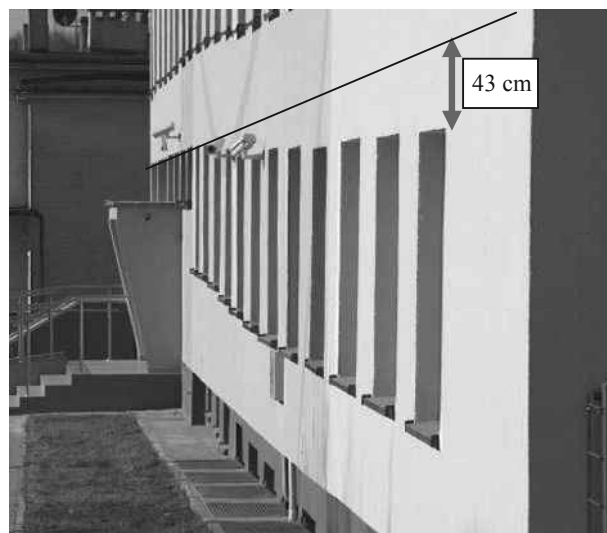


Figure 1. An example of a construction failure resulting from the swelling-shrinkage cycles.

2. EXPANSIVE CLAY CHARACTERISTICS

The building substratum surface zone in Central European conditions is exposed to specific factors, e.g. those related to climate, atmosphere, anthropopressure, etc. Highly dispersed clays with expansive properties are particularly sensitive to the impact of these factors. Expansive clays, typical to nearly half of Poland's territory, are represented by tertiary deposits of the Poznań series.

They form the substratum in the area of the Polish Lowland, see Figure 2.



Figure 2. Occurrence of expansive soils in Poland.

Clays are characterised by a high variability of graining and mineralogical composition. The mineral with strong expansive properties, i.e. *beidelite*, *montmorillonite* prevails in the mineralogical composition. The contents of clayey minerals are as follows:

- smectite: 11 % to 23 %, with the exchangeable ion Ca, Na,
- illite: 5 % to 9 %,
- kaolinite: 6% to 11 %,
- other minerals, chlorite, silica.

The thickness of the Poznań series usually does not exceed 20 m in Poland.

In Poland, the areas of occurrence of tertiary clays of the Poznań series are characterised by a slight overlayer of quaternary formations over the Cenozoic, Kumor (1993). From above, usually a thin (0 to 1.4 m) layer of new deposits occurs that covers the basic complex of Neogene formations. The Neogene has been exposed mainly along erosive river valleys of the Vistula, the Odra, the Brda, the Drwęca and the Warty. It is developed in the form of Mio-Pliocene mottled clay (Poznań series clays), locally washed away and broken.

1.1 Expansive clay properties

The clays of the Poznań series are characterised by expansive properties that are atypical in comparison with other genetic clays of Poland's region, Kumor (2006), Niedzielski (1993). Below, expansive indices values as typical for the Poznań series in Poland are presented (see Table 1).

Table 1. Same parameters of Polish expansive clays.

Parameter	Max.	Medium
Swelling time t_p (h)	>340	24 - 36
Swelling pressure p_c (kPa)	1200	200-400
Shrinkage v_o [%]	44.1	32-34
Swelling humidity w_c [%]	137.0	80-99
Liquid limit LL [%]	148.5	82.1
Swelling index v_p [%]	62.0	~21.7

The expansive indices of clays are not constant, they depend on temperature and increase as temperature rises, e.g. a swelling

index 18 % higher was obtained as the temperature rose from +20 to +55° C.

Expansive clays of Northern Poland can be classified as very highly expansive due to the contractibility range $(LL - SL) = 82.1\% > 50\%$. Characteristic features of an expansive substratum in the natural condition are high cohesion (strength) and low compressibility.

In the general opinion, these are soils with favourable geotechnical properties, basically half-compact or, at most, rigid-flexible. In real geological conditions, there are always differentiated, genetic zones of irrigated or dried soils. These problems naturally impede the execution of foundation works. The photograph shows a spatial open excavation where clays simultaneously dried in local zones and irrigated in others have been found, see Figure 3.

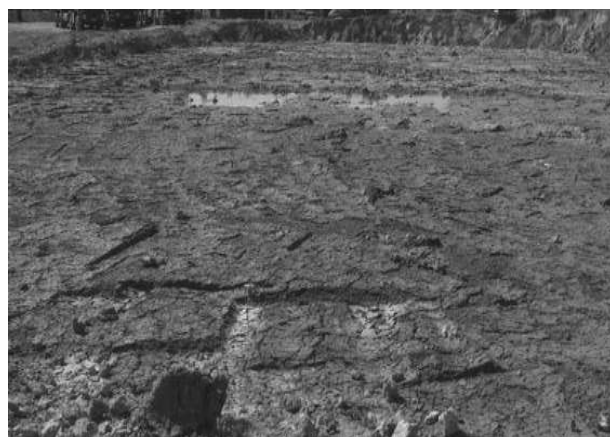


Figure 3. The view of the typical system of clays in the gout of the large building excavation with extremely diverse proprieties, simultaneous drying - the nearer zone, and irrigating - the background.

An example of destruction of expansive clay in the excavation bottom is the situation when under the influence of natural drying, a macro-disintegration of the clay massif is initially noticed, then a volumetric shrinkage and a further granular disintegration of the massif leading to the occurrence of breccia, see Figure 3, 4 and 5.

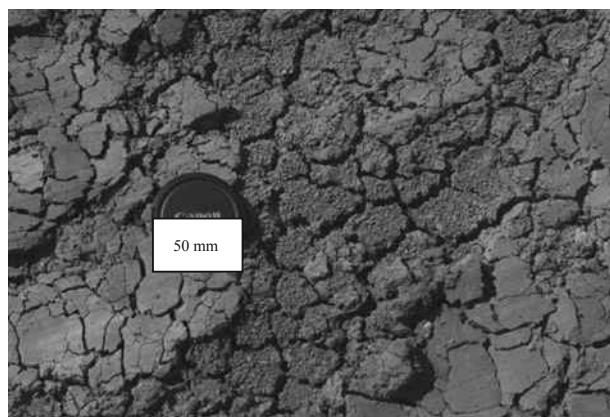


Figure 4. The granular disintegration of dry clay in natural conditions, after 24 hours.

It follows from the presented photograph that the process of clay drying (shrinkage) is characterised by a volumetric shrinkage and the occurrence of a breccia structure with numerous separated grains and deep shrinkage fissures. One of geotechnically undesirable effects is a locally differentiated rigidness and deformability of such substratum. By monitoring the foundation settlement (upon uniform loading), differences ranging from $\Delta s = 0.0$ mm (a dried zone) to $\Delta s = 30.0$ mm in humidified clay zones, see Figure 3.



Figure 5. The natural process of clay drying after exposure of the excavation's bottom, (after 2 hours) the superficial macrodisintegration fully developed.

Detailed examination of the shrinkage and swelling phases of expansive clays, as presented in this paper, were conducted from the constructional substratum of the city of Bydgoszcz situated in the northern region of Poland, (Niedzielski, Kumor 2009).

Expansive clays are characterised by extremely high numerical values of expansive parameters against a background of other tertiary clays in Poland. They should be classified in respect to the contractibility range ($LL - SL$) = 82.1% > 50 %, as very highly expansive soils.

1.2 Shrinkage phase examination

The soil contractibility examination is a determination infrequently performed for practical purposes. The volumetric shrinkage (V_s) of expansive clay was examined according to the following see Eq.:

$$V_s = (V' - V'') / V' \quad (\%) \quad (1)$$

where:

V' – soil sample initial volume [cm^3],

V'' – final volume of the sample after drying [cm^3].

Typical results of the shrinkage progress are presented in Figure 6 and see Table 2.

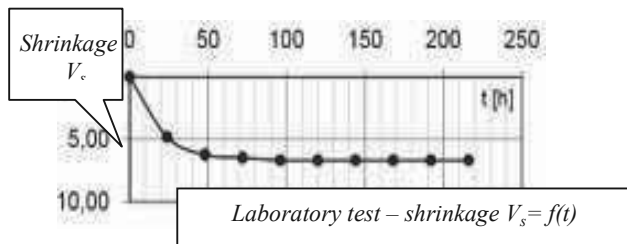


Figure 6. A typical progress of expansive clay shrinkage over time.

Table 2. Same shrinkage parameters of Polish expansive clays.

Parameter	Max.
Shrinkage time t_s (h)	50 -96
Volumetric shrinkage V_s [%]	23-24
Shrinkage limit SL [%]	18.2-18.9

The shrinkage examination results reveal losses in the massif volume and a high sensitivity of expansive clays to changes in humidity in a short time after the start of drying, see: Figure 5 and Figure 6.

The volume loss ΔV_s for the shrinkage phase of the expansive clay in the Northern Poland can be determined by means of see Eq. 2:

$$\Delta V_s = 0,783 * (w_o - w_k) \quad \text{at } w_k > SL \quad (2)$$

where:

w_o – initial humidity, [%]

w_k – final humidity [%].

From a practical point of view, it is important to learn the volumetric shrinkage values. The form of the function between relative volumetric shrinkage (V_s) and humidity is linear for expansive clay, with statistical significance $R^2 = 0.9545$:

$$V_s = -3,5731 + 0,783 * w \quad (3)$$

where:

V_s – relative volumetric shrinkage [%],

w – humidity, > w_s [%].

In the process of drying and with humidity lowered by the value of $\Delta w = (w_o - w_k)$, the volume of the relative volumetric shrinkage is important for forecasting post-consolidation settlements of newly erected buildings, as well as for preventing failures in buildings used for many years. It follows from observations Kumor (2006), that differences in actual shrinkage settlements under a damaged building amount to tens of millimetres, Figure 1.

The parameters describing the shrinkage process, as well as swelling, are individual *material features* of each expansive soils.

1.3 Swelling phase

The swelling parameters of expansive clays were examined using methods applied in soil mechanics laboratories.

Table 3. Swelling parameters of expansive clays.

Parameter	Max.	Medium
Swelling pressure p_c (kPa)	1200	200-400
Swelling humidity w_c [%]	137.0	80-99
Contractility range ($w_L - w_s$) [%]	130.5	99.1
Swelling index v_p [%]	62.0	~21.7

The following values were measured: swelling humidity - w_c , swelling index - V_p , according to Vasiliev, swelling pressure - p_c , in a consolidometer, swelling time - t_p . Exemplary testing results are presented in Table 3, Figure 7, 8.

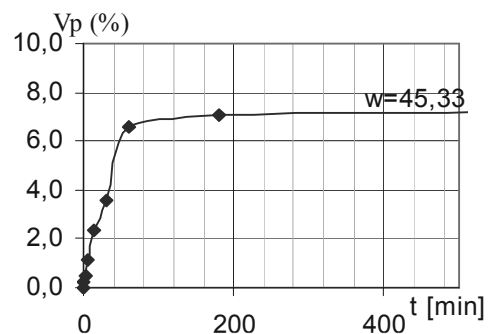


Figure 7. Dependence of the swelling index - V_p , on the expansive clay swelling time - t .

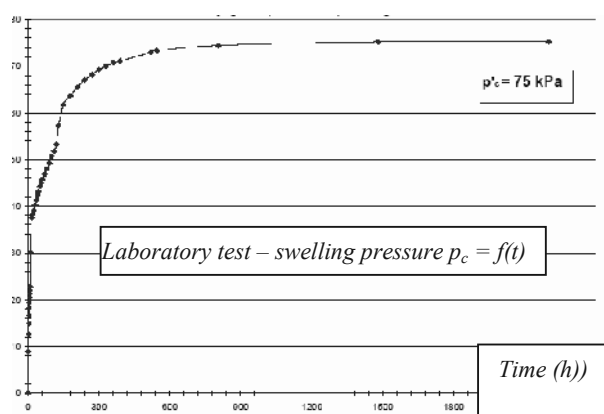


Figure 8. Progress of the expansive clay swelling pressure in time.

Characteristics obtained of the progress of potential volumetric change in the examined clays in relation to the humidity condition are presented see Figure 9. The dependence between the swelling index and humidity can be written down in a general form:

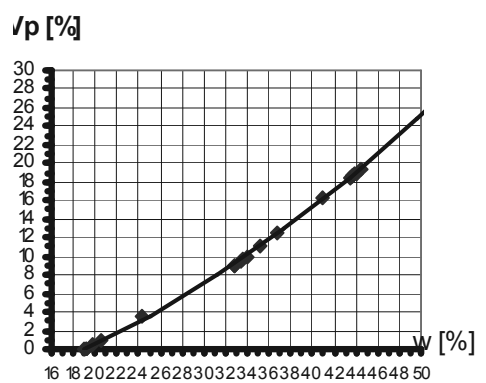
$$V_p = f(w) \quad (4)$$

where: V_p – swelling index,
 w – humidity.

$$V_p = (h_k - h_o) : h_o \quad (5)$$

The function form of the potential expansiveness change characteristics for the examined clay with statistical significance $R_{xy} = 0.912$ is as follows:

$$V_p = 3E-05w^3 + 0,011w^2 + 0,102w - 5,867 \quad (6)$$


 Fig. 9. Dependence of the swelling index – V_p , on the swelling expansive clay humidity - w .

The characteristics of dependence of the swelling index in relation to humidity, as presented in Fig. 9, allows one to determine the progress and expansiveness phase characteristics as well as changes in clay deformation values during swelling. Knowing the final swelling humidity of a particular clay w_k , and anticipating the direction of the humidity change on the basis of initial humidity w_o , we know that one expansiveness phase will occur, i.e. $(+\Delta w)$ – humidity increase – swelling phase, when $(-\Delta w)$ – drying – shrinkage phase.

Having the characteristics of potential volumetric changes in relation to humidity as determined experimentally for a particular type of clay, one can relatively easily forecast in practice the range of substratum displacements.

A potential increase in the swelling clay volume can be calculated from the obtained relationship (6) in the following form:

$$\Delta V_p = V_p(w_o) - V_p(w_k) \quad (7)$$

For the case of humidity increase by value $\Delta w = (w_o - w_k)$, we will determine a positive swelling index – $(+\Delta V_p)$ in relation to the positive condition on the basis of the swelling phase characteristics.

In the clay shrinkage phase, during the swelled massif drying, we obtain from the characteristics and calculations made according to formula (7), a negative value $(-\Delta V_p)$ – shrinkage, in relation to the initial state after the completed swelling, with humidity w_o .

2 ENDING AND CONCLUSIONS

The results obtained enable forecasting building behaviours in relation to determined natural fluctuations in the expansive substratum humidity. On the basis of already conducted expansiveness studies (25 cases) one can predict that for other clays, material differences are related only to the range of limit values, i.e. the limit of swelling contractibility and humidity (R – space), swelling index and swelling pressure, Kumor (2006). When analysing the results obtained for numerical values of expansiveness parameters for the Poznań series Neogen clays from the northern Poland, one can describe them as very cohesive, with a very high and extremely high plasticity as well as a very high and extremely high swelling. They point to the necessity for very careful forecasting of dislocations of buildings when they are founded in expansive clays.

The presented results point to the need for determining characteristics of potential expansiveness for various types of clays, depending on humidity, swelling pressure as well as the time and chemistry of the environment.

3 REFERENCES

- Chen F.H. (1988): Foundations on Expansive Soils. Dev. In Geot. Eng. Elsevier, Amsterdam.
- Danilov A.A. (1964): Grafik dla rozdelenia gruntov na obuhnye, prosadochnye i nabukhajushhie. Osn. Fund. i Mekh. Gruntov, 5: pp. 26-26.
- Kumor M.K., (2006): Investigation of shrinkage-swelling of clays as the potential tool to predict deformation of expansive subsoil. XIV Krajowa Konferencja Mechaniki Gruntów i Inżynierii Geotechnicznej, (in Polish) Białystok, Vol. 1, pp. 234-242.
- Niedzielski A., (1993): Factors Determining swelling Pressure and Free Swelling of Posnanian and Varved Clays. Rozprawy Akademii Rolnicza zeszyt nr 238, Poznań.
- Niedzielski A., Kumor M.K., (2009): Geotechnical Problems of a Foundation on Expansive Soils in Poland. Inżynieria Morska i Geotechnika 3/2009, (in Polish), pp. 180-190.
- Przystański J., and all, (1991): Foundation on Expansive Soils, (In Polish) Zeszyt Politechniki Poznańskiej Rozprawy 224, Poznań.
- Sorochan E.A. (1974): Stroitelstvo Sooruzenij na Nabukhajushhikh Gruntach. Stroizdat, Moskva.

Effect of Particle characteristics on K_0 Behavior for Granular Materials

Effet des caractéristiques particulières sur le comportement des matériaux granulaires K_0

Lee J., Park D., Kyung D., Lee D.

School of Civil and Env. Eng., Yonsei University, Seoul, Korea

ABSTRACT: The Jaky's K_0 equation is commonly used for the estimation of K_0 in practice, which is given as a function of the internal friction angle of soils. As the friction angle is a state-dependent variable, uncertain aspect still exists regarding the value of to be adopted for the Jaky's K_0 equation. In this study, the application of the Jaky's K_0 equation for granular materials is investigated with focus on the effect of particle characteristics, which are related to strength, on K_0 behavior. Particle shape and angularity of particles are considered for the investigation of K_0 behavior. Experimental testing program is established to measure K_0 under various mechanical and physical conditions of test materials. Sand particles and spherically shaped glass beads with and without etched particle surfaces are employed into the experimental testing program.

RÉSUMÉ: L'équation K_0 de Jaky est couramment utilisée pour l'estimation de K_0 en pratique, ce qui est donnée en fonction de l'angle de frottement interne du sol. Comme l'angle de frottement est une variable dépendant de l'état, l'aspect incertain existe encore au sujet de la valeur qui sera adoptée pour l'équation K_0 de Jaky. Dans cette étude, l'application de l'équation K_0 de Jaky pour les matériaux granulaires est étudiée, en particulier par rapport à l'effet des caractéristiques des particules, qui sont liés à la résistance sur le comportement de K_0 . La forme et l'angularité des particules sont considérées comme des enquêtes sur le comportement de K_0 . Le programme d'essais expérimentaux est prévu pour mesurer le K_0 dans différentes conditions mécaniques et physiques des matériaux d'essai. Les particules de sable et de billes de verre en forme sphérique avec et sans surfaces des particules gravées sont utilisées dans le programme d'essai expérimental.

KEYWORDS: coefficient of lateral earth pressure, granular materials, internal friction angle, thin wall oedometer test

1 INTRODUCTION

In-situ stresses are important state soil variables that are necessary for geotechnical analysis and design. The Jaky's K_0 equation (Jaky 1944) is commonly used in practice to evaluate K_0 based on the internal friction angle of soils. In fact, it was analytically derived assuming equilibrium condition for a sand pile with the statically admissible stress state. From the analysis using various assumptions on the stress distribution, it was confirmed that the Jaky's K_0 equation is valid and sufficiently accurate for general geotechnical purposes (Michalowski 2005). According to the Jaky's K_0 equation, K_0 is given as a sole function of the friction angle of soils, while K_0 itself represents the state soil variable that defines the geostatic stress state before failure.

The friction angle of soils can be differently defined. Common definition adopted in geotechnical engineering includes the peak, critical state, and dilatancy friction angles. The peak friction angle ϕ'_p corresponds to the maximum peak strength, and is composed of the critical state friction angle ϕ'_c and the dilatancy friction angle ψ_p . The dilatancy friction angle ψ_p is state-dependent varying as a function of the confining stress and relative density. The critical state friction angle ϕ'_c is an intrinsic soil variable that is dependent only on the inherent soil characteristics such as mineralogy, particle shape, and angularity. From the Jaky's K_0 equation, the highest and lowest K_0 values would be obtained from the critical state friction angle ϕ'_c and the peak friction angle ϕ'_p , respectively.

In this study, the application of the Jaky's K_0 equation for granular materials is investigated focusing on the effect of various particle characteristics such as particle shape, surface roughness, and relative density on the ϕ' - K_0 correlation. The

variation of K_0 calculated using different types of friction angles is analyzed. For this purpose, an experimental testing program is established to measure K_0 under various soil and stress conditions. The tested granular assemblies include natural sand particles and spherical glass beads with and without etched particle surfaces.

2 CORRELATION OF K_0

Jaky (1944, 1948) presented the well-known K_0 equation based on the stress analysis of a geometrically symmetric sand wedge, assuming a limit stress state. K_0 is defined as a function of the internal friction angle ϕ' of soils as given by:

$$K_0 = (1 - \sin \phi') \frac{1 + \frac{2}{3} \sin \phi'}{1 + \sin \phi'} \quad (1)$$

Where ϕ' = internal frictional angle of soils. Eq. (1) can be further simplified as a form that has been a norm in current practice, given as follows:

$$K_0 = 1 - \sin \phi' \quad (2)$$

It is indicated that the denser the sand, the higher the ϕ' , resulting in lower K_0 values. The effect of stress history on K_0 is significant. K_0 for overconsolidated (OC) condition is greater than for normally consolidated (NC) condition (Wroth 1973). In order to reflect the effect of stress history on K_0 , modifications into Eq. (2) have been proposed in a form of:

$$K_0 = K_{0,NC} \cdot OCR^\alpha \quad (3)$$

where $K_{0,NC} = K_0$ for normally consolidated conditions, OCR = overconsolidation ratio, and α = exponential parameter. Mayne and Kulhawy (1982) and Mesri and Hayat (1993), for example, proposed $\sin\phi'$ for α .

Although the Jaky's K_0 equation have been widely used and modified to better reflect the actual stress state, uncertainties still exist, which has not been fully clarified yet. Examples are the effect of particle characteristics, which differ inherently for different materials, and the correlation to the friction angle ϕ' that is not constant but varies state-dependently.

3 TESTING PROGRAM

3.1 Materials

A series of laboratory tests were conducted to investigate and analyze the K_0 evolution of granular materials. Three different granular materials were adopted, which include Jumunjin sand, spherical glass beads (GB), and etched glass beads (EGB). The use of different granular materials was aimed to investigate the effect of inherent particle characteristics, such as particle shape, surface roughness, and angularity, on K_0 in more straightforward and systematic manner. Jumunjin sand particles were sieved, and the particles ranging between 0.425 mm and 0.85 mm (e.g., sieves #40 and #20) were collected and used in the test, to presumably minimize the effect of fines content and to make test samples equivalent in size hence directly comparable to glass beads. The grain size distribution of Jumunjin sand is shown in Fig. 1, in comparison with glass beads. Sand particles are uniformly distributed with the mean particle size D_{50} of around 0.56 mm.

The soda lime glass beads ($\text{CaO-MgO-Na}_2\text{O-SiO}_2$) with spherical particle shape and smooth surface ($D_{50} \sim 0.5$ mm) were selected as comparative materials. The surface roughness was controlled by etching clean, smooth glass bead surfaces with hydrogen chloride-based solution (HCl , $\text{pH}=1\pm0.5$). Glass beads were submerged in the hydrogen solution for 15 seconds and were thoroughly rinsed with deionized water followed by oven-drying at 80°C . This process produced uniformly etched glass beads with rough surfaces, while the same spherical shape as clean glass bead particles was maintained. Three samples were then subjected to the assessment of K_0 measurements.

The values of ϕ'_c were 37.1° , 26.7° , and 30.0° for Jumunjin sand, glass beads, and etched glass beads, respectively. The values of ϕ'_p varied, depending on the relative densities and particle characteristics considered in the tests.

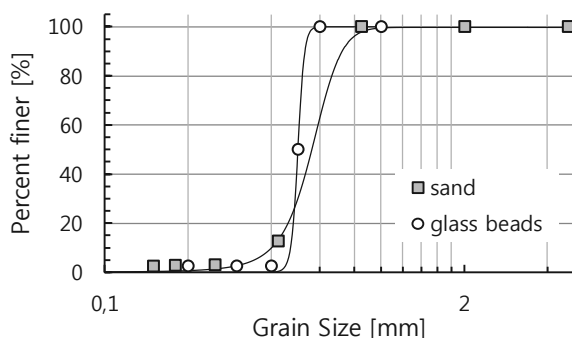


Fig. 1 Grain size distribution curves of test materials.

3.2 Determination of K_0

The various test devices have been proposed to measure K_0 that

satisfy the radial strain limit, smaller than 5×10^{-5} (Okochi and Tatsuoka, 1984). The radial expansion of either membrane or thin-wall tube can be servo-controlled to maintain the zero lateral strain condition whereas the complex stress path and multiple measurement systems are involved. Alternatively, the strain gauges are attached to the thin-wall tube and the horizontal stress is directly related to the cell deformation within acceptable strain regimes (Zhu et al. 1995; Kolymbas and Bauer 1993).

The oedometer method to measure K_0 adopted in this study consists of a thin aluminum cylinder of 0.13 mm in thickness and 66 mm in diameter. A pair of strain gauges (120ohm, CEA-13-240UZ-120, Vishay) was attached at the middle height of the cylinder outside and another pair of dummy gauges was included for temperature compensation forming the full-bridge circuit. The sample height is about 40 mm with a height to diameter ratio equal to 0.61. Fig. 2 illustrates the experimental configuration and peripheral electronics. The voltage response of strain gauges was calibrated and correlated to the horizontal stress using a water-filled balloon inside the cylinder, assuming that the applied vertical stress is equal to the horizontal stress. The linear calibration factor was then obtained and given as follows:

$$\sigma = 0.0476 \cdot V - 4.8338 \quad (4)$$

where σ is the horizontal stress in the unit of kPa and V is the voltage out in the unit of mV.

Dry samples were placed in the thin-wall oedometer to achieve a target relative density and were subjected to loading-unloading-reloading cycles. The maximum loading reached 111 kPa and reloading ran up to 143 kPa beyond the preconsolidation stress. The voltage response was recorded every 1 sec and each loading step lasted 5 to 10 minutes. The horizontal stress for a given loading step was computed by averaging voltage response and by applying the calibration factor. The test specimens were prepared at different relative densities of $D_R = 33\%$ and 84% for Jumunjin sand and $D_R = 56\%$ and 80% and $D_R = 55\%$ and 81% for glass and etched glass beads, respectively.

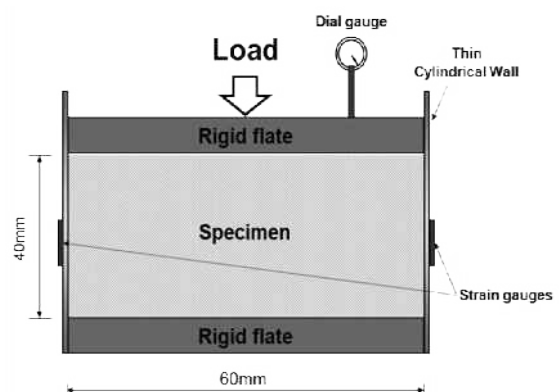


Fig. 2 Configuration of thin-wall K_0 test.

4 TEST RESULTS AND ANALYSIS

4.1 K_0 for Different Test materials

The changes of K_0 with σ'_v for the test materials measured from the thin-wall oedometer are shown in Fig. 3 during the whole loading cycles. As shown for the loose (LS) and dense (DS) sands with $D_R = 33$ and 84% , respectively, the K_0 values of the dense sand run below the loose one at all the loading stages. The dense sand provides the strong force chain along the

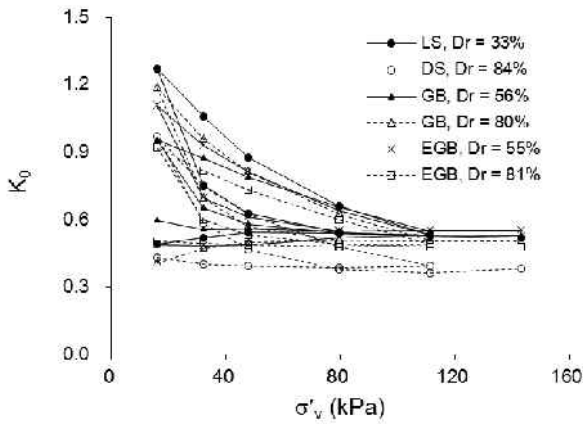


Fig. 3 Values of K_0 with σ'_v for test granular materials of Jumunjin sand (LS and DS) and glass beads with and without etching (GB, EGB).

vertical stress direction due to higher interlocking, which leads to lower K_0 during loading. The horizontally interlocked stress induces the increase of K_0 during unloading whereas the partial release of the horizontal stress during reloading makes the evolution of K_0 run between loading and unloading stages. Once the stress reaches the preconsolidation stress, K_0 remains constant as the normally consolidation condition prevails.

The effect of particle shape and angularity on K_0 was investigated by directly comparing the test results from Jumunjin sand and glass beads. In Fig. 3, the values of K_0 for the dense sand (DS) are compared with those for glass beads (GB) according to the vertical stress. The lower values of K_0 for the dense sand are manifest presumably due to the angularity effect. The particle surface roughness effect on K_0 values can be analyzed by comparing the test results from glass beads (GB) and etched glass beads (EGB). No marked difference of K_0 between GB and EGB is observed during loading, while GB exhibits higher K_0 values during unloading and reloading. The ratio between the artificially created surface dents (e.g., ~ 5 to $10 \mu\text{m}$) and particle diameter ranges from 0.01 to 0.02 for EGB, which impose insignificant impact on K_0 during loading, while the particle geometry is predominant.

4.2 Correlation to Strength

The shear strength of granular materials can be described using different definitions of friction angle. The critical state friction angle ϕ'_c , the peak friction angle ϕ'_p , and dilatancy angle ψ are the typical examples. The inter-particle surface friction angle ϕ'_s can be regarded as another type of friction angle that contributes to the overall shear strength of granular materials.

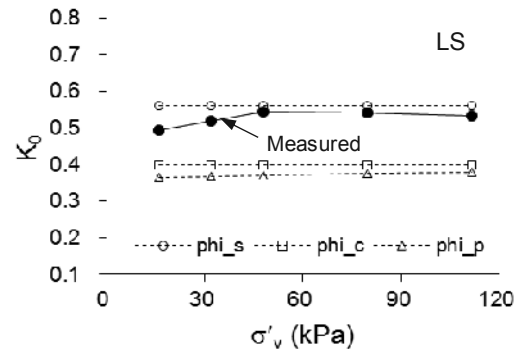
The confining stress within the specimen for the thin-wall oedometer tests continuously increases upon loading. The changes in confining stress result in changes in dilatancy and thus in the peak friction angle. Following Bolton (1986), the effect of confining stress and relative density on the peak friction angle can be evaluated using the following relationship:

$$\phi'_p = \phi'_c + R_D \cdot I_R \quad (5)$$

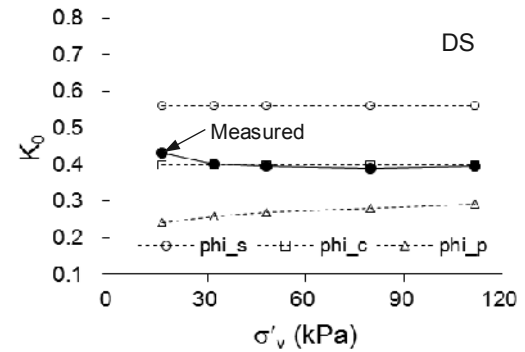
where R_D = dilatancy ratio = 3 and 5 for triaxial and plane-strain conditions, respectively. The dilatancy index I_R is defined as:

$$I_R = I_D \left[Q - \ln \left(\frac{100 \cdot \sigma'_{pm}}{p_A} \right) \right] - R \quad (6)$$

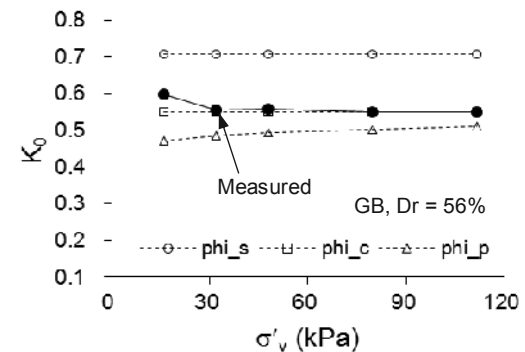
where I_D = relative density as a number between 0 and 1; p_A = reference stress = 100 kPa; σ'_{pm} = mean effective stress at peak in the same unit as p_A ; and Q and R = intrinsic soil variables.



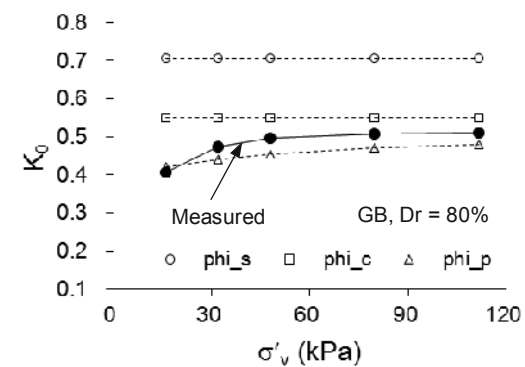
(a)



(b)



(c)



(d)

Fig. 4 Measured and calculated K_0 values with σ'_v for (a) loose sand; (b) dense sand; (c) medium etched glass beads; and (d) dense etched glass beads.

Using Eqs. (5) and (6), the variation of ϕ'_p during loading in the thin-wall oedometer tests for the test materials were obtained. The peak friction angle ϕ'_p decreased from 49.5° to 45.8° and 39.5° to 38.5° in the range of σ'_v from 16.0 to 111.5 kPa for the dense (DS) and loose (LS) sands, respectively. Using ϕ'_p , ϕ'_c and ϕ'_s for Jumunjin sand, K_0 was calculated and compared with the measured K_0 values in Fig. 4. The ϕ'_s was assumed equal to 26° and 17° for Jumunjin sand and glass beads, based on the values presented by Procter and Barton (1974) and Andrawes and El-Sohby (1973). Note that K_0 from ϕ'_c and ϕ'_s is constant as these are intrinsic soil variables. ϕ'_p produces the lowest range of K_0 values, while the upper bound is given by ϕ'_s . It is also noticed that the K_0 values measured during loading follows quasi constant, while decreases in ϕ'_p is certainly expected as indicated in Eqs. (5) and (6). From Fig. 4, it is seen that the measured K_0 values for the loose and dense sands are close to those calculated using ϕ'_s and ϕ'_c , respectively. This implies that the application of ϕ'_p is likely to produce underestimated K_0 values. For glass beads, the application of critical state friction angle ϕ'_c produces close match to the measured K_0 values. Similar results were observed for etched glass beads.

5 CONCLUSION

In this paper, the values of K_0 were investigated for different granular materials focusing on the effect of various particle characteristics. For this purpose, laboratory tests using the thin-wall oedometer were conducted to measure the values of K_0 under various test conditions. Sand particles, glass beads with and without etched particle surfaces were used in the testing program.

From the test results, it was observed that the effect of material density on K_0 was greater in OC stress state than in NC stress state, and in particular becomes more pronounced when unloaded. Regarding the effect of particle shape and angularity, the lower values of K_0 were observed from Jumunjin sand particles than from glass beads, due to the higher angularity and interlocking effects.

For sands, the values of the friction angle employed into the Jaky's K_0 equation to match the measured K_0 values were different for loose and dense sands. From measured and calculated K_0 values, it was found that the measured K_0 values for the loose sand were close to the calculated values using the inter-particle friction angle ϕ'_s . For dense sand, on the other hand, ϕ'_c produced close match to the measured K_0 values. This indicates that the application of the peak friction angle ϕ'_p is likely to result in underestimation of K_0 . For glass beads, calculated K_0 values using ϕ'_c were in good agreement with measured results for both relative densities.

6 ACKNOWLEDGEMENTS

This work was supported by Basic Science Research Program through the National Research Foundation of Korea (NRF) grant funded by the Korea government (MEST) (No. 2011-0030845).

7 REFERENCES

- Andrawes, K. Z. and El-Sohby, M. A. (1973). "Factors affecting coefficient of earth pressure K_0 ." *Journal of the Soil Mechanics and Foundations Division*, 99(SM7), 527-539.
- Bolton, M.D. (1986). "The strength and dilatancy of sands." *Geotechnique*, 36(1), 65-78.
- Jaky, J. (1944). "The coefficient of earth pressure at rest. In Hungarian (A nyugalmi nyomás tényezője)." *Journal of the Society of Hungarian Architects and Engineering*, 355-358.
- Jaky, J. (1948). "Pressure in silos." *Proceedings of 2nd International Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, 103-107.
- Kolymbas, D. and Bauer, E. (1993). "Soft oedometer: A new testing device and its application for the calibration of hypoplastic constitutive laws." *Geotechnical Testing Journal*, 16(2), 263-270.
- Mayne, P.W. and Kulhawy, F.H., (1982). "K0-OCR relationship in soil." *Journal of the Geotechnical Engineering Division*, 108(GT6), 851-872.
- Mesri, G. and Hayat, T. M. (1993). "Coefficient of earth pressure at rest." *Canadian Geotechnical Journal*, 30(4), 647-666.
- Michalowski, R. L. (2005). "Coefficient of earth pressure at rest." *Journal of Geotechnical and Geoenvironmental Engineering*, 131(11), 1429-1433.
- Okochi, Y. and Tatsuoka, F. (1984). "Some factors affecting K_0 -values of sand measured in triaxial cell." *Soils and Foundations*, 24(3), 52-68.
- Procter, D. C. and Barton, R. R. (1974). "Measurements of the angle of interparticle friction." *Geotechnique*, 24(4), 581 - 604.
- Sadrekarami, A. and Olson, S. M. (2011). "Critical state friction angle of sands." *Geotechnique*, 61(9), 771-783.
- Wroth, C. P. (1973). "General theories of earth pressure and deformation." *Proceedings of 5th European Conference on Soil Mechanics and Foundation Engineering*, Vol. 2, Madrid, Spain, 33-52.
- Zhu, F., Clark, J. I., and Paulin, M. J. (1995). "Factors affecting at-rest lateral stress in artificially cemented sands." *Canadian Geotechnical Journal*, 32(2), 195-203.

Duncan-Chang - Parameters for Hyperbolic Stress Strain Behaviour of Soft Bangkok Clay

Duncan-Chang - Paramètres de comportement contrainte-déformation hyperbolique d'argile molle de Bangkok

Likitlersuang S.

Chulalongkorn University, Bangkok, Thailand

Surarak C., Balasubramania A., Oh E.

Griffith University Gold Coast Campus, Queensland, Australia

Syeung Ryull K.

Asian Institute of Technology, Bangkok, Thailand

Wanatowski D.

University of Nottingham, United Kingdom

ABSTRACT: This paper is on the analyses of the stress strain data of soft and stiff Bangkok Clays carried out at the Asian Institute of Technology. A comprehensive series of triaxial tests both in compression and extension was carried out and the results were compared with a number of stress-strain theories as based on critical state concepts. The finite element software PLAXIS contains the Hardening Soil Model as an extension of the Duncan-Chang hyperbolic stress strain model. In this paper, the parameters for the hyperbolic stress strain model are evaluated from the data gathered previously in testing soft and stiff Bangkok clays. The testing program includes two series of undrained and drained tests performed on isotropically consolidated triaxial samples both under compression and extension conditions. These testing results can be used to determine the undrained and drained shear strength parameters for soft and stiff Bangkok Clays. Finally, two set of undrained and drained shear strength and stiffness parameters for the Hardening Soil Model of soft and stiff Bangkok Clays are presented.

RÉSUMÉ: Cet article porte sur les analyses des données de contrainte-déformation des argiles molles et rigides Bangkok menées à l'Institut asiatique de technologie. Une série complète d'essais triaxiaux portant à la fois sur la compression et l'extension a été réalisée, et les résultats ont été comparés avec un certain nombre de contraintes-déformations, théories basées sur des concepts d'état critique. Le logiciel PLAXIS contient le modèle Hardening Soil comme une extension du modèle de contrainte hyperbolique de Duncan-Chang. Dans le présent document, les paramètres du modèle hyperbolique de contraintes sont évalués à partir des données recueillies pour tester les argiles douces et raides Bangkok. Le programme comprend deux séries d'essais non drainés et drainés, effectués sur des échantillons triaxiaux isotropes consolidés à la fois dans des conditions de compression et d'extension. Ces résultats d'essai peuvent être utilisés pour déterminer les paramètres non drainés et drainés de résistance au cisaillement des argiles molles et raides de Bangkok. Enfin, deux résultats de la résistance au cisaillement, non drainée et drainée, et des paramètres de rigidité pour le modèle des argiles molles et rigides Bangkok sont présentés.

KEYWORDS: shear strength parameters, hardening soil model, triaxial tests, Bangkok clay, finite element analysis

1. INTRODUCTION

Bangkok subsoils are one of the most well-known sedimentary soils and have been studied extensively in the past by many research students at the Asian Institute of Technology under the supervision of the fourth author. The experimental work was on isotropically and anisotropically consolidated triaxial tests both in compression and in extension. The results were primarily used to verify the critical state theories as developed for normally and overconsolidated clays (Balasubramaniam & Chaudry, 1978; Balasubramaniam *et al.*, 1978, 1992; Balasubramaniam & Hwang, 1980). Recently, soil models used in PLAXIS such as the Hardening Soil Model (HSM), and the Hardening Soil Model with Small Strain Behaviour (HSS) were studied by Surarak (2010) on the applications in the design and performance of deep excavations and tunnelling works in Bangkok MRT project. The HSM have been developed under the framework of the Duncan-Chang hyperbolic stress strain theory. This paper presents the work on stiffness and strength parameters based on Duncan-Chang theory.

2. GEOLOGICAL CONDITIONS OF BANGKOK SUBSOIL

The Bangkok subsoil forms a part of the larger Chao Phraya Plain and consists of a broad basin filled with sedimentary soil deposits. These deposits form alternate layers of sand, gravel

and clay. While the depth of the bedrock is still undetermined, its level in the Bangkok area is known to vary between 400 m to 1,800 m depth. Based on extensive field and laboratory studies carried out in the past by numerous researchers at AIT, the following descriptions have been proposed for the Bangkok clays: (1) Weathered Crust – the upper most layer (1 – 3 m), (2) Very Soft to Soft Bangkok Clays (3 – 12 m), (3) Stiff to Very Stiff Bangkok Clays (15 – 35 m). The index properties for Weathered, Soft and Stiff Bangkok Clays are summarised in Table 1.

Table 1. Index Properties of Bangkok Clays

Properties	Weathered Clay	Soft Clay	Stiff Clay
Natural water content (%)	133 ± 5	122 – 130	20 – 24
Natural voids ratio	3.86 ± 0.15	3.11 – 3.64	1.10 – 1.30
Grain size distribution			
Sand (%)	7.5	4.0	23
Silt (%)	23.5	31.7	43
Clay (%)	69	64.3	34
Specific gravity	2.73	2.75	2.74
Liquid limit (%)	123 ± 2	118 ± 1	46 ± 2
Plastic limit (%)	41 ± 2	43 ± 0.5	19 ± 2
Dry unit weight (kN/m ³)	15.8 ± 0.3	16.5	15.5 -16.5
Consistency	Soft	Soft	Stiff

Colour	Dark grey	Greenish grey	Greenish grey
Degree of saturation (%)	95 ± 2	98 ± 2	94 – 100

3. HARDENING SOIL MODEL

The PLAXIS finite element software became popular in geotechnical analysis and design. Constitutive models used in PLAXIS are in line from the linear and non-linear elastic models until the hardening double surface plasticity models (Schweiger, 2009). One of the most well-known hardening type models is the Hardening Soil Model (HSM). The HSM was introduced in the PLAXIS program as an extension of the Mohr-Coulomb model to allow for the pre-consolidation pressure to be taken into account. Indeed, the HSM has been developed under the framework of the plasticity theory. The hyperbolic formulation (Duncan & Chang, 1970) is used to define the stress-strain relationship. The total strains are calculated using a stress-dependent stiffness, which is different for both loading and unloading/reloading. The hardening is assumed to be isotropic, depending on the plastic shear and volumetric strains. A non-associated flow rule is adopted when related to frictional hardening and an associated flow rule is assumed for the cap hardening. The following explanation provides a brief summary of the hyperbolic stress-strain and stiffness response of HSM.

The stress-strain relationship, due to the primary loading, is assumed to be a hyperbolic curve in the HSM. The hyperbolic function, as given by Duncan & Chang (1970), for the drained triaxial test can be formulated as:

$$\varepsilon_1 = \frac{q_a}{2E_{50}} \frac{q}{q_a - q}, \text{ for } q < q_f \quad (1)$$

where ε_1 is the axial strain, and q is the deviatoric stress. The ultimate deviatoric stress (q_f) is defined as:

$$q_f = \frac{6 \sin \phi'}{3 - \sin \phi'} (\sigma'_3 + c' \cot \phi') \quad (2)$$

and the quantity (q_a) is the asymptotic value of the shear strength, in which $q_a = q_f/R_f$. The R_f is the failure ratio. Figure 1 shows the hyperbolic relationship of stress and strain in primary loading.

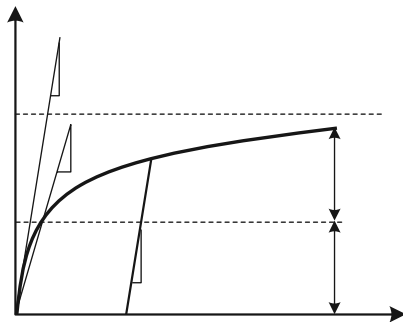


Figure 1. Hyperbolic stress-strain relationship in primary loading for a standard drained triaxial test (Schanz *et al.*, 1999)

The stress strain behaviour for primary loading is highly non-linear. The parameter E_{50} is a confining stress dependent stiffness modulus for primary loading. E_{50} is used instead of the initial modulus E_0 for small strain which, as a tangent modulus, is more difficult to determine experimentally, and is given as:

$$E_{50} = E_{50}^{ref} \left(\frac{c' \cos \phi' - \sigma'_3 \sin \phi'}{c' \cos \phi' + p^{ref} \sin \phi'} \right)^m \quad (3)$$

where E_{50}^{ref} is a reference stiffness modulus corresponding to the reference stress p^{ref} (100 kN/m²). The actual stiffness depends on the minor effective principal stress σ'_3 , which is the effective

confining pressure in a triaxial test. The amount of stress dependency is given by the power m .

The stress dependent stiffness modulus for unloading an reloading stress paths is calculated as:

$$E_{ur} = E_{ur}^{ref} \left(\frac{c' \cos \phi' - \sigma'_3 \sin \phi'}{c' \cos \phi' + p^{ref} \sin \phi'} \right)^m \quad (4)$$

where E_{ur}^{ref} is the reference modulus for unloading and reloading, which corresponds to the reference pressure p^{ref} .

Another input parameter, the reference oedometer modulus (E_{oed}^{ref}), is used to control the magnitude of the plastic volumetric strains that originate from the yield cap. In a similar manner to the triaxial moduli, the oedometer modulus (E_{oed}) obeys the stress dependency law.

Schanz *et al.* (1999) explained in detail, the formulation and verification of the HSM. A total of 10 input parameters are required in the Hardening Soil Model, as tabulated in Table 2.

Table 2. Hardening Soil Model Input Parameters

Parameter	Description	Parameter evaluation
ϕ'	Internal friction angle	Slope of failure line from MC failure criterion
c'	Cohesion	y-intercept of failure line from MC failure criterion
R_f	Failure ratio	$(\sigma_1 - \sigma_3)_f / (\sigma_1 - \sigma_3)_{ult}$
ψ	Dilatancy angle	Function of ε_a and ε_v
E_{50}^{ref}	Reference secant stiffness from drained triaxial test	y-intercept in $\log(\sigma_3/p^{ref})$ - $\log(E_{50})$ space
E_{oed}^{ref}	Reference tangent stiffness for oedometer primary loading	y-intercept in $\log(\sigma_1/p^{ref})$ - $\log(E_{oed})$ space
E_{ur}^{ref}	Reference unloading/reloading stiffness	y-intercept in $\log(\sigma_3/p^{ref})$ - $\log(E_{ur})$ space
m	Exponential power	Slope of trend-line in $\log(\sigma_3/p^{ref})$ - $\log(E_{50})$ space
ν_{ur}	Unloading/reloading Poisson's ratio	0.2
K_o^{nc}	Coefficient of earth pressure at rest (NC state)	$1 - \sin \phi'$

3. EVALUATION OF STRENGTH AND STIFFNESS PARAMETERS FOR BANGKOK CLAYS

All the test results analysed in this study were determined for undisturbed samples taken at the appropriate depths for soft clay, medium stiff clay and stiff clay. The 25.4 mm diameter thin walled sample tubes were used for soft and medium stiff clays are used for triaxial tests in weathered, soft and medium stiff clays.

3.1 Triaxial Tests

The results of several series of compression and extension tests carried out on weathered, soft and stiff clays are analysed. Test specimens were approximately 72 mm in height and 36 mm in diameter. Several series of isotropically consolidated drained and undrained compression (CID, CIU) and extension (CIUE, CIDE) tests carried out at the Asian Institute of Technology were re-analysed in this study. Most of the CID, CIU, CIDE and CIUE tests were carried out under strain controlled conditions (Hassan 1976; Balasubramaniam & Uddin, 1977). In addition, some load controlled CID and CIU tests were also considered (Balasubramaniam & Chaudhry, 1978).

3.2 Mohr-Coulomb Strength Parameters

Table 3 presents a summary of the Mohr-Coulomb strength parameters of the Bangkok subsoils (i.e. weathered clay, soft clay, stiff clay and hard clay) obtained from consolidated isotropically drained and undrained triaxial compression (CID and CIU) and extension (CIDE and CIUE) tests reported in the literature. The notations for the triaxial tests identified in Table 3 are explained. It can be seen that the differences in the applied stress path have the most significant effect on the Mohr-Coulomb strength parameters. Initial conditions at the consolidation state (i.e. isotropic or anisotropic), as well as the drainage conditions during shear (i.e. drained or undrained), also have an effect on the strength parameters, but to a lesser magnitude. Therefore, it needs to be emphasised that the strength parameters should be carefully selected according to the applied stress path, resulting from the construction sequences.

Table 3. Summary of Mohr-Coulomb Strength Parameter of Bangkok Subsoils

Reference	Depth (m)	Test type	ϕ' (°)	c' (kN/m ²)
Weathered Clay				
Balasubramaniam & Uddin (1977)	2.5 to 3.0	CIUE _U	28.9	0
		CIU	22.2	0
Balasubramaniam <i>et al.</i> (1978)	2.5 to 3.0	CID	23.5	0
		CIUE ^U	29	0
Soft Clay				
Balasubramaniam & Chaudhry (1978)	5.5 to 6.0	CIU	26	0
		CID	21.7	0
Balasubramaniam <i>et al.</i> (1978)	5.5 to 6.0	CIU	24	38
		CID	23.5	0
		CID ^P	23.7	0
		CIUE ^L	26	0
		CIUE ^U	21.1	58.7
		CIDE ^L	26.2	0
		CIDE ^U	23.5	31.8
Stiff Clay				
Balasubramaniam <i>et al.</i> (1978)	16.0 to 16.6	CID	26	30
		CIUE ^L	18	54
		CIUE ^U	25	54
		CIDE ^U	16.6	11
Hassan (1976)	17.0 to 18.0	CIU	28.1	11.4
		CID	26.3	32.8

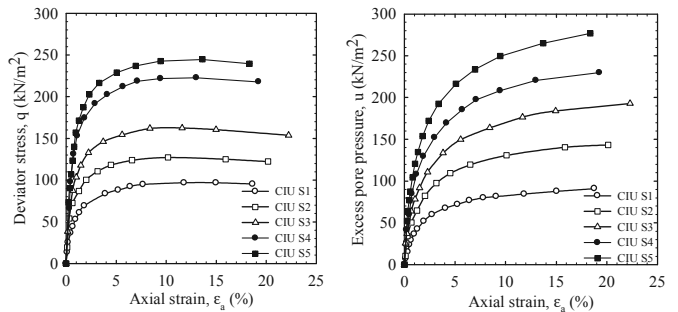
3.3 Stiffness and Strength Parameters of Soft and Stiffness Bangkok Clays

3.3.1 Soft Bangkok Clay

Two series of isotropically consolidated triaxial compression tests, CIU and CID, conducted by Balasubramaniam & Chaudhry (1978) on soft and stiff Bangkok clay, were analysed in this study. The soil samples were taken from a depth of 6.0 m below the ground surface. The confining pressures, σ'_3 used for both CIU and CID series were 138, 207, 276, 345 and 414 kN/m² for tests S1 to S5, respectively. The angle of the internal friction (ϕ') obtained from the CIU and CID tests were 27° and 23.6°; whereas, the cohesion (c') was zero for both series. The drained strength parameters are summarised in Table 4.

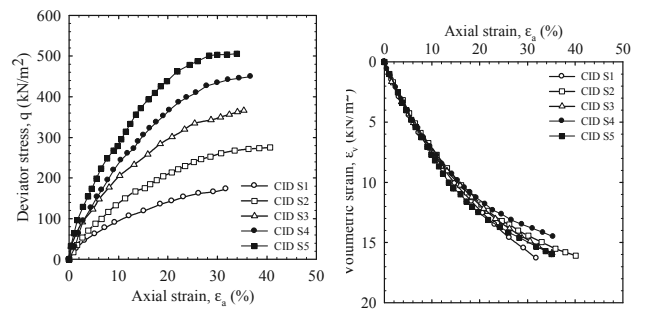
The results of the CIU triaxial tests carried out on the soft clay are plotted in Figure 2. The (q, ϵ_a) and (u, ϵ_a) relationships are shown in Figures 2(a) and 2(b), respectively. The deviator

stress and excess pore pressures versus the axial strain relationships show typical normally to lightly overconsolidated clay behaviour, where the deviator stress and excess pore pressure reaches their ultimate values at a relatively large strain. Moreover, all the excess pore pressure plots were located in the positive range.



(a) Deviator stress vs axial strain (b) Pore pressure vs axial strain
Figure 2. Results of CIU triaxial tests on soft Bangkok clay.

The results obtained from the CID triaxial tests for the soft clay are shown in Figure 3, with the relationships of (q, ϵ_a) and (ϵ_v, ϵ_a) plotted in Figures 3(a) and 3(b), respectively. It can be seen that, during the deviator stress applied, the volume of the soil specimen gradually reduces. The volumetric and axial strain curves of all the tests seem to coincide up to 10% axial strain, after that they tend to divert slightly.



(a) Deviator stress vs axial strain (b) Volumetric strain vs axial strain
Figure 3. Results of CID triaxial tests on soft Bangkok clay

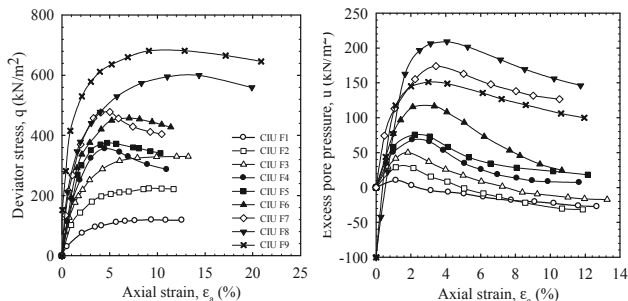
These values are also summarised in Table 4 together with the reference initial modulus ($E_i^{ref}, E_{i,50}^{ref}$), the reference moduli at 50% of strength ($E_{50}^{ref}, E_{u,50}^{ref}$), and the failure ratio (R_f) resulting from CID tests as well as the shear strength parameters (c', ϕ') for Soft Bangkok Clay.

3.3.2 Stiff Bangkok Clay

The two series of isotropically consolidated triaxial compression tests, CIU and CID, conducted by Hassan (1976) on stiff Bangkok clay, are re-interpreted in this study. The undisturbed soils samples were collected from a depth of 17.4 to 18 m below the ground surface. The pre-shear consolidation pressures ranged from 17 to 620 kN/m² and 34 to 552 kN/m², for the CIU and CID series, respectively. The angles of the internal friction (ϕ') from the CIU and CID series were 28.1 and 26.3 degrees; whereas, the values of cohesion (c') were 11.4 and 32.8 kN/m², respectively. The drained strength parameters are summarised in Table 4.

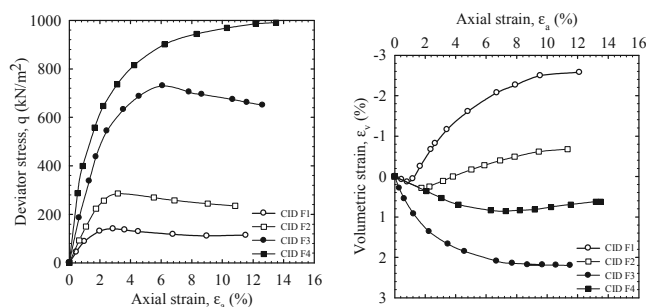
Figure 4 shows the results of CIU tests on the stiff Bangkok clay. It can be seen from Figure 4(a) that (q, ϵ_a) relationships, up to a pre-shear confining pressure of 138 kN/m² (tests CIU F1 to F3), exhibit no strain softening. At a level of confining pressure from 207 to 414 kN/m² (tests CIU F4 to F7), these clay

samples behaved as heavily overconsolidated clay showing a clear peak deviator stress at a low axial strain, followed by a strain softening. Beyond the confining pressure of 552 kN/m² (tests CIU F8 and F9), these samples behaved as lightly overconsolidated clay.



(a) Deviator stress vs axial strain (b) Pore pressure vs axial strain
Figure 4. Results of CIU triaxial tests on stiff Bangkok clay

The relationships between the excess pore pressure and the axial strain are shown in Figure 4(b). For all clay samples (CIU F1 to F9), the excess pore pressure increases as the deviator stress increases, until the peak values are reached at 1 to 4% axial strain, depending on the confining pressure. The peak excess pore pressure seems to be reached at a higher axial strain as the confining pressure increases. As the sample was further sheared, the excess pore pressure gradually reduced to the minimum value, at approximately 12% axial strain. Only the first three samples (tests CIU F1 to F3) reached negative excess pore pressures.



(a) Deviator stress vs axial strain (b) Volumetric strain vs axial strain
Figure 5. Results of CID triaxial tests on stiff Bangkok clay

The results of CID triaxial tests carried out on the stiff Bangkok clay are shown in Figure 5. The deviator stress versus the axial strain relationships of the stiff clay are shown in Figure 5(a). The pre-shear confining pressures of 34, 103, 414 and 552 kN/m² were applied. None of the stiff clay samples demonstrated a well defined peak. However, samples CID F1 to F3 (with confining pressure of 34, 103 and 414 kN/m²) illustrate some degree of strain softening after the peak deviator stresses are reached at axial strain levels of 3 to 5%. The plots of the volumetric versus the axial strain are given in Figure 5(b). The specimens with a confining pressure of 34 and 103 kN/m² (tests CID F1 and 2) start to dilate at about 1.2 and 3.5% axial strain. The specimen at 414 kN/m² confining pressure consolidates up to an axial strain level of 8%. After that, the volumetric strain seems to be constant with an increase in axial strain. The last specimen with a confining pressure of 552 kN/m² consolidates up to 7% of the axial strain, and then it tends to dilate.

The values of E_{50}^{ref} and $E_{u,50}^{ref}$ together with the deformation moduli and the failure ratios resulting from the CIU and CID series are also summarised in Table 4. It can be observed from Table 4 that the failure ratio (R_f) falls in a narrow range with an average value of 0.88. The power m for both the initial and the 50% moduli are approximately 0.5.

Table 4. Stiffness and strength parameters from CID and CIU tests for Bangkok Clays

Parameters	CID	CIU
Soft Clay		
Confining pressure (kN/m ²)	138 – 414	138 – 414
Initial $E_i^{ref}, E_{u,i}^{ref}$ (kN/m ²)	1343	7690
m	1.0	1.2
50% $E_{50}^{ref}, E_{u,50}^{ref}$ (kN/m ²)	690	4831
m	1.1	1.0
R_f	0.72	0.94
ϕ'	23.6	27.0
c' (kN/m ²)	0	0
Stiff Clay		
Confining pressure (kN/m ²)	34 – 552	17 – 620
Initial $E_i^{ref}, E_{u,i}^{ref}$ (kN/m ²)	29676	30109
m	0.52	0.46
50% $E_{50}^{ref}, E_{u,50}^{ref}$ (kN/m ²)	14398	11104
m	0.48	0.53
R_f	0.89	0.88
ϕ'	26.3	28.1
c' (kN/m ²)	32.8	11.4

4. CONCLUSION

In this study, the experimental data on soft and stiff Bangkok clays available in the literature was reanalysed in order to obtain the Duncan-Chang stiffness and strength parameters required for the Hardening Soil Model. Undrained and drained behaviour of Soft and Stiff Bangkok Clays was modelled using these parameters.

5. REFERENCES

- Balasubramaniam A.S. and Hwang Z. M. 1980. Yielding of weathered Bangkok clay, *Soils and Foundations*, 20(2), 1-15.
- Balasubramaniam, A.S. and Chaudhry, A. R. 1978. Deformation and strength characteristics of Soft Bangkok Clay. *Journal of Geotechnical Engineering Division, ASCE*, 104, 1153 - 1167.
- Balasubramaniam, A.S. and Uddin, W. 1977. Deformation characteristics of weathered Bangkok Clay in triaxial extension. *Géotechnique*, 27(1), 75 - 92.
- Balasubramaniam, A.S., Handali, S., and Wood, D. M. 1992. Pore pressure-stress ratio relationship for soft Bangkok clay, *Soils and Foundations*, 32(1), 117-131.
- Balasubramaniam, A.S., Hwang, Z. M., Waheed U., Chaudhry, A. R. and Li, Y. G. 1978. Critical state parameters and peak stress envelopes for Bangkok Clays, *Quarterly Journal of Engineering Geology*, 1, 219-232.
- Duncan, J.M. and Chang, C.M. 1970. Nonlinear analysis of stress and strain in soils. *Journal of Soil Mechanics and Foundations Division, ASCE*, 96(SM5), 1629-1653.
- Hassan, Z. 1976. Stress-strain behaviour and shear strength characteristics of stiff Bangkok Clays. Master Thesis, Asian Institute of Technology, Thailand.
- Schanz, T., Vermeer, P.A., and Bonnier, P.G. 1999. The hardening soil model: formulation and verification. Beyond 2000 in Computational Geotechnics. Rotterdam.
- Schweiger, H.F. 2009. Influence of constitutive model and EC7 design approach in FEM analysis of deep excavations. In: *Proceeding of ISSMGE Int. Seminar on Deep Excavations and Retaining Structures*, Budapest, 99 - 114.
- Surarak, C. 2010. Geotechnical aspects of the Bangkok MRT blue line project. Ph.D. Thesis, Griffith University, Australia.

Laboratory investigation of seismic effects of nanoparticle dispersions in saturated granular media

Étude en laboratoire des effets sismiques des dispersions de nanoparticules dans les milieux granulaires

Luke B.

University of Nevada Las Vegas, Las Vegas, Nevada, USA

Werkema D.

US Environmental Protection Agency, Las Vegas, Nevada, USA

Andersen S.

US Environmental Protection Agency and University of Nevada Las Vegas, USA,

ABSTRACT: Nanomaterials used in industrial applications and consumer products are widespread, thereby increasing the likelihood of unintended environmental release. The fate and transport of nanoparticles in the environment and their effects on the environment and human health are not well understood. This research investigates the potential to use seismic methods for such fate and transport studies. A test cell using piezoceramic bender elements was constructed to investigate how nanoparticles dispersed in the pore fluid of a saturated glass bead medium affect seismic wave propagation. Test cell design addresses optimal seismic wave propagation, uniformity and repeatability of the placement of the granular media and uniformity of fluid flow. Time histories were produced from two tests optimized for shear wave propagation. The first (baseline) test used deionized (DI) water. This test demonstrated the need to stabilize the sample before making measurements by first flushing several liters of liquid through the system. The second test, conducted on a new sample, used a solution of 0.05% nano Zinc Oxide (nZnO) in DI water after first flushing with DI water. Comparison of results between the two tests shows only weak repeatability between test specimens. Despite this, results of the second test still indicate a significant change in response in the presence of nZnO, particularly in signal amplitude. Studies are ongoing to increase experimental reliability and sensitivity, and to more closely approximate expected field conditions.

RÉSUMÉ : Les nanomatériaux utilisés dans les applications industrielles et produits consommation sont très répandus ; il est donc probable que ces substances se retrouvent disséminées dans l'environnement. Le destin et le transport des nanoparticules dans l'environnement et leurs effets sur l'environnement méritent un étude approfondie. Cet article étudie la possibilité d'utiliser des méthodes sismiques pour étudier ces effets. Une cellule a été construite pour voir comment les nanoparticules dispersées dans le fluide interstitiel du verre change la propagation d'ondes. La conception de la cellule d'essai permet d'étudier la propagation des ondes sismiques, le positionnement des milieux granulaires et l'uniformité de l'écoulement du fluide. Les temps de parcours de la propagation des ondes sismiques dans une dilution d'oxyde de zinc nano 0,05 % (NZnO) dans une matrice de billes de verre sont présentés et comparés à ligne de base. Nous avons trouvé une légère réduction de la vitesse de cisaillement et de compression en présence de NZnO par rapport aux valeurs initiales. Nous proposons des études plus complexes qui se rapprocheraient des conditions dans la nature.

KEYWORDS: nanoparticles, seismic, fate and transport, piezoceramic, bender elements.

1 INTRODUCTION

The use of nanoparticles in industrial applications and consumer products has become widespread and continues to grow. As applications of nanoparticles increase, so does the likelihood of unintended environmental release, including the possibility of a large-scale spill event. The fate and transport of nanoparticles dispersed in the environment are largely unknown (Conlon, 2009; Klaine et al., 2008), and their effects on the environment and human health are also not well understood. Consequently, methods are needed for detecting, characterizing, and monitoring subsurface transport of nanoparticles. The capability of electrical geophysical methods has shown some promise in the spectral induced polarization (SIP) response to select nanoparticles in saturated sand laboratory columns (Joyce et al., 2012). This paper investigates the seismic response to nanoparticles in a similar laboratory setting, in order to complement the SIP results and evaluate another geophysical method.

We have developed a test cell that uses piezoceramic bender elements to investigate how nanoparticles dispersed in the pore fluid of a glass bead matrix can affect seismic wave propagation characteristics. To minimize chemical interactions between the granular medium and the nanoparticle solution and to provide uniform grain morphology, non-reactive glass beads are used for the granular medium. Seismic wave characteristics (spectral content, travel time, signal amplitude) can be scrutinized for distinguishing characteristics. Results may suggest whether seismic methods are suitable for nanoparticle fate and transport studies.

This paper reports on the test cell design and development of experimental procedures. Some preliminary travel time and

signal amplitude results using 0.05% nZnO solution are included.

2 TEST CELL DESIGN AND TESTING PROTOCOLS

The test cell design is based on preliminary work by Rajabdeen et al. (2012). The sample or experimental treatment housing is a translucent 15.2-cm inner-diameter PVC cylinder with custom end caps that are fitted with piezoceramic bender elements (Figure 1). The bender elements serve as seismic transmitter and receiver. The elements used in this study are two-piezo layer transducers made with PSI-5A4E piezoceramic (a Lead Zirconate Titanate (PZT) piezoceramic), parallel-poled, nickel electrodes and using brass center reinforcement (Piezo Systems Inc.).

The bender elements are 12.7 mm square and 0.5 mm thick. Elements are potted in vinyl caps using epoxy which are placed inside small-diameter PVC tubes that pierce and affix to the centers of the end caps (Figure 1). This configuration is intended to be robust while also creating impedance traps to encourage transmission of seismic energy through the sample rather than through the test apparatus. The cantilever length (protrusion into the sample) is 4 mm, approximately one-third the length of the element. A short cantilever length reduces dependency of the system resonance frequency on the sample matrix properties (Lee and Santamarina, 2005). The bender elements are installed in-plane such that the S-wave energy propagates along a direct, straight-line path while the strongest P-wave arrival at the receiver would be reflected from the cylinder walls (Figure 1). The test cell design integrates the same considerations of signal attenuation and near-field effects that have been applied to bender-element testing in oedometers

(e.g., Dyvik and Olsen (1991), Zeng and Ni (1998), Lee and Santamarina (2005), Lee et al. (2007)). The bender element tip-to-tip separation is 6.2 cm, yielding a ratio of test specimen diameter to sensor separation of approximately 2.45. The sample height is 15.8 cm. Figure 2 contains a view of the test system including key ancillary equipment.

Glass beads used for the granular testing matrix are 0.5 mm in diameter. Consideration is given to optimize uniformity and repeatability of the placement of the glass beads, in order to minimize unwanted acoustic impedance contrasts in the medium, to discourage preferential fluid flow pathways, and to facilitate comparisons among tests. The oven-dried glass beads are vibrated into place using a vibratory table operating at 60 Hz for 20 minutes with a surcharge load of 28 kg. The surcharge mass was selected based on the methodology laid out by ASTM D4253 - 00(2006) Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table. The surcharge mass is kept in place throughout testing. It approximates a vertical overburden stress of 15 kPa.

The glass beads are flooded with de-ionized (DI) water which is introduced through upward flow distributed across the cross-section of the experimental column. The water is plumbed into the bottom of the test chamber to reduce the amount of entrapped air and is plumbed under gravity flow with total head not exceeding 0.3 meters. The fluid-flow gradient is kept low to discourage entrainment of air bubbles and approximate laminar flow. The fluid passes through a set of baffles and a perforated disc before entering the sample (Figure 1). This process is intended to encourage a uniform wetting front and discourage fingering and the formation of preferential flow pathways. A somewhat loose-fitting top cap allows the cell to be completely flooded and allows the overburden stress to act directly on the specimen. Excess water exits the test apparatus through a port above the cap. Satisfactory dispersion of liquid through the glass-bead-filled test chamber was demonstrated using colored dye (Figure 3). The capacity of the fluid-delivery system including test cell is approximately 1.7 liters.

Data are collected with a Data Physics (Data Physics, Inc.) dynamic signal analyzer. Single sine pulses are created using a function generator. The sampling interval is 9.301 microseconds. The signal is not filtered during data capture. The reported result is an average from 350 pulses, which are repeated at 0.7-second intervals.

3 BASELINE TESTING

Through resonance testing and experimental trials, optimum frequencies to test for shear and compression were determined to be 1 kHz and 8 kHz, respectively. The compression measurements have not yet been resolved satisfactorily because of complications with electrical crosstalk between source and receiver and are not presented here. Results of baseline testing for shear (using DI water, with no experimental treatment) are shown in Figure 4. Four datasets are collected. The first dataset is collected after initial inundation of the sample. Repeat collections occur after 4, 8 and 12 additional liters of DI water are flushed through the sample. The time history from the initial measurement is dramatically different from those collected later. Results demonstrate that up to four liters of fluid need to be flushed through the system before the response stabilizes. The flushing process is likely to improve the signal by expelling entrained air.

The zero time in Figure 4 corresponds to source initiation. Based on first arrival by visual interpretation, the shear wave velocity of the specimen is approximately 170 m/s. Considering the timing for the first troughs in the signals, velocity estimates among the three measurements (after flushing 4, 8 and 12 liters of DI water) would vary by approximately +/- 3 %. Amplitudes at the first trough vary by +/- 17%.

4 NANOPARTICLE EFFECTS

The test sequence was repeated on a new sample, in which a solution of 0.05% (by weight) nZnO solution was introduced. The nanoparticle treatment solution is made through sonication to uniformly disperse the nanoparticles in DI water. Eight liters of clean DI water were flushed through the test chamber first, followed by 4 liters of the nanoparticle solution.

Results are shown in Figure 5. Again, the dataset conducted before flushing was not representative of results after flushing. The time histories collected after flushing but before introducing the nZnO treatment would ideally be the same as those collected in the baseline test. However, comparison with Fig. 4 demonstrates that the tests are not closely repeatable between samples, despite careful efforts to replicate test conditions. For example, consider the difference in signal amplitude between the two figures. This is an unfortunate discrepancy that requires further investigation. From Figure 5, considering the time histories gathered after flushing 4 and 8 liters of DI water, the shear wave velocity is approximately 150 m/s, which represents a decrease of 12 % with respect to baseline (Figure 4). Also, for the later test (Figure 5), results do not stabilize as fluid flushing occurs to the extent seen in baseline testing. Repeating analyses of the timing and amplitude of the first trough, velocity estimates among the measurements after flushing 4 and 8 liters would vary by approximately +/- 9 % and amplitudes by +/- 3 %. With respect to the baseline test, the percentage variabilities are higher for velocity and lower for amplitude.

Despite the variability observed with DI water, the trial conducted after introducing the nZnO experimental treatment (labeled in Figure 5 as "12") shows distinctly different results from those collected earlier on the same test specimen. Signal amplitude is dramatically increased, and timing is delayed. To effectively quantify differences, cross-correlation of traces, signal matching or coda wave analysis might be applied (Lee and Santamarina 2005, Dai et al. 2012).

The significant increase in signal amplitude might be due to agglomeration of nZnO onto the glass bead matrix (Klaine et al., 2008) which would enhance contacts in the skeletal structure of the glass bead matrix, thereby reducing signal attenuation. The smaller decrease in velocity is not as readily explained. One hypothesis (which is yet untested) is that the flushing process allows sufficient displacement of the glass beads to permit introduction of nanoparticles between contacts. This possibility seems unlikely given the slow influent flow rate and the amplitude of the surcharge pressure.

5 CONCLUSIONS

A laboratory apparatus has been developed to study seismic response to the presence of nanoparticles dispersed in the pore fluid of saturated granular media. Uniform placement of the solid sample matrix is encouraged through the use of a vibratory table and a surcharge load. A perforated plate and baffle system for introduction of fluid at the bottom of the test cell encourage laminar flow and uniform fluid distribution which is demonstrated using dye. Baseline testing in clean water using sine pulses at 1 kHz, chosen to optimize transmission of shear energy, demonstrated that by first flushing several liters of fluid through the sample, repeated measurements would yield similar results. Testing a new sample demonstrated that close repeatability of results is not assured between samples.

The measurements revealed sensitivity of the nanoparticle treatment to shear wave energy propagation. After flushing a sample with a treatment containing 0.05% nZnO, measurements revealed a dramatic increase in signal amplitude and a slight increase in travel time. Changes might be due to agglomeration of nanoparticles on the granular matrix which enhances grain-to-grain contacts, possibly coupled with insertion of nanoparticles between grains which reduce system stiffness.

Further experimentation is required to assess if the observed responses are indeed trends. Once accomplished, the research will expand to increase experimental sensitivity and broaden scope, and to more closely approximate conditions in nature.

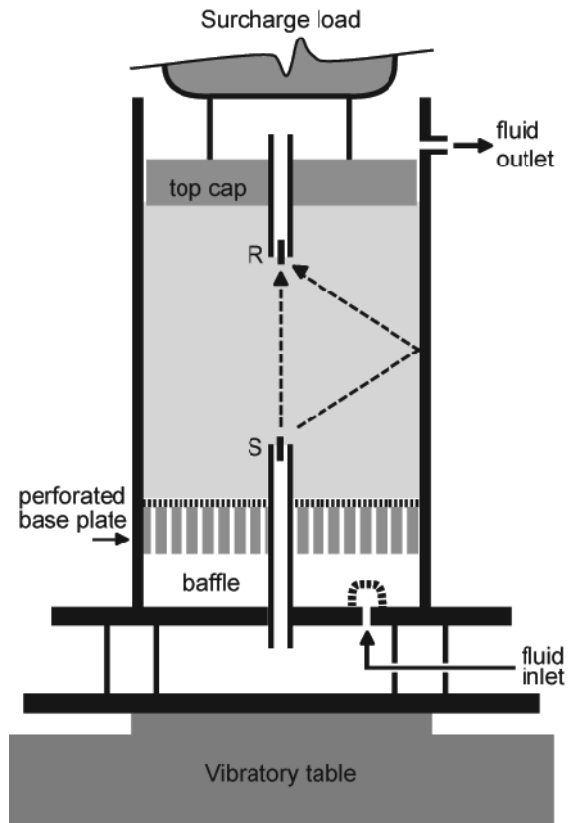


Figure 1. Schematic cross section of test cell. S, R: source and receiver bender elements, respectively. Direct-transmission S-wave path and reflected P-wave path are shown.

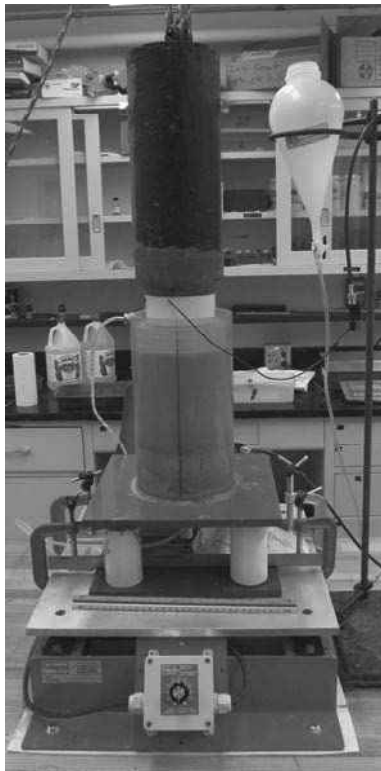


Figure 2. Photo of test cell and ancillary apparatus. Test cell is in center, situated on vibratory table. Surcharge mass is suspended from above. Reservoir for gravity feed of fluid appears as white container in the upper right of the photo.

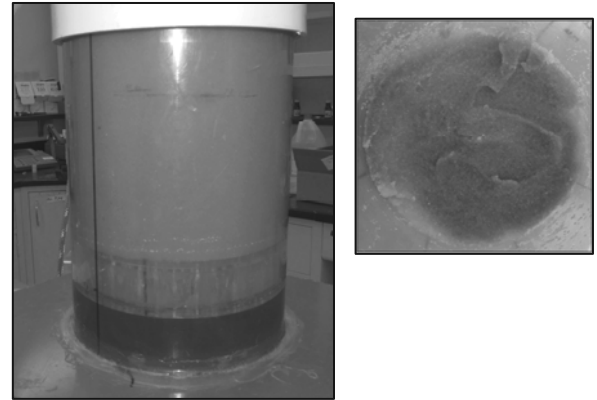


Figure 3. Dye test demonstrates uniformity of distribution of influent. Left: View of column from side. Fluid-filled baffle zone is indicated by dark shade at bottom; just above is perforated base plate. Dye appears to be distributed evenly throughout the sample. Right: View from above of glass beads inside test cell during disassembly.

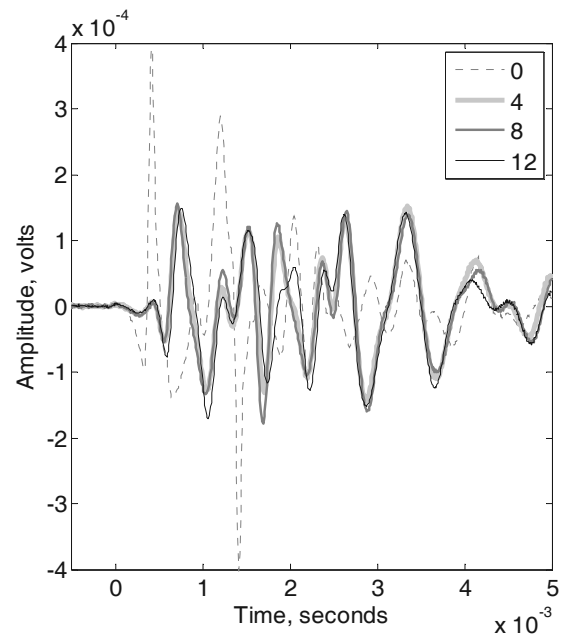


Figure 4. Results of first test: baseline. Legend refers to volume of DI water flushed through the specimen before testing, in liters.

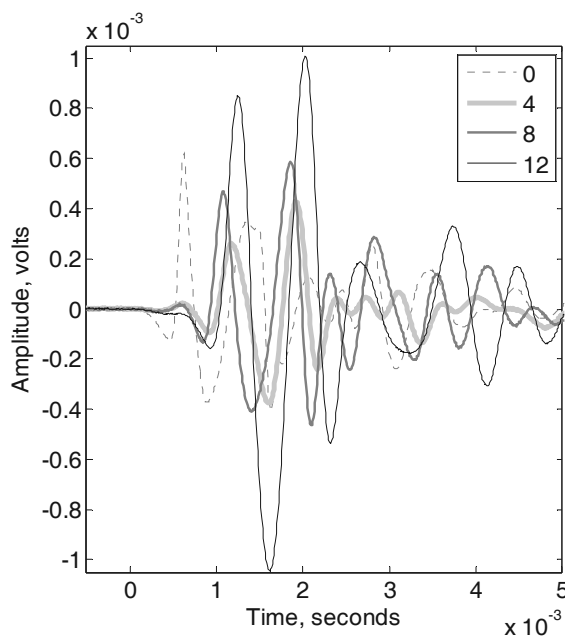


Figure 5. Results of second test. Legend refers to volume of fluid flushed through the specimen before testing, in liters. First eight liters were DI water; final four liters contained experimental treatment (i.e., trial marked “12” describes effect of experimental treatment). Note difference in vertical scale with respect to Figure 4.

6 ACKNOWLEDGMENTS

This work was funded through the U.S. EPA Office of Research and Development contracts EP09D000305 and EP10D000763. It was reviewed by EPA and approved for presentation; it may not necessarily reflect official Agency policy. Mention of trade names or commercial products does not constitute endorsement or recommendation by EPA for use. Practical advice from Carlos Santamarina and colleagues on preparation and use of bender elements is greatly appreciated.

7 REFERENCES

- Conlon, M., (2009). EPA science in action: Nanotechnology research program, National Exposure Research Laboratory, US EPA Office of Research and Development. Retrieved 09/09/12 from http://www.epa.gov/nanoscience/quickfinder/pdf/nanotech_nanomaterials.pdf
- Dai, S., Wuttke, F., Santamarina, J. (2012). Coda wave analysis to monitor processes in soils. *Journal of Geotechnical and Geoenvironmental Engineering*, doi: 10.1061/(ASCE)GT.1943-5606.0000872.
- Dyvik, R., Olsen, T. S. (1991). Gmax measured in oedometer and DSS tests using bender elements. Publication - Norwegian Geotechnical Institute, 181 pp.
- Joyce, R. A., Glaser, D. R., Werkema Jr., D. D., Atekwana, E. A., (2012). Spectral induced polarization response to nanoparticles in a saturated sand matrix, *Journal of Applied Geophysics*, vol. 77, pp. 63-71. doi: 10.1016/j.jappgeo.2011.11.009.
- Klaine, S.J., Alvarez, P.J.J., Batley, G.E., Fernandes, T.F., Handy, R.D., Lyon, D.Y., Mahendra, S., McLaughlin, M.J., Lead, J.R., (2008). Nanomaterials in the environment: behavior, fate, bioavailability, and effects. *Environmental Toxicology and Chemistry*, vol. 27, no. 9, pp. 1825-1851.
- Lee, J., Santamarina, J. C. (2005). Bender elements: Performance and signal interpretation. *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 131, no. 9, pp. 1063-1070.
- Lee, C., Lee, J.S., Lee, W., Cho, T. H. (2007). Experiment setup for shear wave and electrical resistance measurements in an oedometer. *Geotechnical Testing Journal*, vol. 31, no. 2, pp.149-156.
- Rajabdeen, N., Luke, B, and Werkema, D.D., Jr. (2012). Evaluation of the Seismic Characterization of Select Engineered Nanoparticles in Saturated Glass Beads. U.S. Environmental Protection Agency Report EPA/600/R-12/547.
- Zeng, X., and Ni, B. (1998). Measurement of Gmax under anisotropic loading condition using bender elements. *Geotechnical Earthquake Engineering and Soil Dynamics III*, ed. P. Dakoulas and M. Yegian. American Society of Civil Engineers, Reston, Virginia, USA *Geotechnical Special Publication* 75, 189-200.

The SCS Double Hydrometer Test in dispersive soil identification

Essai SCS de double hydrométrie pour l'identification des sols dispersifs

Maharaj A., Paige-Green P.
CSIR Built Environment, Pretoria, South Africa

ABSTRACT: The standard testing procedures for the Soil Conservation Service (SCS) Double Hydrometer test, the Pinhole Test, Crumb test and chemical analyses for the identification of potentially dispersive soils have recently been studied and problems/anomalies identified. Recent testing suggests that many of the shortcomings related to testing of dispersive soils may have been overlooked during past routine investigations. A comparative study involving the testing of three samples using one standard laboratory test, namely the SCS double hydrometer test was carried out and some potential means of overcoming the problems have been identified. The investigation has highlighted the differences that can be obtained on a single soil as a function of the variation in test procedures. The variability of the results obtained from the double hydrometer test in particular appears to be the cause of many of the ambiguities and discrepancies in the classification systems studied during this research. Problems related to the double hydrometer tests pose the potential for misleading results since the test has been used as the basis for identifying the potential dispersiveness of soils during the development of rating systems.

RÉSUMÉ : Les procédures de test standard pour le Soil Conservation Service (SCS) test hydromètre double, le test sténopé, le Crumb test et des analyses chimiques pour l'identification des sols potentiellement dispersifs ont récemment été étudiés et des problèmes ou anomalies identifiés. Des tests récents indiquent qu'un bon nombre de défauts liés à l'analyse des sols dispersifs peut avoir été négligé au cours des enquêtes de routine conduites dans le passé. Une étude comparative entre l'essai de trois échantillons à l'aide d'un test de laboratoire standard, à savoir le test SCS hydromètre double, a été effectué et des moyens possibles de surmonter les problèmes ont été identifiés. L'enquête a mis en évidence les différences qui peuvent être obtenues pour un sol unique en fonction de la variation dans les procédures de test. La variabilité des résultats, obtenus à partir du test SCS hydromètre double en particulier, semble être la cause de bien des ambiguïtés et des incohérences dans les systèmes de classification étudiés au cours de cette recherche. Les problèmes liés aux tests hydromètre double présentent le potentiel pour des résultats erronés, puisque le test a été utilisé comme base pour déterminer la dispersivité potentielle des sols lors de l'élaboration des systèmes de notation.

KEYWORDS: Dispersive soils ; hydrometer ; soil testing

1 INTRODUCTION

The failure of dams and embankments as a result of the use of dispersive clays has been recognized by engineers and geologists in South Africa and internationally for many years. A serious problem, however, still lies in the early identification of dispersive soils (Paige-Green, 2008). Despite all the information gained over the years, there is still no quick, simple and reliable means of conclusively identifying dispersive soils. Many methods have been proposed including the pinhole, double hydrometer, crumb and chemical tests, either individually or in combination. These, however, have not always been entirely reliable and it was considered possible that the reason lay in the actual testing procedures.

The standard testing procedures for the Soil Conservation Service (SCS) Double Hydrometer test, the Pinhole Test, Crumb test and chemical analyses have recently been studied and problems/anomalies identified. Although no discussion regarding these anomalies has been found in the literature, the recent testing suggests that many of these shortcomings may have been overlooked during past routine investigations. This paper summarizes a comparative study involving the testing of three samples using one standard laboratory test, namely the SCS double hydrometer test and discusses some potential means of overcoming the problems identified. The other tests have been discussed separately (Maharaj, 2010a: 2010b: 2011: 2012).

2 HISTORY OF THE SCS DOUBLE HYDROMETER TEST

The Soil Conservation Service (SCS) double hydrometer or dispersion test has been identified as one of the most appropriate tests for classifying dispersive soils. The test evaluates the dispersibility of a soil by measuring the natural tendency of the clay fraction to go into suspension in water. The procedure involves the determination of the percentage of particles in the soil that are finer than 0.005 mm using the standard hydrometer test. A parallel test is also carried out, in which no chemical dispersant is added and the solution is not mechanically agitated. The quantity of particles finer than 0.005 mm in the parallel test is expressed as a percentage of this fraction determined in the standard test, which is defined as the dispersion ratio or dispersivity of the soil (Walker, 1997). Dispersion ratios greater than 50% are considered highly dispersive, between 30 and 50% are moderately dispersive, between 15 and 30% are slightly dispersive and less than 15% are non-dispersive (Elges, 1985). Similar systems with different limits were utilized by Gerber & Harmse (1987) and Walker (1997).

The dispersion test was first described by Volk (1937) as a means of determining the potential dispersiveness of soils. The test has since been used extensively in this regard with minor modifications. Volk's test compared the weight of soil grains, 0.005 mm or smaller that slaked free when air-dried lumps of soil were soaked in quiet, distilled water, with that of the entire soil. This was expressed as the percentage dispersion. Measurements of the clay in the soil-water suspension were made by the pipette method (Volk, 1937).

The pipette method itself was introduced by Middleton in 1930 as a means of determining the erosion potential of a soil. The difference in the methods was that the samples collected in the pipette consisted of particles of a maximum diameter of 0.05mm (Middleton, 1930). Volk's method was, however, preferred because studies in the southwest of the United States indicated that the dispersion of the clay fraction (< 0.005 mm diameter), was more significant in assessing the piping potential of soils (Decker and Dunnigan, 1977).

The SCS involvement in earth dam construction in the United States increased in the 1940's and early 1950's. The dispersion test was employed as a routine procedure for all samples submitted to SCS Laboratories. It was during this period that the test procedure was adapted to use a hydrometer instead of a pipette (Decker and Dunnigan, 1977). The procedure is specified in ASTM standard D422-63 for Particle Size Analysis of soils (ASTM International, 2007a & b).

With the increased use of the dispersion test and the growth of the SCS testing facilities in the 1950's and 1960's, a few Table 1: Difference in testing procedures used for the determination of the clay fraction of a soil.

Property	ASTM- D422-63	BSi- BS 1377-2: 1990	TMH1- 1986 (A6)
Amount of dry soil required for the test:	100g of sand sized particles (i.e.: particles less than 2mm in size) or 50g of soil fines if material has a high percentage of silt/ clay.	Depends on type of soil, i.e.: 100g for sandy soil, 50g for silty soil and 30g for clayey soil.	100g of soil fines (i.e.: particles less than 0.425mm in size) or 50g of soil fines if material has a high percentage of silt/ clay.
Pre-treatment	None	With hydrogen peroxide if organic matter present.	None
Dispersing agent	125ml sodium hexametaphosphate solution at 40g/l.	100ml sodium hexametaphosphate solution comprising 33g Na-hexa. + 7g Na-carbonate in distilled water to make 1 l of solution. NB: If soil does not disperse completely, allow soil to settle, decant as much of the water as possible and then add a further 100ml of dispersant.	5 ml each of sodium silicate and sodium oxalate solutions.
Soaking	Samples soaked for minimum of 16 hrs.	Soak in solution for min of 4hrs or overnight.	Minimum period of 2 hrs but preferably overnight.
Hydrometer readings	Taken at 2min; 5min; 15min; 30min; 1h; 4h and 24h.	Taken at 8min; 30min; 2h; 8h & 24h.	Taken at 18s, 40s & 1h.
Analysis	Equations used to calculate maximum diameter of particles in suspension	Equations used to calculate maximum diameter of particles in suspension.	Maximum particle size calculated. At 1h, max size is 0.005mm, at 40s max size is 0.05mm and at 18s, max size is 0.075mm.
Temperature	Constant temperature at or near 20°C is required.	Constant temperature bath of 25°C, to an accuracy of $\pm 0.5^\circ\text{C}$.	20°C when readings are taken or a temperature correction has to be applied.

Close observation of these test procedures illustrates little variation in the method of determination of the dispersion ratio, except with regard to the types of dispersing agents used. The ASTM and BSi standards specify that sodium hexametaphosphate be used as a dispersing agent. However, the solution is prepared differently and at different proportions in each standard. The volume required to disperse the sample in the test is also very different.

TMH1 specifies that a combination of sodium silicate and sodium oxalate be used as the dispersing agent. A study of the past revisions of the South African standards shows that the combination of sodium silicate and sodium oxalate was specified in the 1958 and 1986 test methods, whereas, sodium hexametaphosphate was specified as the standard dispersant in the 1970 version. There is no reason found as to why the combination of sodium silicate and oxalate is used instead of sodium hexametaphosphate or why there was a change in the standard in 1970.

It should also be noted that TMH1 states that the one hour hydrometer reading indicates the percentage of the clay fraction in the sample. Analyses using Stoke's Law show that at one

inconsistencies were detected, such as, the test was not always reproducible in the same laboratory, and there appeared to be differences in the results between laboratories. Studies revealed that water quality and moisture contents of the samples were responsible for these errors and as such distilled water and in situ moisture contents were specified (Decker and Dunnigan, 1977). Since that time, there appear to have been no discussions in the literature regarding any anomalies encountered during testing.

3 EVALUATION OF CURRENT TESTING METHODS

The test methods for hydrometer analysis currently in use are the American standards (ASTM International, 2007), the British standards (BSi, 1990) and the South African Technical Methods for Highways - TMH1 (NITRR, 1986). Table 1 summarizes procedures carried out for each test standard, focusing on the main aspects of the testing procedure.

hour, particles in suspension in all samples are in the range of 0.006 – 0.007 mm, which is silt-sized and not clay-sized.

4 TESTING EQUIPMENT AND METHODS USED

A study was conducted to evaluate the effects of the different methods on the test results. For the study, testing procedures that were carried out were the South African standard (TMH1) and the American standard (ASTM), the primary difference in the procedures, being the variation in dispersing agents. The tests were carried out on three samples, one non-dispersive, one highly dispersive and a third that was presumed to be moderate to slightly dispersive (based on past experience and field observations).

The dispersing agents used were combinations of sodium hexametaphosphate (33g) plus sodium carbonate (7g) in a liter of water and a sodium silicate (5ml) plus sodium oxalate (5ml) solution. Samples were left to stand for approximately 16 hours after dispersion before being mechanically agitated and then the hydrometer readings were taken. Time intervals for the hydrometer readings were at 1 hour, 40 seconds and 18 seconds as specified in TMH1 Method A6. Duplicate samples were

prepared according to the parallel test procedure, with no chemical dispersant and mechanical agitation.

5 RESULTS

The one hour readings as well as the calculated dispersion ratios for the two dispersing agents are summarized in Table 2.

Table 2: Results obtained from 1 hour hydrometer readings and dispersion ratios

Sample	Control	Na silicate & oxalate	Dispersion ratio %	Na Hexameta-phosphate	Dispersion ratio %
ND309	1.9	7.9	17.7	13.9	10.1
UM108	2.6	11.6	22.4	16.6	15.7
ZT114	19.9	15.9	125.2	24.9	79.9

The test results show that there is significant variation in the apparent clay fraction between the two dispersants. The samples that were dispersed with sodium hexametaphosphate produced more realistic dispersion ratios than those of the other samples.

This could mean that the samples were not completely dispersed with this dispersing agent (sodium silicate + oxalate) or that there was some variation in sample preparation. However, as the samples were prepared together with utmost care to ensure ideal representatives, the latter is unlikely.

The maximum dispersion should occur when a chemical dispersant is added to the sample ensuring a total disaggregation of all flocs of soil. A high dispersion ratio indicates that the sample breaks down significantly without the use of a chemical dispersant and is thus dispersive. In theory, the 0.005 mm fraction for the parallel test cannot be higher than that of the standard test with the use of a chemical dispersant. This is only likely to occur if there are inconsistencies in the testing procedure or if the chemical dispersant does not act fully on the material. The dispersion ratio of 125.2% shown for the sodium silicate/oxalate dispersant (Table 2) indicates that the dispersant in the standard test did not completely disperse the particles or possibly caused some flocculation to occur.

An additional observation is that the classification obtained for the two dispersants do not correlate. Sample ND309 falls into two different categories of dispersivity based on the classification by Elges (1985). It is classified as being slightly dispersive using sodium silicate/oxalate and non-dispersive using sodium hexametaphosphate, which can lead to uncertainty regarding treatment requirements, should it be used for construction.

6 DISCUSSION

Although the results of only limited testing is reported in this paper, it is clear that the incorrect classification of dispersive soils as a result of test variations appears to be common.

Inconsistencies noted in the available literature include the variations in test methods. Observations have found that different authors indicate different particle sizes for the clay fraction. TMH1 and ASTM use the 0.005 mm fraction as the boundary for the clay fraction, whereas BSI uses the 0.002 mm. Many authors quote the 0.005 mm fraction as the clay fraction

when determining dispersivity of a soil. A Dictionary of Geology (1972) defines the clay fraction as a mineral particle having a diameter less than 0.004 mm (1/256 mm). According to Reeves et al (2006), the ASTM standards define the clay fraction as being less than 0.005 mm and Japan defines the fraction as less than 0.006 mm. However, the majority of the countries listed define the clay fraction as particle sizes less than 0.002 mm. Once again there is no standard definition with regards to the unit size for clay particles although a scan of the literature shows that 0.002 mm is used more widely. As the 0.002 mm fraction is also the basis for classification of South African soils according to Brink and Bruin (2002), this size fraction should be taken as the upper limit of clay-sized particles for future studies in South Africa.

The literature also indicates that during studies of dispersive soils the initial indicator of dispersivity of the material is generally classified on the basis of the double hydrometer test by means of various indicator graphs/plots. Many workers (Gerber and Harmse, 1987; Bell and Maud, 1994; Walker, 1997) have then proceeded to indicate that no single test (including the double hydrometer test) can be used to identify dispersive soils, and then propose classification rating systems using a number of tests. It is postulated that many of the ambiguities (i.e., the inconsistencies of results among workers) are the result of the incorrect initial classification of the dispersivity of materials as a result of variations introduced in the double hydrometer test.

Most of the rating systems used currently in South Africa seem to have been based on the initial classification of dispersiveness by the double hydrometer test. Gerber and Harmse (1987) used the test as a primary parameter when developing the ESP-CEC chart. Walker (1997) included the ESP-CEC chart as a parameter in the rating system and studies carried out by Bell and Walker (2000) also make use of the double hydrometer test when initially classifying the dispersive soils.

This has resulted in the overlap of results within single classification bands. Although it is assumed that in these investigations, the materials have been tested following uniform and standard procedures, preliminary testing has indicated spurious results when sodium silicate/oxalate (the South African road standard) is used as the dispersant (NITRR, 1986). It is also noted that the dispersant standard in South Africa has changed over time, possibly affecting the results, if they were obtained from different laboratories over a prolonged period of time. It can thus be assumed that this would be particularly more so in projects carried out over short periods at various times related to the general use of different dispersants with time.

7 CONCLUSIONS

An investigation into the double hydrometer test method used for the identification of dispersive soils has highlighted differences that can be obtained on a single soil as a function of the variation in test procedures. This is due to the relatively ambiguous state of the test procedures resulting in different interpretations of the test methods, and consequently misleading results.

The double hydrometer test is a good example of misinterpretations due to ambiguities. Despite the test being an ASTM standard (ASTM D4221-99), many laboratories just duplicate the standard hydrometer analysis procedure (TMH1 and ASTM D422-63), which invariably produces incorrect results. The variability of the results obtained from the double hydrometer test appears to be the cause of many of the ambiguities and discrepancies in the classification systems studied during this research. The incorrect classification of the dispersiveness in the early stages of the investigations would influence the entire analysis process negatively.

Problems related to the double hydrometer tests pose the potential for misleading results since the test is associated with a number of different parameters in the rating systems. Inaccurate results from the double hydrometer test can significantly affect the correlation of the final rating, particularly when this test method is used as the reference methods for the preliminary classification of the dispersivity of soils (Gerber and Harmse, 1987; Bell and Maud, 1994; Walker, 1997).

It is therefore necessary to stress the importance of developing and following a standard protocol for the test. In order to reduce the variation/inconsistencies in results, it is essential that the test method is reviewed and the optimum procedure developed. The procedure should be simple and have as few ambiguities as possible so that no misinterpretations can occur.

8 REFERENCES

- ASTM International. 2007a. *Standard test method for dispersive characteristics of clay soil by double hydrometer*. ASTM D4221-99. ASTM Pennsylvania.
- ASTM International. 2007b. *Standard test method for particle-size analysis of soils*. ASTM D422-63. ASTM Pennsylvania.
- Bell, F.G. & Walker, J.H. 2000. A further examination of the nature of dispersive soils in Natal, South Africa. *Quarterly Journal of Eng. Geology & Hydrogeology* 33:187-199.
- British Standard (BSi). 1990. *Methods of test for soils for civil engineering purposes: Part 2: Classification tests*. British Standard 1377-2: 1990, British Standards Institute, London.
- Decker, R.S. & Dunnigan, L.P. 1977. Development and use of the Soil Conservation Service Dispersion Test. In: Sherard, J.L. & Decker, R.S. (Eds). *Proceedings symposium on dispersive clays, related piping and erosion in geotechnical projects*. ASTM Special Publication 623: 94-109.
- Elges, H.F.W.K. 1985. Problem Soils in South Africa- State of the Art. *The Civil Engineer in South Africa* 27(7): 347-353.
- Gerber, F.A. & Harmse, H.J. von M. 1987. Proposed procedure for identification of dispersive soils by chemical testing. *The Civil Engineer in South Africa*, 29: 397-399.
- Maharaj, A and Paige-Green, P. 2010a. The impact of inconsistencies in the interpretation of soil test results on the repeatable identification of dispersive soils. *Proceedings IAEG2010, Auckland, New Zealand*, September 2010.
- Maharaj, A. 2010b. Preliminary observations of shortcomings identified in standard tests for dispersive soils. *Proceedings IAEG2010, Auckland, New Zealand*, September 2010.
- Maharaj, A. 2011. The Use of the Crumb Test as a Preliminary Indicator of Dispersive Soils. *Proceedings 15th ARC on Soil Mech and Geotech Eng*, Maputo, Mozambique.
- Maharaj, A. 2012. Problems associated with the chemical analysis of dispersive soils. *Proceedings 2nd European Conference on Unsaturated Soils (E-UNSAT 2012)*. Naples, Italy.
- Middleton, H.E. 1930. The properties of soils which influence erosion. *U.S. Dept. Agri. Tech. Bull.* 178: 1-16.
- NITRR. 1986. *Standard methods of testing road construction materials- Technical methods for highways (TMH) 1*. NITRR. Pretoria. 21-25.
- Paige-Green, P. 2008. Dispersive and Erodible Soils – Fundamental differences. *SAIEG/ SAICE Problem Soils Conference, Midrand*, Nov 2008 pp 59-67.
- Volk, G. M. 1937. Method of determination of degree of dispersion of the clay fraction of soils. *Proceedings Soil Science Society of America* 2: 561-567.
- Walker, D.J.H. 1997. Dispersive soils in KwaZulu-Natal. Unpublished MSc thesis. University of Natal, Durban.

Correlation between deflections measurements on flexible pavements obtained under static and dynamic load techniques

Corrélation entre les déflexions de revêtements flexibles mesurées sous chargement statique et dynamique

Murillo Feo C.A.

Ph.D., Civil Engineering, Department of Civil and Agricultural Engineering. Universidad Nacional de Colombia, Av. NQS 45-03 Bogotá, Colombia

Bejarano Urrego L.E.

Civil Engineering, Universidad Nacional de Colombia, Av. NQS 45-03 Bogotá, Colombia

ABSTRACT: Over the last 40 years several nondestructive techniques have been developed for determining the structural capacity of flexible pavements as a function of the deflections produced by the application of a load. The techniques most used in Colombia to measure pavement deflections are the Falling Weight Deflectometer (FWD) and the Benkelman beam, the first one works under dynamic loading and the second device under static loading. However, for over 10 years, the use of devices under static loading has not been recommended by several design methodologies, including AASHTO, but these are still used widely in many countries, for this reason it was necessary to establish the correlation between the deflections obtained from FWD and Benkelman beam, specially on deteriorated pavement structures; for this purpose, it was selected a section of flexible pavement road with presence of different types of deterioration. The influence of type of deterioration, the temperature of the asphalt layer and the presence of nearby drainage structures and vegetation in the measurements were evaluated. The obtained results showed that both devices have high correlation, and it is possible to obtain FWD deflections as a function of Benkelman beam deflections.

RÉSUMÉ : Dans les quarante dernières années plusieurs techniques ont été utilisées pour déterminer la portance des chaussées en utilisant les bassins de déflexion générée par l'application d'une charge. Le déflectomètre FWD est devenu aujourd'hui l'appareil de référence international pour la détermination de la portance des chaussées. Cependant, en Colombie, la poutre Benkelman est la technique plus utilisée, bien que l'utilisation d'appareils sous charge statique n'ait pas été recommandée par plusieurs méthodologies. Par conséquent, il était nécessaire d'établir la corrélation entre les déflexions obtenues à partir de la poutre Benkelman et le FWD. Plusieurs tests ont été menés afin de réaliser la comparaison entre les déflexions obtenues par la charge statique ainsi que la charge dynamique. Les essais ont été réalisés sur chaussée souple avec différents types de détérioration. L'influence du type de déficience, la température, la végétation ont été pris en compte. Les résultats ont montré qu'il est possible d'établir une corrélation entre les deux appareils.

KEYWORDS: Non-destructive test, Benkelman beam, FWD, deflections, flexible pavement, structural number, resilient modulus, backcalculation.

1 INTRODUCTION

Most of the road network in Colombia has a significant level of deterioration and therefore requires major rehabilitation projects; in general, most of these rehabilitation activities involve a new asphalt layer on the original pavement structure; knowledge and analysis of structural capacity of the pavement is essential to perform a durable and economical rehabilitation design.

It is possible to quantify the structural capacity of the pavement by means of the structural number (SN), which, in this case, is obtained in function of the deflections generated on the surface of the pavement by a process of backcalculation. The most used equipment in Colombia for measuring deflections on pavement structures are the Falling Weight Deflectometer (FWD) which works under dynamic loading and the Benkelman beam which works under static loading.

The Benkelman beam was one of the first methods developed for measuring deflections on pavements, is economical, readily

available and has been widely used in the world, however, its performance is slow, has high degree of uncertainty taking data and mainly it operates under a static load which does not really represent the effects exerted by moving vehicles, presenting low reliability of results. On the other hand the FWD, although it is more expensive, has a high performance, is automated and operates under a dynamic load, this is the most efficient equipment and advanced technically exists to measure the deflections of a pavement structure simulating the action of a moving load.

Different associations like the AASHTO do not recommend the use of deflectometers under static load, but in several countries, including Colombia which presents damage in the most of the road network, these devices are still in use especially the Benkelman beam, not only for structural evaluation but also for design of pavement structures; this is due especially to difficult acquisition, unfamiliarity and cost of falling weight deflectometer.

Therefore, it is important to determine the degree of correlation between these two devices to be able to obtain FWD deflections as a function of Benkelman beam deflections.

2 DEFLECTOMETRY

The deflection of an asphalt pavement structure is the vertical displacement of the surface in response to application of an external load. When this load is applied on the surface, all layers are deflected, developing stress and strain in each layer, as shown in Figure 1.

The shape and dimension of the deflection basin covers important information about the structural characteristics of both the pavement and subgrade. Deflections measured towards the end of the basin reflect the condition of the subgrade, while the measurements taken in the center of load application reflect the condition of the surface layer. Figure 1 shows the evaluation of the pavement according to the length (L_o) and a maximum depth (D_o) of the basin.

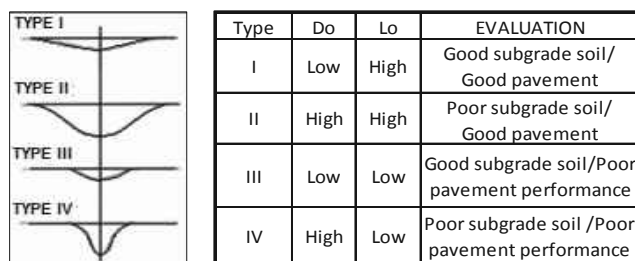


Figure 1. Characteristics of deflection basin

2.1 Admissible deflection value

The maximum values of deflection for design purposes estimated by Hveem (1995) are presented in Table 1.

Table 1. Maximum deflection values estimated by Hveem, 1995.

Type of pavement structure	Thickness (mm)	Maximum deflection value (microns)
Concrete pavement	200	300
Cement treated base	150	300
Asphalt pavement	100	425
Asphalt pavement on base course (plant mixed)	75	500
Asphalt pavement on base course (plant mixed)	50	625
Asphalt pavement on base course (In-situ)	25	925
Surface treatment pavement	13	1250

3 BACKCALCULATION METHODOLOGY

Table 2 shows the basics of back calculation methodology. The back calculation outputs are the modulus of elasticity of the pavement structure, effective structural number of the pavement layers, and subgrade soil resilient modulus.

Table 2. Representation of back calculation methodology

DIRECT CALCULATION	
E, D, μ	$\rightarrow d, \sigma, \epsilon$
BACKCALCULATION	
E, σ, ϵ	$\leftarrow d, D, \mu$

where E : elastic modulus of the materials, μ : Poisson's ratio, d : deflection of the pavement structure, σ : stress on each layer of the structure, ϵ : strain, and D : layers thickness. There are several backcalculation methodologies, most of them carried out in function of the deflections obtained only under dynamic load including the AASHTO and SASW methodology which was used for the analysis of the deflections obtained from FWD. (Murillo et al, 2009).

There exist few methodologies developed from deflections under static load, because these procedures do not simulate adequately the real effects of moving loads. Mario Hoffman, in 1975, presented a methodology based on the "Hogg Model", which was used in the present study for backcalculation procedure based on the deflections obtained using the Benkelman Beam.

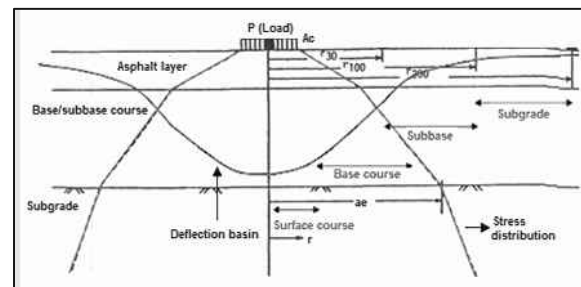
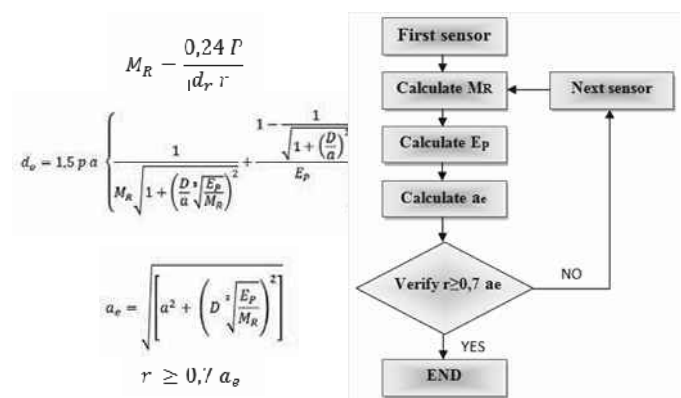


Figure 2. Algorithm backcalculation of M_R - AASHTO.

3.1. AASHTO Methodology (for FWD)

The guide for designing of pavement structures AASHTO establishes a procedure to calculate the resilient module (M_R) and thereafter the effective structural number (pavement structural capacity) as shown in the flowchart of Figure 2.



where M_R : resilient modulus of the subgrade (psi), P : applied load (pounds), r : distance from the center of the load (inch), d_r : deflection at a distance "r" from the center of the load (inch), a_c : radius of the bulb of pressure representing the subgrade level (inch), a : radius of the loading Ring (inch), D : thickness of the pavement structure above the subgrade (inch), E_p : equivalent modulus of all pavement layers above the subgrade (psi), d_o : deflection at the center of the load plate, adjusted to a temperature of 20 ° C (inch), p : pressure of load plate ($P/\pi a^2$). The effective structural number (SN_{eff}) is calculated based on the total thickness of the pavement and its effective modulus as shown below.

$$SN_{eff} = 0.0045 H_T (E_p)^{1/3} \quad (1)$$

where H_T : total thickness of the pavement structure (in), E_p : equivalent modulus of pavement structure above the subgrade (psi).

3.2. Hogg Model Methodology (for Viga Benke.)

In 1944, Hogg presented the mathematical solution of the model which is known by his name. This assumes that the pavement layers are characterized by a thin plate with a certain bending stiffness.

The subgrade is represented by an elastic, linear, homogeneous and isotropic medium (Figure 3). Hoffman, in 1977, presented the computerized solution of the model, which is summarized below in Figure 5.

Figure 3. Scheme of Hogg mode
Hoffman, Mario. 1985

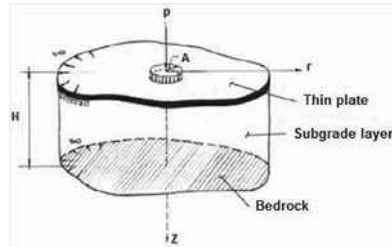


Figure 5. Methodology of Hogg Model for calculation of subgrade modulus

$$\Delta = \sqrt{\frac{P/2}{\pi p}} \rightarrow R_s = R \frac{A^c - R}{\left[A \left(\frac{D_0}{D_R} - 1\right)\right]^c - B} \rightarrow l_0 = Y R_s + \sqrt{(Y R_s)^2 - 4 A X R_s}$$

$$E_0 = \left(\frac{K * I * P}{l_0 * D_0}\right) \frac{S_0}{S} \leftarrow \frac{S_0}{S} - 1 - M \left(\frac{A}{l_0} - 0.1\right)$$

where A: radius of the contact circular footprint, P: load on the double rim (1/2 of the load on back axle. Example 80 KN / 2 = 40KN), p: inflation pressure, R: distance which deflection D_R is measured, D_0 : maximum deflection, D_R : deflection at a distance R, R_s : distance from the geometric center of the double rim along until obtaining the relation $D_R/D_0=0.5$, l_0 : characteristic length of the deflection basin, S_0 : stiffness for theoretical point load, S: stiffness of the pavement, E_0 : modulus of subgrade (kg/cm^2). I, K, M, X, Y, A, B, C: numerical coefficients developed for the model (see Ref 5).

The effective structural number (SN_{eff}) is calculated depending on the characteristic length and the modulus of subgrade as shown below:

$$SN_{eff} = 0,0364 l_0 (E_0)^{1/3} - 0,5$$

where E_0 : modulus of subgrade (MPa), l_0 : characteristic length (cm).

It is possible to calculate the equivalent modulus of pavement layers by means of Ullidtz proposal.

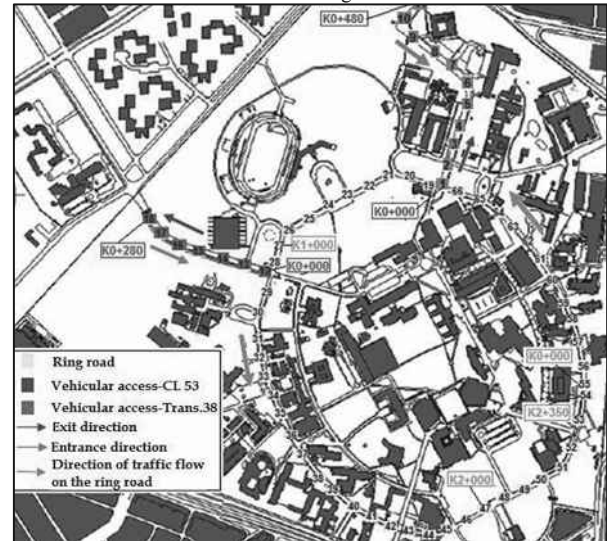
4 STUDY AREA

The study area is located on the campus of the Universidad Nacional de Colombia-Bogotá, it includes three sections of flexible pavement structure which are part of the road network of the university, these are: the main Ring road with a length of 2375 meters, vehicular access Calle 53 with length of 480 meters and vehicular access Transversal 38 with a length of 280 meters.

The deflection measurement was taken in 66 points, as shown in Figure 4. The area presented various types of damage including longitudinal failures, fatigue cracking (alligator cracking), interventions of asphalt patching, edge cracking and small potholes. Moreover vegetation influence is quite evident

negatively causing transverse and block cracks by the action of the root system.

Figure 4. Study area. Road network at Universidad Nacional de Colombia -Bogotá



5 MEASUREMENT DEVICES

5.1 Benkelman beam

Benkelman beam (Figure 5) is a device which operates on a simple lever arm principle, the unit consists of a rigid support beam, pivot, one or two measurement probe beams and dials indicator. It is a convenient and practical device for measuring deflection of flexible pavements under the action of wheel loads and works in conjunction with a suitable loaded vehicle (back axle loaded with 80 KN). The probe beam is placed between the dual tires of a test vehicle, and deflection is measured as the vehicle passes over the test area to beyond the end of the probe beam.



Figure 6. Benkelman beam (two-part probe beam)

The measurements were taken at 0 (l_0), 75, 150 and 300 cm, the end of the two probe beams were separated 25 cm each other, which means the readings were estimated at 0, 25, 75, 100, 150, 175, 300 and 325 cm from the center of load application. The temperature was taken with a manual thermometer.

4.2 FWD

The falling weight deflectometer, FWD works by dropping a controlled weight and transmitted to the pavement structure through a circular plate as shown in Figure 6. A set of geophones (deformation sensors) mounted radially from the center of the load plate measure the deflection in response of load pulse. The distribution of deflection sensors are shown in Table 3.

The FWD used in the tests was model JILS-20, programmed with a load pulse of 9000 pounds and three impact tests at every point. The equipment had an infrared sensor to measure the temperature of the asphalt layer.

Figure 6. FWD a) Equipment mounting b) Load cell and deflection sensors system



Table 1. FWD - Sensor configuration

FWD Geophones configuration									
FWD Sensor number	# 9	# 1 (do)	# 2	# 3	# 4	# 5	# 6	# 7	# 8
Offset from FWD load plate (cm)	-20	0	20	30	45	60	90	120	150

5 EXPERIMENTAL RESULTS

5.1 Deflection basins

Figure 7 shows representative results of deflection basins acquired in the study. The tendency of the deflection curves are deep and of short length, which means that the subgrade corresponds to a poor quality soil and deficient pavement performance. It was observed that the deflection basins obtained from the Benkelman beam are much deeper (12 to 232 mm⁻²) than those obtained using FWD (31,29 to 164,14 mm⁻²) giving more critical quality of the structure, although largely this is due to static charging system (Figure 8); that is, the analysis performed with the Benkelman Beam is more unfavorable, being that a deeper deflection curve indicates a deficient performance not only of the pavement structure but also of the subgrade.

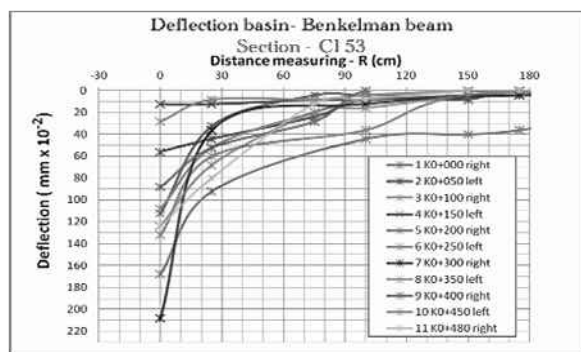


Figure 7. a) Deflection basin - Benkelman Beam, CL 53.

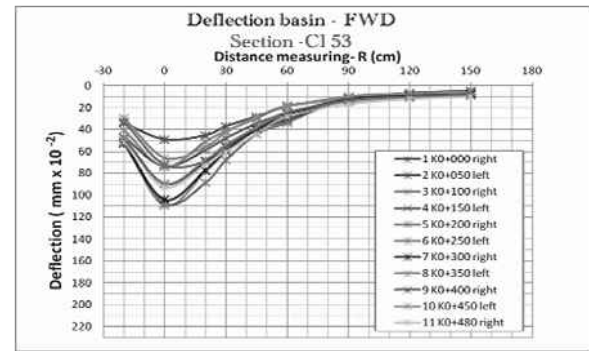


Figure 7. b) Deflection basin-FWD, CL 53.

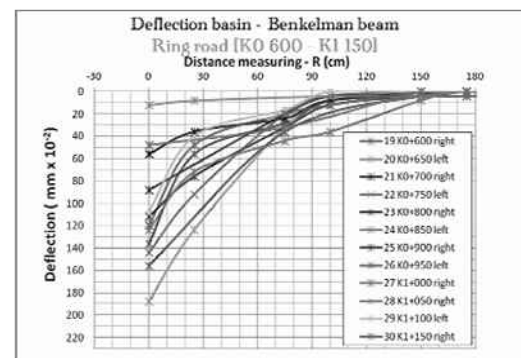


Figure 7c) Deflection basin-Benkelman Beam, Ring road (K0+600-K1+150).

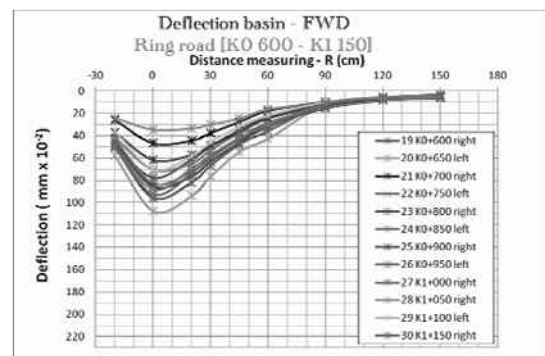


Figure 7d) Deflection basin-FWD, Ring road ((K0+600-K1+150))

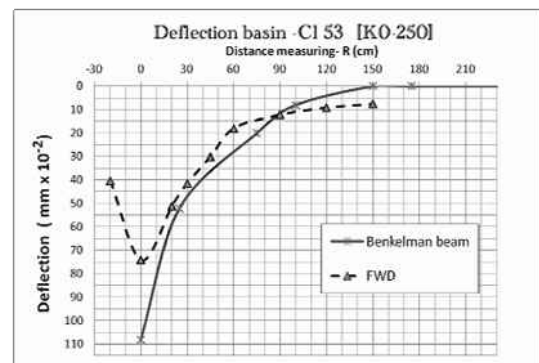


Figure 8. Typical deflection basin throughout the area, obtained using Benkelman beam and FWD

Figure 9 shows that presence of damages such as block and transverse cracking (mostly caused by tree roots) strongly affect the measurement resulting deflection values lower than average. The Benkelman beam technique is more susceptible to this cause.

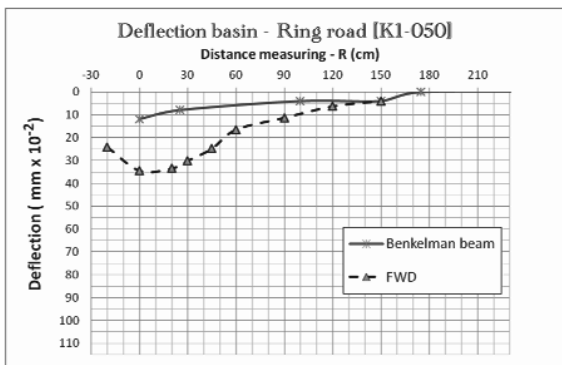


Figure 9. Deflection basin on areas affected by block cracking

In areas intervened with asphalt patching also the curve tends to be shallower than the average (Figure 10). In contrast, Figure 11 shows that on areas with longitudinal failures the deflection measurements resulted be higher than average, over 0.8 mm. The presence of other type of damage or drainage structures also caused high deflection measurements, over 1 mm.

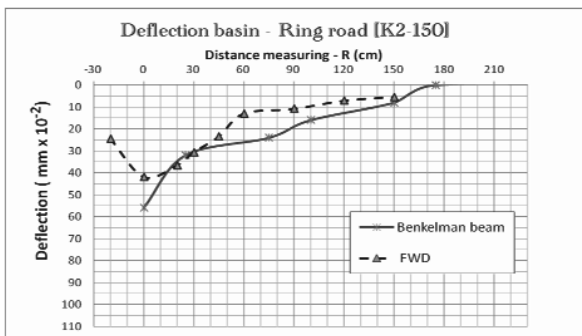


Figure 10. Typical deflection basin on areas with asphalt patching interventions

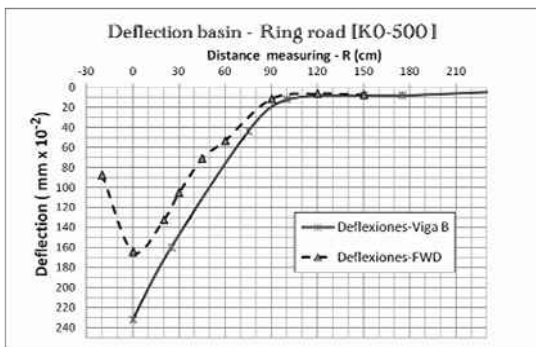


Figure 11. Typical deflection basin on areas with presence of longitudinal cracks

5.2 Maximum deflection value (D_0)

The pavement structure of the test roads has an asphalt layer thickness of 100 mm on average, according to Table 1 the permissible maximum deflection value corresponds to 42,5 mm². Figure 14 shows the maximum deflection profile in the Ring road and the permissible value.

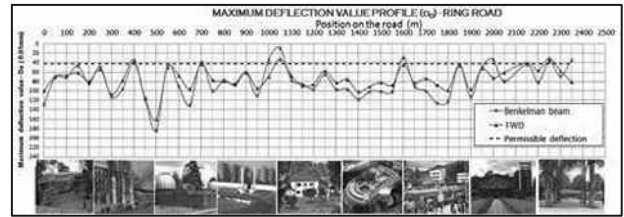


Figure 12. Maximum deflection value profile (D_0) - Ring road.

Figure 12 clearly shows deflections obtained from both devices follow the same trend. Both deflection profiles obtained using the Benkelman and FWD are outside the range of the maximum allowable deflection. The deflection values furthest from the trend correspond to test points affected by transverse cracking (usually caused by tree roots), mainly those measurements values obtained with the Benkelman beam.

Table 4. Maximum deflection - average per section

MAXIMUM DEFLECTION NORMALIZED TO 20°C			
SECTION	Position on the road	Benkel. Beam 0,01 mm	FWD 0,01 mm
Acceso CI 53	[K0+000 - K0+480]	78.46	72.43
Acceso Trans. 40	[K0+000 - K0+280]	96.44	77.50
Anillo vial	[K0+600 - K1+150]	78.96	71.40
Anillo vial	[K1+200 - K1+750]	93.85	78.93
Anillo vial	[K1+800 - K2+350]	71.53	59.55
Anillo vial	[K0+000 - K0+550]	87.54	81.02

The average of maximum deflection values (D_0) is around 0.8 mm, which means a pavement structure of low stiffness and subgrade of low-bearing capacity. As shown in Table 4, the section in best condition is the Ring road [K1+800 – K2+350].

5.3 Comparison between Benkelman beam and FWD measurements

The correlation between the deflection data obtained from FWD and Benkelman beam is shown in Figure 15. Equations (3) and (4) were obtained in order to convert Benkelman beam deflections to FWD deflections taking into account falling weight deflectometers simulate better the real effects made by moving loads.

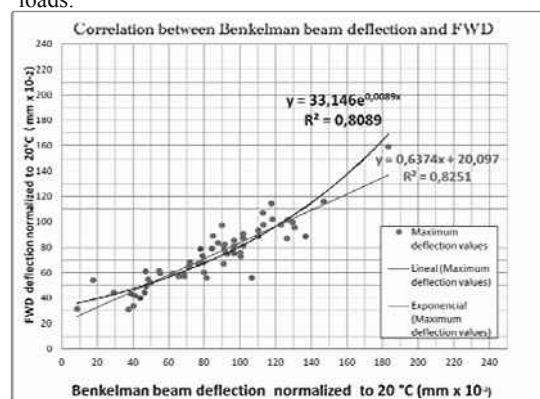


Figure 12. Correlation between deflections obtained with Benkelman beam and FWD

$$FWD = 33,146 e^{0,0089B} \quad (3)$$

$$FWD = 0,6374 B + 20,097 \quad (4)$$

where FWD: maximum deflection value (D_0) obtained from FWD normalized to a standard temperature of 20°C (68°F), B =

maximum deflection value (Do) obtained from Benkelman beam normalized to a standard temperature of 20°C (68°F). The correlation equations (3) and (4) are optimal for deflections measurements taken on flexible pavement structures of low stiffness, it is possible to use them in deteriorated structures, preferably without presence neither transverse nor block cracking (especially that caused by surrounding vegetation). These equations are recommended for deflections between 0.3 and 1.8 millimeters.

5.4 Subgrade soil modulus

Both subgrade soil resilient modulus and effective structural numbers obtained using AASHTO methodology prove to be higher than those obtained using the methodology of the Hogg Model; that is the results obtained with the Benkelman beam still are more unfavorable just as in the analysis of the deflection basins.

Table 5. Subgrade soil modulus obtained in each section

SECTION	Position on the road	SUBGRADE MODULUS (Mpa)			
		BENK. BEAM	c (%)	FWD	c (%)
Acceso CI 53	[K0+000 - K0+480]	42.2	59.8	77.9	14.2
Acceso Trans. 40	[K0+000 - K0+280]	38.3	39.3	63.4	29.5
Anillo vial	[K0+600 - K1+150]	44.4	25.0	55.1	20.0
Anillo vial	[K1+200 - K1+750]	35.1	41.0	62.3	26.3
Anillo vial	[K1+800 - K2+350]	42.5	56.6	59.8	17.4
Anillo vial	[K0+000 - K0+550]	40.7	33.4	50.9	26.0
AVERAGE		40.5	42.5	61.6	22.2
c: Variation coefficient					

As shown in Table 5, on average in all test area, the modulus of subgrade were 40,5MPa (5880 psi) obtained using the Benkelman Beam and 61,6 MPa (8931 psi) with FWD. These subgrade modulus values correspond to a fine-grained soft soil such as fat clays and silts.

5.5 Effective structural number (S_{Neff})

On average, the effective structural number (S_{Neff}) was 1,6 obtained using the methodology of the Hogg Model for Benkelman beam measurements and 2,4 obtained using AASHTO methodology for FWD measurements; it is demonstrated that the structural assessment carried out with the Benkelman beam is more critical than the one carried out with the FWD, this is because the deflections obtained using Benkelman beam are higher for being taken under static load.

As shown in Table 6, these effective structural number values are very low, representing the low pavement structural capacity and the need for implementation of a rehabilitation project.

Table 6. Effective structural number obtained with both techniques

SECTION	Position on the road	EFFECTIVE STRUCTURAL NUMBER, S _{Neff}			
		BENK. BEAM	c (%)	FWD	c (%)
Acceso CI 53	[K0+000 - K0+480]	1.6	104.2	3.0	11.9
Acceso Trans. 40	[K0+000 - K0+280]	1.0	76.7	2.3	11.9
Anillo vial	[K0+600 - K1+150]	2.0	44.3	2.3	11.7
Anillo vial	[K1+200 - K1+750]	1.6	51.7	2.4	20.0
Anillo vial	[K1+800 - K2+350]	1.6	87.9	2.2	11.3
Anillo vial	[K0+000 - K0+550]	1.9	39.4	2.3	16.0
AVERAGE		1.6	67.4	2.4	13.8

6 CONCLUSIONS

-Deflections under static load are higher than those generated by dynamic load; this is due to longer duration of load application.

Therefore, the results obtained from deflection values under static load do not represent accurately the effects made by moving loads (moving vehicles), so the structural analysis made using static loading equipment may generate higher costs in rehabilitation projects.

-Most deflection basins obtained are deep and short extension, which means presence of poor subgrade soils and low performance of pavement layers (low structural pavement capacity).

-Backcalculation of subgrade soil modulus is a simple non-destructive procedure, more practical and faster than calculation by laboratory tests; and more reliable than using correlations based on other parameters as CBR.

-Exist a correlation ($R^2=0,82$) between the deflections obtained from Benkelman beam and FWD. The estimated equations are recommended to use in structural analysis of low stiffness pavement. It is possible to use them on deteriorated structures, preferably without presence neither transverse nor block cracking (especially that caused by surrounding vegetation).

-It was observed that presence of damages such as block and transverse cracking (mostly produced by the effect of tree roots) as well as asphalt patching interventions markedly influence the measurement, resulting deflection values lower than average. In contrast, presence of longitudinal failures as well as drainage structures nearby caused deflection measurements higher than average. It is not recommended to take deflections measurements on these areas.

-The standard deviation and coefficient of variation of data obtained using Benkelman beam are higher than those obtained using FWD.

7 REFERENCES

1. AASHTO, 1993. American Association of State Highway and Transportation Officials. Guide for Design of Pavement Structures.
2. Beena Sukumara N. 2002. Suitability of using California bearing ratio test to predict resilient modulus. Rowan University.
3. Crespo del Río, Ramón. Cálculo de refuerzo de firmes según el método AASHTO. AEPO, Ingenieros Consultores S.A.
4. Crespo del Río, Ramón. 2004. Valores de la deflexión según el equipo de medida. AEPO, Ingenieros Consultores S.A.
5. Hoffman, Mario. 1985. Estudio de evaluación estructural de pavimentos basados en la interpretación de curvas de deflexiones (ensayos no destructivos). Louis Berger Internacional Inc.
6. INVIAS - Instituto Nacional de Vías. Guía metodológica para el diseño de obras de rehabilitación de pavimentos asfálticos de carreteras. Segunda edición. Colombia, 2008.
7. Murillo, C.; Thorel, L. & B. Caicedo. A Miniature Falling Weight device for Non Intrusive Characterization of Soils in the Centrifuge. Geotechnical Testing Journal. ISSN: 0149-6115 (July 2009).
8. Petersen Acevedo, Miguel. Relación entre módulo resiliente determinado mediante deflectómetro de impacto y el de laboratorio. Universidad Técnica Federico Santa María. Valparaíso, Chile.
9. Sturm, H. and Skinner, Michael. Development of a pavement management and prioritization framework for three active municipal landfills. Pavement Performance Case Studies Session of the 2012 Conference of the Transportation Association of Canada Fredericton, New Brunswick, 2012.

Comparison of permeability testing methods

Comparaison des différentes méthodes sur les tests de perméabilité

Nagy L., Tabácks A., Huszák T., Mahler A., Varga G.
Budapest University of Technology and Economics

ABSTRACT: Coefficient of permeability is known as the most variable soil property. Its value can vary over an order of magnitude even in case of relatively homogenous layers. So determining this value is a complicated, complex engineering task. There is a vast number of laboratory and in situ tests to determine the permeability coefficient. Each method has its own advantages, drawbacks and limitations, so different methods should be preferred in different situations. The permeability coefficients of a sandy silt and a silty sand layer have been determined by means of Khafagi probe, Menard probe, water filtration method, constant head laboratory test and falling head laboratory test. The permeability coefficients have also been estimated by the equation proposed by Hazen (1895). The measured values are summarized and evaluated in the paper. Special emphasis is given on the reliability of the methods, on the capability to sense the layer boundaries and their estimation accuracy.

RÉSUMÉ : Le coefficient de perméabilité est connu comme étant la propriété du sol la plus variable. Sa valeur peut varier, même dans le cas de couches relativement homogènes. Ainsi, la détermination de cette valeur est une question difficile, une tâche d'ingénierie complexe. Il existe de nombreux laboratoires et des tests *in situ* pour déterminer le coefficient de perméabilité. Chaque méthode a ses avantages, ses inconvénients et ses limites, ainsi certaines méthodes doivent être privilégiées en fonction du contexte. Les coefficients de perméabilité d'un limon sableux et d'une couche de sable silteux ont été déterminés par la méthode Khafagi, par l'essai pressiométrique Ménard, par la méthode constante et tomber test de perméabilité à la tête. Les coefficients de perméabilité ont également été estimés par l'équation proposée par Hazen (1895). Les valeurs mesurées sont résumées et évaluées dans le document. L'accent est mis sur la fiabilité des méthodes, sur la faculté de détecter les limites des couches et sur la précision de leur estimation.

KEYWORDS: coefficient of permeability, laboratory test, in situ test

1 INTRODUCTION

Coefficient of permeability (also known as hydraulic conductivity, denoted by 'k') is a highly variable soil property. Previous studies have shown that its coefficient of variation can be as high as 240 % (Lumb, 1966., Uzielli, 2008., Mlynarek, 2010.). Additionally the chosen testing method has also high influence on measured results.

The two main factors that determine the order of magnitude of the permeability coefficient are: grain size and cleavage (secondary interstices). These two properties can already have significant spatial variability, but other influencing factors make the determination of permeability coefficient even more complex. The impact of the factors listed below is inferior, but still not insignificant:

- grain shape and orientation,
- quantity and connection of interstices,
- uniformity coefficient,
- water content and saturation conditions before seepage begins,
- the properties of the passing liquid (water),
- hydraulic conditions (hydraulic gradient, Reynolds number etc.),
- transient phenomena (migration, wash-out and wash-in of grains).

Section S3 of Annex S to EUROCODE 7: Geotechnical Design Standard highlights the role of saturation, which may cause a change of up to three orders of magnitude in the coefficient of permeability of certain soil types.

It is fair to say therefore that the coefficient of permeability of soils can depend on a large number of factors of different

character, which is why general relationships (formulas or graphs) based on a few simple quantities are not expected to provide accurate k values. Based on these considerations, it is not a good practice to use values taken from tables of universal validity. No one can guarantee, for instance, that soils with the correlation feature $I_p=30\%$ have identical coefficients of permeability at all sites.

1.1 Background

There is no method specified either as a Hungarian Standard or in a Technical Guideline for calculating the coefficient of permeability. Coefficient of permeability values can be determined by on site or laboratory measurements or indirectly from empirical correlations based on grain size distribution. Small as it is, even a country such as Hungary has failed to come to a common understanding about the test.

Kézdi (1976) expresses a preference for laboratory tests for determining the coefficient of permeability and indeed the following laboratory methods are available for determining the value of k:

- by constant water head test
- by falling water head test,
- by capillary permeability test, and
- from a consolidation test.

Rózsa (1977) rejects the laboratory method and recommends pumping from a well to determine the coefficient of permeability:

"The coefficient of permeability is one of the physical properties that cannot be determined at the required accuracy using laboratory methods. Frequently, a 10-50 fold accuracy of

the k factor would be sufficient, but even that is beyond the scope of laboratory tests."

The handbook of groundwork prefers pumping tests performed on site, but fails to mention what to do in layers above the ground water tables. Kovács (1972) takes a different view and recommends using the grain distribution curve.

"Based on a brief description and a critical analysis of laboratory and on site tests, it is underlined once more that formula based calculation should normally be recommended as the method for determining the coefficient of permeability, not only because this is the simplest technique but also because its reliability reaches and in most cases even surpasses that of other methods. In-laboratory and on-site measurements are justified only in case it is our intention to describe a unique stratification property of a layer. That would require laboratory analysis of undisturbed drill cores, advanced percolation tests or pumping tests using several observation wells."

The coefficient of permeability of a rather large basket of soil types, i.e. ones not characterised by grain size distribution, is left undetermined this way. The following remarks allow us to conclude that there the industry lacks consensus on how to determine the coefficient of permeability value.

Section S3 "Evaluation of test results" of the annex to the EUROCODE 7 standard specifies the following requirements for evaluating test findings:

„There are four widely used methods to determine the coefficient of permeability (hydraulic conductivity):

- field tests, such as pumping and borehole permeability tests;
- empirical correlations with grain size distribution;
- evaluation from an oedometer test;
- permeability tests on soil specimens in the laboratory."

So it can be concluded that there are many laboratory and in situ methods to obtain the coefficient of permeability. However there is no universally applicable method; each method is valid within certain limits, which we need to identify to render investigations easy to target and plan.

Measurements have tended to take the form of site investigation as soil mechanics have developed in the past 15-20 years. That way multiple damage to samples can be avoided and results will better reflect local conditions. It is internationally accepted that local investigation provides more accurate site specific values. The question arises whether or not this statement also holds for the determination of coefficient of permeability values.

1.2 Aims of study

We set out to determine the coefficient of permeability of transitional and fine grained soils (ranging from fine sand through sand meal and miry sand to silt). The following boundary parameters were assumed for the purposes of our test series:

- We selected methods whose range of validity matched in principle the type soil selected for the tests.
- Homogeneous isotropic strata were assumed for the purposes of the test despite the likelihood of periodic sedimentation of coarser and finer grains during layer formation, and an apparently homogeneous layer may be composed of a network of more conductive and more watertight lenses seams.
- Potential filtration anomalies at layer boundaries are ignored.
- The increased conductivity due to atmospheric effects and human intervention of a layer of top soil, which can be up to 0.6-0.8 m thick, is also neglected.

- Most tests determine the coefficient of permeability on a relatively small sample of soil. It would, however, be a mistake to generalise the value achieved that way for the whole layer represented by the sample.

2 TESTING METHODS

The following methods were used to measure the coefficient of permeability of fine grained and transitional soils on site:

- Horizontal permeability can be measured with a Menard probe inserted into a vertical bore hole. The radial infiltration of water into the soil is facilitated by packers and by the injection of water below and above the measurement section.
- Water absorption test across a trickling head lowered through a Khafagi probe to determine the coefficient of permeability. Soil conditions are taken into account for the purposes of dimensioning the trickling head to be used and the calibrated container.
- Depending on ground water level, one or more boreholes may be lowered for pumping or water absorption. Soil conditions must be taken into account for determining the layout and dimension of the boreholes. Serious errors may occur if the liner fails to connect properly to the hole bottom, as water will not only trickle into the soil across the bottom but along a sleeve of unknown length.

Equipment of constant or falling water head may be used in laboratory measurements depending on the coefficient of permeability.

We have also determined the value of coefficient of permeability indirectly (by empirical correlation based on grain size distribution) to compare and verify local measurements. Different authors have identified different relationships to be used in the indirect method of calculation and have partially combined these methods with a variety of status descriptors. A shared feature of these methods involves plotting a grain size distribution curve typically identifying the grain diameter (d_{10}) associated with ten mass percentages passing and this value is normally on the power of two. This paper presents the results received from calculations using formula (see Figure 2).

3 TEST LOCATION, SOIL TYPES

Tests were performed at five locations, but this paper only covers the findings testing section 54+260 of the left bank of the Danube near Ráckeve. The tests were performed on the protected side 10 meters from the toe of the flood control dyke. Exploratory drilling identified the following order of layers:

- the upper layer from 0.0 to 3.4 m contains yellow and yellowish grey silt with silty sand of low water content ($7\% < w < 14\%$) and with moist density at around $\rho = 1.76 \text{ g/cm}^3$. The grain size distribution curve shows that the fine content makes up 80-90% of soil particles. The coefficient of uniformity vary between $C_u = 8.6-12.3$ (see figure 1).
- the layer from 3.4 to 5.0 m contains sand with grey silt. The water content of this well graded layer is 20% on average. Wet bulk density is around $\rho = 1.86 \text{ g/cm}^3$. The layer is understood to be much looser than the one above. The examination of grain size distribution suggests that the sand fraction makes up 70-75% with silt at 25-30% (see figure 1). The coefficient of uniformity is at $C_u = 30-33$.

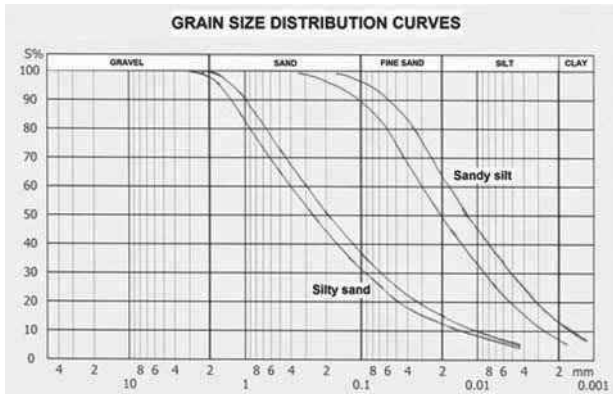


Figure 1. Grain size distribution curve envelopes of the tested soils

4 TEST RESULTS

The results of the performed permeability tests are summarized in Figure 2.

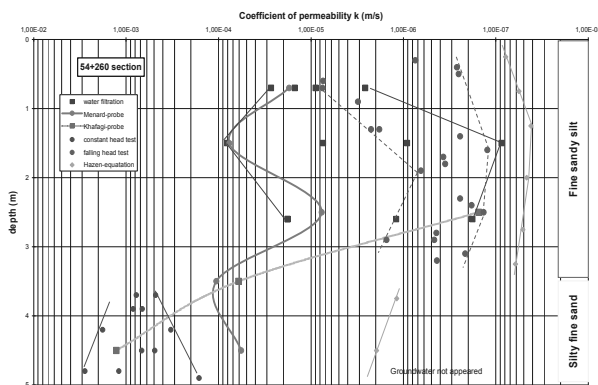


Figure 2. Measured permeability coefficients

The following conclusions can be drawn from a practical comparison of various measurement methods on the basis of determining the applicability of the methods (see Figure 2).

- The majority of measurement methods identified the change of soil along the two sides of the formation boundary.
- The results showed higher than expected scatter and the errors seem to be regular in character and attributable to the method of measurement.
- Each method is likely to have a relative error of one order of magnitude, disregarding the examination of samples taken from the vicinity of the surface.
- The error of water absorption was especially large: three orders of magnitude at the depth of 1.7 meters and only two at 2.6 meters. This method seems to lend itself to erroneous measurements.
- The scope of validity of the test performed with falling water head test did not cover the bottom layer, as water flow through the specimen rapidly. As a result equipment maintaining constant water head test had to be used to examine the lower layer.
- Regardless of the type of soil, measurements with the Menard probe returned values varying between $k = 10^{-4}$ - 10^{-5} m/s and it seems to be insensitive to changes of soil. But it must be also noted that inappropriate device might have caused the experienced error. Unfortunately we couldn't repeat the test to reveal the reason of this trend, so these results have not been analyzed in this study.
- The trend of the permeability coefficient determined with the Khafagi probe and by calculations based on the grain size distribution curve is identical to the findings of laboratory measurements. Each of the three methods

sensed the rise of the coefficient of permeability at the depth of 3.4 meters.

- Compared to the coefficient of permeability findings of local measurements and laboratory tests, calculations from the grain distribution curve produced slight and more pronounced overestimations for soils of poor and better permeability, respectively.

The tests performed in the 54+260 km section and at other locations demonstrated that Khafagi probes lowered by pumping (or in receiver probes) are best suited to routine tests. The advantages are laid out below:

- potential to use both above and below ground water,
- can be lowered with any type of probe,
- simple device,
- relatively low measurement costs,
- no complicated measurement and evaluation methods,
- relatively rapid measurement,
- measurement length is adjustable to soil,
- measurement findings include both horizontal and vertical coefficients of permeability.

Naturally, the studies described above still keep us in the dark about the exact value of the coefficient of permeability at the location we examined, but we have measured approximations, which we know deviate from true values in a certain direction due to measurement error.

5 CONCLUSIONS

The theories established on the basis of various studies offer a solution for determining the coefficient of permeability for instance on the basis of the grain size distribution curve or for evaluating the findings recorded by the Khafagi probe or the Menard probe, but we can still entertain doubts about whether or not the values determined that way are appropriate, the measurements replicate nature or reveal the k value characteristic of natural permeation. We have conducted local and laboratory measurements of the coefficient of permeability, soil mechanics explorations and identification studied to resolve these questions.

It is necessary to increase the accuracy of measuring the k factor, because the relative error of calculations, processing and geometric dimensioning, etc. is orders of magnitude smaller than what we can determine for the coefficient of permeability. Greater accuracy is required because uncertain measurements lead to unjustified over-dimensioning at times and to running unnecessarily large risks at other times. It is extremely important to know the true value of the k factor to avoid that. There are several methods for the in-situ determination of the coefficient of permeability. This study and this paper aimed at a practical comparison of the results of the different methods. Figure 3 presents the range of validity of each measured value and method of calculation based on measurement findings and processing the literature. The same figure shows the classification of soil types by permeability for the purposes of various technical interventions.

The studies allow us to establish two important findings:

- There is no universally applicable method and each method of determining the coefficient of permeability has its own range of validity. This principle was adopted in the Hungarian Standard 15295, which was issued in 1999.
- If there the coefficient of permeability of the layers is no material different, it is not practical to use different measurement methods in a single borehole and one must not use different measurement methods at different depths within one and the same layer, because swapping methods may result in measured findings showing

greater differences than the ones actually present in the soil.

Based on the foregoing, one must try to ensure that the value of the coefficient of permeability is determined at least to ± 20 -30% accuracy.

When evaluating measured values, one must bear in mind that no measurement is perfect and each method of measurement carries an error, hence each will distort measured values one way or another. Drilling (or pressing the probe into position) may destroy the structure of the soil in in-situ measurements. When drilling in floating sand or sand meal, fresh water should always be replenished and the bit has to be withdrawn slowly enough to prevent the soil from breaking into the borehole due to the emanating piston effect. When performing laboratory measurements of an undisturbed sample, taking samples with a chipped sampler vessel is a dangerous source of errors as we will measure the volume of water flowing past the mantle. When evaluating measured values, one must pay attention to the direction of change in the measured value triggered by the measurement error. A chipped sampler will lead to measuring a larger value of permeability, while the soil compacted by the probe will reduce the measured value.

We cannot say what the coefficient of permeability of the soil is, we can only state the value we received by performing a certain type of measurement. And even if we do that, we need to take into account the disturbance of the soil sample, the errors of our method, etc. One must examine the method used to produce the result as well as the distribution of the result and the error lodged in the examination.

Presenting the above measured values helps us orientate ourselves among the mysteries of the coefficient of permeability. The poorest conclusion one can draw is that the value of the coefficient of permeability is uncertain and therefore there is no need to determine it. One should not follow that road: on the contrary, one should underpin soil mechanics opinions by sound measurements.

6 REFERENCES

- Casagrande A. 1938. Seepage through dams, J.N. Engl. Water Works Association, L1(2), pp 131 -172.
- Darcy H. 1856. *Les fontaines publiques de la ville de Dijon*. Dalmont, Paris.
- Galli L. 1968. Planning of dikes, (in Hungarian), VIZITERV tervezési segédletek, 11. kötet, Vízügyi Tervező Vállalat.
- Hazen A. 1895. The filtration of public water-supplies, New York.
- Kézdi, Á. 1976. Soil mechanics, (in Hungarian), Tankönyvkiadó, Budapest
- Kovács Gy. 1972. Seepage hydraulics, (in Hungarian), Akadémiai Kiadó, Budapest
- Lumb P. 1966. The variability of natural soils. Canadian Geotechnical Journal Nr 3.
- Mlynarek, Z. 2010. Quality of in situ and laboratory tests contribution to risk management.
- Nagy L. 200. Geotechnical questions of flood dikes, (in Hungarian). Vízügyi Közlemények, (2000), LXXXII. évf., Vol. 1., pp. 121-146.
- Nagy L. 2007. Flood defence against dike slope slide, (in Hungarian). Innova-Print Kft. nyomda, ISBN 978-963-87073-9-0.
- Nagy L. 2008. A comparative study of the permeability (in Hungarian), Kézdi memorial conference, (2008), pp. 97-106, Budapest, ISBN 978-963-06-5725-9.
- Rózsa L. (editor) 1977. Foundation engineering handbook (in Hungarian), Műszaki kiadó.
- Uzielli M. 2008 Statistical analysis of geotechnical data. Proceedings of Geotechnical and Geophysical Site Characterization Conference, Balkema, Taiwan, Taylor, Frances 2008.

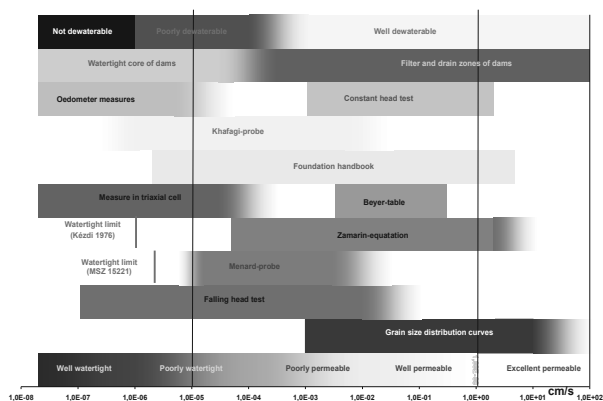


Figure 3. Soil permeability characteristics and the ranges of determining the coefficient of permeability

Oscillation of Acceleration Accompanying Shear Band and Subsequent Time-Dependent Behavior in Overconsolidated Clay under Undrained Plane-Strain Conditions

Oscillation de l'accélération accompagnant la formation de bandes de cisaillement et comportement dépendant du temps dans une argile surconsolidée en déformations planes et conditions non drainées.

Noda T.
International Member, Nagoya University, Japan

Xu B.
Student Member, Nagoya University, Japan

ABSTRACT: In this paper, a compression test on a rectangular overconsolidated clay specimen under plane strain, constant cell pressure, and undrained conditions was simulated using a soil-water coupled finite deformation analysis code taking inertial loads into consideration, and the following results were obtained: 1) In order to reproduce uniform deformation in a perfect specimen with no initial geometric imperfection, it is necessary to have conditions in which the “permeability coefficient is zero” so that pore water migration does not virtually occur, as well as to apply an initial distribution of velocity, acceleration and pore water pressure to the interior as well as the boundary in accordance with the vertical velocity of the top/bottom ends so as to satisfy the undrained conditions; 2) In specimens with initial geometric imperfection set to a very small amplitude primary cosine curve, when a vertically symmetric vertical displacement was applied under displacement control from static conditions, shear banding occurred in the specimen, and accelerations were generated from that part; 3) After the load became unstable, by changing to load control and maintaining the load constant, creep behavior under undrained conditions was exhibited by the specimen.

RÉSUMÉ: Un essai de compression sur une éprouvette rectangulaire d'argile surconsolidée, en déformations planes et conditions non drainées est modélisé en utilisant une analyse couplée en grandes déformations, prenant en compte les forces d'inertie. Cette analyse a apporté les résultats suivants: 1) En vue de reproduire une déformation uniforme dans une éprouvette parfaite sans défaut géométrique initial, il est nécessaire d'avoir des conditions dans lesquelles le « coefficient de perméabilité est égal à zéro » de telle sorte qu'il n'y a aucun écoulement d'eau, ainsi que d'appliquer une distribution initiale de vitesses, d'accélération et de pressions interstitielles à l'intérieur ainsi qu'aux limites de l'éprouvette en accord avec la vitesse verticale de la base et du haut de celle-ci en vue de satisfaire les conditions de non drainage ; 2) Pour des éprouvettes présentant un défaut géométrique initial de très faible amplitude, quand un déplacement vertical symétrique est appliqué à déplacement contrôlé, des bandes de cisaillement se développent dans l'éprouvette et des accélérations se développent à partir de là ; 3) Après que le chargement soit devenu instable, en changeant pour un chargement contrôlé en effort et en maintenant l'effort constant, l'éprouvette présente un comportement de fluage en conditions non drainées..

KEYWORDS: imperfection, shear band, acceleration oscillation.

MOTS CLES : imperfection, bande de cisaillement, oscillation de l'accélération

1 INTRODUCTION

In bifurcation analysis or shear strain localization analysis of soil specimens, calculations are normally carried out on a uniform deformation field assuming quasi-static conditions that ignore the effect of inertia forces (Ikeda et al. 2003, Wan et al. 1990). The authors have developed the GEOASIA code, which is capable of performing analyses without distinction dynamic/static problems, extending the static soil-water coupled finite deformation analysis that has been carried out to date (Asaoka et al. 2007, Noda et al. 2008). In this paper, the strain localization problem of rectangular specimens of saturated overconsolidated clay under undrained and constant side pressure conditions was addressed, checking the conventional solutions that were obtained under quasi-static assumptions and taking the problem to be a dynamic problem that considers inertial term. This paper introduces several phenomena such as the generation of accelerations and time-dependent behavior (creep) associated with the occurrence of shear banding as newly found calculation results using GEOASIA.

2 CALCULATION CONDITIONS

Fig. 1 shows the finite element mesh and boundary conditions used in the calculations. The calculation assumed 2-dimensional plane strain conditions and a saturated rectangular specimen

with 3.5 cm wide and 8 cm high in the isotropic consolidation state. The specimen was a fully remolded overconsolidated clay. As the elasto-plastic constitutive model in the analysis, the SYS Cam-clay model (Asaoka et al. 2002), the degree of structure was $R^*=1.0$ and there was no effect of the initial anisotropy and induced anisotropy ($\zeta_0=0$, $br=0$).

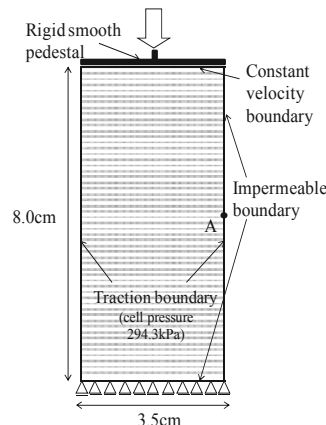


Fig. 1 Finite element mesh and boundary conditions

Table 1 shows the soil elasto-plastic constants and the set of initial values used in the calculations. Apart from the

overconsolidation degradation index m , which controls the overconsolidation behavior, the elasto-plastic constants used were the same values as used by Asaoka et al. 1994. Vertical constant loading rate was applied on the top surface. The boundary conditions were assumed to be constant lateral pressure and undrained conditions, with no friction at the top and bottom and with complete freedom of movement in the horizontal direction. Calculation under these conditions cannot be realized with quasi-static analysis that ignores inertial forces. The mesh subdivision was 70 elements laterally by 160 elements vertically.

Table 1 Specimen elasto-plastic constants and initial values

<i>Elasto-plastic parameters</i>	
Critical state index M	1.55
NCL intercept N	2.0
Compression index $\lambda, \sim$	0.108
Swelling index $\kappa, \sim$	0.025
Poisson's ratio ν	0.3
<i>Evolution parameters</i>	
Degradation index of OC m	0.2
<i>Initial conditions</i>	
Specific volume v_0	1.747
Stress ratio η_0	0.0
Degree of structure $1/R_0 \square$	1.0
Degree of overconsolidation $1/R_0$	5.0
Degree of anisotropy ζ_0	0.0
Soil particle density ρ_s (g/cm ³)	2.65
Permeability coefficient k (cm/s)	3.7×10^{-8}

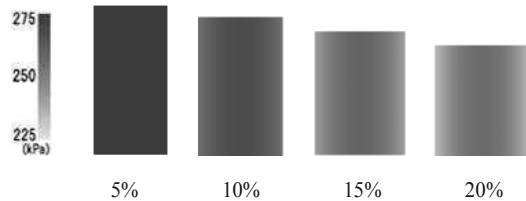


Fig. 2 Change in specimen deformation and pore water pressure distribution under uniform deformation

3 REPRODUCTION OF A UNIFORM FIELD USING THE GEOASIA ANALYSIS CODE

Consider undrained compression deformation of a perfectly rectangular specimen with no initial material or geometric imperfections. The top and bottom of the specimen were free in the horizontal direction, and after fixing the bottom in the vertical direction, a constant uniform vertical displacement was applied to the top. In accordance with the u - p Formulation, when solving without ignoring inertial forces, a uniform deformation field satisfying element-wise undrained conditions in a rectangular specimen can only be realized when the permeability coefficient is $k = 0$, although the theoretical proof (Noda et al. 2013) is omitted. In order to realize a uniform deformation field using this analysis code, it is necessary to set a velocity distribution that is proportional to the height and the velocity applied to the top of the specimen (not all are zero), an acceleration distribution to maintain the rectangular shape, and pore water pressures that exhibit a parabolic distribution in the horizontal direction as initial conditions in addition to the coordinates of the finite element nodes at the boundary and interior (Noda et al. 2013).

In this section, for the case with $k = 0$ first, a constant vertical velocity of 10^3 cm/s was applied on the top to illustrate the calculation results when a uniform deformation field is achieved. Fig. 2 shows the change in specimen deformation and the parabolic pore water pressure distribution, and Fig. 3 shows the horizontal and vertical component of acceleration generated in the center of the right side surface of

the specimen. However, when the theoretical initial values are set for the velocity and acceleration as initial conditions, small vibrations occur around time $t=0$. Therefore, for no vibration the initial velocities and accelerations are set slightly smaller than the theoretical values. See Noda et al. 2013 for the method of obtaining the reduced values.

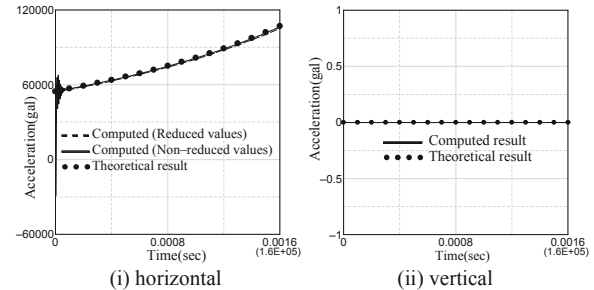


Fig. 3 Difference in acceleration generated at the right side surface of the specimen

4 OCCURRENCE OF ACCELERATION ASSOCIATED WITH SHEAR BANDING

In the following, the initial conditions were changed from the conditions appropriate to realize a uniform deformation field to velocities, accelerations, and pore water pressures that are all zero. The calculation was carried out with (a) no initial geometric imperfections and (b) initial geometric imperfections applied to the specimen. In the case of (b), a half wavelength cosine curve (primary mode) with a small amplitude of 10^{-5} cm was applied to the side surfaces of the specimen in accordance with Asaoka et al. 1994. In static analysis, as in Asaoka et al. 1994, this is the shape of the induced initial imperfection, and it changed to the primary mode with reduction in load (imperfection-sensitive bifurcation behavior). In this section, in order for the specimen to maintain vertical symmetry in the case of (a), a vertical displacement at the constant rate of 0.5×10 cm/s was applied to both the top and bottom of the specimen in the compression direction. Also, the permeability coefficient was changed from zero to the values shown in Table 1. The calculation results are shown below.

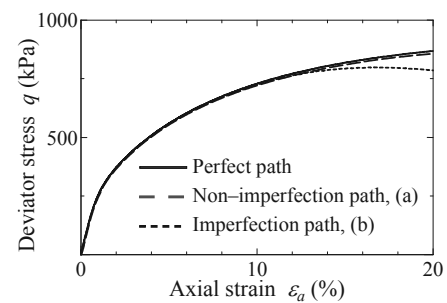


Fig. 4 Relationship between apparent q - ε_a with differences in initial imperfections

4.1 VERTICALLY ASYMMETRIC DEFORMATION INDUCED BY INITIAL IMPERFECTIONS

Fig. 4 shows the apparent axial differential stress q - axial strain ε_a relationship, and Fig. 5 shows the specimen shear strain distribution. q is the total increment of equivalent nodal forces obtained on the top divided by the area of the top at each time, and ε_a is the vertical displacement divided by the initial height. In the case of (a), q was virtually the same as the "perfect path" (= response of the constitutive equation) obtained in the uniform deformation field, and the specimen maintained left to right and top to bottom symmetry from the beginning to end. In contrast, in the case of (b), the deformation virtually maintained left to right and top to bottom symmetry up to an

ε_a of about 12%, but thereafter, vertical symmetry was broken. Associated with this deformation, q deviated from the perfect path and exhibited small values.

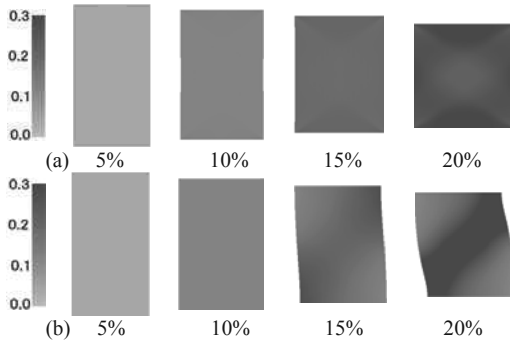


Fig. 5 Change in shear strain distribution in a specimen with initial imperfection in top loading

(a) No initial imperfections. (b) With initial imperfections.

4.2 ACCELERATIONS ASSOCIATED WITH SHEAR BANDING AND THEIR FOURIER AMPLITUDES

Fig. 6 shows the distribution of the horizontal component of acceleration generated in the specimen in (b). The acceleration distribution is symmetrical left to right and top to bottom up to about an ε_a of 12%. These are the accelerations generated due to the compression from the top and bottom as described above. In contrast, after the breakdown of vertical symmetry, localized

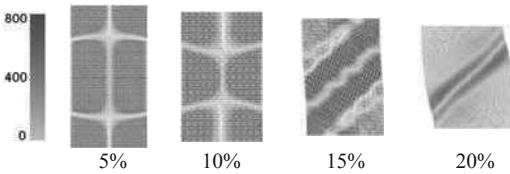


Fig. 6 Occurrence of shear banding associated with horizontal components of generated accelerations (gal)

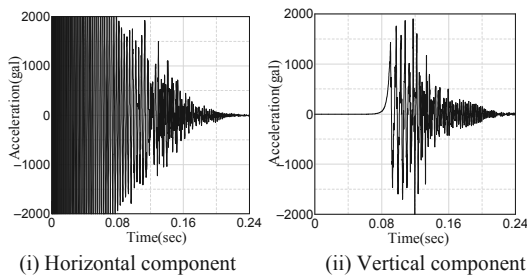


Fig. 7 (i) Horizontal component and (ii) Vertical component of acceleration generated at the side surface of the specimen

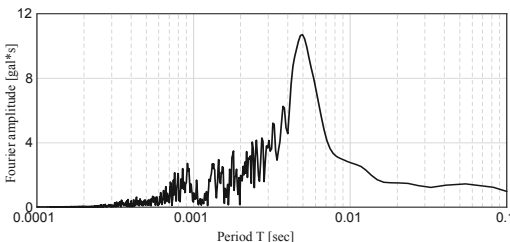


Fig. 8 Fourier amplitude of the vertical component of accelerations generated at point A

shear banding developed like reverse faults, and accelerations were generated along the shear bands. Fig. 7 shows (i) the horizontal component and (ii) the vertical component of acceleration generated at the node A shown in Fig. 1. Different from the horizontal component of acceleration, the vertical component was the component normal to the central axis of the specimen, and kept to be zero until about 0.08 sec after the start

of loading, in other words, until the ε_a was about 10%. Thereafter, as the shear banding started, new accelerations were generated with a maximum value of about 2000 gal. Also, after exhibiting the maximum value of acceleration, each component tended to converge as ε_a increased. Fig. 8 shows the Fourier amplitude of the acceleration up to $\varepsilon_a = 30\%$ for the side surface (point A) of the specimen. From these figures, it can be seen that accelerations are generated predominantly with a period of around 5.0×10^{-3} sec.

5 LOADING RATE EFFECT

An investigation into the effect of displacement rate was carried out for compression under displacement control by applying a geometric initial imperfection in (b) of section 4. Fig. 9 shows the results of a comparison of the Fourier amplitudes of the accelerations obtained at point A on the side surface of the specimen for displacement velocities of 2.5 cm/s, 5 cm/s, and 10 cm/s. In all cases, the specimen deformed as in Fig. 5(b) (figures omitted). From Fig. 9, it can be seen that the Fourier amplitudes increase with velocity, as a loading rate effect, and that the predominant vibration amplitude is about 5.0×10^{-3} sec with almost no variation.

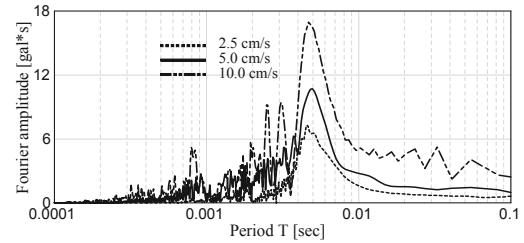


Fig. 9 Fourier amplitude of the vertical component of acceleration generated at point A on the right side surface of the specimen (load rate effect)

6 UNDRAINED CREEP BEHAVIOR UNDER CONSTANT LOAD

In this section, firstly all the initial and boundary conditions as well as the initial imperfection are same as (b) in section 4. The calculation performed was continued after deviation from the perfect path until (i) $\varepsilon_a = 13\%$ (0.104 sec after the start) and (ii) $\varepsilon_a = 18.75\%$ (0.150 sec after the start), and then was altered to load control at the top, maintaining the load constant, and continuing with displacement control on the bottom edge but stopping the vertical displacement. For load control, the conditions for the pedestal with no friction were calculated using the constraint conditions on the finite element nodes by the Lagrange method of undetermined multipliers, as in Asaoka et al. 1998. The results of the calculations are described below.

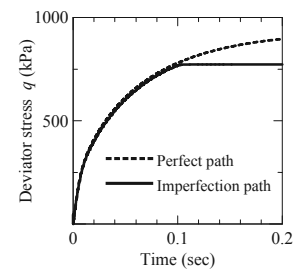


Fig. 10 Load control after displacement control ("creep")

Fig. 10 shows the relationship between the calculated specimen apparent top q and the elapsed time from displacement control, and Fig. 11 shows the change in axial strain from the start of displacement control. However, in Fig. 10, q was obtained by dividing by the initial area of the top of the specimen. Also, the $\circ$ in Fig. 11 indicates the point of

change to load control. Figs. 12 and 13 show the vertical component of the velocity and acceleration, respectively, of the top of the specimen (in both cases, upwards is positive). In the case of both (i) and (ii), immediately after the change to load control, large changes in acceleration were seen, but in the case of (i), the changes soon converged. In contrast, in the case of (ii), after temporarily converging, the vertical velocity increased at around 0.5 sec to 0.9 sec in association with an increment in ac-

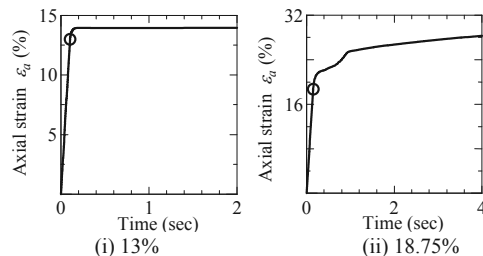


Fig. 11 Development of axial strain; (i) and (ii) are the axial strain after changing to load control from displacement control)

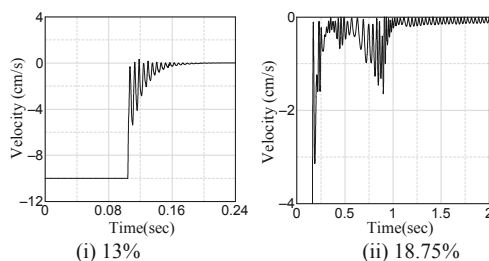


Fig. 12 Relationship between vertical velocity of the top and time

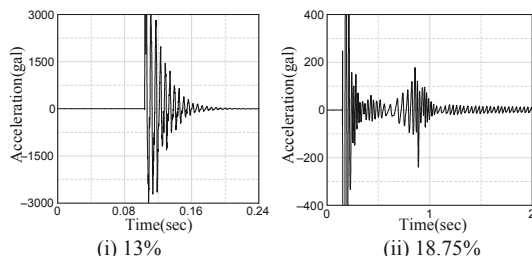


Fig. 13 Relationship between vertical acceleration of the top and time

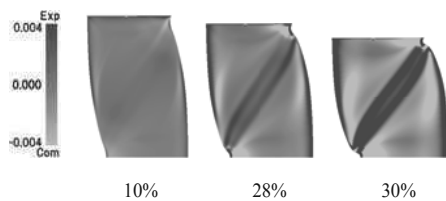


Fig. 14 Changes in specific volume (swelling shear bands)

celeration and then reduced again, and thereafter, the axial strain slowly increased. In order to show the state occurring within the specimen at this time, Fig. 14 shows the change in specific volume within the specimen from the initial state. During the displacement control that included $\varepsilon_a = 10\%$, the loading was comparatively fast, so there was almost no migration of pore water within the specimen; however, under load control with the load constant, migration of pore water proceeded, causing flow into the shear band and swelling (generating swelling bands and acceleration).

7 CONCLUSION

This paper describes examples of calculation using the dynamic/static soil-water coupled finite deformation analysis code, GEOASIA in accordance with the u - p Formulation on the

plane strain rectangular overconsolidated specimen under constant cell pressure and undrained conditions. The following is a summary of the new knowledge obtained:

1) Firstly, it was shown that a uniform deformation field in saturated soil under element-wise undrained conditions is possible when the permeability coefficient is zero, and that the output acceleration and pore water pressure field are in virtual agreement with the theoretical values. In order to realize this, it is necessary to apply a distribution of initially reduced velocities and accelerations, as well as pore water pressure within the specimen derived from theoretical considerations.

2) Next, initial velocity, acceleration, and pore water pressure values of zero were applied, and under vertical displacement control, vertical rates were applied with symmetric compression loading from the top and bottom. In this case, the lateral and vertical symmetry of the specimen broke down, and strain localization occurred. Also, in association with the occurrence of shear banding, when a vertical rate of 5 cm/s was applied, accelerations with a maximum of slightly less than 2000 gal were generated from the shear band with a predominant period of around 5.0×10^{-3} sec.

3) The load rate effect was investigated thirdly. The results showed that although the magnitudes of the Fourier amplitudes of the generated accelerations were different, the predominant period in all cases was unchanged at around 5.0×10^{-3} sec. Together with 2), in dynamic analyses, even though an acceleration time history such as input seismic motions, etc., has been applied to specimens or the ground, the point of view of generation of accelerations during the deformation of the soil or the ground itself has almost never been emphasized.

4) Finally, the specimen exhibited creep behavior under undrained conditions when the displacement control in 2) was changed to load control after the basic path deviated, and the load was maintained constant. When the load was held constant at a more unstable position, the axial displacement increased greatly with increases and decreases in acceleration in association with the development of shear bands associated with swelling (swelling bands). In this calculation, the inherent clay concept of the soil structure was not introduced, but it was shown that due to the existence of the inertia term and soil-water coupled behavior, large deformation associated with time dependence under load instability can be easily simulated.

8 REFERENCES

- Ikeda, K. et al. 2003. Simulation and interpretation of diffuse mode bifurcation of elastoplastic solids, *J. Mech. Phys. Solids* 51 (9), 1649–1673.
- Wan, R.G. et al. 1990. A Finite Element Method for the Analysis of Shear Bands in Geomaterials, *Finite Elem Anal Des* 7 (2), 129–143.
- Asaoka, A. et al. 2007. All soils all states all round geo-analysis integration, *Implementation, Evaluation, and Application*, Hong Kong, China, 11–27.
- Noda, T. et al. 2008. Soil-water coupled finite deformation analysis based on a rate-type equation of motion incorporating the SYS Cam-clay model, *S&F* 48 (6), 771–790.
- Asaoka, A. et al. 2002. An elasto-plastic description of two distinct volume change mechanisms of soils, *S&F* 42 (5), 47–57.
- Asaoka, A. et al. 1994. Imperfection-sensitive bifurcation of Cam-clay under plane strain compression with undrained boundaries, *S&F* 35 (1), 83–100.
- Noda, T. et al. 2013. Realization of uniform deformation of soil specimen based on soil-water coupled finite deformation analysis considering inertia forces, *S&F*, to be submitted.
- Asaoka, A. et al. 1998. Displacement/traction boundary conditions represented by constraint conditions on velocity field of soil, *S&F* 38 (4), 173–181.

Behavior of fine-grained soils compacted with high shear stresses

Comportement des sols fins compactés avec des niveaux de cisaillement élevés

Perez N., Garnica P., Mendoza I., Reyes M.A.
Mexican Transportation Institute

ABSTRACT: One of the parameters used to carry out the quality control of unbound compacted fine-grained materials is the maximum dry unit weight obtained from a Proctor standard or modified test. However, these tests are far from simulating the field compaction mechanism produced by the sheep foot roller equipment. Lately, the gyratory compactor has been put forward as a new laboratory equipment to determine the compaction curves. This paper shows results of Proctor and modified compaction curves as well as the ones obtained from the gyratory compactor. The new method of compaction was evaluated for three soils classified as CH, ML and SM. The controlled variables in the gyratory compactor were the gyration angle, the vertical pressure, and the number of gyrations. The results showed that the optimum water content is reduced as the vertical pressure increases and the opposite happens with the dry unit weight. In addition, it was observed that the Proctor compaction curve is obtained with a vertical pressure of 200 kPa and around 200 gyrations regardless the soil type. On the other hand, it seems that the compaction curves are similar disregarding the rate of gyration and gyration angle.

RÉSUMÉ : Un des paramètres classiques utilisés pour le contrôle de qualité des sols fins compactés non traités est le poids volumique sec obtenu dans l'essai Proctor, standard ou modifié. Cet essai est cependant loin de simuler les mécanismes de compactage in situ produits par des équipements tels que les rouleaux à pieds dameurs. Récemment, le dispositif de compactage giratoire a été mis en avant, en tant que nouveau dispositif de laboratoire pour la détermination des courbes de compactage. Cette communication présente une comparaison des courbes de compactage obtenues aussi bien avec les essais Proctor qu'avec le dispositif giratoire, pour lequel les influences de paramètres tels que la pression verticale, l'angle de giration et le nombre de girations ont été évaluées pour des sols de classe CH, ML et SM. Une discussion est ensuite proposée, montrant que la teneur en eau optimale diminue avec l'augmentation de la pression verticale, une tendance opposée apparaissant pour le poids volumique sec. On a aussi observé que la courbe Proctor classique est obtenue pour une pression verticale de 200 kPa et environ 200 girations, quel que soit le type de sol. Les courbes de compactage obtenues ne semblent cependant pas dépendre de la vitesse ni de l'angle de giration.

KEYWORDS: gyratory compactor, soil compaction, quality control, Proctor test, compaction curves, pavements, unbound materials.

MOTS-CLÉS : compacteur giratoire, compactage des sols, contrôle de qualité, essai Proctor, courbes de compactage, revêtement routier, matériau non traité

1 INTRODUCTION

Soil compaction is a process which is often used in the construction of almost every single engineering structure. Examples such structures are dams, the approaches of bridges, mats for buildings, airports, pavements, etc.

To carry out the quality control of the compacted materials it is necessary to count with two parameters, that is to say, the field dry unit weight and the maximum dry unit weight obtained in a laboratory test which can be Proctor standard or modified (this paper is focused in the evaluation of lab maximum dry unit weight).

To evaluate laboratory properties (i.e. the dry unit weight, resilient modulus, unconfined compression, etc), it is paramount that the test specimens possess as far as possible the same structure that the soil will have in field. The evaluation of the maximum dry unit weight and optimum water content of fine-grained soils with test as the Proctor standard and modified test is an example of an inconsistency between field and lab structure. The field equipment for these cases are the sheepfoot roller which compacts the soil from bottom to top while in the Proctor standard test the soil is compacted by impacts, thus, it is expected to obtain different soil structures. On the other hand, Ping et al. (2003) have found that the field and laboratory compaction curves are completely different. Thus, as a consequence, some researchers (Ping et al., 2003; Milberger y Dunlap, 1996; Mokwa et al., 2008, etc.) have put forward the gyratory compactor as a new equipment to determine the laboratory compaction curves.

In this paper, the compaction curves were obtained with the gyratory compactor and then they were compared with the standard and modified compaction curves. In addition, it was studied the effect on the compaction curve of variables as the vertical pressure, the angle of gyration and the number of gyrations. The procedures, equipments and results are described in the following paragraphs.

2 GYRATORY COMPACTOR

Compaction in this equipment is achieved by the application of vertical stress to a known mass within a mold of 100 or 150 mm internal diameter. The longitudinal axis of the mould is rotated (gyrated) at a fixed angle to the vertical while the platens are kept parallel and horizontal. During the compaction the height of the sample is automatically measured and the mixture density is calculated. The operator can choose whether to compact to a certain number of gyrations, a certain height or until a target density (<http://www.cooper.co.uk>). It is important to mention that even this equipment was designed to compact specimens of asphalt mix, lately it has been utilized to compact fine-grained and granular soils.

3 TEST SOILS

During this research it was collected a series of samples classified as clay (CH), silt (ML) and sand (SM) (Figure 1).

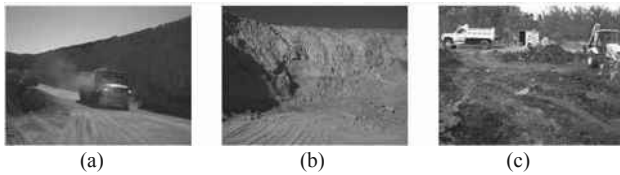


Figure 1. Places from where the soils were sampled. (a) Silt (ML); (b) Sand (SM); and (c) Clay, CH.

The index properties of the soils are listed in Table 1 and the compaction characteristics obtained from Proctor and modified tests are summarized in Table 2 (Figure 2 a 4).

Table 1. Index properties of the test soils.

Soil Type	Atterberg limits			% passing 200 sieve (%)	G _s
	LL (%)	PL (%)	PI (%)		
CH	66	25	41	85.7	2.61
ML	44	33	11	87.0	2.56
SM	NP	NP	NP	37.0	2.52

Table 2. Compaction characteristics.

Soil type	Standard effort (ASTM D698)		Modified effort (ASTM D 1557)	
	W _{opt} (%)	γ _{dmax} (kN/m ³)	W _{opt} (%)	γ _{dmax} (kN/m ³)
CH	30.0	13.32	---	---
ML	30.5	13.33	24.5	14.59
SM	23.54	14.04	19.0	15.17

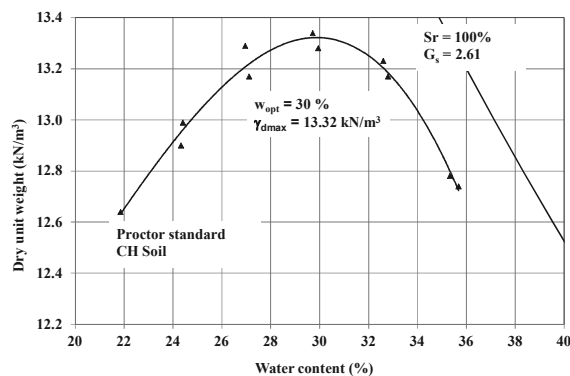


Figure 2. Compaction curve of the CH soil.

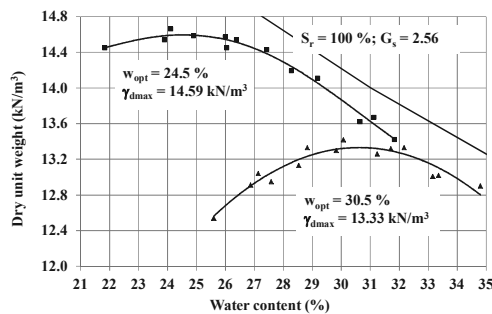


Figure 3. Compaction curves of the ML soil.

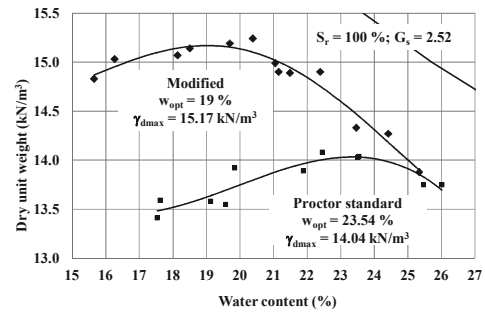


Figure 4. Compaction curves of the SM soil.

4 TEST PROCEDURES.

4.1 Compaction curves. Proctor standard and modified (ASTM D698 and ASTM D 1557)

To obtain the standard and modified compaction curves, the ASTM procedures were followed (ASTM D698 and ASTM D1557). From these procedures, the method A was utilized in both cases since all material tested passed sieve number 4.

4.2 Compaction curves. Gyrotory compactor

As was mentioned in previous paragraphs, in this equipment the operator can control variables such as: vertical pressure, angle of gyration, height of specimen, density, etc. In this study, the controlled variables were as follows:

- Vertical pressure: 200, 300, 400, 500, and 600 kPa.
- Angle of gyration: 1 and 1.25 degrees
- Number of gyrations: 500
- Rate of gyrations: 10, 20 and 30 gyrations/min
- Soil type: 3
- Mass of compacted soil: 2300 g

Once the soils were gathered, the material larger than No. 4 sieve was discarded. The material passing was allowed to dry at environmental conditions and subsequently it was mixed thoroughly and then stored in sacks.

For all three soils, the procedure followed to evaluate the compaction curves was as follows:

1. 180 soil samples of 2300 g of dry soil (of each soil type) were weighted so that to cover all the combinations of the variables to be controlled (5 vertical pressures x 2 angles of gyration x 3 rates of gyration x 1 sample at each point x 6 points on the compaction curve).
2. Different amounts of water were added to each sample so that to cover the range in which the modified and standard tests were found (to develop each curve, six points were considered).
3. The soil samples were stored during 24 hours.
4. An amount of 2300 g of wet soil was placed inside the compaction mold (Figure 5b and 5c), but previously some plastic strips were placed on the interior wall of the mold so that the soil did not stick and the sample could be extracted (Figure 5a). In addition, another plastic circle was placed on top of the soil so that it did not stick against the top compaction platen.



Figure 5. (a) Strips inside the interior wall of the mold; (b) 2300g of wet soil were placed in the mold, and, (c) A plastic circle was placed on top of the mold for the soil not to stick to the compaction platen.

5. The mold was placed inside the machine (SERVOPAC). It was programmed to compact the soil at a required angle of gyration, vertical pressure and 500 gyrations. This step was repeated for the 180 samples of each soil type.
6. After the sample was compacted at 500 gyrations, it was extracted. The dimensions and weight were taken and registered.

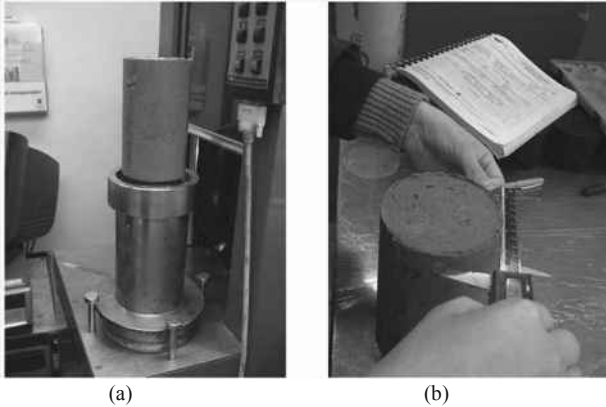


Figure 6. (a) Ejection of the compacted sample; (b) the dimensions of the sample were taken.

7. In the final step the sample was taken apart to obtain samples to determine the water content.
8. With data of water content, height of the specimen and dimensions, the dry unit weight was calculated at every gyration. A typical plot obtained for every single sample is illustrated in Figure 7.

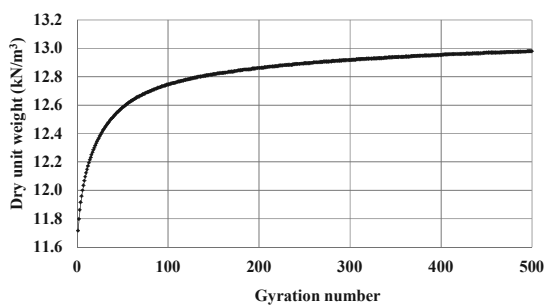


Figure 7. Typical plot of dry unit weight versus gyration number

From Figure 7 data at 100, 200, 300, 400, and 500 gyrations were taken to plot the compaction curves.

5 DISCUSSION OF RESULTS

5.1 Compaction curves from gyratory compactor

Figure 8, 9 and 10 illustrates the compaction curves obtained at different vertical pressures, 1.25 angle of gyration, 10 gyrations per minute and for each soil type. The dry unit weight plotted in these figures was taken at gyration number 500.

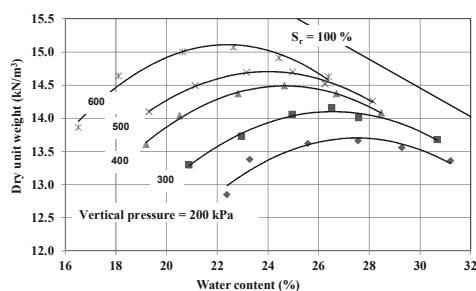


Figure 8. Compaction curves for CH soil.

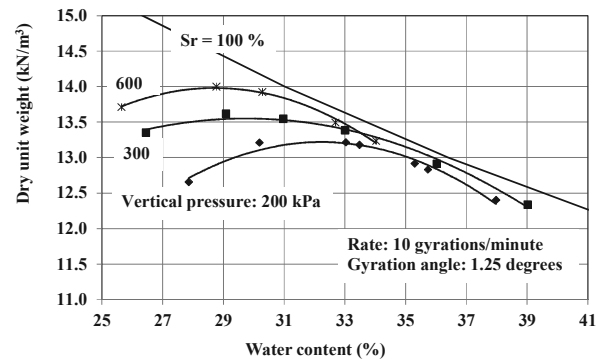


Figure 9. Compaction curves for ML soil.

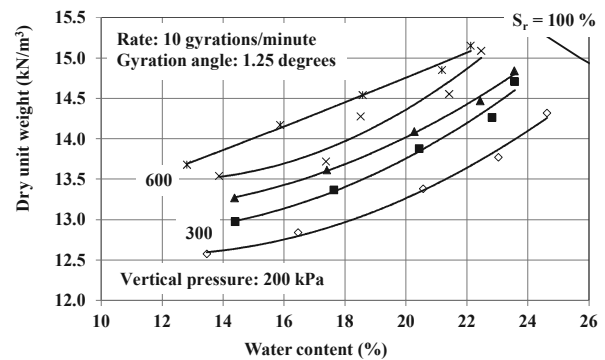


Figure 10. Compaction curves for SM soil.

As can be noted from Figures 8, 9 and 10, the dry unit weight increases and the optimum water content is reduced as the vertical pressure increases, however, this trend is clear for the CH and ML soil. On the other hand, for the SM soil, the dry unit weight increases, but for water contents larger than the optimum of the Proctor standard test, the material cannot be compacted because water starts to flow out of the mold. For this reason, the compaction curves do not show the maximum dry unit weight that can be observed in the standard compaction curve obtained by impacts.

5.2 Effect of different variables on the compaction curve

5.2.1 Gyration number

In Figures 8 to 10 the value of dry unit weight that was plotted was calculated at gyration number 500, however, for each compacted sample, it was obtained a curve of gyration number versus dry unit weight. Thus, to detect the effect of the number of gyrations, some of the compaction curves were plotted on the compaction space together with the standard and modified curves (Figure 11). As seen from this figure and also from Figure 7, the major change in the dry unit weight is noted in the first 100 or 200 cycles. For further gyrations, there is only a slow change in dry unit weight. Thus, it can be concluded that the soil can be compacted with 100 or 200 gyrations.

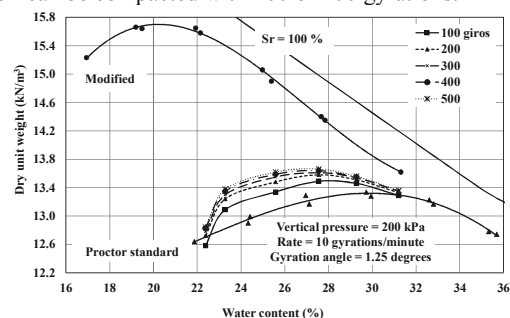


Figure 11. Effect of number of gyrations on the compaction curve (CH soil).

From Figures 8 and 11 it is also clear that to compact material in field, a heavy equipment is more effective than to apply many passes of a light one.

5.2.2 Gyration angle

The data obtained in this study indicates that the compaction curve is independent of the gyration angles, at least for the two gyration angles studied (Figure 12).

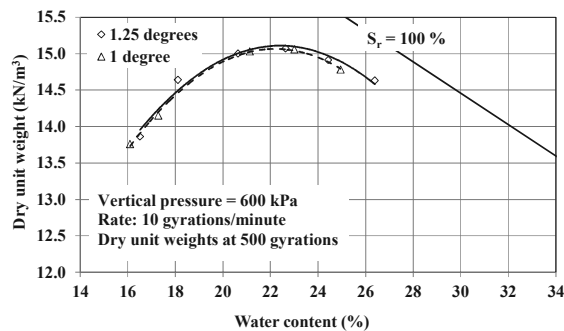


Figure 12. Effect of gyration angle (CH soil).

5.2.3 Gyration rate

The data obtained in this study indicates that the compaction curve is independent of the gyration rate, at least for the three gyration rates studied (Figures 13, 14 and 15). Thus, for practical purposes it is convenient to carry out the tests at the highest velocity tested in this study.

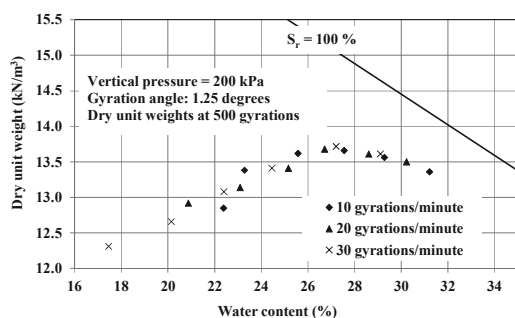


Figure 13. Effect of gyration rate (CH soil).

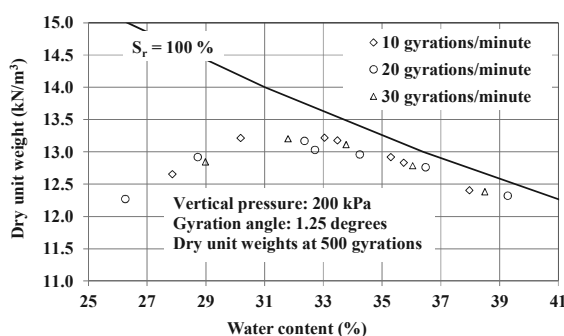


Figure 14. Effect of gyration rate (ML soil).

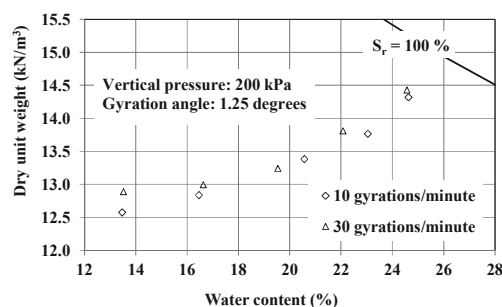


Figure 15. Effect of gyration rate (SM soil).

5.3 Shear stresses from gyratory compaction

The data registered during the compaction in the gyratory compactor not only includes the height of the specimen, the vertical pressure and the gyration angle, it also contains the shear stress developed during the whole process of compaction. Figure 16 shows an example of the magnitude of the shear stresses that can be developed during the process of compaction of a clay with 600 kPa of vertical pressure.

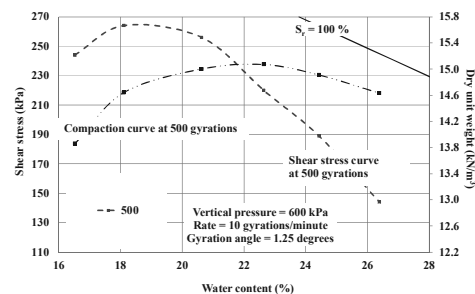


Figure 16. Shear stress curve for CH soil (600 kPa of vertical pressure).

From Figure 16 it can be observed that the shear stress tends to increase when the compacted soil is on the dry side of the compaction curve, however, close to the optimum, the shear stresses are reduced. It was expected because as the water content increases the contacts between grains are reduced and as a consequence the shear stresses are reduced.

6 CONCLUSIONS

The study of the behavior of compacted soils is of paramount importance because this kind of material is involved in the construction of almost all civil engineering structures. Its quality control has been determined by the evaluation of two parameters, that is to say, the field and laboratory density. To evaluate the second one, the standard or modified Proctor tests have been in use long time ago, however, these test are far from developing the same soil structure provided for example for a sheepfoot roller which are utilized to compact fine-grained soils. For this reason, the gyratory compactor has been put forward as a new lab equipment to determine the standard and modified compaction curves. This paper presented a series of compaction curves developed in this equipment. It was observed that there are a series of combinations of variables that can be controlled in the gyratory compactor to obtain the standard compaction curve. On the other hand, the compaction curves seems to be independent of variables as the gyration angle and the gyration rate, at least for the two gyration angles studied and the three velocities. In addition, it is clear that to increase the dry density of a soil, it is more effective to utilize heavy equipment that many passes of a light one.

7 REFERENCES

- Milberger, L., Dunlap, W. (1966). A gyratory compactor for molding large diameter triaxial specimens of granular materials. Research report number 99-2. Texas Highway department in cooperation with the U.S. Department of Commerce.
- Mokwa, R., Cuelho, E., y Browne, M. (2008). Laboratory Testing of Soil Using Superpave Gyratory Compactor. Transportation Research Board, 2008.
- Ping, W.V., Xing, G., Leonard, M., y Yang, Z. (2003). Evaluation of Laboratory Compaction Techniques for simulating field soil compaction (Phase II). Report No. FL/DOT/RMC/BB-890(F). http://www.cooper.co.uk/info/index.asp?page=crt_gyr_en_gyratory_compactor_33.
- Norma ASTM D 698-07e1, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12 400 ft-lbf/ft³ (600 kN-m/m³)).

Influence of Minerals on the Elastic Behaviour of Cohesive Soil

Influence des minéraux sur le comportement élastique des sols cohésifs

Sarma D., Sarma M.D

M-AGS, M-ISSMGE, Independent Consultant (Southern Africa)

ABSTRACT: The trend of predicting the design parameters of soil from the index properties is continuing since decades, however, those correlations do not focus much on the prediction of elastic behaviour of cohesive soils. Moreover, the influence of mineral(s) for the validation of such correlations is either ignored or presumed to be covered by the Atterberg's limits. As the outcomes of the research, pertaining to the title of the paper, influence of a prime constituent mineral has been identified and the extent of its influence is derived. The constitutive modelling, an outcome of the correlative study, is analysed based on its typical pattern. The model thus derived within the set scope of the investigation has led to the conclusion that the pattern is unique for cohesive soils and is under the possible domination of the existence of a common mineral and extent of its weathering process. It leads to the indication of the profound influence of the mineral and its weathering state upon elastic behaviour of cohesive soils. The model pointed out that some of the engineering and geological properties are interdependent. The model has been calibrated with other available empirical correlations and its civil engineering applications and limitations are derived. Outlines of subsequent research areas are also indicated.

RÉSUMÉ : La tendance de prédire les paramètres de conception du sol des propriétés de l'index se poursuit depuis des décennies, cependant, ces corrélations ne se concentrent pas autant sur la prédiction du comportement élastique des sols cohésifs. En outre, l'influence de minéraux (s) pour la validation de ces corrélations est soit ignorée, soit présumée être couverte par des limites d'Atterberg. Comme les résultats de la recherche, portant sur le titre du document, l'influence d'un minéral constituant principal a été identifiée et l'étendue de son influence en découle. La modélisation constitutive, le résultat de l'étude correlative sont analysés en fonction de son modèle typique. Le modèle ainsi obtenu dans le cadre d'ensemble de l'enquête a conduit à la conclusion que le modèle qui est unique pour les sols cohésifs et est sous la domination possible de l'existence d'un minéral commun et l'étendue de son processus de vieillissement. Elle conduit à l'indication de l'influence profonde de la matière minérale et de son état intempéries sur le comportement élastique des sols cohésifs. Le modèle a fait remarquer que certaines des propriétés techniques et géologiques sont interdépendants. Le modèle a été calibré avec d'autres corrélations empiriques disponibles et ses applications en génie civil et les limites sont dérivées. Les contours des domaines de recherche sont également indiqués.

KEYWORDS: Index properties, elastic behaviour, influence of minerals, cohesive soil, constitutive modelling, weathering process.

1 INTRODUCTION

The trend for prediction of design parameters of soil from the index properties is continuing since decades for avoiding complex, time-consuming, and relatively costly tests. Many empirical correlations are already published and some are being used in practice. Preliminary literature survey indicates that those correlations do not focus much on the prediction of elastic behaviour of cohesive soil. Moreover, the influence of mineral(s) for the validation of such correlations is either ignored or presumed to be covered by the Atterberg's limits. In a postgraduate research of engineering geology, formulated by the Author¹, it was intended to establish the empirical correlation for the prediction of elastic behaviour of cohesive soil from the index properties and any influence of mineral(s). Subsequent development was to use the correlation for fast-track design of road and airport pavements, foundation of structure, and for slope stability analysis within restricted deformation based on known geological formations.

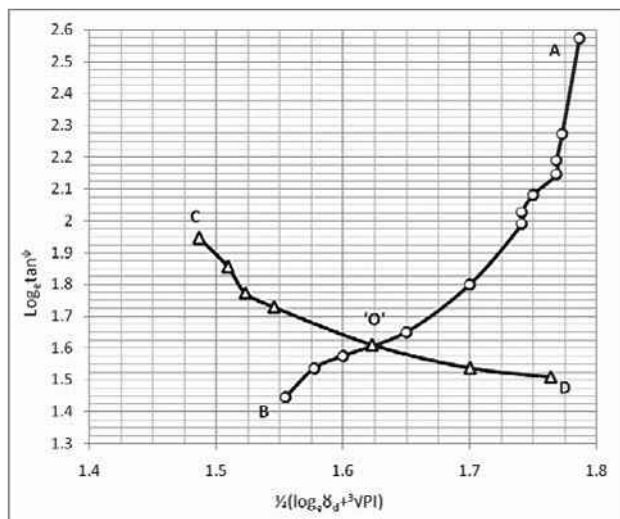
In this research, a detailed literature study and its inference was drawn for the prevailing empirical correlations with particular emphasis on the coverage of elastic property, directly or indirectly. Further, this research included brief methodologies of samplings collected from various locations to the extent necessary for common civil engineering structures, schedule of laboratory investigations on index properties, stress-strain responses of shear tests, and relevant geological studies.

Laboratory investigations for civil engineering properties and geological studies were carried out at state Road Research Laboratory and Geological Science Laboratory of the state Public Works Department and University respectively. Based on the outcomes of these studies the inference of all such investigations has been drawn. Furthermore, constitutive modelling for the empirical correlation and influence of minerals has been investigated.

2 INFERENCES

This research was carried out with an objective of correlating geological nature of soil and engineering properties, with particular emphasis on the prediction of elastic behaviour of cohesive soils vis-à-vis the elastic settlement of structures. Significant outcome of the research is the constitutive model (Ref Figure-1), which is based on some sensitive parameters, namely, dry density (γ_d), Plasticity Index (PI), and initial tangent modulus ($\tan \psi$), where ψ is the angle between initial tangent modulus and abscissa and presented in graphical form. The graphical model has been obtained by plotting two factors, as shown in the abscissa and ordinates, based on the results from about 8 sites in northeast India, which is famous for highest rainfall, flood, earthquake, and complex geological formations. The result yields two curves that intersect at nearly 90 degree depicting two classified characteristics of the model. Hence, it was found necessary to verify whether the graphical

model holds good for the parameters of other sites of the region. Accordingly, supplementary test-data were obtained from the samples of various sites, where investigations were undertaken by the Road Research Laboratory (RRL). Based on the data, so obtained from RRL, values of the two distinguished factors as shown in the abscissa and ordinates of Fig 1 were plotted and were found corroborating with the model. This simulation exercise had encouraged further investigations on the interpretation of data and development of correlation, exploring



various geochemical and other technical evidences in context to the two classified characteristics of the model.

Figure 1. The constitutive model of elastic and index properties of cohesive soil distinguished by geological characteristics

3 INTERPRETATION

The constitutive model, as shown in Fig 1, was required to be interpreted with severe setbacks invading it. However, the expectation that the engineering characteristics of soil depend on the geological properties was given due attention. Accordingly, for clayey soil it had been the expectation that the values of PI , γ_d , and ψ were under the profound influence of the texture, structure, composition, and other geological parameters.

A most common form of geo-chemical test was carried out and found to be of much help in interpretation of the parameters that influence the nature of the graphical model.

The ultimate extent of interpretation was to investigate how the engineering properties of the clays of the study area have the bearing on physical, chemical, and biological weathering processes of rock and how such information on geological variations of the soil samples could be correlated for estimation of the elastic settlement of structures.

4 CORRELATION

Correlation among the various geological and engineering properties was found to be a complex process, which nonetheless was attempted within the limited scope of this research through geochemical analysis. The samples were identified as medium to highly plastic inorganic clay of semi-pervious to impervious nature. This identification process was based on the results obtained from the laboratory tests carried out to investigate the engineering properties of the samples under study.

The study samples were predominantly Kaolinite and Illite (formed by decomposition of Potash Feldspar), Biotite (mostly altered to Chlorite and Serpentine), partly weathered Quartz, and possibly Montmorillonite. Although the exact crystal structure of clay minerals could not be known in thin slides, nonetheless, geochemical and other indirect evidences proved

these to be clays obtained predominantly by the decomposition of Feldspar.

5 GEOCHEMICAL EVIDENCE

The mineralogy of sedimentary rocks were characterised by two distinct types of minerals, first, the resistant mineral obtained from the mechanical breakdown of the parent rock, and, second, the minerals newly formed from the products of chemical decomposition. The latter minerals were generally hydrated compounds. "Goldich (1938) pointed out that the order of the stability of minerals of igneous rock towards weathering is the reverse of their order in the reaction series of Bowen (1915 a & b)" - Mason and Moore 1991. The identity of arrangement between Bowen's reaction series and Goldich's stability series indicates that the last-formed minerals of igneous rock are more stable in subtropical temperature than the minerals formed at an early stage of crystallisation. In other words, the difference between the conditions at the time of formation and those existing at the surface reflects the order of stability of common silicate of igneous rocks.

Quartz and Feldspar are the abundant and dominant minerals. Whereas Quartz is very resistant to the chemical attack, feldspar is less resistant under identical scenario. Although Feldspar may persist indefinitely in sedimentary rocks, they are chemically decomposed by prolonged weathering. In particular, Feldspars give rise to clays with Potash Feldspar reacting in the presence of water to give Illite and Plagioclase Feldspar reacting in a similar manner to give Montmorillonite.

The samples under study were predominantly a mix structure of Kaolinite and Illite with the presence of Ferromagnesian minerals and weathered Quartz. However, the presence of Montmorillonite was doubtful as because the studied samples did not show the typical expansive characteristics.

The specific gravity is generally low when rock contains light coloured minerals like Quartz and Feldspar and is high when rocks contain dark coloured minerals, for example, Ferromagnesian. However, clay minerals generally have a mean specific gravity value of about 2.7, but the samples that were studied showed a range of values of specific gravities from 1.99 to 2.65. It was expected that the samples would contain some amount of organic matter and possibly more decomposed Feldspar than decomposed Biotites as they are more resistant than Feldspar or other Ferromagnesian minerals. The only exception being the sample which lies at the point of intersection between two curves of the model.

6 INDIRECT EVIDENCE

The clay sample of the study area typically Quaternary deposits, which were the derivation of the Precambrian Granite and Quartz Feldspathic Gneisses, had been influenced by all three types of weathering processes, namely, physical, chemical, and biological, of which chemical is dominant.

The Precambrian Granitic rock, which includes the Quartz Feldspathic Gneisses of the area, are composed of predominantly Quartz, Feldspar, and Biotite as primary minerals. Therefore, it is obvious that decomposition of these minerals have led to the formation of clays. Since Quartz and partly Biotite are resistant to chemical weathering, the role of Feldspar stands out in this regard. The Feldspar easily decomposes in the presence of rain water and in presence of carbon-dioxide in atmosphere. The product of the decomposition is clay which plays an important role in the formation of soil of the study area. Quartz remains unchanged in the process of chemical decay and therefore presence of some amount of silt and also sand at depth greater than 15m is notable. Biotite on decomposition yields yellowish clay, the yellow colour being due to the iron content in Biotite. The

decomposition sequence of these minerals is also in agreement with the stability order of minerals by Goldich (1938).

7 INTERFERENCE DRAWN FROM CORRELATION

The correlation between the engineering and geological properties infers the conclusion that the pattern of the curves obtained in the model (Fig 1) is apparently dominated by the content of Feldspar and extent of decomposition of Feldspar in relation to other constituent minerals.

The samples represented by the points along the curve A-B show a continuous decomposition of Feldspar and alteration of Biotite to Chlorite with the increase of the values of $\frac{1}{2}(\log_e \gamma_d + \sqrt[3]{PI})$ and $\log_e \tan \psi$. Therefore it has been inferred that the initial tangent modulus is apparently related to the content and extent of decomposition of Feldspar and hence is expected as a major influential factor in estimating the elastic behaviour of cohesive soil.

The points along the curve C-D shows some exception to the above inference and possibly suggests a reversal of the above, whereby Ferromagnesian minerals are undergoing the process of decomposition with increase in the values of $\log_e \tan \psi$ and decrease in the values of $\frac{1}{2}(\log_e \gamma_d + \sqrt[3]{PI})$. The point number "O" is obviously an intermediate point for both the curves of the model.

Correlation of engineering and geological properties based on specific gravity analysis shows that the point no "O", that is, the intersection point has the highest value of specific gravity being 2.65, thereby suggesting presence of more or less an intermediate composition of light and dark coloured minerals. The bottom point of the curve A-B shows a specific gravity of 1.99 and the upper point of the curve A-B shows 2.19, indicating enhancement of decomposition and alteration of both light and dark coloured minerals with the increase in the values of $\frac{1}{2}(\log_e \gamma_d + \sqrt[3]{PI})$ and $\log_e \tan \psi$. The bottom and top points of the curve C-D shows specific gravity values of 2.19 and 2.4 respectively, thereby indicating that Ferromagnesian minerals have increased with the increase of $\log_e \tan \psi$ and decrease of $\frac{1}{2}(\log_e \gamma_d + \sqrt[3]{PI})$ values.

8 CONCLUSION

As the outcomes of the research, influence of a prime constituent mineral has been identified and the extent of its influence is derived. The constitutive modelling, an outcome of the correlative study, is analysed based on its typical pattern. The model thus derived, within the set scope of investigation, has led to the conclusion that the pattern is unique for cohesive soils and is under the possible domination of a common mineral and extent of its weathering process. It leads to the indication of the profound influence of mineral and its weathering state upon elastic behaviour of cohesive soils. The model pointed out that some of the engineering and geological properties are interdependent. The model has been calibrated with other available empirical correlations and its application in civil engineering and limitations are derived. Outlines of subsequent research areas are also indicated.

9 REFERENCES

- Bowen N. L. 1915a. *The crystallisation-differentiation in silicate liquids*, American Journal of Science, Series 4, 39, 175-191.
- Bowen N. L. 1915b. *The crystallisation of haplobasaltic, haplodioritic, and related magmas*, American Journal of Science, Series 4, 40, 161-185.
- Goldich S. S. 1938. A study in rock weathering. *Journal of Geology*. 46, 17-58. *A carefully documented account of the chemical and mineralogical changes produced by weathering on a granite gneiss, two diabases, and an amphibolite.*
- Hussain M. 1996. A Correlative Study of Geological Nature and Engineering Properties of Clay Samples to Interpret their Influence

upon elastic Behaviour of Cohesive Soil. *A dissertation submitted for the Degree of Master of Science in Geology*. Gauhati University, India.

Mason B. and Moore C. B. 1991. *Principles of Geochemistry*. Wiley Eastern Limited. 4th Edition. P 155

10 NOTATIONS

γ_d : Dry Density

PI: Plasticity Index

Tan ψ : Initial Tangent Modulus

ψ : Angle between initial tangent modulus and abscissa.

11 ACKNOWLEDGEMENTS

Author¹ wishes to thank Dr M. Chakravarty, Senior Lecturer, Department of Geology, Cotton College, Guwahati, India for agreeing to the area of research for joint supervision and for scrutinising the geological aspects of the dissertation [Hussain 1996]. Further, Author¹ wishes to thank Ms Mehjebeen Hussain nee Simi Deka, the Research Scholar for accomplishing the R&D work as per the expectation. Furthermore, Author¹ wishes to acknowledge the immense support he received from the entire team of Road Research Laboratory, Guwahati, India, where he worked as in-charge of the pavement laboratory. Last but not the least, Author¹ wishes to record the contribution of Author² in context to her contributions pertaining to this paper in general literature review and from her international publications on the correlation of partially saturated clays with simple index properties. Besides general technical review she specifically contributed to the translation aspects, being a French literate.

Experimental Analysis on the Influence of Surcharge Filters on Safety Against Hydraulic Heave

Analyse expérimentale de l'influence d'un filtre de surcharge sur la stabilité contre des soulèvements d'eau d'une fouille de construction

Schober P., Boley C.

Institut für Bodenmechanik und Grundbau, Universität der Bundeswehr München

ABSTRACT: When designing a deep excavation pit, verifying hydraulic heave safety is in many cases crucial for determining the necessary embedment depth of the pit wall. To reduce the required embedment depth, a surcharge filter can be installed on the pit bottom. However, theoretical analyses show that the conventional approaches are not adequate for verifying hydraulic heave safety if the embedment depth of the wall is low and a surcharge filter is installed on the pit bottom. Therefore, a theoretical approach with an extended unstable block was developed by Odenwald and Herten. For further analysis and verification of this extended theoretic approach, big scale laboratory experiments were carried out and the critical hydraulic difference in dependence of the embedment depth and the surcharge filter thickness was determined. In the process it became apparent that the calculated results were qualitatively accord with the experimental results of the test series, yet far more conservative. For a better understanding of the effectiveness of surcharge filters on hydraulic heave safety, one dimensional flow-test with variation of the filter thickness and the relative density of the sand were carried out. This paper presents the results of the experimental series und compares them to theoretical approaches.

RÉSUMÉ : Pour des excavations à proximité immédiate des canaux de navigation en service, la vérification de la stabilité contre des renards est déterminante pour la longueur nécessaire de la cloison de l'excavation. En installant un filtre de surcharge au fond de l'excavation, on peut réduire la profondeur nécessaire de la cloison. Pourtant, analyses conduites sur la base des calculs numériques des courants de l'eau souterraine montrent que les vérifications conventionnelles ne sont pas applicables pour des excavations avec un filtre au fond de l'excavation, parce qu'elles ne prennent pas les courants verticaux au-dessous du pied de la cloison en compte, qui, en ce cas, sont très significatifs. C'est pour cette raison qu'une méthode fiable a été développée sur la base des calculs numériques des courants de l'eau souterraine pour déterminer l'épaisseur nécessaire du filtre de surcharge qui prend les forces des courants en compte. Pour vérifier cette approche et analyser le mécanisme de la défaillance, de nombreux d'essais de laboratoire ont été exécutés et exploités moyennant de différentes méthodes.

KEYWORDS: hydraulic heave, seepage failure, embedment depth, groundwater flow, safety

1 INTRODUCTION

If the water level in an excavation pit is lowered to its base, the difference between the groundwater level outside of the excavation pit and the water level inside of the excavation causes a groundwater flow from the higher to the lower level. This leads to an upward flow from the base of the retaining wall to the pit bottom. In this case, the hydraulic heave safety has to be verified.

If the vertical flow force S in front of the pit wall suspends the buoyant weight of the soil body G' as well as other possible stabilizing forces R (Figure 1), a hydraulic heave will result. This can lead to a rapid flooding of the pit due to regressive erosion around the toe of the wall and eventually to a pit collapse.

Based on the German geotechnical code, the hydraulic heave safety has to be verified according to Terzaghi/Peck (Terzaghi and Peck 1948) or according to Baumgart/Davidenkoff (Baumgart and Davidenkoff 1970). Both approaches simply compare the acting flow forces S with the buoyant weight of the soil G' . Possible friction forces R are neglected. In Terzaghi/Peck's approach a rectangular area is defined as unstable block. The height of the rectangular area corresponds to the embedment depth t of the wall from the bottom of the excavation pit to the toe of the excavation wall. The width of the rectangular area has the half of the embedded depth of the wall $t/2$. The approach according to Baumgart/Davidenkoff uses an unstable block with negligible width and a height corresponding to the embedment depth t of the wall.

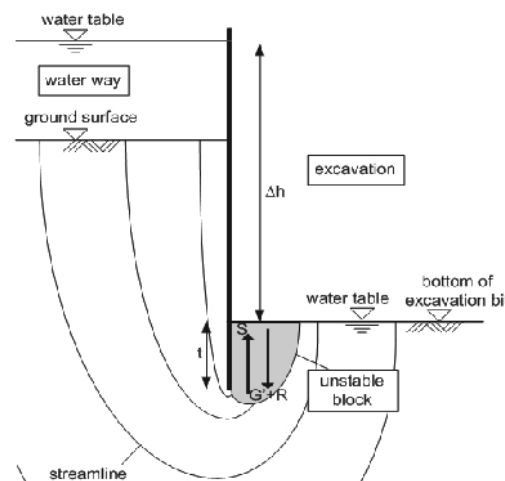


Figure 1. Hydraulic heave at an excavation pit

In both approaches a surcharge filter is considered as an extra load acting at the bottom of the excavation pit and which has no significance for the shape of the unstable block. The magnitude of the extra load corresponds to the weight of the surcharge filter G_F above the unstable block. The surcharge filter only leads to a slight decrease of the water potential.

2 MOTIVATION FOR EXPERIMENTAL ANALYSIS

Motivated by a case of damage at an excavation pit next to a waterway, Odenwald and Herten (Odenwald und Herten 2008) started numerical flow computations respective (?) to the

hydraulic heave safety with surcharge filters. Preliminary they did their calculations according to Terzaghi/Peck's and Baumgart/Davidenkoff's approach and developed a functional relation between $t/\Delta h$ and $d_F/\Delta h$ (t : embedment depth of the pit wall; d_F : thickness of the surcharge filter; Δh : potential difference). A ratio of $\gamma_F/\gamma_W = 1.0$ (γ_F : unit weight of the surcharge filter material; γ_W : unit weight of water) and $\gamma_S'/\gamma_W = 1.0$ (γ_S' : buoyant unit weight of the soil) was assumed. The functional relation is illustrated in Figure 2 (dashed line) for the conventional unstable block according to Baumgart/Davidenkoff's approach.

As expected, the required filter thickness initially rises with constant potential difference Δh and decreasing embedment depth t . However, after reaching a maximum the necessary filter thickness d_F decreases with constant potential difference Δh and continuously decreasing embedment depth t down to zero. Hence, an unstable block that only reaches to the wall toe is inadequate to verify the hydraulic heave safety if a surcharge filter is installed and the embedment depth of the pit wall is small (Odenwald and Herten 2008).

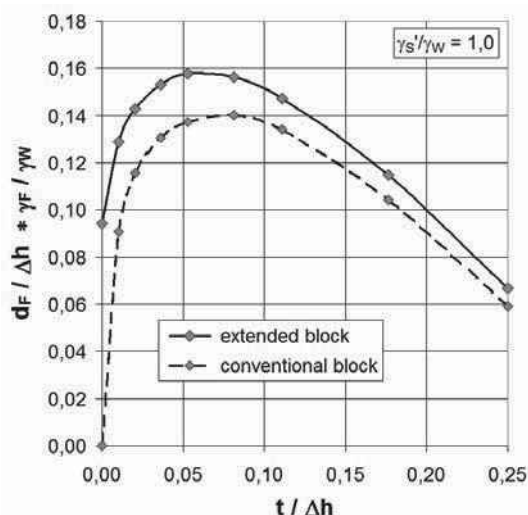


Figure 2. Required thickness of the surcharge filter d_F (Odenwald and Herten 2008)

Therefore, Odenwald and Herten developed an unstable block which additionally considers the vertical flow forces below the wall toe. Hence, the unstable block according to Davidenkoff was extended to the depth where the specific hydraulic gradient i_z (vertical hydraulic gradient) and the limiting gradient i_{gr} (hydraulic gradient corresponding to the ratio γ_S'/γ_W) have the same value ($i_z = i_{gr}$).

Figure 2 (solid line) also shows the results of the computation with the extended unstable block ~~also~~ as a functional relation between $t/\Delta h$ and $d_F/\Delta h$. However, a maximum is also reached here, which means that with constant potential difference Δh , a further reduction of the embedment depth t requires a smaller thickness of the surcharge filter d_F .

To verify the results of the numerical approach with the extended unstable block, numerous large scaled laboratory experiments were performed.

3 SIMULATION OF HYDRAULIC HEAVE

In order to verify and improve these theories, several large-scale experiments to simulate hydraulic heave with a low embedment depth t and installed surcharge filter were carried out at the Institute of Soil Mechanics and Foundation Engineering of the Bundeswehr Universität München. In these experiments, the embedment depth as well as the thickness of the surcharge filter was varied. The aim of the experiments was

to appoint the critical potential difference Δh_{crit} as a function of the embedment depth t and the surcharge filter thickness d_F .

The test rig (Figure 3) consists of two parts: the water supply, which is used to increase the potential difference continuously, and the test box. The rectangular test box has the following dimensions: length x with x height = 1.70 m x 0.40 m x 1.50 m. It mainly consists of 4 acrylic glass walls, a base plate and a vertically moveable partition acrylic panel in the middle of the box. The partition wall simulates the retaining wall in the laboratory test. An inlet connects the box with the water supply. On the feed stream side of the test box, three, 3 cm in diameter, pipes allow free drainage.

Sand (as basic material) and a mixture of coarse sand and fine gravel (as filter material) were used as test materials for the simulations of hydraulic heave with filter layers at the excavation side of the wall.

The behavior of the soil during the experiment was monitored via displacement transducers, water pressure sensors, water quantity measurement and the Particle Velocimetry Method (PIV).

A more detailed description of the employed measurement techniques, the test materials and the construction of the experimental rig is given in (Schober, Boley and Odenwald 2011).

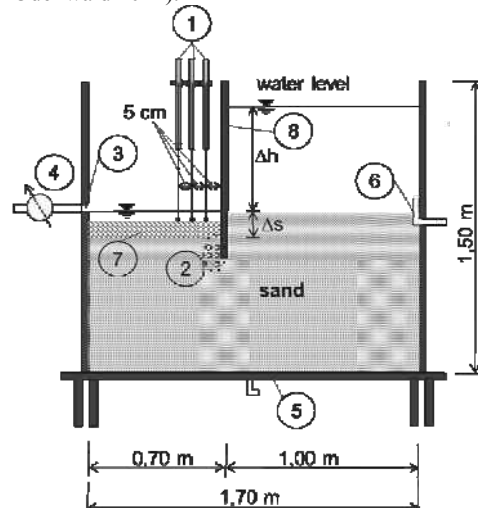


Figure 3. Schematic of experimental rig: (1) displacement transducer, (2) water pressure sensors, (3) outlet, (4) water meter, (5) base plate, (6) inlet, (7) surcharge filter, (8) partition panel

Overall, 24 tests were carried out. The embedment depth was varied between $t = 0$ cm and $t = 5$ cm in 0.5 cm and 1.0 cm steps. Moreover, the surcharge filter was installed in different sizes, with a thickness of $d_F = 2, 4$ and 6 cm.

3.1 Test results

Due to different boundary conditions of the numerical model (Odenwald and Herten 2008) and the experimental rig (Figure 3), it was not possible to compare the results directly to each other. Therefore, it was necessary to adapt the theoretical approach to the boundary conditions of the experiments series. This was done by numerical calculations with the same boundary conditions as the experimental model (Schober and Odenwald 2012).

Figure 4 shows the results of the test series the adapted numerical calculation as a function relation of $t/\Delta s$ and $\Delta h_{crit}/\Delta s$ (Δs : level difference between up and down stream side of the wall, Figure 3).

From Figure 4 it can be seen that the experimental results agree qualitatively well with the results of the numerical calculation. Both the experimentally determined and the

numerically calculated curves reach a minimum at a certain ratio $t/\Delta s$ where the lowest critical potential differences Δh_{crit} arise for a constant thickness of the surcharge filter d_F . Starting from this minimum, further reduction of the embedment depth t enables higher critical potential differences Δh_{crit} . The laboratory experiments confirm the findings of the extended theoretical approach of Odenwald and Herten.

However, the critical potential differences Δh_{crit} achieved during laboratory tests are significantly higher than the critical potential differences Δh_{crit} , calculated by Odenwald & Herten, which lie on the safe side. Moreover, the difference between the results of the experimental series and the theoretical approach increases inversely proportionately with increasing thickness of the surcharge filter (Figure 4).

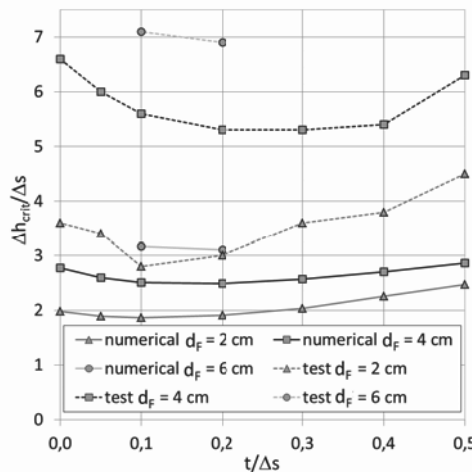


Figure 4. Comparison of test results with the results of the extended theoretical approach

Basically the difference between the numerical calculations and the experimental test results is due to the highly simplified unstable block, which is applied in Odenwald and Herten's calculation approach. Secondly, in the numerical calculations only the weight of the sand and the surcharge filter was recognized as retaining force, frictional forces were neglected.

It has to be kept in mind, that when installing a surcharge filter on the pit bottom frictional forces are acting in the shear zone of the surcharge filter and in the base material in critical condition, too.

In addition, it was found out, that the size of the unstable block increases with increasing surcharge filter thickness. Since, in conventional procedures for verifying the safety against hydraulic heave, the thickness of the surcharge filter is only considered as additional weight in calculations, but with no impact on the shape and size of the unstable block. These methods do not offer any realistic result if a surcharge filter is installed (Figure 5).

To achieve more realistic and therefore, more economic calculation results, an approach that brings the unstable block closer to the realistic failure body and also takes into account the frictional forces in the surcharge filter and in the base material has to be developed.

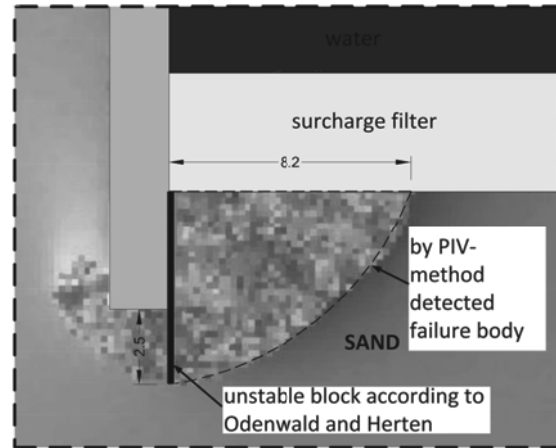


Figure 5. Comparison of test results with the results of the extended theoretical approach

The observation of the experiments and the evaluation of the results also showed that the thickness of the surcharge filter has significant influence on the failure mechanism and shape of the unstable block. In order to analyze the hydraulic heave with surcharge filter more detailed, multiple uniaxial flow-tests were carried out.

4 UNIAXIAL FLOWED SAND COLUMN

To analyze the influence of the surcharge filter thickness on the hydraulic heave more detailed, numerous uniaxial flow-tests were performed. For this purpose a specific apparatus was designed to simulate uniaxial flow through a sand column (Figure 6). A more detailed description of the test materials, the construction of the test rig and the used measurement technique is given in (Schober and Boley 2012).

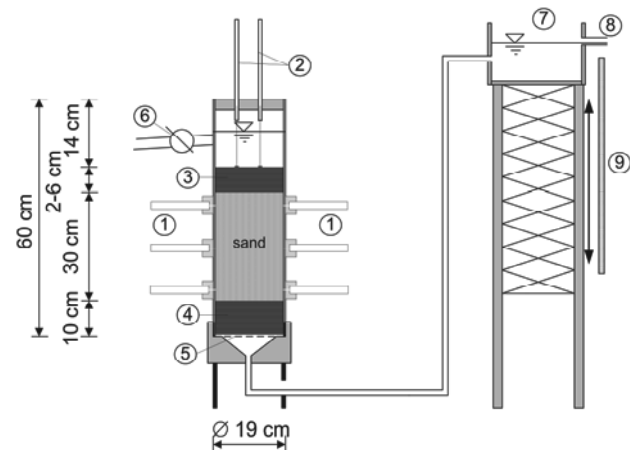


Figure 6. Schematic of flowed cylinder: (1) water pressure sensors, (2) displacement transducers, (3) surcharge filter, (4) filter, (5) perforated plate, (6) water meter, (7) water supply, (8) over fall, (9) staff gauge

The aim of the test series was, among other things, to determine investigate the influence of the surcharge filter thickness d_F and the relative density D of the test material on the critical hydraulic gradient i_{crit} . The results of the uniaxial flow-tests are used for a better understanding of the results of the hydraulic heave experiments.

Altogether, 25 tests were carried out. The relative density was varied between $D = 0.1$, 0.5 and 0.8 . Moreover, the surcharge filter was installed in different sizes, with a thickness of $d_F = 1, 2, 3, 4, 5$ and 6 cm. Tests without a surcharge filter were carried out too. In the test series, different relative

densities of the sand were combined with different sizes of the surcharge filter d_F .

5 TEST RESULTS

The critical hydraulic gradient i_{crit} was determined in each test, so that the dependence of the critical hydraulic gradient i_{crit} on the relative density D and the thickness of the surcharge filter d_F could be shown.

Figure 7 shows the critical hydraulic gradient i_{crit} as function of the surcharge filter thickness d_F for the different relative densities D . Moreover, the figure shows the theoretical critical hydraulic gradient $i_{crit,theoretical}$ calculated according Formula (1) (l : length of the sand column). The figures also display the difference between the test results and the theoretical approach.

$$i_{crit,theoretical} = \frac{d_F \cdot \gamma'_F}{l \cdot \gamma_w} + \frac{\gamma'_s}{\gamma_w} \quad (1)$$

The results of the test series show that the critical hydraulic gradient i_{crit} strongly depends on the thickness of the surcharge filter d_F . With increasing thickness of the surcharge filter d_F , the critical hydraulic gradient i_{crit} increases inverse proportionately.

The relative density D of the sand has a significant influence on the critical hydraulic gradient i_{crit} too. For a sand with a low relative density ($D = 0.1$), the critical hydraulic gradient i_{crit} is smaller than theoretically calculated (Figure 7). The results of the test series with a relative density of $D = 0.5$ and 0.8 show, that the critical hydraulic difference Δh_{crit} and the critical hydraulic gradient i_{crit} respectively is always higher than the gradient theoretically predicted (Figure 7).

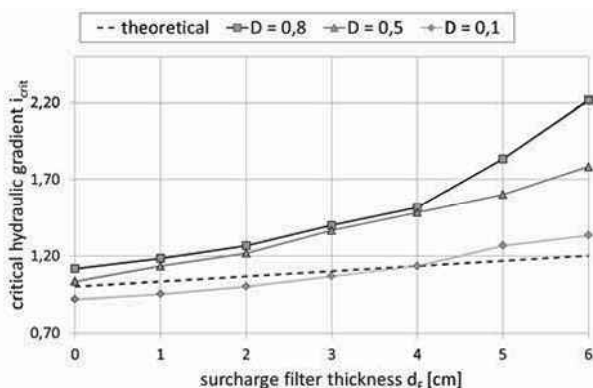


Figure 7. Critical hydraulic gradients of uniaxial flow-test

The disproportional increase of the critical potential difference Δh_{crit} with increasing thickness of the surcharge filter d_F which was detected by the big-scale experiments could be confirmed by the uniaxial flow-tests. The reasons for this behavior will be analyzed in more detail by the Discrete Element Method (DEM) by the Institute of Soil Mechanics and Foundation Engineering of the Bundeswehr Universität München.

The influence of the relative density D on hydraulic heave safety, if a surcharge filter is installed on the pit bottom, needs further investigation by experimental tests. Nevertheless, by the analysis of the uniaxial flow-tests it becomes apparent, that the relative density D of the basic material has a significant influence on the hydraulic heave safety even if a surcharge filter is installed on the pit bottom.

6 CONCLUSIONS

The experiments carried out at the Institute of Soil Mechanics and Foundation Engineering of the Bundeswehr Universität München strives for the verification of the approach with an extended unstable block (Odenwald and Herten 2008). The results of the big scale experiments fit quantitatively to the results of the theoretical analysis. Thus, it could be observed

that there is an unfavorable embedment depth for a pit wall with a surcharge filter on bottom of the construction pit floor. The safety towards hydraulic heave increases when the embedment depth is reduced or enhanced. According to that, the safety towards hydraulic heave can rise due to reducing the embedment depth. The numeric results are however more conservative than the experimental results. This could be due to the simplified failure shape as well as the neglected friction forces in the sand and surcharge filter. Moreover, the difference between the results of the experimental series and the theoretical approach increases proportionally with increasing thickness of the surcharge filter

Furthermore, one dimensional flow-tests were carried out to determine the influence of filter thickness and relative density of the test sand on the critical hydraulic gradient. The experiments showed an inverse proportional correlation between the critical hydraulic gradient and the thickness of the surcharge filter. This was confirmed by the uniaxial flow-tests.

The object of further studies will be to verify and improve the theoretical approach by means of the experimental results towards the hydraulic surcharge and one dimensional flow-tests. The improved approach has to consider friction forces and a more realistic unstable block so that results get closer to reality and the disproportional increase of critical potential difference Δh_{crit} with increasing filter thickness will be considered.

7 ACKNOWLEDGEMENTS

The project is supported by the Federal Waterways Engineering and Research Institute Karlsruhe, Germany.

8 REFERENCES

- Davidenkoff, R. (1970): Unterläufigkeit von Stauwerken. Werner Verlag.
- Odenwald, B., Herten, M. (2008): Hydraulischer Grundbruch: neue Erkenntnisse. Bautechnik, Vol. 85, Heft 9, S. 585 – 595.
- Schober, P., Boley, C., Odenwald, B., (2011): Hydraulic Heave Safety at Excavations with Surcharge Filters. 3rd International Symposium on Geotechnical Safety and Risk (ISGSR). 02. - 03. Juni 2011, Munich, Germany, S. 677 – 684.
- Schober, P., Odenwald, B. (2012): Der Einfluss eines Auflastfilters auf die Bruchmechanik beim hydraulischen Grundbruch. BAWMitteilungen Nr. 95 „Johann-Ohde-Kolloquium“, Karlsruhe, S. 151 – 162.
- Schober, P., Boley, C. (2012): Fracture behavior of uniaxial flowed sand column with surcharge filter. 2nd IAHR, 27. - 29.06.2012, Munich, Germany.
- Terzaghi, K., Peck, R. B. (1948): Soil Mechanics in Engineering Practice, John Wiley and Sons, New York.

Coupled THM mechanical model for porous materials under freezing condition

Modèle mécanique couplé THM pour les matériaux poreux dans des conditions de congélation

Shin H.

University of Ulsan, Republic of Korea

Ahn J.-H.

Pusan National University, Republic of Korea

Kim Y.-T.

Pukyong National University, Republic of Korea

Lee S.-R.

Korea Advanced Institute of Science and Technology, Republic of Korea

ABSTRACT: Recent growing interests associated with frozen ground have required to advance fundamental theories and to precede systematic researches of soil behavior under freezing conditions. Unlike the well-established soil mechanics' theory, temperature variation and phase change of pore-water cause water migration to cold side, ground heaving, sharp increase in earth pressure, etc., and they bring about serious problems to freezing geotechnical structures. Elasto-plastic mechanical constitutive model for frozen/unfrozen soil subjected to fully coupled THM phenomena is formulated based on a new stress variable that is continuous in frozen and unfrozen regions. Numerical simulations are conducted to discuss numerical reliability and applicability of the developed constitutive model. The numerical results show that developed model can efficiently describe complex THM phenomena of frozen soil, and it can be utilized to analyze and design the geotechnical structures under freezing conditions, and predict long-term behavior of them.

RÉSUMÉ : Récents intérêts croissants associés à un sol gelé ont besoin pour faisons avancent des théories fondamentales et de pour précédent recherches systématiques du comportement du sol sous des conditions glaciales. Contrairement à la théorie de la mécanique des sols bien établis, les variations de température et de changement de phase de l'eau interstitielle migration de l'eau à cause de côté froid, soulèvement du sol, forte augmentation de la pression de la terre, etc, et ils produisent de sérieux problèmes à geler les structures géotechniques. Élasto-plastique modèle mécanique constitutive d' frigorifié / soluble sol à charge des phénomènes parfaitement couplés THM est formulé à partir d'une variable nouvelle contrainte qui est continue dans les régions frigorifié et soluble. Des simulations numériques sont accomplies pour discuter de fiabilité numérique et l'applicabilité du développé constitutif modèle. Les résultats numériques montrent que le constitutif modèle peut décrit des phénomènes complexes de THM sol gelé, et cela utilisé pour analyse et dessiné les structures géotechniques dans des glaciales conditions, et de prédire le comportement à long terme d'entre eux.

KEYWORDS: frozen soil, mechanical constitutive model, THM coupling, heaving pressure.

1 INTRODUCTION

Soil behavior during a freezing process has been intensively studied mainly in Canada, the United States, Russia, and Japan from the 1920s. In Korea, a recent Antarctic bases and gas-pipeline negotiations to introduce Russia's natural-gas line with 1100km long have attracted attention to understand fundamental phenomena of natural or artificial frost ground.

Typically, freezing region within soil is divided into frozen zone, frozen fringe, and unfrozen zone in Figure 1. New ice lens formation occurs on the boundary between frozen zone and frozen fringe, called a freezing front. The frozen fringe is the very thin layer with 1mm ~ 10mm thick, and soil type and freezing rate control its thickness. Cryogenic suction due to temperature gradients within the frozen fringe absorbs water from unfrozen zone, and form ice crystals onto the freezing front to separate between the soil particles, called segregation freezing. Konrad and Morgenstern (1981) defined the ratio of a thermal gradient within the frozen fringe to liquid inflow rate as segregation potential, and empirical correlations between segregation potential and basic properties of the soil were proposed to evaluate freezing susceptibility (Konrad, 1999).

Silty soil with relatively high hydraulic conductivity and triggering high suction is prone to freeze and form an ice-lens due to temperature drop. Slowing freezing and resulting low temperature gradient forms thicker ice crystals in the soil (Lawrence, et al., 2005). Konrad and Morgenstern (1982) suggested segregation potential function to consider the effect of an external load on the inflow rate into the freezing fringe due to a thermal gradient. It was known that overburden pressure limits

the amount of heaving, but the presence of shut-off pressure restraining further expansion is still debating (Arvidson & Morgenster, 1977). Electrolyte in the soil pore water reduces total suction, and plays a role to decrease the amount of frost heaves of the soil.

Phase change between liquid water and solid ice in porous material intimately affects deformation characteristics, as water and heat flow do. Thus, fully coupled THM (Thermo-Hydro-Mechanical) phenomena in the porous material require well-established governing equations, and necessitate solving nonlinear equations and complex numerical correlation between constitutive models. Up to date, many numerical studies have been conducted to imitate fundamental characteristics of frozen soil. However, most cases performed combined TH analysis without considering mechanical effects (Tan, et al., 2011; Painter, 2011). Lately mechanical analyses based on the total stress was attempted for frozen soil (Michalowski and Zhu, 2006), and frozen ground was assumed as a linear elastic material and interpretations was carried out (Liu & Yu, 2011; Thomas, et al., 2009), except for the case of Nishimura et al. (2009).

In this paper, THM elasto-plastic mechanical constitutive model is presented to reproduce freezing action in porous material consisted of soil particles, unfrozen water, and ice. The adopted new stress variable represents the sum of ice and soil skeleton stress to maintain continuity across the frozen-unfrozen transition regions, and stress-strain relationship is derived as the form of incremental formulation. After conjunct with pre-developed THM finite element program, THM numerical analyses for freezing process in porous materials were performed to evaluate the performance of the mechanical model.

2 GOVERNING EQUATIONS

2.1. General assumptions and mass balance equations

The governing equations for porous materials under freezing action were assumed the following to define deformation characteristics associated with THM phenomena.

First, the void of soils is fully saturated with water or water/ice. That is, ice and unfrozen water fill the pore under frozen condition, and the void is fully saturated with liquid water above freezing temperature.

Second, porous material consisted of soil particles, water, and ice is under local thermal equilibrium conditions.

Finally, a freezing porous medium, in the context of theory of mixtures, is viewed as a mixed continuum of three independent overlapping phases of solid and liquid (Bear & Bachmat, 1991). For every phase, its mass conservation equations can be obtained according to the principles of continuum mechanics and mixture theory.

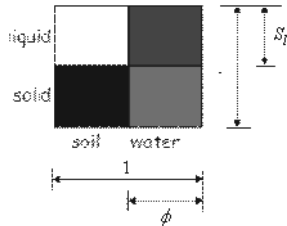


Figure 1. Phase diagram for frozen soil

The macroscopic balance of any species or property per unit volume in a continuum can be expressed by the following generalized partial differential equations (Lewis & Schrefler, 1998).

$$\frac{\partial \rho^w}{\partial t} + \nabla \cdot \tilde{j}^w = 0 \quad (1)$$

where ρ^w is a species in porous material (e.g. soil, water), and ρ^w is mass per unit volume of each species. $\tilde{j}^w$ is mass flux of each species which can include advective and non-advective components.

2.2. Soil mass balance equation

Soil particles only exist in solid-state, so mass balance equation can be summarized as follows: mass density $\rho^s = \rho_s(1-\phi)$, mass flux $\tilde{j}^s = \rho^s \tilde{u} = \rho_s(1-\phi)\tilde{u}$ in Equation 1.

$$\frac{\partial}{\partial t} [\rho_s(1-\phi)] + \nabla \cdot [\rho_s(1-\phi)\tilde{u}] = 0 \quad (2)$$

where ρ_s is mass density of soil particles, ϕ is soil porosity, and $\tilde{u}$ is velocity of soil particles.

2.3. Water mass balance equation

Since water can exist in liquid water or solid ice, the governing equation was derived from generalized law of mass conservation ($\rho^w = \rho_l^w \phi(1-S_l) + \rho_i^w \phi S_l$). Solid ice is assumed to be an immobile substance which can make phase change from solid to fluid.

$$\frac{\partial}{\partial t} [\rho_l^w \phi(1-S_l) + \rho_i^w \phi S_l] + \nabla \cdot [\rho_l^w \phi(1-S_l)\tilde{u} + \rho_l^w \phi S_l \tilde{u} + \rho_l^w \tilde{q}] = 0 \quad (3)$$

where ρ_l^w is mass density of liquid water, S_l is degree of liquid saturation in the void of material. $\tilde{q}$ is flow rate of liquid water from Darcy's law.

2.4. Energy balance equation

Although the energy balance is expressed by enthalpy balance in most cases, it is preferable to express it in terms of internal energy (Olivella, et al., 1996; Lewis, et al., 1998). If thermal equilibrium between phases is assumed, the temperature is the same in all phases and only one equation of total energy is required. Adding the internal energy of each phase, the total internal energy per unit volume of the porous medium becomes,

$$\begin{aligned} & \frac{\partial}{\partial t} [(1-\phi)\rho_s E_s + \phi(1-S_l)\rho_l^w E_i^w + \phi S_l \rho_l^w E_l^w - \phi(1-S_l)\rho_l^w L_f] \\ & + [(1-\phi)\rho_s E_s + \phi(1-S_l)\rho_l^w E_i^w + \phi S_l \rho_l^w E_l^w - \phi(1-S_l)\rho_l^w L_f] \nabla \cdot \tilde{u} \\ & + \nabla \cdot [\tilde{i}] + \nabla \cdot [\rho_l^w E_l^w \tilde{q}] = 0 \end{aligned} \quad (4)$$

where E_s is internal energy of soil per unit mass, and E_i^w is internal energy of water in solid phase per unit mass. Energy transfer by heat conduction in porous materials was estimated using from the Fourier's law ($\tilde{i} = -\lambda \nabla T$). Last term $-\phi(1-S_l)\rho_l^w L_f$ in first partial derivative represents internal energy loss due to water phase change (Thomas, 2009; Tan, 2011; Jane, 1980).

2.5. Static equilibrium equation

Neglecting the inertial effects over all phases, the momentum conservation equation reduces to the static stress equilibrium based on the total stress.

$$\nabla \cdot \tilde{\sigma} + \rho_m \tilde{g} = 0 \quad (5)$$

where $\tilde{\sigma} = \sigma_{ij}$ ($i, j = 1, 3$) is total stress, average mass density is $\rho_m = (1-\phi)\rho_s + \phi(1-S_l)\rho_l^w + \phi S_l \rho_l^w$, and gravity direction is $\tilde{g}_i = [0, 0, -1]$.

2.6. Numerical implementation

Substituting Eq. 2 into Eq. 3 and 4, the differential equation governing non-isothermal liquid flow through frozen-nonfrozen porous material is obtained. The primary variables are displacement components $\tilde{u}$, liquid pressure P_l , and temperature T from fully coupled governing equations. The material derivative with respect to the solid velocity field will be very useful to obtain the final expressions for balance equations and equilibrium equation. The material derivative relative to the a phase is given by

$$\frac{d}{dt}(\bullet) = \frac{\partial}{\partial t}(\bullet) + \nabla \cdot (\bullet) \cdot \tilde{u} \quad (6)$$

Generalized trapezoidal rule (Eq. 7) is used to perform time integration between $t^{(n)}$ and $t^{(n+1)}$ of coupled governing equations, and they use discrete approximations to take advantage of Newton's iterative process.

$$\begin{aligned} \int_{t^{(n)}}^{t^{(n+1)}} P_l dt & \approx (t^{(n+1)} - t^{(n)}) [\theta P_l^{(n+1)} + (1-\theta)P_l^{(n)}] \\ & \approx \Delta t [\theta P_l^{(n+1)} + (1-\theta)P_l^{(n)}] = \Delta t [P_l^{(n)} + \theta \Delta P_l + \theta \Delta P_l] \end{aligned} \quad (7)$$

where θ is an integration parameter to govern stability and accuracy of the solution, and the solution is unconditionally stable if $\theta \geq 1/2$.

A volume integration of all governing equations then leads to a weighted residual approximation to the governing equations, based on the Galerkin method. After all governing equations are discretized, the final system of algebraic equations can be expressed in matrix form with respect to primary variables $\mathbf{D}(u, P_g, P_l, T)$.

$$\mathbf{K}^{(n,i)} d\mathbf{D}^{(n,i+1)} = (\mathbf{F}_{\text{EXT}})^{(n+1)} - \mathbf{F}_{\text{INT}}(\mathbf{D}^{(n,i)}) \quad (8)$$

where $\mathbf{K}^{(n,i)}$ is non-symmetric tangential stiffness matrix at $t^{(n,i)}$, and the right term is residual load vectors.

3. MECHANICAL CONSTITUTIVE MODEL

The proposing mechanical constitutive model is based on the following assumptions to define deformation characteristics associated with THM phenomena.

First, the void of soils is fully saturated with water or water/ice. That is, ice and unfrozen water fill the pore under frozen condition, and the void is fully saturated with liquid water above freezing temperature.

Second, porous material consisted of soil particles, water, and ice is under local thermal equilibrium conditions

Based on these assumptions, this study proposes a mechanical constitutive model for two-phase porous materials, with a similar framework of BBM (Barcelona Basic Model) for unsaturated soils (Alonso, et al. 1990). Stress and strain are positive value for compression.

3.2. Stress variables

Mechanical constitutive law to describe deformational behavior can be expressed by various stress variables. In saturated soils, only effective stress is a stress variable to determine soil behavior (Terzaghi, 1936). However, selection of stress variables in unsaturated soils is still critical issue (Shin, 2011). The most previous studies for frozen soils performed TH coupled analysis so that alteration of hydraulic and thermal properties from deformation cannot be considered, or performed simple linear-elastic analyses for frozen soil. Recently, Nishimura et al. (2009) used net stress $p_n = p - \max(P_i, P_l)$ and suction $s = \max(P_i - P_l, 0)$ to simulate nonlinear deformation behavior of frozen ground. However, ice pressure P_i below freezing temperature shows such a rapid increase that it could be greater than the sum of surface loads, soil skeletal force and water pressure. These stress variables can lose physical meaning and become discontinuous across freezing temperature, and it inhibits numerical stability, along with emission of latent heat of fusion.

In the developed model, a new stress variable $\bar{\sigma}$ defines the sum of soil skeletal stress and ice stress in Eq. (9), and internal variable suction S (difference between ice pressure and fluid pressure) only affect the pre-consolidation stress.

$$\bar{\sigma} = \sigma' + (1 - \chi) P_i \quad 1 + \chi P_l = \bar{\sigma} + \chi P_l \quad (9)$$

where $\bar{\sigma} = \sigma_{ij} (i, j = 1, 3)$ is total stress, σ' is effective stress carrying soil skeletal force, P_i is ice pressure from Clausius-Clapeyron equation, and P_l is fluid pressure. $1 = \delta_{ij}$ is Kronecker's delta tensor, and χ is Bishop's coefficient of effective stress which can be simplified to degree of liquid water saturation. The new stress variable $\bar{\sigma} [= \sigma' + (1 - \chi) P_i]$ is the same as effective stress above the freezing temperature. It can be useful stress variable to develop mechanical constitutive model for unsaturated freezing soils including air and gas phase.

3.3. Yield surface and plastic potential function

New stress variable $\bar{\sigma}$ is used to define yield surface and plastic potential function which describe deformational behavior of frozen and unfrozen soils. The proposed yield function is based on MCC (Modified Cam Clay) model which has an oval

shape of yield surface, taking account of bonding strength due to ice (Figure 2).

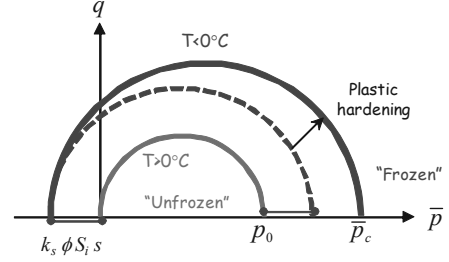


Figure 2. Yield surface of porous material in frozen and unfrozen regions

The increase in the bonding strength is assumed to be equal in the direction of compression and tension, and be proportional to strength ratio k_s (Nishimura, et al., 2009) and pore volume fraction of ice $\phi S_i = \phi(1 - S_l)$.

$$f = q^2 - M^2 (\bar{p} + k_s \phi S_i s) (\bar{p}_c - \bar{p}) = 0 \quad (10)$$

where pre-consolidation pressure can be expressed as

$$\bar{p}_c = p_r \left(\frac{p_0}{p_r} \right)^{\frac{\lambda(0) - \kappa(0)}{\lambda(s) - \kappa(s)}} + k_s \phi S_i s \quad (\text{Alonso, et al. 1990}).$$

Above freezing temperature, $\bar{p}_c$ is the same as p_0 , however it evolves nonlinearly below freezing temperature from suction and ice pressure P_i development. p_r is reference stress, and M is the slope of critical state line in $\bar{p}$ - q space. Compression index at various suction is $\lambda(s) = \lambda(0) [1 - r] \exp(-\beta s) + r$, and swelling index $\kappa(s)$ is assumed to have the same function of suction as compression index for numerical stability.

Mechanical plastic potential function determining incremental direction of plastic deformation follows associated flow rule $g = f$, and strain-rate-dependent characteristic of frozen soil is not considered in this model.

3.4. Strain components

Total strain increment can be divided into mechanical strain from stress variable, thermal strain, and phase change between liquid water and solid ice in the pore. In addition, mechanical strain increment has elastic and plastic mechanical strain parts.

$$d\bar{\epsilon} = d\bar{\epsilon}^{<m>} + d\bar{\epsilon}^{<T>} + d\bar{\epsilon}^{<ph>} = d\bar{\epsilon}^{e<m>} + d\bar{\epsilon}^{p<m>} + d\bar{\epsilon}^{<T>} + d\bar{\epsilon}^{<ph>} \quad (11)$$

where elastic mechanical strain increment is $d\bar{\epsilon}^{e<m>} = D^{e-1} : d\bar{\sigma}$ (Sheng et al., 2008), and

$D^e = (K - 2G/3) 1 \otimes 1 + 2GI$ is (elastic stiffness tensor). K and G are elastic bulk and shear modulus, and $1 \otimes 1 = \delta_{ij} \delta_{kl}$, $I = (\delta_{ik} \delta_{jl} + \delta_{il} \delta_{jk}) / 2$.

Temperature-dependent volumetric change in elastic region is determined from thermal elastic contractile coefficient (α_T),

$d\bar{\epsilon}_v^{<T>} \approx d\bar{\epsilon}_v^{e<T>} = \frac{\partial \bar{\epsilon}_v^e}{\partial T} dT = 3\alpha_T dT$. And volumetric strain due to phase change is the following from conservation of water mass.

where ρ_l^w and ρ_i^w are mass density of water in liquid and solid phase, and strain due to phase change can be expressed as

$d\tilde{\varepsilon}^{<ph>} = a^{<ph>} dT + b^{<ph>}$ (Liu & Yu, 2011). Frost heave can be inhibited by overburden stress (Konrad, 2005; Michalowski & Zhu, 2006). This sensitivity of volumetric expansion to overburden stress can be expressed as $d\tilde{\varepsilon}_i^{<ph>} \propto \sigma_i^{-a}$, conserving increment of volumetric strain from Eq. (11).

Incremental relations of the effective stress and strain can be expressed as follows:

$$d\tilde{\sigma} = D^e : [d\tilde{\varepsilon} - d\tilde{\varepsilon}^{p<m>} - d\tilde{\varepsilon}^{<T>} - d\tilde{\varepsilon}^{<ph>}] \quad (12)$$

Plastic flow rule determines increment of mechanical plastic deformation in a direction perpendicular to plastic potential function with a magnitude of non-negative scalar multiplier $d\lambda$.

$$d\tilde{\varepsilon}^{p<m>} = d\lambda \frac{\partial g}{\partial \tilde{\sigma}} \quad (13)$$

3.5. Hydraulic characteristics of frozen soils

Darcy's law using the slope of total head has been adopted to describe fluid flow in porous material. Many experimental results on frozen soils showed liquid flow in the direction to lower temperature even at the same total head (Hoekstra, 1966; Mageau & Morgenstern, 1980). Water flow due to thermal gradient could be estimated by introducing cryogenic suction from water-fluid interfacial tension (Thomas et al., 2009; Hansson et al., 2004; Liu & Yu, 2011) or segregation potential (Tan et al., 2011).

This study used segregation potential method which can directly calculate thermal water flow through the laboratory tests. And Darcy's law considering abundance of liquid water in frozen soils can be summarized as follows.

$$q_l = -\frac{k_l}{\gamma_l} \left(\nabla P_l - \gamma_l g \right) - \phi S_l SP_0 \nabla T \quad (14)$$

where SP_0 is segregation potential, the ratio of fluid velocity of unfrozen water to thermal gradient (Konrad & Morgenstern, 1981). The effect of applied stress is well considered in the variation of porosity as a function of mean stress (Konrad & Morgenstern, 1984).

The relationship between temperature and volumetric water content in frozen soil is called freezing characteristic function. It can be estimated from empirical equation through laboratory experiments (12a, Andersland & Ladanyi, 2004) and using similarity with SWCC in unsaturated soils (22b, van Genuchten, 1980; Nithimura et al., 2009; Tan et al., 2011). Empirical formula (22a) was partially modified due to inordinate prediction in temperature range $-1.0^\circ C < T < 0.0^\circ C$ (Thomas, et al., 2009).

$$S_l = [1 - (T - T_0)]^\alpha \quad (15a)$$

$$S_l = \left[1 + \left(\frac{P_l - P_0}{P_0} \right)^{\frac{1}{1-\lambda}} \right]^{-\lambda} \quad (15b)$$

where T_0 is freezing temperature of water, and α is determined from pore size and chemical composition of the pore fluid (Thomas et al., 2009). P_0 and λ are material parameters in van Genuchten model (van Genuchten, 1980), and .

Ice pressure P_i in frozen soil can be calculated from Clausius-Claperyron (Eq. 23), assuming that thermodynamic equilibrium is satisfied at the contact surface between ice and liquid water in the pore (Henry, 2000).

$$P_i = \frac{\rho_i^w}{\rho_l^w} P_l - \rho_i^w L_f \ln(T/273.15) \quad (16)$$

where L_f is specific latent heat of fusion of water, $3.34 \times 10^5 J/kg$.

4. CONCLUSIONS

Recent increasing interest in the frozen soil has raised the demand to advance theoretical establishment and numerical tools to interpret fully coupled thermal-hydro-mechanical phenomena in naturally or artificially freezing ground. However, previous numerical analysis of freezing soil usually disregarded mechanical characteristics or assumed freezing porous material as a linear elastic material.

In this paper, THM elastoplastic constitutive model for porous materials is derived for the frozen-unfrozen soil consisted of soil particles, unfrozen water, and ice. The new stress variable, the sum of skeletal stress and ice pressure, has continuity in the frozen-unfrozen transition condition, and it can easily be applicable to unsaturated freezing soil. Stress increment due to strain and temperature change was derived as the form of incremental formulation. Proposed mechanical model was implemented into THM FEM code, and it was applied to numerical examples to confirm stability of solution and applicability of the model. Numerical results effectively described complex THM phenomena related with frozen soil, where governing equations had high nonlinear and constitutive models were inter-coupled.

5. REFERENCES

- Alonso, E. E., Gens, A., and Josa, A. (1990), "A constitutive model for partly saturated soils", *Géotechnique*, Vol.40, No.3, pp.405-430.
- Andersland, O. B., Ladanyi, B. (2004), "Frozen Ground Engineering", John Wiley & Sons.
- Bear, J., Bachmat, Y. (1991), "Introduction to modeling of transport phenomena in porous media", Kluwer Academic Publisher, p. 553.
- Konrad, J. M., Morgenstern, N. R., (1981), "The segregation potential of a freezing soil", *Can. Geotech. J.*, Vol.18, pp.482-491.
- Nishimura, S., Gens, A., Olivella, S. and Jardine, R. J. (2009), "THM-coupled finite element analysis of frozen soil: formulation and application", *Géotechnique*, Vol.59, No.3, pp.159-171.
- Shin, H. S. (2011), "Formulation of Fully Coupled THM Behavior in Unsaturated soil", *Journal of Korean Geotechnical Society*, Vol.27, No.3, pp.75-83.
- Terzaghi, K. (1936), "The shear resistance of saturated soils", *Proceedings for the 1st. International Conference on Soil Mechanics and Foundation Engineering*, Cambridge, MA, pp.54-56.
- Van Genuchten, M. Th. (1980), "A closed-form equation for predicting the hydraulic conductivity of unsaturated soils", *Soil science society of America journal*, Vol.44, pp.892-898.

Correlation between drained shear strength and plasticity index of undisturbed overconsolidated clays

Corrélation entre la résistance au cisaillement des sols drainés et l'indice de plasticité des argiles surconsolidées non perturbées

Sorensen K.K.

The Danish Geotechnical Institute (GEO) / Department of Engineering, Aarhus University

Okkels N.

The Danish Geotechnical Institute (GEO)

ABSTRACT: A number of triaxial compression tests have been performed by The Danish Geotechnical Institute over the past decades on undisturbed overconsolidated Danish clays; ranging from clay till of low plasticity to extremely high plasticity marine Tertiary clays. The test results confirm that the drained peak angle of shearing resistance can be related to the plasticity index, though a large scatter is generally seen. Based on the results and a review of published data a conservative relationship between drained peak angle of shearing resistance and plasticity index for undisturbed overconsolidated clays is proposed. The proposed relationship and the interrelation to the effective cohesion are discussed.

RÉSUMÉ: De nombreux essais de compression triaxiale ont été effectués par l'Institut danois de géotechnique au cours des dernières décennies sur les argiles danoises surconsolidées non perturbées, allant des argiles morainiques ayant une plasticité faible à des argiles marines tertiaires ayant une plasticité extrêmement élevée. Les résultats des essais confirment que l'angle de résistance maximale au cisaillement des sols drainés peut être lié à l'indice de plasticité si une forte dispersion est généralement observée. Sur la base de ces résultats et à partir d'un ensemble de données déjà publiées, une relation conservatrice entre l'angle de résistance maximale au cisaillement des sols drainés et l'indice de plasticité pour les argiles surconsolidées non perturbées est proposée. La relation proposée et les liens à une cohésion effective sont discutés.

KEYWORDS: Plasticity index, Drained shear strength, Laboratory testing, Overconsolidated clay

1 INTRODUCTION

Empirical correlations are widely used in geotechnical engineering practice as a tool to estimate the engineering properties of soils. Useful correlations exist between the index properties obtained from simple routine testing and the strength and deformations properties of cohesive soils among others. For practical purposes the results of routine index tests and correlations can be used as a first approximation of the soil parameters for use in preliminary design of geotechnical structures, and later as a mean to validate the results of laboratory tests. Results from several index tests obtained for a given site can be used to assess the variation in the properties of the soil mass.

This study is aiming to provide a conservative correlation between the effective peak angle of shearing resistance and plasticity index I_p for natural undisturbed overconsolidated Danish clays based on the results from a large database of triaxial compression tests performed by The Danish Geotechnical Institute (GEO) over the past decades.

1.1 Stress-strain behavior and effective strength of overconsolidated clays

In contrast to normally consolidated (NC) clays highly overconsolidated (OC) natural clays typically show a distinct strain softening behavior in drained triaxial compression, which can be related to the breakdown of interparticle bonding and the dilatant behavior of the compact clay structure (Burland 1990). With further shearing in the post peak region OC clays (as well as NC clays) may experience an additional reduction in strength due to the alignment of the platy clay particles on the failure plane (residual state). The stress-strain behavior of OC clays compared to NC clays is illustrated in Figure 1a.

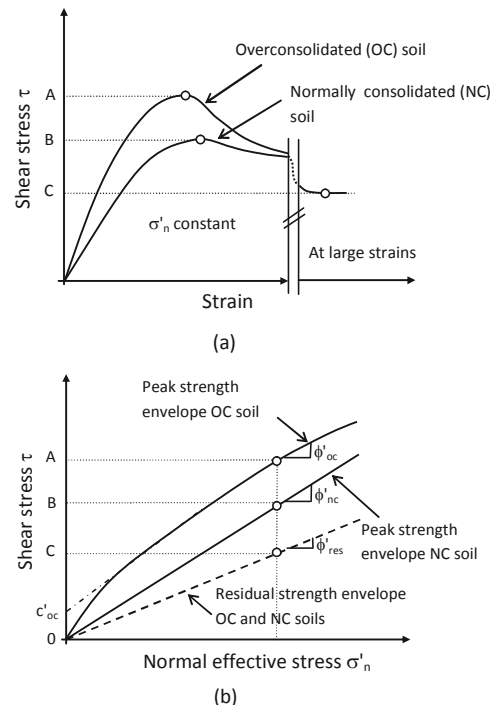


Figure 1. Peak and residual shear strength for normally consolidated and overconsolidated soils (a) Typical stress-strain curves (b) Failure envelopes showing the associated angle of shearing resistance, ϕ'

Stiff fissured overconsolidated clays may fail along preexisting fissures in which case the strength is governed by the fissure strength. Skempton (1957) found the fissure strength to correspond to the fully softened strength i.e. shear strength of the reconstituted normally consolidated soil, which is less than

the peak strength of the OC clay, but greater than the residual strength.

It is generally accepted that the effective strength of uncemented saturated clays is frictional and the strength envelope is nonlinear. Hence the strength envelope will pass through origin, and so the true cohesive intercept $c'=0$ kPa (Burland 1990). However, over the typical range of stress levels met in practice (~ 50 – 400 kPa) the effective peak strength for NC and OC clays can be approximated by a linear relationship between effective normal stress at failure σ'_f and shear strength τ_f using the Mohr-Coulomb strength equation:

$$\tau_f = \sigma'_f \tan(\phi') + c' \quad (1)$$

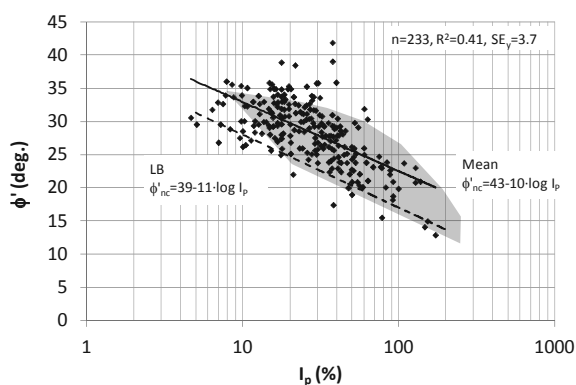
where ϕ' and c' respectively are the tangent drained angle of shearing resistance and the apparent cohesive intercept, as illustrated in Figure 1b. For OC clays $c'_{oc} > 0$ kPa and for NC clays $c'_{nc} \approx 0$ kPa. While the angles of shearing resistance for OC and NC clays are typically found not to differ much.

Generally the frictional resistance to shearing, as expressed by ϕ' , can be expected to decrease as the content of platy clay minerals increase in the soil mass. With increasing content of platy clay particles both the liquid limit w_L and the plasticity index I_p will increase, and hence a correlation between ϕ' and w_L or I_p can be expected.

1.2 Existing relationships between effective shear strength and plasticity index

Several studies have been reported in the literature with regards to the correlation between the effective angle of shearing resistance ϕ'_{peak} and plasticity index I_p (Brooker and Ireland 1965, Ladd et al. 1977, Stark and Eid 1997, Terzaghi et al. 1996 among others). These studies are however mainly focused on normally consolidated reconstituted or undisturbed natural clays, while only little has been reported for overconsolidated undisturbed clays.

Figure 2 shows collected data from the literature in a plot of ϕ'_{nc} vs. I_p (single log plot) for primarily NC clays (I_p range 5–240%). ϕ'_{nc} generally represent a peak secant value with the assumption that c'_{nc} is zero. A very significant scattering of the data points is seen, e.g. at $I_p=20\%$ the value of ϕ'_{nc} is found to vary between approximately 25deg. and 35deg. However,



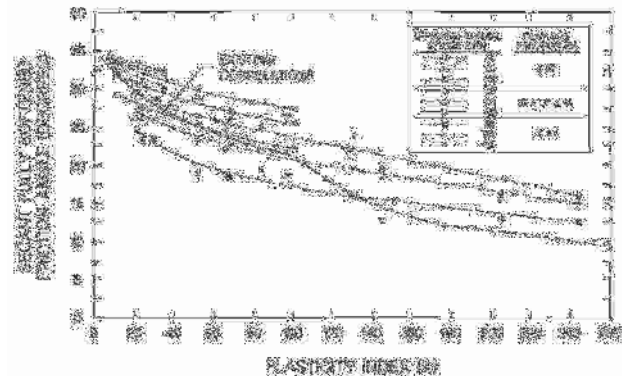
despite the significant scatter a trend of reducing ϕ'_{nc} with increasing I_p is seen, and the data furthermore suggest the existence of a lower bound value for ϕ'_{nc} at given value of I_p .

Figure 2. ϕ'_{nc} vs. I_p for primarily normally consolidated reconstituted and undisturbed clays after Ladd et al. 1977 (with data from Kenney 1959 and Bjerrum and Simons 1960), Terzaghi et al. 1996 and Brooker and Ireland 1965.

The shaded area in Figure 2 represents the range of results reported by Stark and Eid 1997 from a large series of ring shear tests on 24 different reconstituted normally consolidated natural soils ($I_p=8$ – 244% , Clay-size fraction $CF=10$ – 88% , normal effective stress $\sigma'_n=50$ – 400). Based on the data, relationships

between ϕ'_{nc} and I_p were proposed which were dependent on clay-size fraction and normal effective stress, as seen in Figure 3. By taking account of clay-size fraction and stress level Stark and Eid showed a significantly reduced scatter around the mean trend lines. A downward shift in the trend lines were observed with increasing stress levels and increasing clay-size fraction. The findings by Stark and Eid suggests that the observed scatter in the reported data found in the literature, as shown in Figure 2, to a large extent can be explained by variations in stress level due to a non-linear strength envelope and additionally clay-size fraction, as both soil mineralogy and clay-size fraction are not accounted for solely by the variation in the index properties.

Figure 3. ϕ'_{nc} vs. I_p for reconstituted normally consolidated soils as a



function of clay-size fraction and normal effective stress (Stark and Eid 1997)

Based on the literature data a cautious lower bound (LB) estimate of the relationship between ϕ'_{nc} and I_p for NC clays can be derived together with a best estimate from the best-fit regression line through the data, as indicated in Figure 2 and given below.

$$\text{Cautious LB estimate: } \phi'_{nc} = 39 - 11 \cdot \log I_p \text{ (deg.)} \quad (2)$$

$$\text{Best estimate: } \phi'_{nc} = 43 - 10 \cdot \log I_p \text{ (deg.)} \quad (3)$$

The lower bound estimate, which correspond roughly to the 5% fractile, also approximately match the lower bound of the range of results reported by Stark and Eid for clay-size fractions above 50% and a stress level of 400 kPa. Hence for clay-size fractions below 50% and stress levels below 400 kPa the effective angle of shearing resistance ϕ'_{nc} can be expected to be significantly greater than estimated from eq. 2 (up to approximately 12deg. for $CF < 20\%$ and $\sigma'_n = 50$ kPa, as seen from Figure 3).

2 SOIL DESCRIPTION AND TEST PROCEDURES

A number of triaxial compression tests have been performed by GEO on various undisturbed overconsolidated clays over the past decades. Test data have been collected from older tests (> 30 years) and more recent test series as listed in Table 1.

2.1 Soil description

The tested soils range from very low plasticity clay tills to extremely high plasticity Eocene clays. The recent tests include a test series in connection to the 1992 Great Belt bridge (GB) ground investigation, which provides a significant amount of test data for very low plasticity clay till. While the newly completed 2011 Fehmarnbelt (Fixed Link) (FB) ground investigation contribute significantly to the understanding of the strength behavior of very high to extremely high plasticity Eocene and Paleocene marine clays from the Røsnes, Ølst and Holmehus clay formations. The majority of the investigated clays from the Fehmarnbelt (Fixed Link) ground investigation have been assessed to be situated within glacial folded strata. A

series of recent tests in connection to a ground investigation at Esbjerg Harbour (EB) highlights the strength properties of Mica clay – a Miocene marine clay of high plasticity.

The additional old test data covers glacial clay till, a few glacial/late glacial meltwater and late glacial marine clays and furthermore a wide range of Tertiary (Palaeogene) marine clays of late Miocene age to Palaeocene age: Mica clay, Septarian clay, Søvind marl, Lillebælt clay, Røsnes clay, Ølst clay, Tarras clay and Holmehus clay.

The majority of the very high plasticity Palaeogene clays from the listed test series are found to be fissured in nature.

Figure 4 shows the outline of the index properties of the tested clays in Casagrandes classification chart. The classification parameters have generally been determined in accordance with BS 1377:Part 2:1990 using the Casagrande method to determine the liquid limit. The data points for the different soils generally fall close around straight lines and above the A-line corresponding to clays of very low ($4\% < I_p < 7\%$) to extremely high plasticity ($I_p > 100\%$).

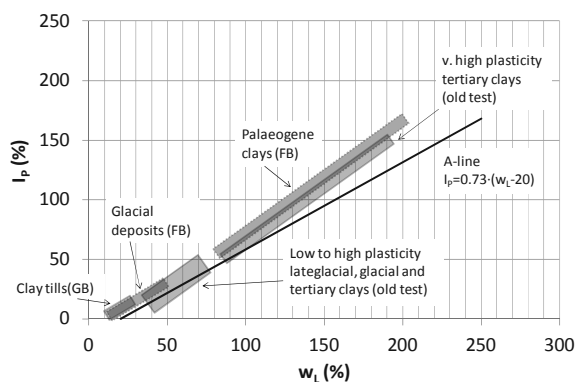


Figure 4. Outline of index properties of the tested clays shown in Casagrandes classification chart I_p vs. w_L .

The classification parameters incl. clay-size fraction (CF) and natural water content w_n for the tested clays are also listed in Table 1 alongside test types and number of tests (n). For the consistent test series (GB, FB and EB) the mean values of the classification parameters are listed and the standard deviations are shown in brackets. A more detailed description of the Palaeogene clays incl. mineralogy is given by Fehmarnbelt (Fixed Link) 2011.

2.2 Test procedures

The recent triaxial compression tests have generally been performed on nominally undisturbed specimens with a diameter of approximately 70mm and a height to diameter ratio of 1. Smooth end platens have been used. Samples have been extracted by means of a push-in Shelby-tube sampler (called A-tube in Denmark) with an inner diameter of 70mm.

Samples are saturated using backpressure and typically preloaded to reduce the effects of possible sample disturbance and bedding effects. The preloading are in most cases performed under anisotropic K_0 conditions. After the preloading and unloading of the sample, the specimen is sheared in either drained or undrained compression to failure. Multiple tests are carried out, consisting of preloading, unloading to a new and higher stress level followed by a drained or undrained compression test (these steps are repeated 2 or more times).

The applied rate of straining during shearing is chosen according to the soil type to ensure slow enough rates to achieve full equalization of pore water pressures in the specimen. For the high plasticity clays this means that the applied strain rate is around 0.02-0.05%/hr, while a strain rate of approximately 0.1-0.7%/hr has been used for specimens of clay till.

For specimens of very high plasticity clay care is generally taken during all test stages to avoid that the effective stress reduces below the in-situ stresses to prevent swelling, which

may lead to destructuration of the micro-structure (Leroueil and Vaughan 1990).

Older triaxial compression tests have in contrast typically been performed with a height to diameter ratio of 2:1 and a diameter of approximately 36 mm. Samples were usually extracted using a 42mm inner diameter sampler. Preloading was in some cases carried out under isotropic conditions and in some cases omitted. Saturation was in most cases carried out without the application of backpressure and compression was performed with the pore water pressure kept equal to zero. Hence, undrained compression was achieved by adjusting the cell pressure during testing to obtain constant volume displacement.

The applied rate of straining during the compression stage was generally higher than what would be recommended today to ensure full equalization of pore water pressures within the samples of especially high plasticity clays. Hence, the actual effective stresses at the failure state are somewhat uncertain because the pore water pressure is unknown. Nevertheless, the old triaxial tests constitute a very comprehensive database of strength parameters for low to high plasticity clays, which can be compared to the recent and presumably more reliable tests results.

Table 1. Overview of classification parameters and larger series of triaxial compression tests on undisturbed overconsolidated clays performed by GEO.

Project	Soil type	$w_n^{\#}$ [%]	$w_L^{\#}$ [%]	$I_p^{\#}$ [%]	$CF^{\#}$ [%]	Test types	n
GB (1992)	Clay till	11 (2)	16 (2)	6 (2)	-	MCAU _{u=0} and CAU	45
	Upper till	9 (-)	19 (1)	7 (1)	26 (9)	CAU and CAD	5
	Chalk till	9 (-)	20 (1)	6 (0)	23 (-)	CAU and CAD	5
	Lower till	13 (5)	28 (4)	16 (3)	24 (3)	CAU and CAD	8
FB (2011)	Røsnes	35 (4)	147 (28)	117 (27)	70 (7)	CAU and CAD	48
	Ølst	45 (11)	140 (25)	106 (23)	51 (15)	CAU and CAD	5
	Holmehus	42 (6)	133 (23)	98 (25)	61 (7)	CAU and CAD	5
	Palaeogene clays	37 (6)	145 (27)	114 (27)	67 (9)	CAU and CAD	58
	Mica clays	28 (3)	58 (9)	36 (8)	-	MCAU, MCAD, MCIU	10
Other recent test	Tertiary clays	-	-	19-85	-	MCAD CAU CU	8
GEO old test (> 30 yrs)	(Late) Glacial & Tertiary clays	-	-	5-151	-	CAU _{u=0} CAD _{u=0} CU _{u=0} CD _{u=0}	108

[#] mean values with the standard deviation shown in brackets

An overview of the number of tests (n) and test types are given in Table 1. The following abbreviations are used:

- CAU/CAD Anisotropically (K_0) Consolidated. Undrained/Drained compression.
- MCAU/MCAD Anisotropically (K_0) Consolidated. Undrained/Drained compression. Multiple test on the sample.
- CU/CD Isotropically Consolidated Undrained /Drained compression.
- u=0 denotes older test procedures with no backpressure and pore water pressure kept at zero kPa.

The drained strength parameters; angle of shearing resistance ϕ' and effective cohesion c' can be derived from the results of two or more compression tests (either using multiple testing on the same sample or sets of two or more compression tests on samples with similar properties). Alternatively, the strength parameters can be interpreted from the undrained compression effective stress path, since the effective stress path for overconsolidated clays will tend to climb the strength envelope as the soil dilates and the pore water pressures decrease.

Generally, test interpretation may be difficult in cases where the specimen experiences destructuration during testing or if the sample is fissured.

3 DRAINED SHEAR STRENGTH AND PLASTICITY INDEX

3.1 Drained peak angle of shearing resistance

Figure 5 shows the relationship between the drained peak angle of shearing resistance ϕ'_{oc} and the plasticity index I_p (single log plot) as derived from triaxial compression tests performed by GEO on the various overconsolidated undisturbed clays shown in Table 1.

ϕ'_{oc} has generally been derived as a tangent value, to minimize the otherwise high influence of stress level resulting from the initially curved failure envelope. Hence, both values of ϕ'_{oc} and c'_{oc} are obtained from the tests. Results from the older tests (> 30 years, open points) are shown separately from the more recent tests (closed points).

The shaded area and dashed lines respectively represents results of the series of triaxial compression tests on palaeogene clays and glacial till deposits (Lower and Upper till) performed in connection to the Fehmarnbelt (Fixed Link) 2011 ground investigation. The spans shown in the I_p and ϕ'_{oc} values represent mean values ± 1 standard deviation.

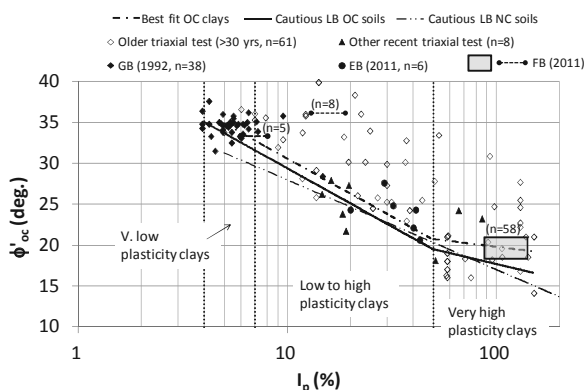


Figure 5. Relationship between peak angle of shearing resistance ϕ'_{oc} and plasticity index I_p for overconsolidated undisturbed clays

A large scatter in the data and a trend of reducing ϕ'_{oc} with increasing I_p is generally seen similarly to what was observed for NC clays in figure 2.

It is observed that the old triaxial test results generally show a much larger spread than the results of the presumably more reliable recent triaxial tests. Furthermore, the new test data tend to be found in the lower half of the range of results from the old tests. This indicates that ϕ'_{oc} is likely to be overestimated by the old tests. The use of high strain rates and the lack of full saturation in the old undrained $CU_{u=0}$ triaxial compression tests are likely to have lead to some uncertainties in the actual effective stress state at failure. This may explain some of the observed differences between the results from the old and more recent triaxial compression tests.

As for NC clays some of the scatter may arise as a result of variations in clay-size fraction, since this is not accounted for solely by the variations in I_p . Variations in stress level on the

other hand should have less of an influence, since a tangent value of ϕ'_{oc} is derived from a failure envelope which is approximately linear within the typical stress range (100-600kPa), as seen in Figures 8-10. As mentioned previously, difficulties in test interpretation and influence of fissures and destructuration may have influenced some of the results.

Based on previous experience and much of the old data shown in Figure 5 within the I_p range of 6%-106%, GEO (Knudsen 1981) has since the early 80's suggested using the following correlation to get a cautious lower bound estimate of ϕ'_{oc} on the basis of a mean value of I_p :

$$\phi'_{oc} = 45 - 15 \cdot \log I_p \text{ (deg.)} \quad (4)$$

In the light of the more recent test data, which extends the I_p range, especially in the high I_p end, to 4%-151%, it is suggested that the lower bound values of ϕ'_{oc} should be slightly less than previously predicted by eq. 3 for very low I_p clays and somewhat higher for high plasticity clays ($I_p > 50\%$). Hence, it is suggested to use the following revised cautious lower bound (LB) estimate of the relationship between ϕ'_{oc} and the plasticity index I_p as given by eq. 5 and 6, and shown in Figure 5 (solid line):

Cautious LB estimate:

$$4 < I_p < 50 \quad \phi'_{oc} = 44 - 14 \cdot \log I_p \text{ (deg.)} \quad (5)$$

$$50 \leq I_p < 150 \quad \phi'_{oc} = 30 - 6 \cdot \log I_p \text{ (deg.)} \quad (6)$$

For an I_p value of 100%, which is typical for e.g. Røsnes clay, this means that the lower bound estimate increases from a peak value of 15 deg. using eq. 4 to 18 deg. using eq. 6.

The best estimate given by the best-fit regression line through the recent test data is shown in Figure 5 (chain dotted line) and is given by:

Best estimate:

$$4 < I_p < 50 \quad \phi'_{oc} = 45 - 14 \cdot \log I_p \text{ (deg.)} \quad (7)$$

$$50 \leq I_p < 150 \quad \phi'_{oc} = 26 - 3 \cdot \log I_p \text{ (deg.)} \quad (8)$$

Eq. 5-8. are believed to be applicable to most overconsolidated natural clays with clay-size fractions below 80%. For soils with clay-size fractions higher than 80% the above relationships should be used with caution until its validity is confirmed by additional tests.

It should be noted that some cases may dictate a mobilized angle of shearing resistance which is lower than the above estimated peak values, e.g. when progressive failure is considered in connection to slope stability analysis in high plasticity clays (Skempton 1977, Burland 1990).

The lower bound estimate for NC clays is shown in Figure 5 (dash-double-dot line) for comparison. It is observed that the lower bound estimates for NC and OC clays do not deviate much.

3.2 Effective cohesion

Figure 6 shows the relationship between the cohesive intercept of the strength envelope c'_{oc} and the plasticity index I_p (single log plot). Data from recent tests and older tests (>30 years) are separated. Two sets of c'_{oc} values have been plotted: the derived values from the tests and estimated values of c'_{oc} . The derived values have been interpreted from the tests results and are paired with the ϕ'_{oc} values shown in Figure 5. While the estimated value of c'_{oc} is found from each failure point (σ'_f, τ_f) by subtracting the stress dependent "frictional" strength contribution $\sigma'_f \tan(\phi'_{oc})$ from the shear strength τ_f . This can also be expressed in terms of mean effective stress $s'_f = \frac{1}{2}(\sigma'_1 + \sigma'_3)$ and shear stress $t_f = \frac{1}{2}(\sigma'_1 - \sigma'_3)$ from the following equation.

$$c'_{oc,est} = \frac{\sigma'_f - \sigma'_f \cdot \sin \phi'_{oc,est}}{\cos \phi'_{oc,est}} \quad (9)$$

where $\phi'_{oc,est}$ is the the lower bound estimate of the angle of shearing resistance, as given by eq. 5. and 6. on the basis of I_p . The estimated c'_{oc} may show negative values in cases where the estimation of ϕ'_{oc} is too high.

The intercept of the strength envelope is very sensitive to the interpretation of the test results, and factors like destructuration or influence of fissures may have great impact on the failure points and hence the value of c'_{oc} . The scatter in the data points will therefore be very significant, much more so than what was observed for ϕ'_{oc} in figures 2 and 5. Despite the scatter a weak trend of reducing lower bound value of c'_{oc} with increasing I_p is seen for I_p greater than 7%. Part of the data from the old tests has been excluded since it is not certain if the meltwater and late glacial marine deposits are heavily overconsolidated (overconsolidation ratio OCR>4).

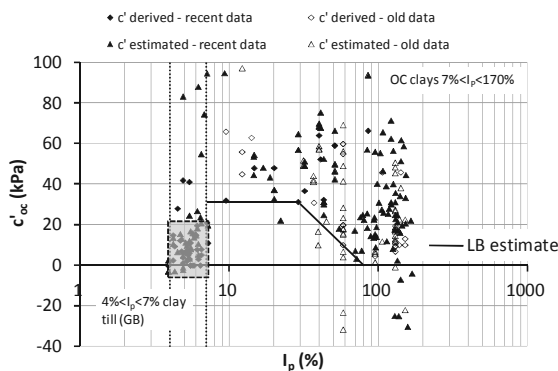


Figure 6. Relationship between effective cohesion c'_{oc} and plasticity index I_p for overconsolidated undisturbed clays

The test results from the Great Belt bridge ground investigation (GB) on very low I_p ($4\% < I_p < 7\%$) clay till are seen to deviate significantly from the general trend by showing relatively low c'_{oc} values; typically between 0 and 15 kPa. This is true for both the derived and estimated values of c'_{oc} . One or several factors might explain this, among others: a) Some of the tested clays are possibly ablation tills that are deposited in a nearly normally consolidated state. b) The clay till may have been subject to postdepositional geological processes (e.g. subsequent passing of ice sheets, meltwater, heave etc.), which may have disturbed the structure and erased the effects of overconsolidation. c) Generally, the tested clay tills have a high content of silt, sand and gravel, and the soils can therefore be characterized as transitional soils, which in terms of strength may behave more like a frictional soil ($c' \approx 0$ kPa) than clay. This is further supported by the fact that the I_p value is likely to be overestimated, as particles larger than 0.425 mm have been removed prior to classification in accordance with BS1377:part 2:1990. An estimated 20-40% of the soil particles have been removed. Hence sand till or gravel till may in most cases be a more representative classification for the tested till deposits.

The low c'_{oc} values observed at very high I_p values above 60% are likely to be a result of fissures and/or sample destructuration. Destructuration may result from large straining events during sampling, sample preparation, testing or geological history. The Palaeogene clays from the Fehmarnbelt (Fixed Link) project mostly belong to an upper stratum which is folded by glacier. Hence, the measured strengths are likely to be influenced by varying degrees of destructuration caused by glacial disturbance of the otherwise intact clay layers.

The significant variation in the c'_{oc} values seen in Figures 6 and 7 also seem to indicate this. For heavily overconsolidated clays with $7\% < I_p < 80\%$ the test data indicate a cautious lower bound estimate of the relationship between c'_{oc} and the

plasticity index, I_p as shown on Figure 6 (solid line) and given by the following equations depending on the value of I_p :

$$\text{Cautious lower bound estimate:} \quad 7\% < I_p < 30\% \quad c'_{oc} = 30 \quad (\text{kPa}) \quad (10)$$

$$30\% \leq I_p < 80\% \quad c'_{oc} = 48 - 0.6 \cdot I_p \quad (\text{kPa}) \quad (11)$$

$$I_p > 80\% \quad c'_{oc} = 0 \quad (\text{kPa}) \quad (12)$$

While the drained angle of shearing resistance ϕ'_{oc} is more naturally linked to soil mineralogy composition, as expressed partly by the I_p value, the apparent effective cohesion is more naturally linked to the soil structure and dilative tendencies. As the I_p value is determined from reconstituted state it does not take account of soil structure. Hence, the above relationship between c'_{oc} and I_p may not be the most appropriate to use.

As suggested in the previous Danish code of practice for foundations (Danish Standard DS 415) it may be expected that the value of c'_{oc} is better related to the undrained shear strength c_u rather than I_p . Both c_u and c'_{oc} are influenced by soil structure and dilation, but as the stress level is likely to have a greater impact on c_u than c'_{oc} , the relationship will not be unique. Based on a comparison of the drained and undrained bearing capacity in connection to plate loading tests on clay till (Jacobsen 1970), the previous Danish code of practice for foundations suggests the following cautious estimate of c'_{oc} on the basis of c_u :

$$c'_{oc} = 0.1 \cdot c_u \quad (\text{kPa}) \quad (13)$$

Figure 7 shows the relationship between c'_{oc} and c_u based on data from the performed tests. As before both derived values from the tests and estimated values of c'_{oc} are shown. As expected the observed scatter is very significant, but there is a tendency of increasing values of c'_{oc} with increasing values of c_u . Both recent and older data appear to verify that the relationship between c'_{oc} and c_u given by eq. 13 can be used as a cautious lower bound estimate with an upper limit of $c'_{oc} = 30$ kPa for all heavily overconsolidated clays except very low I_p clay till. For very low I_p clay till the effective cohesion c'_{oc} is in the majority of the cases found to be lower than given by eq. 13, as shown by the shaded area in Figure 7. Hence, in agreement with observations from Figure 6 it is suggested to use $c'_{oc} = 0$ kPa for very low I_p ($4\% < I_p < 7\%$) clay till/transitional soils independently of c_u , unless specific triaxial test data is available to suggest otherwise. $c'_{oc} = 0$ kPa should also be assumed for fissured high plasticity clays in cases where the overall mobilized strength may be governed by the fissure strength.

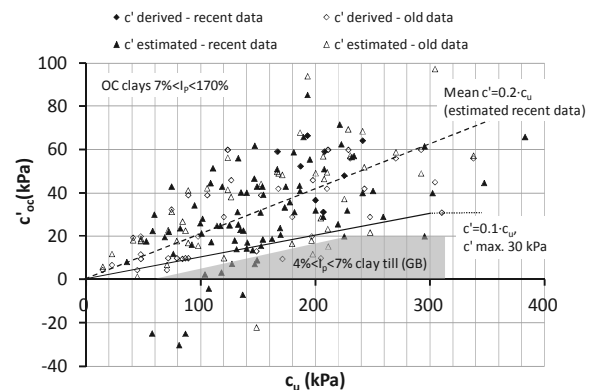


Figure 7. Relationship between effective cohesion c'_{oc} and undrained shear strength c_u for overconsolidated undisturbed clays

Generally it is suggested to estimate c'_{oc} on the basis of eq. 13 for clays with I_p values between 7% and 150%. It should however be noted that if both ϕ'_{oc} and c'_{oc} are estimated cautiously using the above correlations then the estimated combined strength may in some cases be too conservative. On

the other hand, the combined strength found from the proposed correlations may overestimate the strength at low effective stresses (<50 kPa) as the failure envelope curves towards origin (see Figure 1b).

4 VALIDATION OF THE PROPOSED RELATIONSHIPS

The test results from the three test series from the Great Belt (GB), Fehmarnbelt (Fixed Link) (FB) and Esbjerg Harbour (EB) ground investigations are shown in Figures 8-10. The figures show the failure points for each shear stage for all the tested specimens in a plot of shear stress t against mean effective stress s' . For each test series the best-fit line through the data points is shown, and for comparison the cautiously estimated effective strength envelope is also shown (dashed line). The estimated strength envelope is based on the estimated angle of shearing resistance ϕ'_{oc} found from eq. 5-6 on the basis of the mean value of I_p for the tested clays, and the effective cohesion c'_{oc} determined from eq. 13. c_u has been assumed equal to the mean shear stress t_{fm} at failure, as derived from the best-fit regression line through the failure points.

It is observed that the estimated strength envelope in all cases provides a cautious estimate of the strength of the tested clays within the given stress intervals.

5 CONCLUSION

Simple correlations between the plasticity index and the drained peak strength parameters in terms of ϕ'_{oc} and c'_{oc} have been proposed on the basis of a comprehensive database of triaxial compression tests on undisturbed overconsolidated Danish clays of very low to extremely high plasticity. The proposed correlations gives cautious lower bound values of the drained strength parameters, which can be used as a first approximation for use in preliminary design of geotechnical structures. Furthermore, the correlations can be used to evaluate the results of laboratory effective strength tests, and as a mean to assess how well these results represent the entire soil mass at a given site when viewed in connection to the variations of the index properties in the soil mass.

The authors however believe that the proposed correlations should only be used in cases where time and cost constraints do not allow for actual effective strength tests to be carried out. In most other cases the use of effective strength tests will provide a much more reliable and cost effective estimate of the strength properties of the soil in question.

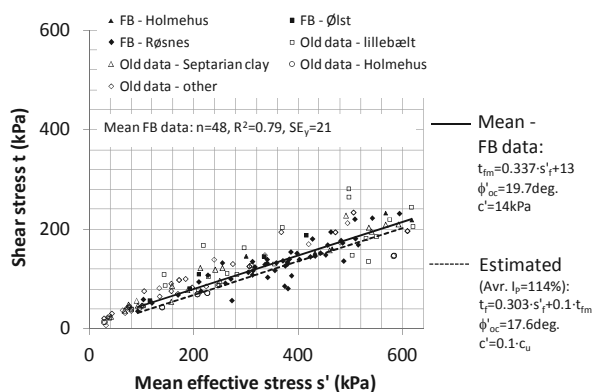


Figure 8. Failure points for high plasticity clays (50%< I_p <170%) shown in a t,s' plot

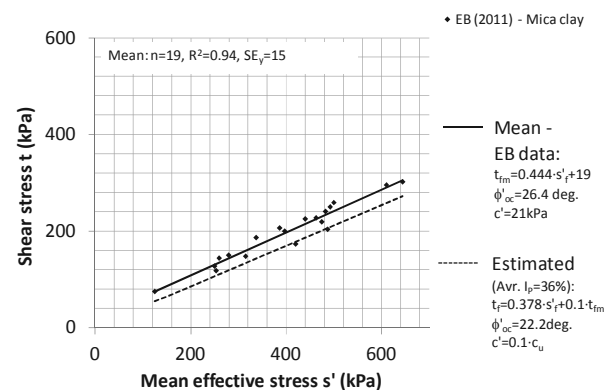


Figure 9. Failure points for mica clay (20< I_p <43%) shown in a t,s' plot

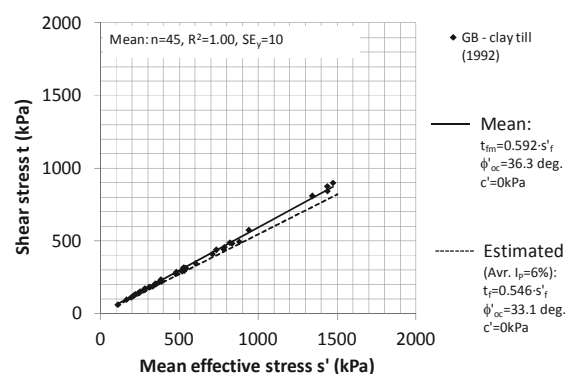


Figure 10. Failure points for clay till (4< I_p <7%) shown in a t,s' plot

6 ACKNOWLEDGEMENTS

The authors would like to acknowledge the support and work done by B. Knudsen (previously The Danish Geotechnical Institute) on the subject.

Data have been made available by Femern A/S (www.femern.com), but findings and conclusions expressed in this paper do not necessarily reflect the views of Femern A/S.

7 REFERENCES

- Brooker E. W. and Ireland H.O. 1965. Earth pressure at rest related to stress history. *Canadian geotechnical journal*, vol. 2 no. 1, pp.1-15
- Burland J. 1990. On the Compressibility and Shear Strength of Natural Clays. *Géotechnique*, vol. 40 no. 3, pp 329-378.
- Fehmarnbelt (Fixed Link) 2011. *Ground Investigation Report*. GDR 00.1-001 with additional data from GEO (2011) GDR 18.0-004, Advanced Laboratory Testing, Glacial Deposits and GDR 18.0-005, Advanced Laboratory Testing, Clays of Palaeogene Origin.
- Knudsen B. (1981). Plastisk ler – Geoteknisk design. Unpublished Internal report (in Danish). *The Danish Geotechnical Institute*.
- Ladd C.C. et al. 1977. Stress-deformation and strength characteristics. *Proc. 9th. Int. conf. soil mech. and found. Eng.* pp. 421-494.
- Leroueil S. and Vaughan P. R. 1990. The general and congruent effects of structure in natural soils and weak rocks. *Géotechnique*, vol. 40 no. 3, pp 467-488.
- Jacobsen M. 1970. Strength and deformation properties of preconsolidated moraine clay. *The Danish Geotechnical Institute. Bulletin 27*. pp 21-45.
- Skempton A. W. 1977. Slope stability of cutting in brown London clay. *7th ICSMFE*. Tokyo vol. 3, pp. 261-270.
- Stark, T.D. and H. T. Eid. 1997. Slope stability analysis in stiff fissured clays. *JGGE*, pp 335-343.
- Terzaghi et al. 1996. *Soil Mechanics in engineering practice*. 3rd. ed. John Wiley & Sons, New York

Mechanisms of binder interactions and their role in strengthening Kuttanad clay

Mécanismes d'interaction des liants et leur rôle dans le renforcement de l'argile de Kuttanad

Suganya K. ,Sivapullaiah P.V.
Indian Institute of Science, Bangalore, India

ABSTRACT: The present investigation is on a soft organic soil from Kuttanad region in Kerala, India wherein a detailed laboratory study was undertaken to contemplate the interaction of various components of soil with cement and cement admixtures. An attempt was made to understand the influence of cement as binder based on parameters such as Clay water/ Cement (W/C) ratio, Initial water content of clay (IW) and curing period by studying the Unconfined Compressive Strength (UCC) of stabilized soil. The results showed that the UCC strength increased with decreasing W/C ratio irrespective of the IW used. Binder combinations involving partial replacement of Cement with Lime, Fly ash and Slag were not effective in improving the strength of soil when compared to the performance of cement alone as binder. On the other hand, addition of Sodium Silicate to cement enhanced the strength of the soil significantly.

RÉSUMÉ : Le travail présenté porte sur un sol mou organique de la région de Kuttanad, dans le Kerala, en Inde, pour lequel une étude de laboratoire détaillée a été menée pour analyser les interactions de différents composants du sol avec le ciment et les adjuvants ajoutés. Une tentative a été faite pour comprendre l'influence du ciment en tant que liant, en se basant sur l'étude de paramètres tels que le rapport eau de l'argile / ciment (W / C), la teneur en eau initiale de l'argile (IW) et la période de cure, en étudiant la résistance à la compression simple (UCC) du sol traité. Les résultats ont montré que la résistance à la compression simple augmente avec la diminution du rapport W / C, quelle que soit la valeur de IW. Différentes formulations de liants basées sur le remplacement partiel du ciment par de la chaux, des cendres volantes et du laitier ne se sont pas avérées efficaces pour améliorer la résistance du sol par rapport au ciment seul. D'autre part, l'ajout de silicate de sodium au ciment améliore de manière significative la résistance du sol.

KEYWORDS: Kuttanad soil, Minerology, Binders, Organic content, Shear strength

MOTS-CLÉS : sol du Kuttanad, minéralogie, liants, teneur en matière organique, résistance au cisaillement

1 INTRODUCTION

Soft soils are more prominently found in coastal regions and low land areas where many important cities and ports are located. And also in today's scenario of increased rate of infrastructural development to match the population growth, it has become mandatory to construct on soft grounds. Soft clays normally exist with high liquidity indices and presence of soft clay as a foundation material in any construction project demands proper engineering judgment as it is associated with inherent problems of excessive settlement and low shear strength. In addition to it presence of organic content in soft clays further worsens the ability of the deposit to support even moderate loads. Such problematic soft soil deposits with inadequate geotechnical properties are present in Kuttanad region of Kerala, India. This soft soil used in the present study also contains organic matter (Thiyyakkandi and Annex 2011) and is found to be acidic (Muralidharan et al. 1999). There are many geotechnical failures reported in Kuttanad region (Ayyar 1966) and hence improving the properties of the soil have been a matter of intense research (Bindu and Vinod 2008, Dipty and Girish 2009).

Compared to other methods of improving the soft ground, introducing deep mixed cement columns and mass stabilization with binders are rapid techniques of ground improvement. Successful case histories are reported on soft grounds improved with soil stabilization techniques (Bergado et al. 1999, Lin and Wong 1999). It is important to note that the success of the method is by the interaction of the binders with the soft clay and the extent of improvement in strength depends on factors such as the type and amount of binder, water content, soil type and curing period (Lorenzo and Bergado 2004, Horpibulsuk et al.

2005). However little is understood about the role of organic matter and clay type on the binder soil interaction. Hence in the present study an attempt is made to evaluate the mechanism of strength development with various binders along with other additives.

2 MATERIALS AND METHODS

2.1 Soil Sample

Clay collected from Kuttanad region, Kerala from a depth of 2 m was used in experimental investigation. The clay is classified as organic clay of high plasticity (OH) according to IS 1498 (Part 1) - 1987. The specific gravity of the soil was determined based on the code IS 2720 (part 3) – 1980. The Cation Exchange Capacity (CEC) of the Kuttanad clay was measured as per the guidelines of ASTM D7503. Organic content of the soil was determined as per the code, ASTM D2974. Unconfined compressive strength of the soil at a water content of 125 % was found to be 10 kPa. The properties of the clay used in the study are given in Table 1.

Table 1. Properties of Kuttanad clay.

Colour	Black
Specific gravity	2.48
Soil classification	OH
Organic content	17 %
Natural water content	100 to 200 %
CEC	17 meq/100 g
pH	3.6

2.2 Soil Mineralogy

2.2.1 X-Ray Diffraction (XRD) Studies

XRD analysis was carried out using Bruker D8 Advance system on powder samples. Samples were scanned using copper K alpha ($\lambda = 1.54 \text{ \AA}$) radiation at a scanning rate of $1^\circ/\text{minute}$. Identification of minerals was done by comparing the XRD patterns got from the soil with the standard data sets of the known minerals (JCPDF, Powder Diffraction File, 1990). The peaks for the Kuttanad soil and its corresponding minerals are shown in Fig. 1.

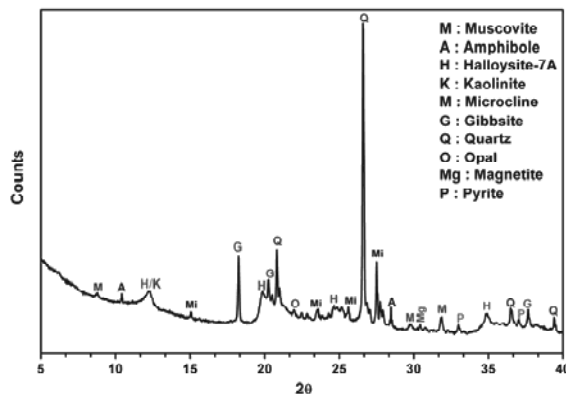


Figure 1. XRD of Kuttanad clay

2.2.2 Scanning Electron Microscopy (SEM)

The Quanta Environmental Scanning Electron Microscope (ESEM) was used in analyzing the fabric of the uncemented and cemented clay. A study of the micrographs of uncemented Kuttanad soil sample prepared by freeze drying technique indicates an open honeycomb type of fabric arrangement, as shown in Fig. 2(a). The fabric shows the presence of frustules, Fig. 2(b & c), which are skeletal remains of diatoms, composed of amorphous silica (opal, presence also verified by XRD analysis). Diatomaceous frustules in marine sediments are known to play a significant role in controlling the geotechnical properties of the soil (Rajasekaran 2006) and this study tries to throw some light on the ability of this amorphous form of silica to participate in the strengthening reactions with binders.

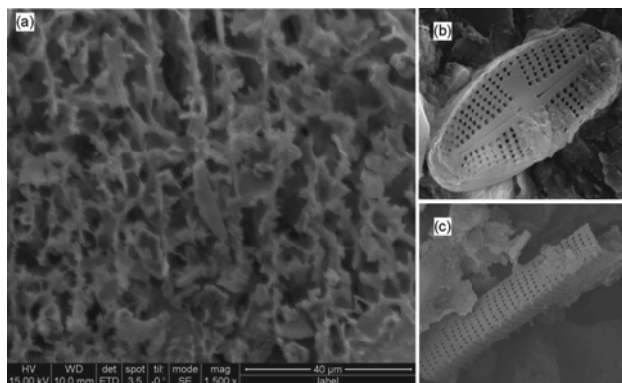


Figure 2. (a) Open fabric of Kuttanad marine clay and (b) & (c) Diatom frustules in Kuttanad clay

2.3 Binders

In the current study ordinary Portland cement of 43 grade was used as the binder. Initially the effect of cement stabilization was evaluated by varying parameters such as IW, W/C and curing period. And then the effect of different composite binder combinations with mineral admixtures and pozzolonic materials such as quick lime (CaO), Ground granulated blast furnace slag

(GGBS) and Fly ash (FA) on Kuttanad soil stabilization was studied. Further for W/C ratio of 3.33 addition of FA, GGBS, Silica fume (SF), Sodium Silicate (SS) and Fine sand to cement on the strength of Kuttanad clay was evaluated.

2.4 Sample Preparation

The clay was thoroughly hand mixed and IW was increased to 165 % and 200 %. The intentional increase in the initial water content before stabilizing was done to simulate the field conditions where there is a raise in the insitu water content because of wet binder dispensing methods adopted during deep mixing (Horpibulsuk et al. 2004). Cement quantity to be added for preparing the mixes was calculated based on W/C ratio. In the present investigation three W/C ratios were used namely 6.5, 5 and 3.33. Cement was added to the soil and mixed thoroughly for about 10 minutes as suggested by Miura et al. 2001. Split moulds having diameter 38 mm and height 76 mm (Length/Diameter = 2) with sides slightly lubricated with oil were used for casting and curing the stabilized mixes. The well mixed paste was transferred to the split mould in three layers. Spreading of the paste into the mould and removal of air entrapped during casting was done by giving mild tamping for each layer. The samples casted in the split moulds were cured for 7, 14 and 28 days in desiccators maintained at 100 % relative humidity. Similar methodology of making samples was adopted for different composite binder combinations adopted in this study.

3 RESULTS AND DISCUSSION

3.1 Cement as binder

Figure 3 presents the UCC strength of cement stabilized samples at different W/C ratio with varying IW and Cement Content (CC) with curing period.

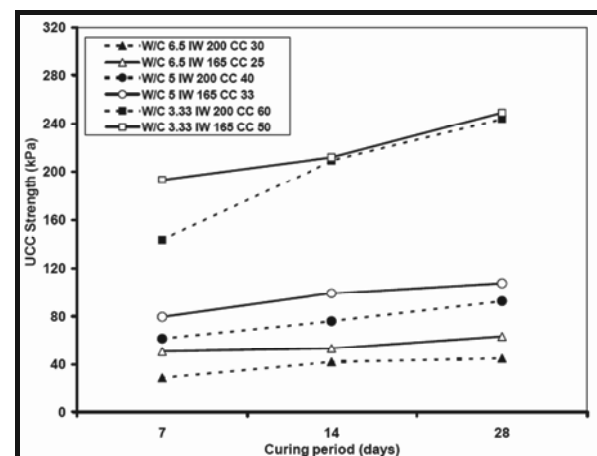


Figure 3. Unconfined compressive strength (UCC) of cement stabilized samples

It can be seen from Figure 3 that with reducing W/C ratio there is an increase in the strength developed. Even though the cement percentage corresponding to the W/C ratios used in the study is high (more than 25 %) the strength attained is very low. This might be due to the organic content present in the soil hindering the cement hydrolysis and subsequent strength development. It can also be inferred that up to W/C ratio of 5 there is a marginal improvement in the UCC strength but when the W/C ratio further reduces to 3.33 the strength gained is significant. This is because the organic components gets enough calcium at lower water cement ratios so that surplus calcium is available in solution to aid the cement hydration and hardening reactions contributing to better strength development. From (Figure 4) the fabric changes happening over reduction in W/C

ratio it can be concluded that the cementation compounds are still insufficient to bind the clay fabric in case of high W/C ratio whereas at the lowest W/C ratio the cement matrix is continuous binding the clay aggregates contributing to a significant strength gain.

It can be seen from Figure 3 that the effect of curing is almost negligible and this is mainly because the organics tending to coat the surface of the soil particles impeding the interaction between clay minerals and the cement hydration products harming the long term strength gain contributed by secondary reactions (Chen & Wang 2006). Results indicate that for the same W/C ratio the variation in initial water content has very little influence on the long term strength gain thus confirming the critical role played by W/C ratio in strength development of cemented clays as suggested by Miura et al. (2001).

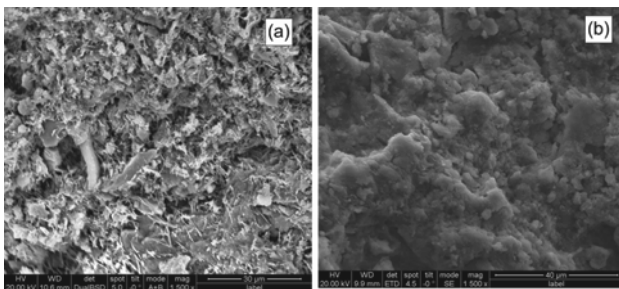


Figure 4. Fabric of Cemented Kuttanad clay (a) W/C 6.5 (b) W/C 3.33

3.2 Cement Composite binder combination

With an effort to economize and reduce the carbon foot print of the proposed ground improvement technique, composite binder combinations such as cement – Lime, Cement – FA and Cement – GGBS were tried.

3.2.1 Cement-Lime

Quick lime as reported in literature (Åhnberg and Johansson 2005) as a possible replacement for cement was tried in different proportion with cement and the UCC tests were done on stabilized soil samples. For IW 165 % and W/C ratio 5, Cement(C) - Quick Lime (L) binder combinations of 75C-25L and 50C-50L were tried and the UCC strength at the end of 14 days curing was found. UCC strength of the sample with only cement as binder was 99 kPa whereas with lime replacement the strengths significantly reduced to 33 kPa (75C-25L) and 25 kPa (50C-50L). Development of strength happens when the added lime reacts with the clay minerals. As soon as lime is added, pH increases and there is De-flocculation of soil fabric and removal of organics adsorbed on to the minerals. The organic matter hinders the reactions between added lime and the clay minerals present in the soil hence there is no beneficial effect of lime replacement observed. Diatom silica is also found not reacting with the added lime confirming the role of components such as organic content and sesquioxides (Van Capellen et al. 2001) present in the soil inhibiting its solubility, finally retarding the pozzolonic reactions responsible for strength development.

3.2.2 Cement - Mineral admixtures

Figure 5, shows the stress – strain characteristics of cement with GGBS and Fly ash binder combination with and with out lime. It can be clearly seen that the effect of replacing a portion of cement with either of the mineral admixtures (GGBS / FA) results in strength loss when compared to the cement alone case. Hence it reinforces the fact that the positive effect of pozzolonic reactions is not enough to offset the role of cement which is replaced. Both lime released and lime added have insignificant

role to play in enhancing the strength of the Cement - Mineral admixture stabilized Kuttanad soil.

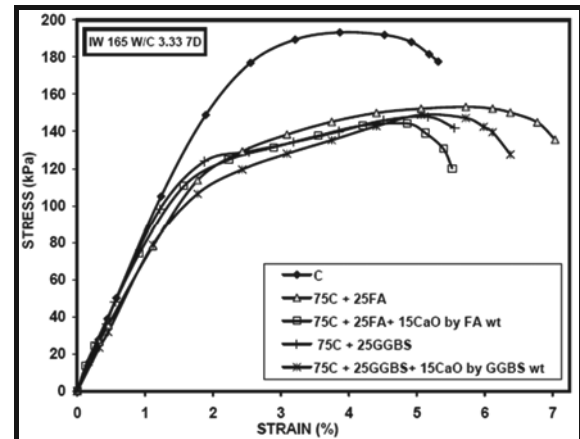


Figure 5. Stress Vs Strain for cement- mineral admixture combination

3.3 Role of Admixtures

To further enhance the strength of soil with cement as binder different additives with varying reactive silica content were considered. Results from UCC tests of different additives such as Fine sand, Fly ash, Silica fume, GGBS and Sodium silicate used along with cement are shown in Figure 6.

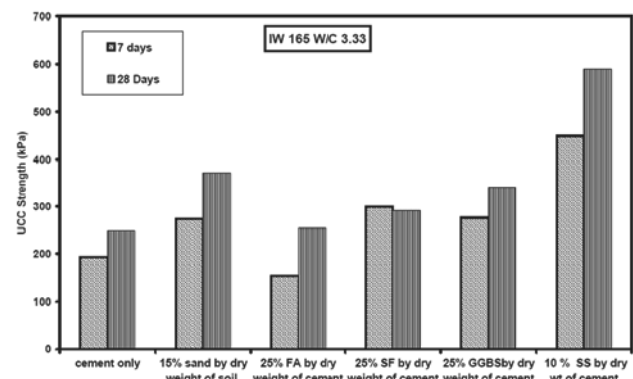


Figure 6. Effect of various additives on UCC strength of cemented soil

It is evident from Figure 6, that addition of fine sand to the mix has enhanced the shear strength of the cement stabilized soil. The mechanism of strength enhancement in this case can be attributed to the matrix of cemented sand supporting the weak soil within it. It is well known that the cement can bind very effectively the coarser sand particles involving lesser contact points than the fine organic soil.

The addition of fly ash resulting in no strength improvement can also be due to the interaction of the strengthening matrix of both soil cement and fly ash cement system leading to a weaker framework.

Silica fume as an additive with cement resulted in 17 % strength increment in case of 28 days cured samples (Figure 7). The strength increment is marginal and can be attributed to the participation of silica fume in the pozzolonic reactions, involving lime liberated from cement hydrolysis.

Adding 25 % GGBS by weight of cement resulted in a strength increase of about 37 % at the end of 28 days curing (Figure 7). The strength increase is due to the formation of cementation products because of the alkali activation of GGBS caused by cement hydrolysis (Taylor 1997). GGBS is coarser and accommodates soil in its matrix and the subsequent production of cementitious products by itself results in effective binding giving an enhanced strength.

Compared to other additives tried along with cement to improve the strength of the stabilized soil, Sodium Silicate added about 10 % by dry weight of cement resulted in maximum increase of UCC strength. From Figure 7 it can be seen that the strength increase compared to cement alone at 28 days curing was about 137 %. Initially when sodium silicate is added to the soil, the negative silicate ions go and sit in the positive edges of the clay and makes the clay completely negative resulting in thorough dispersion of the clay. Then when cement is mixed with soil the calcium ions released as a by-product of cement hydrolysis is utilized by the silicate ions leading to the formation of calcium silicate hydrate which eventually binds the clay minerals together (Moayed et al. 2011). This is expected to be the mechanism behind enormous strength gain in the presence of small percentage of SS with cement.

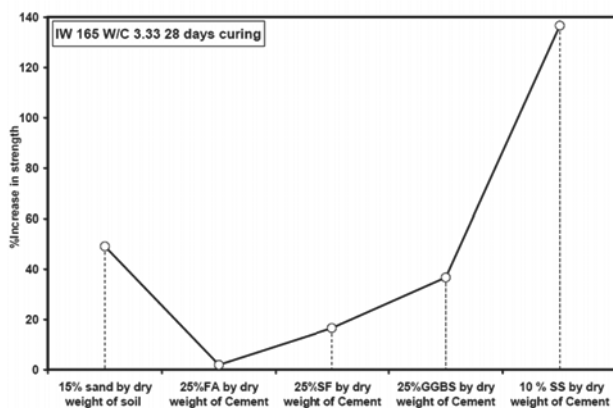


Figure 7. Percentage increase in UCC strength of cement stabilized soil upon adding various additives

4 CONCLUSIONS

The following conclusions can be drawn from the present study

- In case of cement stabilization W/C ratio is the key parameter controlling the strength of the treated soil samples. Higher initial water content requires more cement content to attain desired strength as compared to the soil treated at lower initial water content for the same water cement ratio. Effect of curing is almost negligible because of the organic constituents present in the soil.
- Replacement of cement by quick lime is found to be ineffective as the organic component of the soil inhibits lime from reacting with the clay minerals. Hence lime and lime substitutes will not work in case of stabilizing Kuttanad clay.
- The solubility of Diatom silica is affected by the organic matter and sesquioxides present in the soil making the soil less responsive to lime. Thus supplying reactive silica by other additives along with cement is necessary for improving the strength of the soil.
- Among the additives used with varying quantities of reactive silica, small percentage of Sodium Silicate with cement is found to be most effective in enhancing the strength of cement stabilized Kuttanad soil.

5 REFERENCES

- Thiyyakkandi S. and Annex S. 2011. Effect of Organic Content on Geotechnical Properties of Kuttanad Clay. *Electronic Journal of Geotechnical Engineering*, 16, 1653 – 1663.
- Muralidharan P., Rattan R. K. and Datta S. P. 1999. Phosphorus and iron adsorption characteristics of acid saline soils of Kuttanad, Kerala. *J. Tropical Agric.*, 37(1-2), 46-50.
- Ramanatha Ayyar, T.S. 1966. Strength characteristics of Kuttanad clays. PhD Thesis, University of Roorkee, India.
- Bindu J. and Vinod P. 2008. Mini-plate Load Test on Preloaded Kuttanad Clays. *Proceedings of Indian Geotechnical Conference*, Bangalore.
- Dipty S. I. and Girish M. S. 2009. Suitability of Different Materials for Stone Column Construction. *Electronic Journal of Geotechnical Engineering*, 14.
- Bergado D. T., Ruenkairergsa T., Taesiri Y., and Balasubramaniam A.S. 1999. Deep soil mixing to reduce embankment settlement. *Ground Improvement J.*, 3(3), 145–162.
- Lin, K.Q. and Wong, I.H. 1999. Use of deep cement mixing to reduce settlements at bridge approaches. *J. Geotech. Geoenviron. Eng.*, ASCE, 125(4), 309-320.
- Lorenzo G. A. and Bergado, D. T. 2004. Fundamental parameters of cement-admixed clay-new approach. *J. Geotech. Geoenviron. Eng.*, ASCE, 130(10), 1042–1050.
- Horpibulsuk S., Miura N. and Nagaraj T.S. 2005. Clay–water/cement ratio identity of cement admixed soft clay. *J Geotech Geoenviron Eng.*, ASCE, 131(2), 187–92.
- Rajasekaran G. 2006. Influence of microfossils and pyrites on the behaviour of oceanbed sediments. *Ocean Engineering*, 33,517–529.
- Horpibulsuk S., Miura N. and Bergado D.T. 2004. Undrained shear behavior of cement admixed clay at high water content. *J. Geotech. Geoenviron. Eng.*, ASCE, 130(10), 1096-1105.
- Miura N., Horpibulsok S. and Nagaraj T. S. 2001. Engineering behavior of cement stabilized clay at high water content. *Soils Found.*, 41(5), 33–45.
- Chen H and Wang Q. 2006. The behaviour of organic matter in the process of soft soil stabilization using cement. *B. Eng. Geo. Environ.*, 65(4), 445-448.
- Åhnberg H. and Johansson S.E. 2005. Increase in strength with time in soils stabilised with different types of binder in relation to the type and amount of reaction products. *Proceedings of the International Conference on Deep Mixing*, Stockholm, Vol.1.1.
- Van Capellen P., Dixit S. and van Bennekom A. J. 2001. Processes controlling solubility of biogenic silica and pore water build-up of silicic acid in marine sediments. *Mar. Chem.*, 73, 333– 352.
- Taylor H.F.W. 1997. *Cement chemistry*, Thomas Telford, London.
- Moayed H., Huat B.K., Moayed F., Asadi A., and Parsaie A. 2011. Effect of Sodium Silicate on Unconfined Compressive Strength of Soft Clay. *Electronic Journal of Geotechnical Engineering*, 16, 289-295.

Hardening process of clayey soils with high water content due to thixotropy effect

Processus de durcissement des sols argileux à forte teneur en eau causé par un effet thixotropique

Tanaka H.

Hokkaido University, Sapporo, Japan

Seng S.

Institute of Technology of Cambodia, Phnom Penh, Cambodia

ABSTRACT: Hardening due to thixotropy for clayey soils with high water content was studied by change in stiffness by time, using Bender Element Test. Shear strength was also measured to be compared with the increase of the stiffness measured by the bender element. Following interested findings are obtained: 1) the effect of thixotropy is found significantly at around the liquid limit state and less remarkable at the lower and higher ranges than the liquid limit; 2) the shear modulus at liquid limit after 24 hours resting is around 200 kPa; 3) the increment of the shear modulus developed in the thixotropy process appears noticeably higher than that in the secondary consolidation process. It is believed that these findings are very useful to establish a new theory for the consolidation of the ground filling by very soft clays or dredged soils with extremely high water content as well as to understand the effects of ageing on consolidation properties of natural soils.

RÉSUMÉ : Le durcissement de sols argileux, ayant une teneur en eau élevée et causé par la thixotropie, a été étudié à l'aide de plaques de piézo-céramiques permettant de suivre les changements de rigidité du sol en fonction du temps. La résistance au cisaillement a aussi été mesurée et comparée aux changements de rigidité. Ceci a permis les constatations intéressantes suivantes : 1) l'effet du gain thixotropique devient significatif lorsque l'état du sol est proche de la limite de liquidité lequel serait moindre aux valeurs supérieures et inférieures à la limite de liquidité; 2) le module de cisaillement à la limite de liquidité, après 24 heures au repos, est d'environ 200 kPa; 3) l'augmentation du module de cisaillement causée par la thixotropie apparaît, d'une façon notable, plus élevée que celle pouvant provenir de la consolidation secondaire. Nous croyons que ces constatations sont très utiles pour établir une nouvelle théorie de la consolidation pour les cas de remblayage avec des argiles très molles ou des sols de dragage tout autant que pour comprendre les effets du vieillissement sur les propriétés de consolidation des sols naturels.

KEYWORDS: Thixotropy, Stiffness, Bender Element, Consolidation, Liquid limit.

1 INTRODUCTION

The concept of "effective stress" has been widely accepted by geotechnical engineers and it is believed that geotechnical parameters such as strength or stiffness can be simply correlated with the effective stress. In this concept, these parameters are independent of time and the apparent time dependency during consolidation can be understood by changing in the effective stresses in the process of the dissipation of the excess pore water pressure. On the other hand, some behaviors opposite to the effective stress approach are also recognized, i.e., thixotropic hardening. Soil loses strength or stiffness by remolding, but they are recovered in time. It may be considered that this factor should play an important role in the process of settlement. Thus, this paper will try to understand the thixotropic hardening phenomenon of clays with high water content.

Although some experimental data have been reported on this field, the conventional destructive testing tools such as triaxial, unconfined compression, and laboratory vane apparatus have been employed by previous investigators to measure strength hardening due to thixotropy. Normally, these methods do not guarantee the same testing condition because of using different samples and relatively long preparation time for testing, i.e., difficult to measure properties at very early time, and also disturbance effect occurs during insertion of the vane blade, for example. To overcome these drawbacks, a nondestructive bender element test, which provides the measurement at a very low strain level as small as 10^{-5} (0.001%), is introduced in this paper to study the thixotropic behavior of very soft clays, together with a vane shear test measuring the undrained

strength. In another approach, oedometer test under very low pressure was also carried out, being equipped with the bender element to examine the development of soil stiffness due to consolidation. Correlation between the stiffness and the effective stress will be established, and also study on the changing in the stiffness during the secondary consolidation. Using these test results, the increment of stiffness caused by thixotropic behavior is tried to be evaluated.

2 SAMPLE AND TESTING METHOD

The tests were run on very soft clays using samples obtained from three commercial powder clays and three natural clays. Commercial clays are Fujinomori, Kasaoka, and NSF. The natural ones are Ariake, Hachirogata and Tokuyama. Their index properties are summarized in Table 1. Their liquid limits vary in a large range from low (48.6 %) to very high (246.0 %) value that allows this study to cover the wide different characteristic of materials.

After mixing at a target water content, the sample was poured into a mold with various sizes for different purposes: a plastic cylindrical mold with the height of 10 cm and the diameter of 5 cm for the bender element test, a gallon bucket with 20 cm of the diameter and heights of specimen varied from 7 to 8 cm for the vane test, and a consolidometer cell with diameter of 10 cm and adjustable height from 4 to 15 cm for consolidation test. Vibration was gently applied to drive out air bubbles from a slurry specimen and a sample was compacted when its water content was small and the sample was stiff. The mold with the specimen for the thixotropy test was wrapped by

Table 1. Geotechnical Properties used in the test.

Soil Types	Symbol in Figures	Plastic Limit (%)	Liquid Limit (%)	Plasticity Index	Particle Density (g/cm^3)
Fujinomori	F	21.3	48.6	27.3	2.69
Kasaoka	K	27.5	62	33.5	2.61
NSF	N	29	55	26	2.76
Ariake	A	31.4	72.5	41.1	2.621
Hachirogata	H	96.5	246	149.5	2.43
Tokuyama	T	40	110.6	70.6	2.62

thin plastic film, and then it was kept in a high humidity box to prevent the water evaporation. The elapsed time for thixotropy was accounted just after the sample was poured into the mold.

As input signals for the bender element test, sine and rectangular waves have been alternatively used with wide ranges of frequencies in accordance with material stiffness, in order to attain a clear output waveform. Generally, the high frequency is required in testing stiff soils and vice versa. The “start-to-start” method for determining the arrival time (Δt) and “tip-to-tip” method for determining the travel distances (Δd) of the shear wave were adopted in this study (Yamashita et al., 2009).

The increase in the undrained shear strength during thixotropic hardening was also confirmed by the vane shear test. The vane diameter and height used in this experiment were 20 mm and 40 mm, respectively. The shear rate of the laboratory vane apparatus was constant at 6° rotations per minute. All tests were carried out for the same sample created by a gallon bucket, as already mentioned.

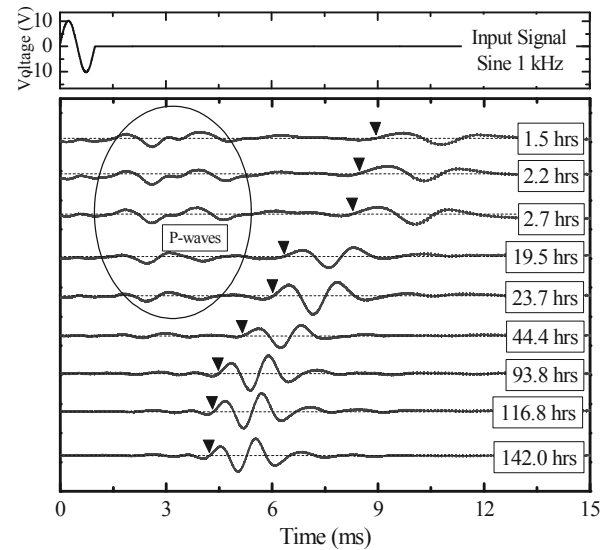
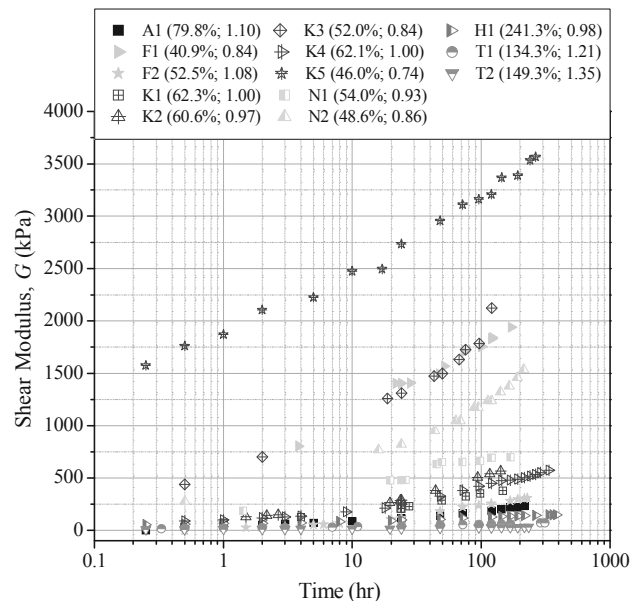
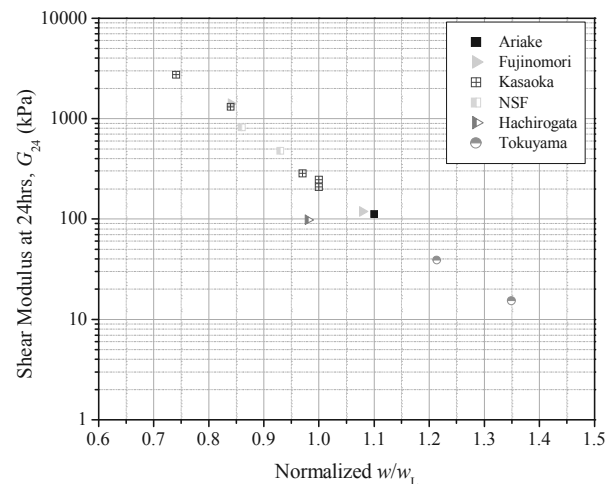
3 THIXOTROPIC EFFECT MEASUREMENT

3.1 Shear Wave Velocity and Shear Modulus by Bender Element Test

Figure 1 shows an example of increase in the shear wave velocity (V_s) with time, measured by the bender element test on a specimen made from Kasaoka clay mixed with 60.6% of water content. Since the water content of the specimen was almost equal to liquid limit state, the received shear wave signals at the beginning of the measurement were hardly identified because of their low amplitude and frequency. P-waves clearly appeared since they could propagate through liquid. As time was proceeded, the soil became stiffer; consequently the arrival times were detected more shortly, in another word the shear wave velocity increased. It may be considered that the increase in shear wave velocity (V_s) corresponds to the increases in the stiffness, which is reflected to the thixotropic phenomenon. The received shear waves became much clearer with high amplitude and frequency after a certain time, while P-waves seemed to decay.

The shear modulus (G) derived from V_s with elapsed time is illustrated in Fig. 2, where the symbols of A, F, K, N, H, and T represent Ariake, Fujinomori, Kasaoka, NSF, Hachirogata, and Tokuyama clays respectively with sample number. The number in the brackets indicates the water content (w) and the normalized water content by the liquid limit (w/w_L). It can be seen from the figures that G values for all conditions increases with time even at over limit liquid states. And, G builds up nearly in proportion to time in the logarithm scale, but this magnitude is certainly depended on types of soils and the amount of water content.

Since G increases in time, G at 24 hrs (G_{24}) will be a represented parameter for identifying characteristics of soil types and influence of water content. G_{24} is plotted in Fig. 3


 Figure 1. An example of shear waves (Kasaoka clay, $w=60.6\%$)

 Figure 2. Relationships between G and time.

 Figure 3. Relationship between w/w_L and G after 24 hours.

against the (w/w_L), considering different types of materials. A clear correlation between these two parameters can be observed

with minor scatters. Obviously, G decreases with increasing the normalized w/w_L , meaning that w_L defines the magnitude of G regardless types of soils. In this case, G_{24} value at w_L appears around 200 kPa.

By extruding data of Hachirogata clay, because of a strange point in the G_{24} and w/w_L relation as shown in Fig. 3, the correlations of hardening G and normalized w/w_L at not only 24 hours but also various times are plotted in Fig. 4 to examine the effect of thixotropy corresponding to wide range of water content. It can be seen in the figure that the increment of G to time at around w_L is the largest, while at smaller water contents than w_L , say, when w/w_L is less than 0.8, the tendency of G increasing against time becomes smaller.

3.2 Undrained Shear Strength by Vane Test

Although the figure is not presented in this paper, it is also observed in laboratory vane test that the shear strength increases with time. The values of s_u measured at 24 hrs (s_{u24}) are plotted with normalized w/w_L in Fig. 5, in the same manner as G_{24} shown in Fig. 3. Almost a linear correlation between s_{u24} in the logarithm scale and w/w_L ratio is also identified, but more remarkable scatters and slightly steeper gradient are recognized than those in Fig. 3. At w_L , the magnitude of s_{u24} varies from about 1 to 2 kPa, whose values are somewhat lower than those suggested by Wood (1990), who did not consider the thixotropic hardening effect and recommended 2 kPa of s_u at w_L .

Relationship between thixotropic hardening G and s_u at various elapsed times is shown in Fig. 6 together with that obtained from cement-treated soil (CTS) material proposed by Seng and Tanaka (2011). They found that G and s_u relation of CTS can also be applicable to most of worldwide natural clays with s_u varied from 10 to 150 kPa. It is observed in Fig. 6 that at very high water content corresponding to low initial strength, s_u remains constant until a certain time unlike G . After G reaches a certain values, s_u starts to increase and the relation of G and s_u seems to approach to the same line as CTS. Indeed, Seng and Tanaka (2011) reported that even CTS material behaves the similar way when the strength of CTS is extremely small. However, when strength of CTS is greater than 1 kPa, the G and s_u correlation for each sample forms linear function, unlikely very soft clays that show a monotonic increase. Both behaviors, the constant values and slow increases of s_u , are quite interesting and might be associated with viscosity or strain rate effect which is obviously an important factor governing soil strengths especially when material remains soft; however further investigation is necessitated to confirm this presumption.

4 SHEAR MODULUS MEASUREMENT DURING SECONDARY CONSOLIDATION

The development of G during secondary consolidation is also an interesting topic. It should be noted that the condition of the secondary consolidation is under a constant effective stress but the volume is changing, while in the thixotropic condition, the volume change does not take place. Therefore, it may be anticipated that the increase rate of hardening G owing to the secondary consolidation should be greater than that of thixotropy, as decreasing void ratio during the secondary consolidation. As shown in Figs. 7 and 8, however, the increase in G during the secondary consolidation is considerably smaller than that in Thixotropy test. Figure 7 shows test results from the thixotropy test, where G is normalized by G at 1 hour after remolding (G_{at1h}). In Fig. 8, change in G during the secondary consolidation is shown. The end of primary consolidation (EOP) was estimated by the root t method, and it is assumed that the effective stress is constant after EOP. Both axes in the figure are normalized by time at EOP (t_{EOP}) and G at EOP

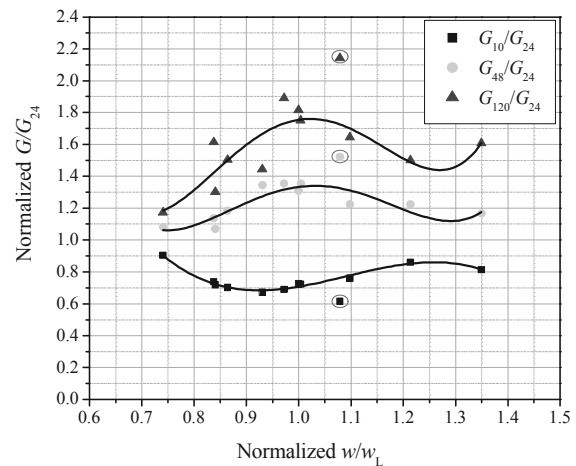


Figure 4. Increase in G due to time at different w .

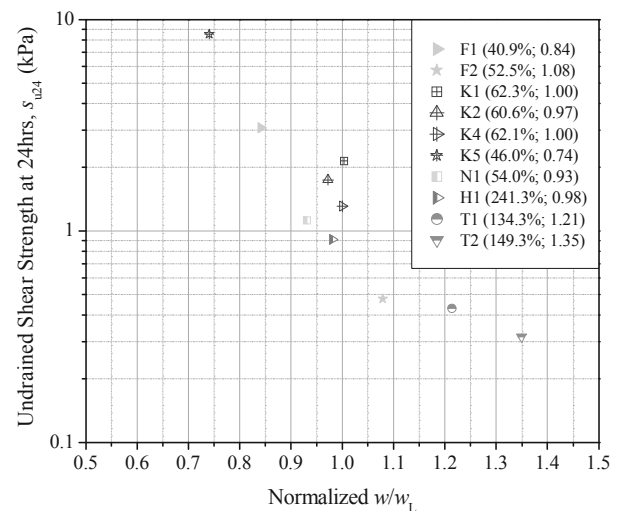


Figure 5. Relationship between w/w_L and s_u measured by vane after 24 hours.

(G_{EOP}). The reason for small increment G during the secondary consolidation might be explained by destruction of the interparticle bounding, which is created during the thixotropy process.

5 CONCLUSIONS

The experimental results of thixotropic phenomenon measured on various clays are presented and compared with values obtained by consolidation test with low pressures. Main conclusions can be drawn as following:

- 1) Bender element test is a powerful tool and an appropriate method for evaluating the thixotropic hardening stiffness of very soft clays, since it are able to detect even small changes in G with an extremely small strain.
- 2) G at 24 hours for w_L is around 200 kPa.
- 3) Regardless of soil types, thixotropy affects the clay most strongly at around liquid limit state and becomes less remarkable at lower and higher water contents.
- 4) The correlation between G and s_u for very soft clays is analogous to that of cement-treated soil proposed by Seng and Tanaka (2011). Additionally, similar behavior is recognized at very low strengths, where s_u appears constant while G increases.

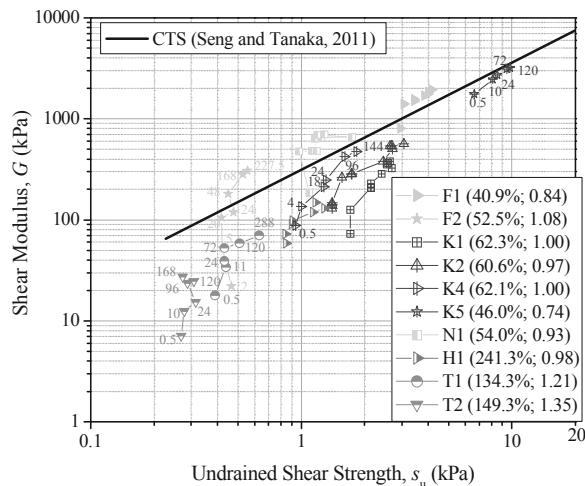


Figure 6. Relation between G and s_u . Numbers in the figure indicate the elapsed time after remolding.

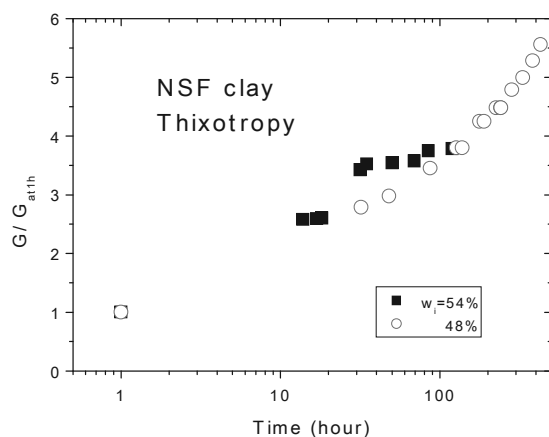


Figure 7. Increase in G during thixotropic test.

5) The increment of the shear modulus under secondary consolidation is shown relatively low, compared with that developed during the thixotropic process. It is suggested that these differences may be caused by the different constraint pressures between the secondary consolidation and thixotropic condition, i.e., the free arrangement of soil particles is prevented under the former condition, while under thixotropic condition, the soil particles can be freely arranged.

From the present study, we can point out the importance of thixotropic hardening phenomenon for understanding soil behavior with high water contents. It is well known that the void ratio or water content in situ is much larger than that predicted by laboratory consolidation tests based on e - $\log p'$ relation at the EOP (for example, Tanaka et al., 2004). If we consider the secondary consolidation effect, in situ void ratio should be smaller than that predicted from e - $\log p'$ relation at the EOP. To cope with such inconsistency, Burland (1990) has proposed a concept of Sedimentation Consolidation Line (SCL), considering the cementation or fabric effect. Study on the thixotropy, which was conducted in the present study, is also expected to provide an important key to understanding such a phenomenon.

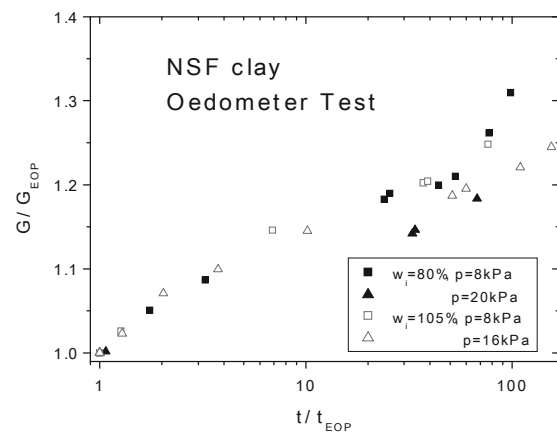


Figure 8. Increase in G during the secondary consolidation.

6 REFERENCES

- Burland, J. B. 1990. On the compressibility and shear strength of natural clays, *Géotechnique* 40(3), 329-378.
- Seng, S. and Tanaka, H. 2011. Properties of cement-treated soils during the initial curing stages, *Soils and Foundations* 51(5), 775-784.
- Tanaka, H., Kang, M. S. and Watabe, Y. 2004. Ageing effects on consolidation properties –Based on the site investigation of Osaka Pleistocene clays–, *Soils and Foundations* 44(6), 39-51.
- Wood, D. M. 1990. *Soil behavior and critical state soil mechanics*, Press Syndicate of the University of Cambridge, USA.
- Yamashita, S., Kawaguchi, T., Nakata, Y., Mikami, T., Fujiwara, T. and Shibuya, S. 2009. Interpretation of international parallel test on the measurement of G_{max} using bender element, *Soils and Foundations* 49(4), 631-650.

Comparative study of long-term consolidation for subsoils under Kansai Airport and Pisa Tower

Etude comparative de la consolidation à long terme pour les sous-sols d'aéroport de Kansai et de tour de Pise

Watabe Y., Sassa S.

Port and Airport Research Institute, Yokosuka, Japan

Udaka K.

Oyo Corporation, Saitama, Japan

ABSTRACT: In both the Kansai International Airport and Leaning Tower of Pisa, long-term consolidation settlement is a very important geotechnical issue. In this study, a series of long-term consolidation tests were conducted for undisturbed samples retrieved from these two sites. The isotache concept observed in the long-term consolidation behavior was successfully modeled by a simple equation, and then the difference in long-term consolidation behavior between the Osaka Bay clay (significant delayed consolidation) and Pisa clay were compared and discussed. Using the isotache model, the long-term consolidation settlement can be quantitatively predicted in association with the strain rate dependency.

RÉSUMÉ: Pour l'aéroport international de Kansai et la tour penchée de Pise, le tassement de consolidation à long terme constitue une problématique géotechnique très importante. Dans cet article, on présente une série d'essais de consolidation à long terme réalisés pour des échantillons intacts prélevés sur ces deux sites. Le concept isotache, observé dans la consolidation à long terme, a pu être modélisé par une équation simple. Les différences de consolidation à long terme observées entre l'argile de la baie d'Osaka (consolidation retardée significative) et l'argile de Pise ont été comparées et discutées. En utilisant le modèle isotache, le tassement de consolidation à long terme peut être quantitativement évalué en association avec la dépendance en vitesse de déformation.

KEYWORDS: long-term consolidation, isotache, strain rate.

1 INTRODUCTION

In both the Kansai International Airport and Leaning Tower of Pisa, long-term consolidation settlement is a very important geotechnical issue. Observed settlements at the two sites are, however, difficult to be directly compared, because their scale and mechanism are different.

In this study, a series of long-term consolidation tests were conducted for undisturbed samples retrieved from these two sites. The test results were interpreted based on the most recent findings from the isotache concept, which considers strain rate dependency in preconsolidation pressure. Then, essential difference between the long-term consolidation behaviors at these two sites was clarified in association with the strain rate dependency.

2 PHYSICAL AND MECHANICAL PROPERTIES OF THE CLAYS

Osaka Bay clay (Kansai International Airport) and Pisa clay have common characteristics, such as soil consistency ($w_L \approx 80\%$, $I_p \approx 50$), grain-size distribution (clay fraction ($< 2 \mu\text{m}$) of 50%, fine particle fraction ($< 75 \mu\text{m}$) of 100%), void ratio ($e \approx 1.5$), overconsolidation ratio (OCR ≈ 1.4), etc, as shown in Figure 1. Dominant clay minerals from X-ray diffraction are different: smectite and kaolinite for Osaka Bay clay and illite for Pisa clay. Micro-fabrics observed by scanning electron microscope (SEM) are shown in Figure 2. Osaka Bay clay is consisted of flaky particles (typically smectite) forming aggregations with abundant microfossils (typically diatoms). Pisa clay is consisted of platy particles (typically illite) with a small number of microfossils.

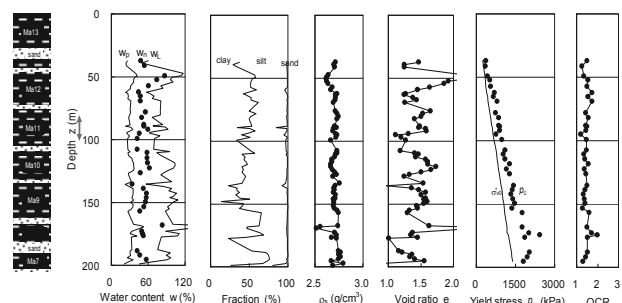


Figure 1a. Depth profiles of soil properties for the Osaka Bay clay.

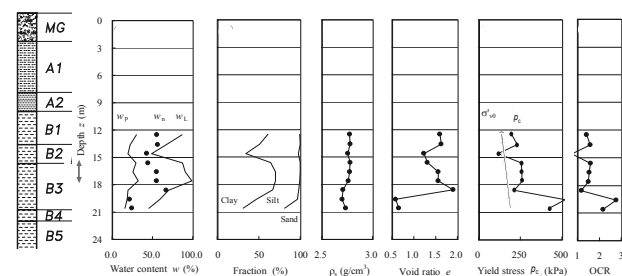
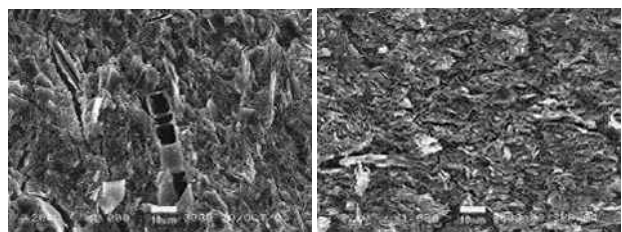


Figure 1b. Depth profiles of soil properties for the Pisa clay.



(a) The Osaka Bay clay. (b) The Pisa clay.
Figure 2. Microfabrics observed by SEM.

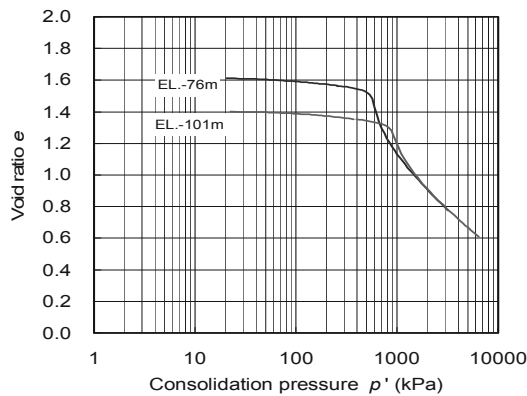


Figure 3a. Compression curves for the Osaka Bay clay.

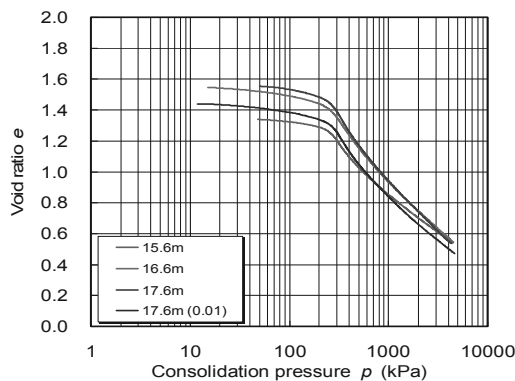


Figure 3b. Compression curves for the Pisa clays.

3 CONSOLIDATION TESTS

Compression curves (e - $\log p$ curves) for the two clays are shown in Figure 3. Undisturbed samples collected from G.L.-76 and -101m (Ma11) for Osaka Bay clay and from G.L.-15.6 and -17.6 m (B3) for Pisa clay were examined. Compression indices C_c at these two sites were commonly 0.7 with very similar compressibility. Preconsolidation pressures σ'_p for Osaka Bay clay and Pisa clay were 600–900 kN/m² and 250 kN/m², respectively. For these values, a common overconsolidation ratio ($OCR = \sigma'_p/\sigma'_{v0}$) of approximately 1.4 were calculated with the overburden effective stress σ'_{v0} of 750 kN/m² and 250 kN/m², respectively.

In the long-term consolidation test, a specimen with 60 mm in diameter and 20 mm in height was trimmed from an undisturbed sample, then it was set in the oedometer with double side drainage condition, then it was preliminary consolidated by 24-h incremental loading up to the overburden effective stress σ'_{v0} , and then a target pressure for the long-term consolidation test was loaded (the overburden effective stress σ'_{v0} , preconsolidation pressure σ'_p , and twice of preconsolidation pressure $2\sigma'_p$). Consolidation curves observed at the target pressures were drawn in Figure 4. In the case of σ'_{v0} , Osaka Bay clay shows significant delayed consolidation with convex curve, which means that the secondary consolidation index $C_{\alpha\epsilon}$ gradually increases with time. Pisa clay, however, shows concave curve, which means that the secondary consolidation index $C_{\alpha\epsilon}$ gradually decreases with time. In the case of $2\sigma'_p$, the both clays continuously shows the secondary consolidation with concave curve after the primary consolidation. In the case of σ'_p , observed behaviors for the two clays were between the above two cases, respectively.

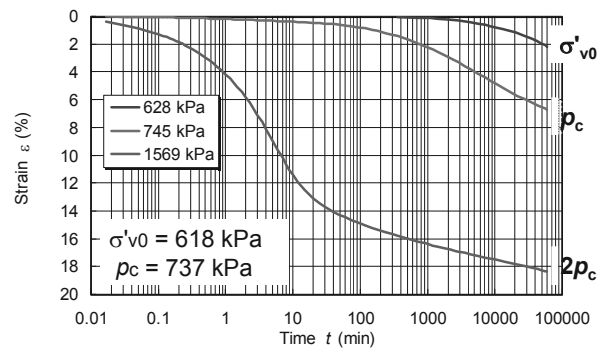


Figure 4a. Consolidation curves for the Osaka Bay clay.

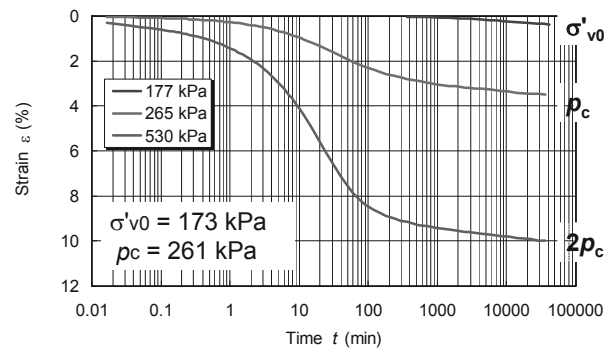


Figure 4b. Consolidation curves for the Pisa clay.

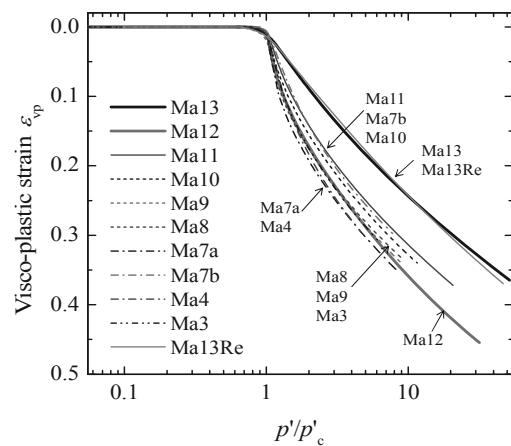


Figure 5a. Reference compression curves for the Osaka Bay clays.

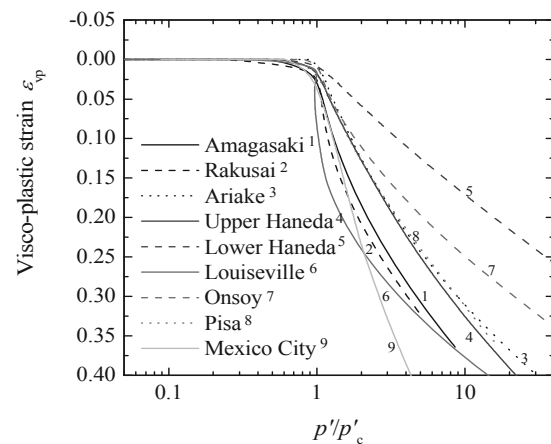


Figure 5b. Reference compression curves for the worldwide clays.

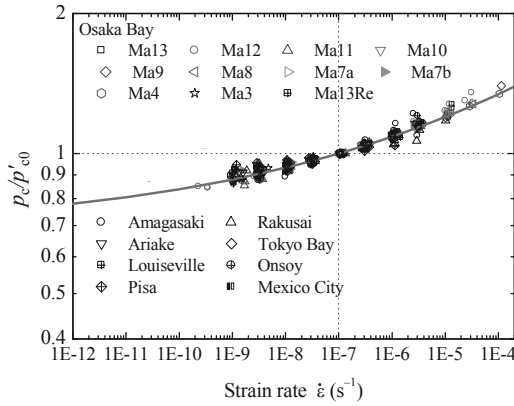


Figure 6a. Strain rate dependency for the worldwide clays.

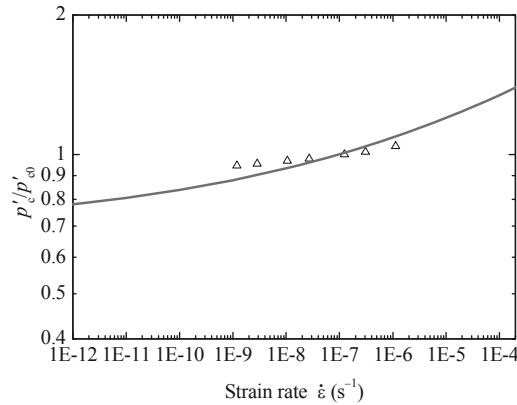


Figure 6b. Strain rate dependency for the Pisa clay.

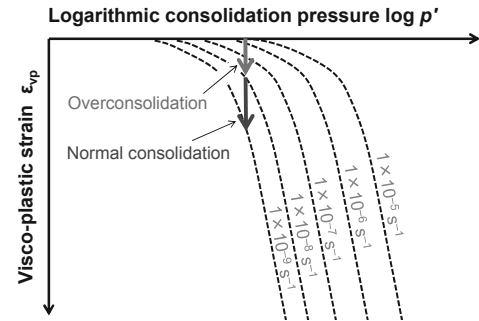
4 MODELLING WITH ISOTACHE CONCEPT

In this study, isotache concept (Šuklje, 1957) is modeled by simple equations proposed by Leroueil et al. (1985), but applied them only to visco-plastic strain ε_{vp} . Watabe et al. (2008) modeled strain rate dependency of preconsolidation pressure σ'_p as Equation (1):

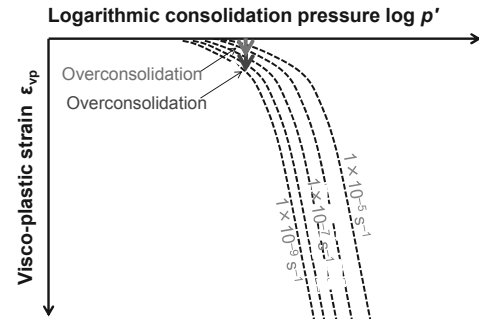
$$\ln \frac{\sigma'_p - \sigma'_{pL}}{\sigma'_{pL}} = c_1 + c_2 \ln \dot{\varepsilon}_{vp} \quad (1)$$

Here, σ'_{pL} , c_1 and c_2 are constants. Equation (1) expresses that the preconsolidation pressure σ'_p converges to a lower limit of σ'_{pL} .

Watabe et al. (2008) investigated the strain rate dependency of overconsolidation pressure for Osaka Bay clays at various depths from Holocene clay (Ma13) to Pleistocene clay (Ma12 to Ma3) up to 300 m depth, and Watabe et al. (2012) examined the applicability of Equation (1) to worldwide clays with various characteristics. Reference compression curves, in which the consolidation pressure σ'_v is normalized by the preconsolidation pressure σ'_p , obtained from the constant rate of strain consolidation tests are drawn in Figure 5. The clays examined show various compressibility. For each clay, long-term consolidation test was conducted in normal consolidation range, then the relationship between preconsolidation pressure σ'_p and strain rate $\dot{\varepsilon}_{vp}$ was obtained (Figure 6a). Here, preconsolidation pressure σ'_p is normalized by a reference value σ'_{p0} that corresponds to a strain rate of $1.0 \times 10^{-7} \text{ s}^{-1}$ (equivalent to 24-h incremental loading oedometer test). Strain rate dependency can be approximated by a unique model curve with parameters $\sigma'_{pL}/\sigma'_{p0} = 0.7$ and $c_1 = 0.935$ for Osaka Bay clays at all of the depths (Watabe et al., 2008). Note here that, when the approximate curve passes a certain point, the parameter c_2



(a) In a case of high strain rate dependency.



(b) In a case of low strain rate dependency.

Figure 7. Illustration of long-term consolidation settlement in over-consolidation domain.

automatically determined by the other two parameters σ'_{pL} and c_1 .

The parameters determined for the Osaka Bay clays are applicable to the worldwide clays examined in the previous study (Watabe et al., 2012). Consequently, the isotache concept can be commonly modeled by the unique approximation curve for the worldwide clays. The unique approximation curve is very useful; however, data for some clays, particularly for Pisa clay, is apart from it. The relationship between preconsolidation pressure σ'_p and strain rate $\dot{\varepsilon}_{vp}$ for Pisa clay is compared to the unique approximation curve in Figure 6b. Preconsolidation pressure for Pisa clay does not decrease so much with decrease of strain rate, indicating that the strain rate dependency of Pisa clay is smaller than that of the other clays.

5 DISCUSSION

The key factor to model the isotache concept is the strain rate dependency of preconsolidation pressure. From the previous studies, it was found out that the strain rate dependency can be expressed by the unique approximation curve. Pisa clay, however, shows particularly smaller strain rate dependency than the other clays. This different dependency strongly influences the long-term consolidation behavior in over-consolidated domain. In practice, consolidation settlement is predicted based on the compression curve corresponding to a strain rate of $1.0 \times 10^{-7} \text{ s}^{-1}$. Because the Osaka Bay clay has high strain rate dependency, overconsolidation with a higher strain rate at the beginning can be eventually changed to normal consolidation with a smaller strain rate. Figure 7a illustrate the mechanism for the significant delayed long-term consolidation when the Osaka Bay clay was loaded in slightly overconsolidation. On the other hand, because the Pisa clay has low strain rate dependency, overconsolidation with a higher strain rate at the beginning can be eventually maintained in overconsolidation with a smaller strain rate. Figure 7b illustrate the mechanism for that the Pisa clay remained in overconsolidation when it was loaded in slightly overconsolidation.

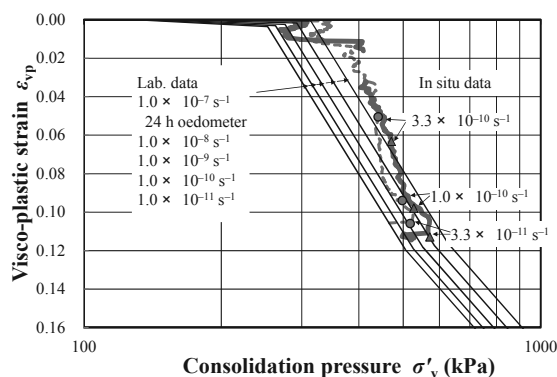


Figure 8. Measured field compression curves in Kansai International Airport Phase 1 (Ma12).

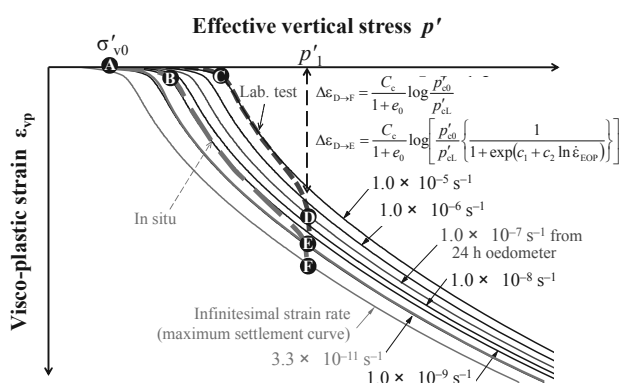


Figure 9. Isotache compression curves in the laboratory and prediction of long-term consolidation settlement.

From comparison between the Osaka Bay clay and Pisa clay, one of the main reasons of those different long-term consolidation tendencies seems to be the existence of microfossils, which are found abundantly in Osaka Bay clay, but barely in Pisa clay. However, it is not true. Because the unique approximation curve was determined for worldwide clays with various characteristics with or without microfossils, it cannot be said that the main reason is the existence of microfossils. From X-ray diffraction, clay minerals in most of the clays examined in the previous studies were smectite and kaolinite. As mentioned above, the Osaka Bay clay also mainly consists of smectite and kaolinite, but only the Pisa clay among the clays examined mainly consists of illite. Therefore, it indicates that illite results in a small strain dependency; i.e., one of the main reasons of different strain dependency is the clay minerals.

For the Pisa clay, because strain rate dependency is not significant, long-term consolidation settlement can be approximately predicted based on the compression curve corresponding to a strain rate of $1 \times 10^{-7} \text{ s}^{-1}$. In fact, settlement of the Leaning Tower of Pisa has been successfully predicted without regard to strain rate effect (Burland et al., 2003), even after about 800 years since its construction. In contrast, because strain rate dependency of the Osaka Bay clay is significant, long-term consolidation settlement predicted based on the compression curve corresponding to a strain rate of $1 \times 10^{-7} \text{ s}^{-1}$ results in underestimation. In situ compression curves observed by sublayered measurements of settlement and pore-water pressure at the first phase of the Kansai International Airport are drawn in Figure 8 together with isotache curves deduced from 24-h incremental loading oedometer test. The in situ measured data crosses the isotache compression curves diagonally from higher to lower strain rates, indicating that the behavior is consistent with the isotache modeling.

Because the strain rate dependency of the Osaka Bay clay is similar to the other worldwide clays, it can be said that the

strain rate dependency has to be considered in prediction of long-term consolidation settlement for most of the clays. Prediction method of field settlement from the overburden effective stress σ'_{v0} to a certain consolidation pressure p_1 is illustrated in Figure 9. The compression curve observed in the laboratory long-term consolidation test follows the path $A \rightarrow C \rightarrow D \rightarrow E$. At the end of primary consolidation, strain rate is in an order of $1.0 \times 10^{-5} \text{ s}^{-1}$, then it passes the point D at $1.0 \times 10^{-7} \text{ s}^{-1}$, and then it reaches the point E at a much smaller strain rate. When long-term consolidation test is conducted, strain rate easily decreases to $1.0 \times 10^{-9} \text{ s}^{-1}$ in 2–4 weeks, but it hardly decreases to $1.0 \times 10^{-10} \text{ s}^{-1}$ because it requires several months. In situ strain rate for the Osaka Bay clay is in an order of $1.0 \times 10^{-11} \text{ s}^{-1}$, which is much smaller than the laboratory strain rate of $1.0 \times 10^{-9} \text{ s}^{-1}$, and the compression curve follows a path $A \rightarrow B \rightarrow E \rightarrow F$. According to a conventional method, the field settlement is predicted as point D based on the result of 24-h incremental loading oedometer test; however, the real field settlement could be point E. In practice, in situ consolidation settlement can be predicted corresponding to the strain rate, which is predicted in association with the thickness of the clay layer. In addition, ultimate consolidation settlement possibly reaches point F. Using the initial void ratio e_0 and compression index C_c , the additional consolidation strains from D to E ($\Delta \varepsilon_{D \rightarrow E}$) and from D to F ($\Delta \varepsilon_{D \rightarrow F}$) can be calculated from geometric relationship (equations are shown in Figure 9). These in situ additional strains in association with the strain rate dependency are significant, particularly in a case of thick clay layer.

6 SUMMARY

In the present study, the isotache concept observed in the long-term consolidation behavior was successfully modeled by Equation (1), in which preconsolidation pressure decreases with decrease of strain rate and converges to a lower limit at an infinitesimal strain rate, then the difference in long-term consolidation behavior between the Osaka Bay clay and Pisa clay were compared and discussed using this model. In the long-term consolidation in an overconsolidation domain, significant delayed settlement was observed in the Osaka Bay clay, but not in the Pisa clay. The significant delayed settlement seems strange; however, it can be said that this strange behavior is inherently natural because it is caused by the common strain rate dependency for the worldwide clays. Therefore, the little delayed settlement observed in the Pisa clay is rather strange. Using the isotache model, the long-term consolidation settlement can be quantitatively predicted in association with the strain rate dependency.

6 REFERENCES

- Burland J.B., Jamiolkowski M. and Viggiani C. 2003. The stabilisation of the Leaning Tower of Pisa, *Soils and Foundations*, 43(5), 63–80.
- Leroueil S., Kabbaj M., Tavenas F. and Bouchard, R. 1985. Stress-strain-strain rate relation for the compressibility of sensitive natural clays, *Géotechnique*, 35(2), 159–180.
- Šuklje L. 1957. The analysis of the consolidation process by the isotache method, *Proc. 4th Int. Conf. on Soil Mech. Found. Engng.*, London, Vol.1, 200–206.
- Watabe Y., Udaka K. and Morikawa Y. 2008. Strain rate effect on long-term consolidation of Osaka bay clay, *Soils and Foundations*, 48(4), 495–509.
- Watabe Y., Udaka K., Nakatani Y. and Leroueil S. 2012. Long-term consolidation behavior interpreted with isotache concept for worldwide clays, *Soils and Foundations*, 52(3), 449–464.

Settlement and shear strength of uncemented coal mine overburden materials placed loose under dry and wet conditions

Tassement et résistance au cisaillement de matériaux de couverture non cimentés extraits de mines de charbon et mis en dépôt en vrac dans des conditions sèches et humides

Williams D.J., Kho A.K.
The University of Queensland, Brisbane, Australia

ABSTRACT: The overburden materials in the coalfields of South East Queensland, Australia, are dominated by essentially uncemented rocks, which rapidly break down on excavation to extract coal. On excavation, these uncemented materials bulk up to a very loose density. On end-dumping by haul truck in spoil piles, these loose materials then undergo three forms of settlement: that due to their self-weight, "collapse" settlement on wetting-up by rainfall, and settlement due to degradation on exposure to weather, resulting in a substantial increase in density. Due to their lack of cementation, these overburden materials degrade rapidly on exposure to the weather, leading to significant settlement, followed by some reversal on re-agglomeration and swell. Collapse and weathering-induced settlements, both being associated with exposure to water, occur simultaneously on wetting-up. Wetting-up also causes a substantial reduction in the shear strength of the materials. The paper quantifies the settlement and shear strength of uncemented overburden materials excavated on open pit mining in the coalfields of South East Queensland.

RÉSUMÉ: Les matériaux de couverture dans les bassins miniers du Sud-Est du Queensland, en Australie, sont principalement constitués par des roches non cimentées, qui se décomposent rapidement lors de l'excavation pour extraire le charbon. Lors de l'excavation, ces matériaux non cimentés sont mis en dépôt en vrac dans un état très lâche (très faible densité). Après mise en dépôt par des camions dans des zones de stockage, ces matériaux subissent trois formes de tassement : celui résultant de leur poids propre, le tassement d'"effondrement" résultant du mouillage par la pluie et le tassement dû à la dégradation lié à l'exposition au climat. Ces tassements entraînent une augmentation substantielle de la densité. En raison de leur non cimentation, l'exposition au climat entraîne une dégradation rapide de ces matériaux de couverture, se traduisant par des tassements significatifs, suivis par des processus de reformation d'agrégats et de gonflement. Les tassements résultant du phénomène d'effondrement et des dégradations dues au climat, tous deux liés à l'exposition à l'eau, se développent simultanément lors du mouillage. Le mouillage entraîne aussi une réduction significative de la résistance au cisaillement des matériaux. La communication présente une quantification du tassement et de la résistance au cisaillement de ces matériaux de couverture excavés dans les mines à ciel ouvert des bassins miniers du sud-est du Queensland.

KEYWORDS: coal mine, compression, degradation, dry, overburden materials, settlement, shear strength, wet.

MOTS CLES : mine de charbon, compression, dégradation, sec, matériaux de couverture, tassement, résistance au cisaillement, humide

1 INTRODUCTION

The net bulking of coal mine overburden materials from their *in situ* state on open cut mining results in an increased volume required to accommodate the excavated spoil. Knowing the bulking and subsequent self-weight, collapse and degradation-induced settlement of the spoil over time is important for estimating the storage volume required in mined-out pits and in out-of-pit piles.

The majority of the self-weight settlement occurs during placement ((Naderian *et al.*, 1996), with only a residual 20%, or so, occurring after the end of construction. Collapse settlement on wetting-up of the placed spoil by rainfall and/or groundwater infiltration requires just sufficient wetting-up to saturate the micro-cracks that occur at highly-stressed particle contacts. On collapse of these particle contacts, the coarse-grained particles break down, creating multiple, less-stressed particle contacts. Further wetting-up of the spoil then essentially fills the pores between the broken particles, without leading to significant further settlement.

Uncemented coal mine overburden materials bulk-up substantially on excavation, but also settle significantly due to their self-weight or the height of spoil, "collapse" on wetting-up, and degradation on exposure to weather cycles. Uncemented weathered rock spoil sampled from Jeebropilly Coal Mine in the Ipswich Coalfields of South East Queensland, Australia is characterised and its shear strength, compression under loading,

and degradation due to exposure to weather cycles are described. Laboratory testing was carried out on scalped specimens at the as-sampled gravimetric moisture content (that is, "dry") and in a water bath (that is, "wet"), which highlighted the high potential for this material to slake and disperse, and to collapse and breakdown, on wetting-up.

2 SAMPLING OF JEEBROPILLY WEATHERED ROCK

The uncemented weathered rock overburden found in the depth range from about 5 to 20 m in an open pit at Jeebropilly Coal Mine was sampled from haul truck dumps. It was shaken through a 19 mm sieve fitted to the top of a 20 litre bucket. The sample passing 19 mm (-19 mm) and the +19 mm oversize were both weighed, and the oversize was photographed for later estimation of its particle size distribution.

3 SPOIL CHARACTERISATION

The characterisation testing of the Jeebropilly weathered rock was broadly carried out in accordance with AS 1289. The as-sampled gravimetric moisture content of the -19 mm scalped Jeebropilly weathered rock was 14.9%, and the Liquid and Plastic limits of the -0.0425 mm fraction were 71.0% and 21.0%, respectively, giving a Plasticity Index of 50.0% and indicating a Unified Soil Classification of CH (Clay of High

Plasticity). Note that the as-sampled moisture state was drier than the Plastic limit. In the as-sampled moisture state, the -2.36 mm scalped sample had a total suction, measured using a WP4 Dewpoint Potential Meter, of 4,320 kPa. The specific gravity of the solids, measured using a helium pycnometer, was 2.60.

The electrical conductivity (EC) and pH of a 5 (deionised water) to 1 (-2.36 mm scalped, dry solids) paste of Jeebropilly weathered rock were 356 $\mu\text{S}/\text{cm}$ and 4.0, respectively. From the measured EC, corrected to 25°C, and the relationship between EC and osmotic suction (after U.S.D.A. 1954), an as-sampled osmotic suction of 10 kPa was obtained, and a matric suction of 4,310 kPa was obtained by subtraction from the measured total suction. The Emerson Class Number of the Jeebropilly weathered rock was 1, implying that the material slakes and disperses completely, and had a high potential for breakdown and erosion.

Mg^{2+} is the dominant cation in the Jeebropilly weathered rock, and its high cation exchange capacity (CEC) of 16 $\text{cmol}(+)/\text{kg}$ and sodium adsorption ratio (SAR) of 0.76 suggest a relatively high clay content, low permeability, limited structure, the potential for compaction under trafficking, and the potential for internal erosion.

The particle size distribution curves obtained from photographs using SplitDesktop (<http://www.spliteng.com/split-desktop/>) or by dry sieving after air-drying of the -19 mm scalped Jeebropilly weathered rock are shown in Figure 1, together with the scalped curves obtained by the removal of the oversize, for use in the various laboratory tests.

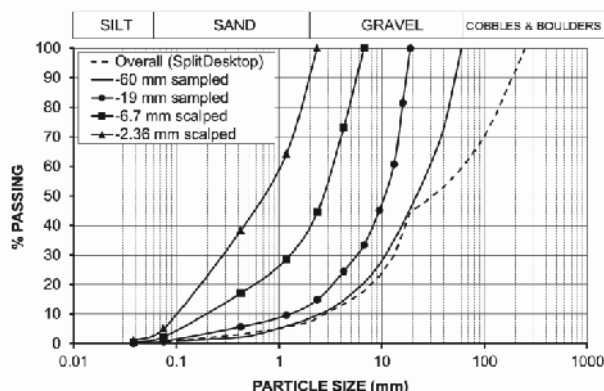


Figure 1. Particle size distribution curves of Jeebropilly weathered rock overall, and scalped for use in various laboratory tests.

The particle size distribution curves obtained for the -19 mm scalped Jeebropilly weathered rock by dry sieving after air-drying or 60°C oven-drying, by wet sieving using tap water without or with dispersant added, and by hydrometer analysis in deionised water without or with dispersant added, are shown in Figure 2.

It is clear from Figure 2 that Jeebropilly weathered rock is extremely prone to breakdown in the presence of water, dominated by the gravel fraction breaking down to sand-size, with no appreciable generation of silt and clay fines. The results of the hydrometer analyses suggest that the fines content is less than 10% by mass, and that the clay content is negligible. However, it is likely that the intrinsic particle size does contain appreciable silt and clay fines, and that these would require more energy to be released than is applied in the standard test procedure. The effect of adding dispersant is minor.

Laboratory Standard compaction testing of -19 mm scalped Jeebropilly weathered rock indicated a low Maximum Dry Density (MDD) of only 1.52 t/m^3 , at an Optimum (gravimetric) Moisture Content (OMC) of 19.0%, close to the Plastic Limit of the material. At the MDD and OMC, the material has a high porosity of 0.42 and a low degree of saturation of 69%.

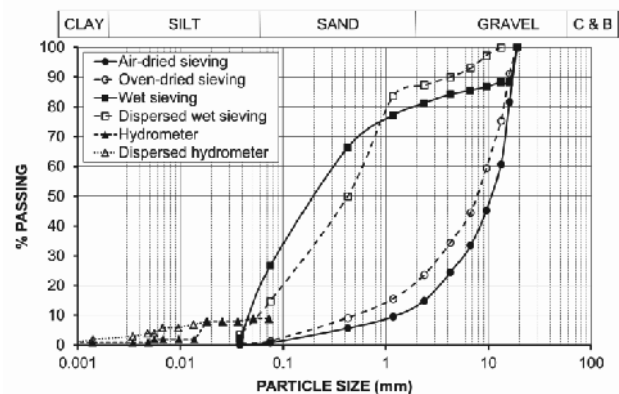


Figure 2. Particle size distribution curves of -19 mm scalped Jeebropilly weathered rock obtained with different pre-treatments and testing methods.

4 SHEAR STRENGTH

Shear strength testing of loosely-placed, -2.36 mm scalped Jeebropilly weathered rock, at its as-sampled gravimetric moisture content of 14.8% (that is, tested “dry”) or tested in a water bath (that is, tested “wet”), was carried out in a 60 mm direct shear box. It demonstrated that the shear strength envelopes obtained were not substantially dependent on the shearing rate over the range from 0.01 mm/min to 1 mm/min; implying that drained conditions existed over this range.

On testing dry, the loose-placed dry density of the Jeebropilly weathered rock averaged 0.908 t/m^3 . This increased to an average 1.226 t/m^3 after 24-hour compression under a normal stress of 500 kPa, and to an average 1.370 t/m^3 (75% of MDD) after shearing. Compression increased the average degree of saturation from the loose-placed value of 20.7 to 34.4% and 36.9% after shearing, as the average porosity decreased from 0.650, to 0.528 to 0.510, respectively.

On testing wet, the loose-placed dry density of the Jeebropilly weathered rock averaged 0.991 t/m^3 . This increased to an average 1.351 t/m^3 after 24-hour compression under a normal stress of 150 kPa, and to an average 1.445 t/m^3 (82% of MDD) after shearing. Compression decreased the average gravimetric moisture content from the loose-placed value of 70.4 to 35.5% and 34.3% after shearing, as the average porosity decreased from 0.647, to 0.480 to 0.471, respectively.

The shear strength parameters obtained for -2.36 mm scalped Jeebropilly weathered rock are summarised in Table 1, in terms of apparent cohesion and friction angle, and friction angle only (assuming zero cohesion). These values are in broad agreement with those recommended for comparable coal mine spoil materials (Simmons 1995).

Shear strength testing of -2.36 mm scalped Jeebropilly weathered rock was also carried out in a 300 mm direct shear box at a shearing rate of 1 mm/min. The shear strengths obtained from the 60 mm and 300 mm direct shear box testing at a shearing rate of 1 mm/min are compared in Figure 3. The results from the smaller shear box cover a far greater range between dry and wet conditions than those from the 300 mm shear box, and appear more plausible.

Table 1. Shear strength parameters for Jeebropilly weathered rock.

State	Dry	Wet
Apparent cohesion (kPa)	29	6.0
Apparent friction angle (deg.)	27.2	19.5
Friction only angle (deg.)	35.2	22.0

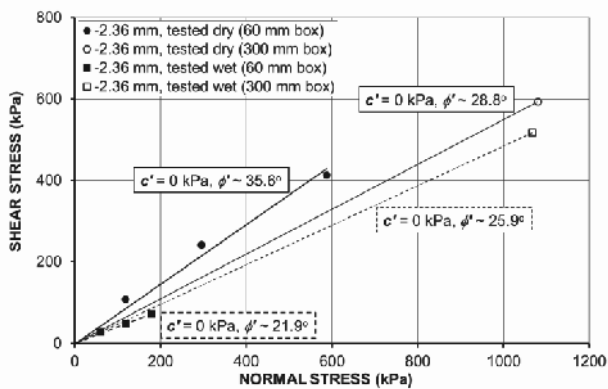


Figure 3. Comparison of friction only strength envelopes for 60 mm and 300 mm direct shear testing of -2.36 mm scalped Jeebropilly weathered rock.

5 COMPRESSION UNDER LOADING

Staged, and single-stage creep, compression testing of loosely-placed, -2.36 mm Jeebropilly weathered rock, at its as-sampled gravimetric moisture content (that is, tested “dry”) or tested in a water bath (that is, tested “wet”), was carried out in a 76 mm diameter oedometer. The compression curves for the staged testing under dry and wet conditions are shown in Figure 4, indicating Compression Indices C_c of 0.408 and 0.271, respectively.

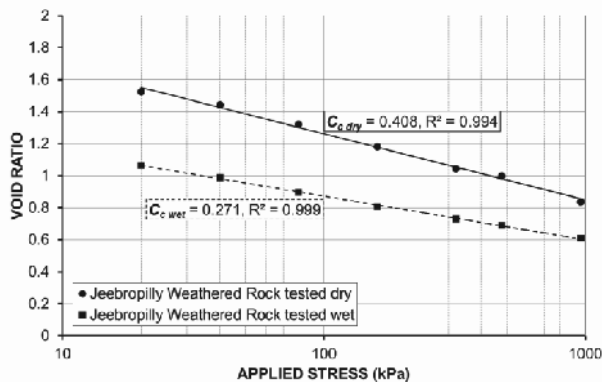


Figure 4. Compression curves for staged testing of loose-placed, -2.36 mm scalped Jeebropilly weathered rock under dry and wet conditions.

The % settlement (relative to the initial loose height) versus $\log_{10}$ time plot for a loosely-placed specimen tested dry under a single-stage creep loading of 500 kPa, over 14 days, shown in Figure 5, indicates a self-weight settlement rate of 4.5%/log₁₀ cycle of time, and a creep settlement rate of 0.5%/log₁₀ cycle of time.

From the staged testing under wet conditions and the creep testing under dry conditions, coefficient of consolidation c_v values may be calculated as a function of void ratio e , as shown in Figure 6. It is noteworthy that the data point from the single-stage creep test carried out under dry conditions lines up with the trendline for the data points from the staged testing under wet conditions. The “dry” specimen, tested at its as-sampled gravimetric moisture content of 14.8% (initial degree of saturation of 24.7%), achieved a degree of saturation of 40.0% by the end of the test. The relatively high final degree of saturation for the dry test goes some way towards explaining why the dry data point lines up with the wet trendline.

From the calculated coefficient of consolidation c_v , and coefficient of volume decrease m_v values, the saturated

hydraulic conductivity k_v values may be calculated from Equation (1).

$$k_v = c_v \cdot m_v \cdot \gamma_w \quad (1)$$

where γ_w is the unit weight of water = 9.81 kN.m⁻³. The resulting k_v values are plotted in Figure 7, which demonstrate the low permeability of wet -2.36 mm scalped Jeebropilly weathered rock, even in a relatively loose state.

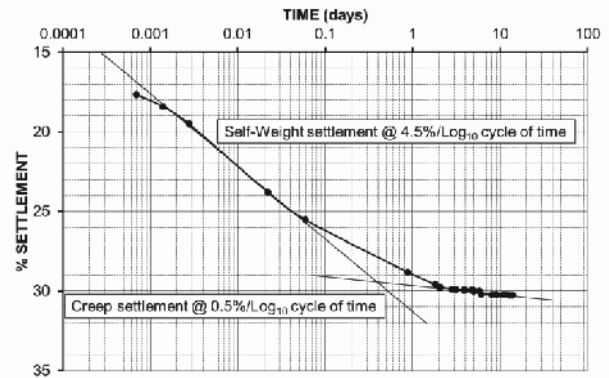


Figure 5. Single-stage creep loading of loose-placed, dry, -2.36 mm scalped Jeebropilly weathered rock under 500 kPa for 14 days.

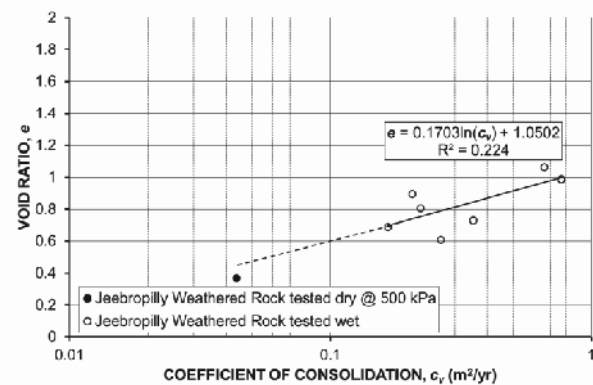


Figure 6. Coefficient of consolidation for loose-placed, -2.36 mm scalped Jeebropilly weathered rock under wet and dry conditions.

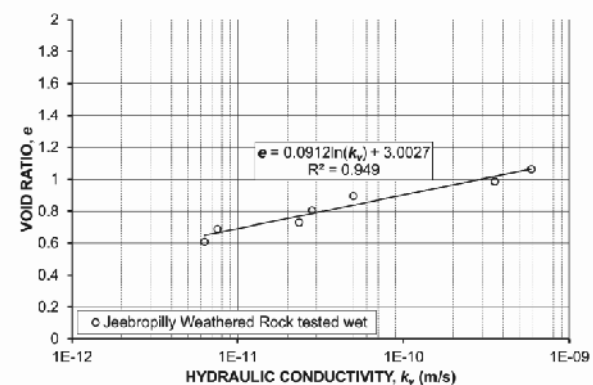


Figure 7. Hydraulic conductivity for loose-placed, -2.36 mm scalped Jeebropilly weathered rock under wet conditions.

6 DEGRADATION ON EXPOSURE

Jeebropilly weathered rock scalped to -19 mm was loose-placed to a nominal depth of 100 mm in a Perspex tray measuring

600 mm square by 150 mm deep and left exposed to the weather for 35 days, during which time it experienced wetting by a number of rainfall events totalling 113 mm, with desiccation between these rainfall events. The rainfall was recorded, the net settlement of the surface was measured regularly at nine points over the area of the specimen, and sub-samples were taken at the same time intervals for air-drying and dry sieving to determine the change in the particle size distribution.

The specimen underwent major particle breakdown, followed by re-agglomeration on desiccation crusting, as shown visually in Figure 8. Changes in particle size distribution during the course of the degradation test are shown in Figure 9. Figure 10 shows the cumulative rainfall and average net % settlement relative to the initial specimen height with time during the degradation test. Both Figures 9 and 10 clearly show the effects of the initial major particle breakdown and subsequent re-agglomeration, leading to reversals in the particle size distribution curves and average net % settlement.

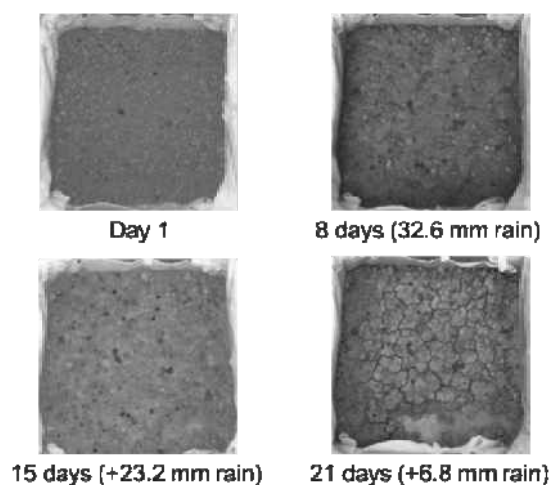


Figure 8. Photographs of major particle breakdown and re-agglomeration on crusting during degradation test on loose-placed, -19 mm scalped Jeebropilly weathered rock.

7 COMBINED SETTLEMENT OF JEEBROPILLY WEATHERED ROCK

Based on the results of the laboratory testing, the self-weight settlement of initially relatively dry, loose-dumped, uncemented Jeebropilly weathered rock spoil under 500 kPa (equivalent to a spoil pile height of about 30 m) could amount to about 15% of the initial spoil height, of 6% (20% of this) would likely occur post-construction. Wetting-up of loose-dumped, uncemented Jeebropilly weathered rock spoil could cause collapse settlement of a further 15% of the initial spoil height. Degradation-induced settlement of loose, uncemented Jeebropilly weathered rock spoil could be 15 to 25% of the initial height, although this might only occur to limited depth. Overall, the combined settlement of a 30 m high pile of loose-dumped, uncemented Jeebropilly weathered rock spoil, also subjected to collapse and degradation on wetting-up, could be up to 36 to 46% of the initial loose spoil height.

Coal mine open pits up to 500 m deep are being planned in the Hunter Valley Coalfields of New South Wales, Australia, which would result in spoil piles up to 600 m high. These could settle about 40% of the initial loose spoil height, resulting in a net bulking of the order of 12% relative to the initial *in situ* dry density of about 1.87 t/m³.

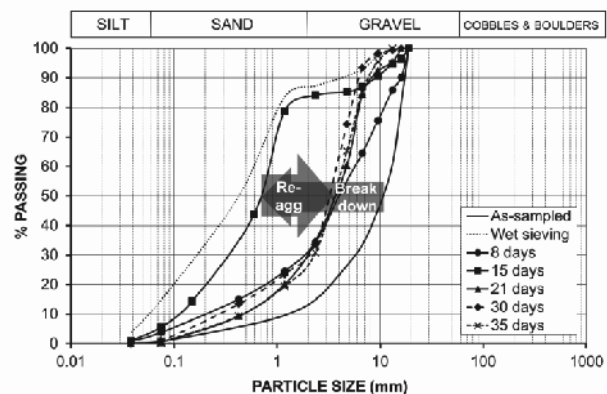


Figure 9. Changes in particle size distribution curves with time during degradation test on loose-placed, -19 mm scalped Jeebropilly weathered rock.

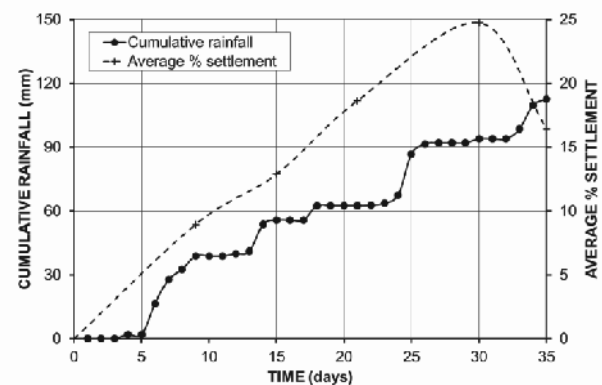


Figure 10. Cumulative rainfall and average net % settlement relative to the initial specimen height with time during degradation test on loose-placed, -19 mm scalped Jeebropilly weathered rock.

8 CONCLUSION

The laboratory testing of uncemented Jeebropilly weathered rock spoil has identified and quantified the components of settlement on loading under dry and wet conditions, and on wetting-up alone. These are substantial and reduce, to a relatively modest amount, the initial considerable bulking on excavation and loose-dumping of the overburden in spoil piles. The final dry density of a high spoil pile of uncemented weathered rock spoil could eventually approach that achieved by laboratory Standard compaction of the material.

9 ACKNOWLEDGEMENTS

The research on which this paper is based forms part of Australian Coal Association Research Program Project C19022, whose funding is greatly appreciated, as are Jeebropilly Coal Mine personnel for assisting with spoil sampling.

10 REFERENCES

- AS 1289. *Testing Soils for Engineering Purposes*. Standards Australia.
- Naderian, A.R., Williams, D.J. and Clark, I.H. 1996. Numerical modelling of settlements in back-filled open-cut mines. *International Journal of Surface Mining, Reclamation, and Environment*, 10, 25-29.
- Simmons J. 1995. Slope Stability for Open Cut Coal Mining. *Australian Geomechanics Journal*, 29, 45-57.
- U.S.D.A. 1954. *U.S.D.A. Agricultural Handbook No. 60, Diagnosis and Improvement of Saline and Alkali Soils*.

Anisotropic strength ratio and plasticity index of natural clays

Étude de la relation entre l'anisotropie et l'indice de plasticité des argiles naturelles

Won J.Y.
Technip

ABSTRACT: A number of K_0 -consolidated triaxial compression and extension test results were collected to re-evaluate the relationship between anisotropic strength ratio and plasticity index of natural clays. The database covers 203 pairs of triaxial tests performed on normally consolidated natural clays from 14 countries, published between 1972 and 2007. Data selection criteria were established for a consistent comparison. The anisotropy was strongly influenced by the definitions of failure in extension tests. Once the anisotropy data are grouped into their depositional environments, no general trend of anisotropy with plasticity index can be observed. The well-known trend that anisotropy decreases with plasticity index cannot be justified. A trend of anisotropy correlated exclusively with plasticity index can be misleading. Anisotropic characteristics of a natural clay should be evaluated by careful consideration of site specific characteristics, spatial variability, depositional and post-depositional environments of the clays.

RÉSUMÉ : Des résultats d'essais triaxiaux K_0 -consolidés en compression et en extension ont été collectés pour réexaminer la relation entre l'anisotropie et l'indice de plasticité des argiles naturelles. Les données consistent en 203 séries d'essais sur des argiles normalement consolidées de 14 pays, et publiés entre 1972 et 2007. Un critère de sélection des données a été établi pour faire des comparaisons cohérentes. L'anisotropie est fortement corrélée avec le type de rupture en extension. En groupant les données en fonction de leurs conditions de dépôts, aucune corrélation entre l'anisotropie et l'indice de plasticité n'est observée. La diminution bien connue de l'anisotropie en fonction de l'indice de plasticité reste inexpliquée. Considérer que l'anisotropie ne dépend que de l'indice de plasticité serait erroné. L'anisotropie d'une argile naturelle doit être évaluée en tenant compte soigneusement des caractéristiques spécifiques du site, de la variabilité spatiale, des environnements de dépôts et post-dépôts de l'argiles.

KEYWORDS: Clay, Undrained Shear Strength, Anisotropy, Plasticity, Triaxial Test, Extension Test.

1 INTRODUCTION

Undrained shear strength of natural clays shows anisotropy, i.e. it displays different undrained shear strengths under different shear stress conditions and directions. Anisotropy in stiffness, permeability, and shear strength comes from stress-induced and inherent anisotropy. Anisotropic characteristics of natural clays play a significant role in many geotechnical applications, such as slope stability, and bearing capacity of shallow and deep foundations both in onshore and offshore applications. Anisotropy is routinely investigated for offshore foundations (driven piles and suction anchors). In onshore projects, however, the anisotropy is seldom investigated because of many reasons. In many cases, thus, geotechnical engineers resort to empirical trends to evaluate anisotropy of the soil of interest.

It is generally known that the anisotropy decreases with plasticity index, i.e. high plastic clays are more isotropic than low plastic clays. The trend was reported by Berre and Bjerrum (1973) and Ladd et al. (1977) decades ago, based on a limited number of test results. Besides the debates whether or not plasticity index alone can represent soil characteristics, there have been numerous anisotropy test results published since then. Therefore, it is worthwhile to collect and analyze the available anisotropy information to re-evaluate the anisotropic undrained shear strength characteristics of natural clays.

A number of published K_0 -consolidated triaxial compression and extension tests performed on normally consolidated natural clays were collected. A total of 203 pairs of K_0 -consolidated triaxial compression and extension test results from 14 countries were analyzed. Different test conditions and methods of the data necessitated consistent data selection criteria. The data were grouped into their regions and/or depositional environments. This paper presents review on depositional environments, discussion on test conditions, and re-evaluation of the generally accepted anisotropy trend.

2 METHODOLOGY

From an extensive literature review, K_0 -consolidated anisotropy test results were collected. The published data and test

conditions were carefully reviewed to select acceptable data. The database was, subsequently broken down into respective regions or depositional environments including Scandinavia, Canada, Europe, Middle East, Japan, and East Asia.

2.1 Anisotropy

In this study, the anisotropy was evaluated by K_0 -consolidated triaxial compression (CK_0UC) and extension (CK_0UE) tests. Anisotropic strength ratio ($K_s = S_{uE}/S_{uC}$) was defined as the undrained shear strength ratio of extension strength (S_{uE}) to compression strength (S_{uC}). The higher the anisotropic strength ratio is, the lower the anisotropy is.

2.2 Data selection criteria

Anisotropy can be evaluated by various test methods such as plane strain, hollow cylinder apparatus, triaxial compression and extension tests as well as field vane tests with different shape and length of blades. Each method measures different aspects of anisotropy. Only CK_0UC and CK_0UE test results on undisturbed natural clays were selected and analyzed. For consistency, data from other test methods were excluded. It was found that many published data did not include detailed test conditions. To avoid unnecessary discussion on the effects of consolidation methods for anisotropic triaxial tests, only the results that followed the recompression concept i.e., consolidate the specimen to an estimated in-situ overburden stress (σ'_{vc}) before the undrained shear, were selected. Many test data for USA clays were excluded because the majority of them were obtained from the SHANSEP (Stress History and Normalized Soil Engineering Parameters) approach. The following data selection criteria were established for a consistent comparison.

- Geologically normally consolidated clays
- Apparently (lightly) overconsolidated clays
- K_0 (or anisotropically)-consolidated undrained triaxial
- Undisturbed natural clays
- Consolidation method: Recompression

- Inorganic clays

The data from the following conditions were excluded.

- Geologically overconsolidated clays
- Rotating angles of tested specimens
- Consolidation method: SHANSEP
- Isotropically consolidated triaxial
- Hollow Cylinder Apparatus test results
- Unconfined compression or unconsolidated undrained
- Organic clays and peat
- Artificial, remolded or resedimented clays

2.3 Anisotropy database

A total of 203 pairs of CK_oUC and CK_oUE test results were collected. The 53 resources reported by Mayne (1983) were also carefully reviewed following the data selection criteria listed above. The database covers 14 countries and data published between 1972 and 2007. A relatively large amount of anisotropy data (86) have been published for Japanese natural clays. Statistical information of the database is summarized in Table 1.

Table 1. Statistical information of the database

Area	Country	Local name	No. data	Plasticity Index	K _s (S _{uE} /S _{uC})
Northern	Finland	Kimola	1	31	0.70
Europe	Norway	Various sites	19	4~32	0.23~0.78
	Sweden	Various sites	4	26~53	0.63~0.79
Europe	Italy	Porto Tolle, Trieste	2	30~47	0.61~0.81
	France	Various sites	3	54~67	0.49~0.64
	UK	Bothkennar	9	28~43	0.36~0.58
North	Canada	NBR and other sites	21	5~57	0.41~0.74
America	USA	Gulf of Mexico	2	33~55	0.78~0.79
Asia	Iraq	Khor Al-Zubair, Fao	10	18~36	0.50~0.89
	Japan	Osaka bay	25	50~71	0.55~0.91
		Izumo	21	25~104	0.69~1.06
		Ariake	13	36~81	0.60~1.00
		Kinkai	11	22~80	0.64~1.08
		Tokyo bay	8	36~50	0.59~0.73
		Various sites	8	35~75	0.48~0.64
	Indonesia	Banjarmasin, Surabaya	3	60~85	0.65~0.83
	Singapore	Singapore	3	34~58	0.88~1.08
	Thailand	Bangkok	12	26~77	0.74~1.27
	Korea	Namak	20	22~41	0.45~0.65
		Shihwa	8	14~25	0.66~0.89
	14 countries		203	4~104	0.23~1.27

2.4 Definitions of failure in extension test

There are two common definitions of the failure for triaxial extension test (Tanaka et al. 2001a).

- Definition-A: strength at the same strain level (typically less than 2 %) as the peak strength from a compression test
- Definition-B: peak strength (necking failure) or strength at 15 % of axial strain if the peak is not observed

It is evident that undrained shear strength values from CK_oUE will be significantly different, depending on the definitions of failure. An example of the different failure definitions is shown in Figure 1: A specimen from 8.4m depth has $S_{uC}/\sigma'_{vc}=0.202$ and the corresponding extension specimen has $S_{uE}/\sigma'_{vc}=-0.118$ and -0.164 following the failure definition-A and -B, respectively. The difference in anisotropic strength ratio between the definition-A ($K_s=0.584$) and B ($K_s=0.812$) is about 40 %. Irrespective of the failure modes (necking or ductile) in extension tests, the difference between the two failure definitions will be substantial. Many published data did not have its definitions of failure for either triaxial compression or extension tests.

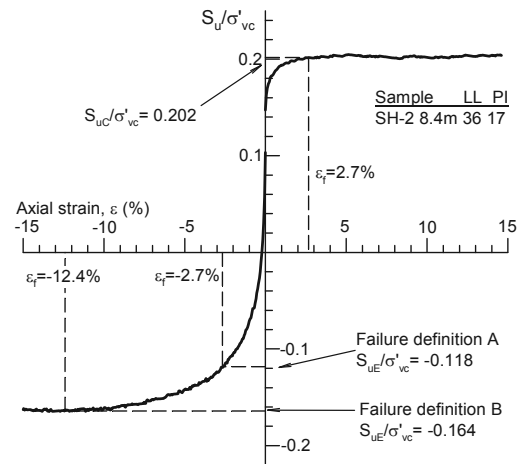


Figure 1. An example of the definitions of failure for CK_oUE test.

2.5 Strain rate effects

It was found that many authors and institutes used different strain rates for the undrained shearing. For example, strain rates of the undrained (compression/extension) shearing were 0.75 %/hr for NGI in Norway, 6.0 %/hr for PHRI in Japan, 0.18 %/hr for UK, 0.5 %/hr for USA. The two Korean marine clays were sheared at 3.0 %/hr. Strain rate effects on anisotropy are not covered in this study and are assumed to be insignificant.

3 DEPOSITIONAL ENVIRONMENTS

For further comparison, it is vital to review depositional and post-depositional environments of the clays presented in this study, with regard to their physical and geotechnical properties.

3.1 Scandinavian clays

Depositional environments and the following geological events (leaching) in Scandinavian clays were well documented by Bjerrum (1954) as the causes of quick clay formation. A typical feature of glacially derived clays is that the clay fraction (< 2 μm) contains considerable quantities of non-clay minerals (rock flour). For example, the rock flour of the Drammen clay was produced by the abrasive action of glacial ice during the Ice Age (Tanaka et al. 2001a). As a result, the Drammen clay has a high clay-size fraction (40-50%) but plasticity index is approximately 20 (Tanaka and Tanaka 1997).

3.2 European and Gulf of Mexico (USA) clays

Unlike the well-documented Bothkennar clay in the United Kingdom (Nash et al. 1992, Tanaka et al. 2003), depositional environments of the other European clays are not readily available. The Bothkennar clay is characterized as a Holocene deposit with low clay fractions. Abundant thin laminations and mottled features are commonly observed. Overconsolidation ratios are about 2 without any stress changes after its deposition, is most probably caused by ageing or cementation. The Bothkennar clay was formed since 8,500-6,000 years B.P. (before present) under an estuary environment. The Italian and French clays in the database are geologically normally consolidated deltaic clays. Information of the depositional environments of the two data points from an offshore site, Gulf of Mexico, USA is not available. However, the marine clay deposits in the Gulf of Mexico are known as geologically normally consolidated Pleistocene or Holocene clays, except for the cases where excess pore pressure is present. Abundant smectite, yet moderate sensitivity (2~4) are the known characteristics.

3.3 Canadian clays

Depositional environments of the Canadian cemented clays were described by Lefebvre et al. (1983), Tanaka et al. (2001b), and Tanaka et al. (2003). Typical Canadian clays are of marine origin, cemented and were lifted to the present elevation due to the isostatic movement after the end of the Ice Age. Overconsolidation ratios over 2 are believed to be a result of cementation. Mineralogy of the Canadian quick clays can be summarized as high amorphous clays and abundant clay-size rock flour (non-clay minerals).

3.4 Japanese clays

Most Japanese marine clays are characterized as non-glacial, pyroclastic and low-swelling smectitic clays with clay fractions of about 50%. A well-developed flocculated structure combined with abundant fossil remains was mostly derived from diatoms (Ohtsubo et al. 2000, Tanaka et al. 2001b, Tanaka et al. 2003). Most of the Japanese marine clays have been developed since 8,000 years B.P., when the rapid sea-level rise commenced in the late Quaternary era (Hanzawa and Tanaka 1992).

3.5 Asian clays

The Bangkok clay is a non-glacial, high-swelling smectite, non-pyroclastic origin clay (Ohtsubo et al. 2000). Microfossils, such as diatom or foraminifera are rare (Tanaka et al. 2001a). Clay fractions of the Bangkok clay are typically over 50%. The Iraqi clays and the Korean marine clays in the database are with non-swelling minerals, non-glacial origin, and non-pyroclastic. Sedimentation time of Iraqi clays is about 5,000 years B.P. (Hanzawa and Tanaka 1992). Depositional and post-depositional environments of the Korean marine clays were described in details by Won and Chang (2007). The Shihwa clay is silt-dominant, whereas the Namak clay is clay-dominant.

4 DISCUSSION

Decades ago, Berre and Bjerrum (1973) and Ladd et al. (1977) reported that anisotropy of clays decreases as plasticity index (PI) increases. In other words, the anisotropic strength ratio ($K_s = S_{ue}/S_{uc}$) increases with PI. This trend was followed by many researchers (for example, Mayne, 1983, Jamiolkowski et al. 1985). Until Mayne (1983) compiled 66 anisotropic data points, the trend was supported only by 16 data points including 4 plane strain data and 12 triaxial data on undisturbed or resedimented samples, mainly from Scandinavian clays and a mixture of recompression and SHANSEP approaches. In the meantime, 53 resources compiled by Mayne (1983) included test results from different conditions and test methods, such as quick sand, remolded specimens, overconsolidated soils, and unconfined compression tests on different trimming angles.

In this study, anisotropic data following the data selection criteria are grouped into their regions and depositional environments (Figure 2). If all the data are plotted in one space, the trend can be biased by the dominant number of test sets, for example Japanese clays. Furthermore, one can treat different clays with the same PI as the similar clays, even though they have different mineralogy, clay structures, and clay fractions, i.e. different deposition environments. Anisotropic strength ratios of the Scandinavian clays in Figure 2(a) show a wide spread within a small range of PI. Anisotropy data for the Drammen clay typically shows strain hardening behavior and does not have peak extension strengths (Berre and Bjerrum 1973, Ladd et al. 1977, Hanzawa and Tanaka 1992), the extension strengths by definition-A resulted in much less K_s than ones by definition-B. In fact, Ladd et al. (1977) and Berre (1982) have mentioned that extension strengths determined by the definition-A can be somewhat too low; hence the K_s values for the Norwegian low PI clays reported by Berre (1982) and

Berre and Bjerrum (1973) must have been underestimated. Moreover, Tanaka and Tanaka (1997) have reported anisotropy data for the Drammen clay (filled circles in Figure 2(a)), which were quite different from the results by Berre and Bjerrum (1973). Tanaka and Tanaka (1997) reported K_s ranging 0.32-0.78 for PI=15-32, whereas the range of K_s was 0.265-0.4 by Berre and Bjerrum (1973) for the same lean and plastic Drammen clays. Tanaka and Tanaka (1997) must have followed the failure definition-B. If the anisotropy data for Scandinavian clays were based on the failure definition-B at the beginning, the anisotropy trend with PI would have been quite different.

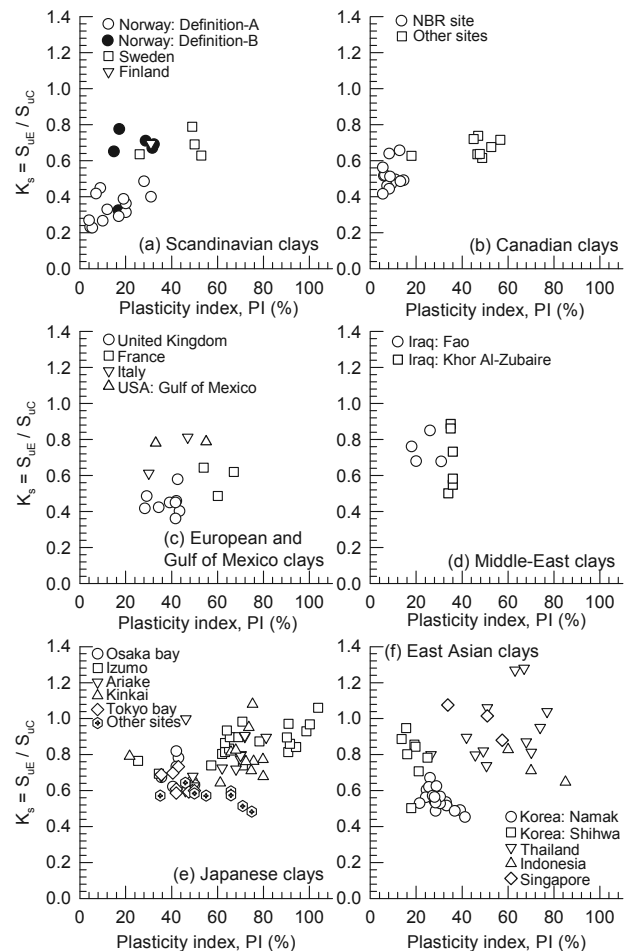


Figure 2. Anisotropic strength ratio versus plasticity index for different depositional environments.

For Canadian clays in Figure 2(b), the published data can be grouped into (1) low PI, sensitive and highly structured clays, and (2) structured clays with high PI. The majority of the low PI group are of the NBR site (Lefebvre et al. 1983), where an intensive test program has been performed on the marine clay with PI=5-15. Among the anisotropy data from the NBR site, only the data that satisfied the data selection criteria are presented. The failure definition-B (necking failure) was used for the NBR site. The K_s values in the NBR site varied between 0.41-0.66, depending significantly on the degrees of structure, within a narrow PI range. The majority of the high PI group data are from the Champlain Sea area. The difference between the low and the high PI groups seems to be originated from mineralogy of the clay size particles; the low PI clays consist of rock flour for clay-size particles, whereas the high PI clays consist of illite, chlorite, and vermiculite (Tanaka et al. 2001b). Distinctively different characteristics of the two groups make it difficult to draw a trend line for the Canadian clays.

Anisotropic data from Gulf of Mexico and the data of European clays are plotted together in Figure 2(c) because their depositional environments seem to be similar. Anisotropic

strength ratios from the Bothkennar site ranged between 0.36 and 0.58 with PI range of 28-43. Peak or necking failures were observed (definition-B) from the samples retrieved by Laval and Sherbrooke samplers. No apparent trends between K_s and PI can be observed in Figure 2(c).

The only anisotropy data from the Middle East are of Iraqi clays as shown in Figure 2(d). The Iraqi clays in the database are normally consolidated young or aged clays. The anisotropy substantially changes ($K_s=0.50-0.89$) within a very narrow PI range (PI=34-36) for the Khor Al-Zubaire clay.

Anisotropic data of Japanese clays are presented in Figure 2(e). It is clear both K_s and PI values significantly vary even within a specific site. For example, the Izumo clay has PI range of 25-100 and K_s values of 0.70-1.06. From Figure 2(e), one could suggest $K_s=0.5-0.8$ for PI=20-60 and $K_s=0.7$ or more for PI over 60, for Japanese clays. Spatial variation within a site as well as locality seems to have strong effects on anisotropy than a single index property, PI.

Anisotropy data from the East Asian countries are shown in Figure 2(f). All the anisotropy data followed the failure definition-B. Anisotropic strength ratios typically ranged between 0.5 and 1.1 for the wide PI range of 14-85. The Namak clay is similar to the Bothkennar clay in many ways including stress history, organic contents, laminated features, and estuary environments. The K_s range of the Namak clay is 0.45-0.67 with PI=22-41 that is comparable to the Bothkennar clay. Of special interest is anisotropy data from the Bangkok clay. Berre and Bjerrum (1973) presented only one data point that was not from the Scandinavian Peninsula and PI over 35: from Bangkok east. In fact, the data point ($K_s=0.52$ & PI=88) was not selected in this study because it appeared to be an organic clay. As for Bangkok clay, published data show a wide range of $K_s=0.74-1.28$ with PI=26-77. As Tanaka et al. (2001a) mentioned, Southeast Asian clays seem to behave more isotropically despite the scatter and their moderate PI values.

When Berre and Bjerrum (1973) and Ladd et al. (1977) reached the conclusion that anisotropy of clays decreases with plasticity index, very limited test results were available. Scandinavian low PI clays with failure definition-A that underestimated S_{ue} formed the left lower end and the two data points from Bangkok organic clay and Atchafalaya clay, USA (PI=75 and definition-B) formed the right end to conclude the trend. Once the anisotropy data with consistent criteria in this study are grouped into their depositional environments, the trend of K_s increase with PI can hardly be observed. The statement "less plastic, and often more sensitive, clays tend to have higher anisotropy than more plastic clays" by Jamiolkowski et al. (1985) appears to be appropriate, only if the less plastic and sensitive clays are Scandinavian and Canadian low PI clays. This study supports the conclusion by Hanzawa and Tanaka (1992) that undrained strength anisotropy is not correlated with plasticity index. Other aspects, such as clay fraction, clay structure, mineralogy, origin, diatoms, spatial variation are the major factors that control anisotropy in shear strength of natural clays. In other terms, depositional and post-depositional environments, and regional variations are the governing factors for anisotropy of natural clays, rather than a single index property like PI. It should be emphasized that index properties are good indicators of those major governing factors in a limited sense. For a given local soil, a carefully selected empirical correlation or a trend based on local data should be valid and useful. However, a comparison between various natural clays solely by a single index property such as plasticity index, without careful consideration of depositional and post-depositional environments can be misleading.

5 CONCLUSIONS

A large number of anisotropic triaxial test results (CK_0UC and CK_0UE) were collected and analyzed to re-evaluate the generally accepted trend between anisotropy and plasticity index. Data selection criteria were established for a consistent

comparison. K_0 -consolidated (recompression) triaxial test results on geologically normally consolidated, undisturbed natural clays were selected. Based on the analysis, the well-known trend that anisotropy decreases with plasticity index cannot be justified. The trend was developed by limited test results and different definitions of failure. Anisotropy was strongly influenced by the definitions of failure in CK_0UE tests. When comparing different natural clays, an anisotropy trend correlated exclusively with plasticity index can be misleading. Clay structure, clay fraction, mineralogy, origin, diatoms, and spatial variation are the governing factors for understanding anisotropy of natural clays. Relationship between anisotropic strength ratio and plasticity index should be evaluated by careful consideration of spatial variability, site characteristics, and depositional and post-depositional environments of an individual clay of interest.

6 REFERENCES

- Berre T. 1982. Triaxial testing at the Norwegian Geotechnical Institute. *Geotechnical Testing Journal* 5 (1/2), 3-17.
- Berre T. and Bjerrum L. 1973. Shear strength of normally consolidated clays. *Proc. 8th ICSMFE*, Moscow 1, 39-49.
- Bjerrum L. 1954. Geotechnical properties of Norwegian marine clays. *Geotechnique* 4 (2), 49-69.
- Hanzawa H. and Tanaka H. 1992. Normalized undrained strength of clay in the normally consolidated state and in the field. *Soils and Foundations* 32 (1), 132-148.
- Jamiolkowski M., Ladd C.C., Germaine J.T. and Lancellotta R. 1985. New developments in field and laboratory testing of soils. *Proc. 11th ICSMGE*, San Francisco 1, 57-153.
- Ladd C.C., Foott R., Ishihara K., Schlosser F. and Poulos H.G. 1977. Stress deformation and strength characteristics. *Proc. 9th ICSMFE*, Tokyo 2, 421-494.
- Lefebvre G., Ladd C.C., Mesri G. and Tavenas F. 1983. *Report of the Testing Subcommittee*. Committee of specialists on the sensitive clays on the NBR complex, SEBJ, Montreal, Canada, Annexe I.
- Mayne P.W. 1983. Discussion: Undrained shear strength anisotropy of normally consolidated cohesive soils. *Soils and Foundations* 23 (4), 143-146.
- Nash, D.F.T., Powell, J.J.M. and Lloyd, I.M. 1992. Initial investigations of the soft clay test site at Bothkennar. *Geotechnique* 42 (2), 163-181.
- Ohtsubo M., Egashira K., Koumoto T. and Bergado D.T. 2000. Mineralogy and chemistry, and their correlation with the geotechnical index properties of Bangkok clay: comparison with Ariake clay. *Soils and Foundations* 40 (1), 11-21.
- Tanaka H., Locat J., Shibuya S., Soon T.T. and Shiwakoti D.R. 2001a. Characterization of Singapore, Bangkok, and Ariake clays. *Canadian Geotechnical Journal* 38 (2), 378-400.
- Tanaka H., Shiwakoti D.R., Mishima O., Watabe Y. and Tanaka M. 2001b. Comparison of mechanical behavior of two overconsolidated clays: Yamashita and Louiseville clays. *Soils and Foundations* 41 (4), 73-87.
- Tanaka H., Shiwakoti D.R. and Tanaka M. 2003. Applicability of SHANSEP method to six different natural clays, using triaxial and direct shear tests. *Soils and Foundations* 43 (3), 43-55.
- Tanaka H. and Tanaka M. 1997. Application of the UC test to two European clays. *Proc. 14th ICSMFE*, Hamburg 1, 209-212.
- Won J.Y. and Chang P.W. 2007. The causes of apparent overconsolidation in the Namak marine deposit, Korea. *Geotechnique* 57 (4), 355-369.

Hydraulic Heave in Cohesive Soils

Rupture hydraulique du sol en terrain cohérent

Wudtke R.-B.

wudtke geotechnik, Weimar, Germany, r.wudtke@wu-geo.de

Witt K.J.

Bauhaus-Universität Weimar, Weimar, Germany, kj.witt@uni-weimar.de

ABSTRACT:

According to DIN EN 1997-1:2009, the shear parameters of the soil are not relevant for stability analysis due to hydraulic heave. Resistance is only activated by the weight of the soil of the failure area. However, during tests on cohesive soils the failure sequence was visually detected as a series of pore widening, initial crack formation and hydraulic induced structure decomposition of the soil in the failure area followed by downstream sudden uplift. On the basis of the test observations a new design approach under consideration of soil weight, shear parameters and an optionally mobilised earth pressure is proposed. The used reference volume represents both the construction as well as the flow situation. Processes that occur before the final uplift are transmitted by the hydraulic gradient i_{crit} on the failure area. The final limit state is interpreted as a shear failure.

RÉSUMÉ:

Pour l'évaluation d'une défaillance par rupture hydraulique du sol, les normes DIN EN 1997-1:2009 sur les propriétés de cisaillement du sous-sol ne sont pas pertinentes. Les résistances sont activées seulement par le poids du sous-sol. Des essais de visualisation du déroulement de la défaillance en sol cohérent ont montré que le processus se caractérise par une succession d'effets de dilatation des pores, de formation initiale de fissures, de destruction structurelle du continuum dans la zone de défaillance, suivis d'une rupture brutale de la surface. Conformément au déroulement de la défaillance, les calculs présentés prennent en compte, en tant que résistances, le poids mort du sol, sa résistance au cisaillement de rupture ainsi que, le cas échéant, l'existence d'une contraction du sol. La mise en évidence se fait sur une zone de défaillance-modèle reproduisant la situation de construction et de courant. Les phases précédant la rupture finale sont transférées à la zone défaillante par les gradients hydrauliques i_{crit} . L'état limite final est interprété comme rupture par cisaillement.

KEYWORDS: stability analysis, design approach, cohesive soil behaviour, test method

1 INTRODUCTION

Stability analysis against hydraulic heave considers in accordance with Eurocode 7 (DIN EN 1997-1:2009) exclusively the soil weight as stabilizing influence. Thus, especially for cohesive soils the design approach takes not the available shear strength into account. In accordance with current regulations, a consideration of shear properties is only possible if special experience with the material is available.

In order to optimize the design approach of hydraulic heave for cohesive soils within the constraints of the valid standard, it is necessary to examine various questions. This includes:

- How to characterize the failure mechanism of hydraulic heave in cohesive soils?
- Which is the characterizing difference of this for non-cohesive soils?
- How to describe or determine an appropriate reference volume for the limit state?
- Which are the controlling material parameters of the limit state?
- What is the effect of the water content on these soil properties?

In order to develop a new design approach, it is also necessary to consider temporal aspects of the failure sequence. An adaptation of the design approach to the problem of a flow around excavation wall must be considered for both drained and undrained soil properties. In addition, a specific consideration of

the supporting effect of the subsoil abutment at sheeting location is possibly positive.

2 CURRENT DESIGN

Changes in groundwater regime are critical for hydraulic heave and for particle transport processes in the soil. The stability of earth structures, excavations and foundations is influenced by both phenomena during and after construction.

From a global point of view, changes in ground water level cause changing pore water pressures, which might affect the general balance within the soil continuum. A violation of the limit state condition leads in this context to failure. On the other hand an increased flow of water through the voids leads to changes in the soil structure due to particle transport. To evaluate hydraulic heave a limit state condition is critical. Particle transport processes have only an indirect effect on the overall stability.

Referred to Eurocode 7 (DIN EN 1997-1:2009) for a stability analysis against hydraulic heave for each possible soil prism the limit state condition according to Equations (1) or (2) has to be evaluated.

$$u_{dst;d} \leq \sigma_{stb;d} \quad (1)$$

$$S_{dst;d} \leq G'_{stb;d} \quad (2)$$

The limiting criterion in Equation (1) is formulated as a comparison of the design values of total stresses $\sigma_{stb;d}$ and pore

water pressures $u_{dst,d}$. Equation (2) refers the context to the flow force $S_{dst,d}$ and the buoyancy weight $G'_{stb,d}$ and therefore to the effective stress.

The following additional considerations of how to make assumptions and define the boundary conditions for hydraulic heave stability analysis and their interpretations are given in DIN 1054:2010. For example, in flow around an excavation wall conform to Terzaghi's approach the reference volume can be taken into account as a rectangle with a width of half of the wall embedment depth. The hydraulic impact may be differentiated on the basis of the soil type. Accordingly, the partial safety factors can be distinguished for soils with flow unfavorable or favorable characteristics. For at least stiff cohesive soils the consideration of cohesion or tensile strength as a material-specific parameter is permitted, if special expertise and experience are available.

In particular, the reference to a possible consideration of additional material specific strength properties indicates that the valid approach lead to a conservative interpretation of the relevant boundary conditions. Thus, according to the current experience the approach is suitable especially for non-cohesive soils. The existing shear strength of cohesive soils is neglected.

3 EXPERIMENTAL ANALYSIS

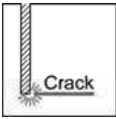
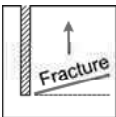
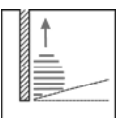
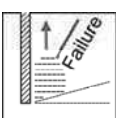
3.1 Visual failure state determination

The valid failure mechanism of cohesive soils is not so far comprehensively analysed. Therefore experiments were carried out to describe the structural failure during a hydraulic heave in cohesive soils on the basis of a visual identification (Wudtke and Witt 2006). The experimental setup and procedure rebuild the water flow around a pit wall. The dividing wall was fixed on frame sides.

The following questions were examined by the experiments:

- Which sequence of events can be characterized as the most important attribute of hydraulic heave in cohesive soils?
- What phenomena are announcing the limit state?
- How the hydraulic decomposition of a cohesive soil can be described?
- Where does the initial damage occur?

Table 1. Failure sequence during hydraulic heave in cohesive soils (Wudtke 2013)

Phase	Description
	- water content increase, - development of an initial crack with origin at the foot of the dividing wall
	- continuing pore widening, - heaving of the downstream surface, - development of a deep-lying fracture
	- continuous crack initiation and closure; - continuing heaving, - locally limited decomposition of the soil structure, dissection of the soil into aggregates
	- continuing rapidly increasing elevation of the soil surface, - sudden failure as hydraulic heave, - local potential equalization, failure body erosion

From visual observations it can be stated that the failure sequence is characterized as pore widening effects, soil structure destroying crack opening and closing followed by a final failure of the downstream surface.

Relative to a real excavation pit, the decomposition of the soil structure at the foot of the flow around pit wall leads to

changing hydraulic routing and flow distribution in the soil. Consequence of the changed hydraulic boundary conditions is increasing hydraulic impacts at the excavation side of the pit wall.

The type of failure sequence is independent of the shear strength of the soil. The failure principle is subjected to the availability of cohesion and tensile strength. The activation of resistances is related to the shear strength of the soil and the stress state.

For low-cohesive soils in the course of crack opening and closing processes a comparatively small resistance is activated. At the same time the influence of shearing increases. How much the resistance can be mobilized depends on the stress state of the reference volume and the hydraulic head difference. An increase of the water content causes plasticity changes and is therefore sensitive to the occurrence of the failure.

3.2 Critical hydraulic gradient

Conventional tensile strength tests show that the tensile strength of cohesive soils decreases with increasing of water content. Basically it can be noted that tensile strength is available only for soils with at least stiff plastic consistency. In these tests due to the mechanically induced sample destruction the conventionally determined tensile strength is only partially representative for the detected hydraulic induced loss of structural integrity of cohesive soils, (Wudtke and Witt 2010).

The development of a new testing method was required to identify adequately the hydraulic induced limit state condition. Objective of the testing method is to identify and quantify for a certain construction status valid critical hydraulic gradient i_{crit} . The parameter represents the hydraulic impact required for a structure decomposition of the soil. There are three main influences on the test results: 1 – representative stress state, 2 – pore water pressure, 3 – soil water content. In contrast to conventional testing methods the test method considers a hydraulic effect as the driving force for specimen destruction. (Wudtke 2013)

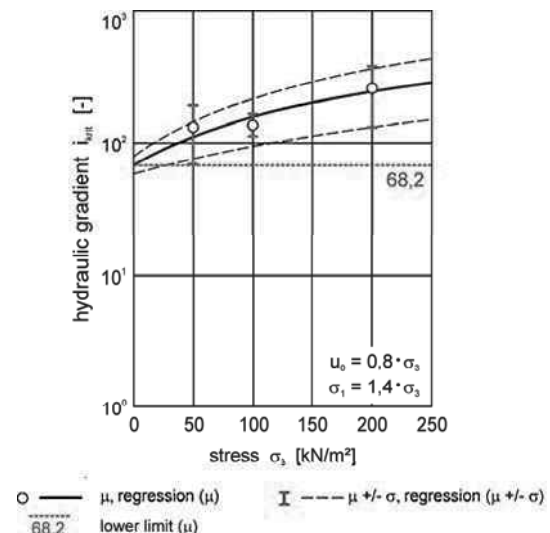


Figure 1. Stress-dependent critical hydraulic gradient, mean and variation.

Exemplary for an inorganic clay of medium plasticity ($\varphi' = 23.5^\circ$ und $c' = 13.7 \text{ kN/m}^2$) the evaluation of the stress-dependent development of i_{crit} is shown in Figure 1. As experimental conditions a principal stress ratio of $\sigma_1 = 1.4 \cdot \sigma_3$ at an initial pore water pressure of $u_0 = 0.8 \cdot \sigma_3$ were defined. From the test results, the lowest limit value of i_{crit} can be determined for this kind of soil, $i_{crit} \geq 68.2$ (Figure 1).

The experimental determination of the parameter i_{crit} is basis for defining the representative reference volume for calculating the

limit state of the hydraulic heave. Considering a construction state depending flow distribution and a valid limit value of i_{crit} there is a failure area, which can be distinguished as hydraulic induced structure decomposition and shear failure. Therefore the failure process consists of two phases, the structure decomposition followed by shear failure.

4 RECOMMENDATION OF A DESIGN APPROACH

4.1 Reference volume

The definition of a representative reference volume is essential for transferring the typical failure sequence of hydraulic heave in cohesive soils to the design approach. For a failure process given resistance activation the spatial arrangement of the resistances shall prevail.

The reference volume is based on the flow situation, which can be determined by the construction state and the soil stratification. The areas of hydraulically induced structure decomposition of the soil, as well as of shear failure are defined as part of the reference volume (Figure 2). The outer boundary of the reference volume is represented by the flow line, which is tangent to the curve of i_{crit} . For determination of the shear body, the relevant flow line is approximated as a parabolic function. A widening of the reference volume beyond the base of the wall, can only occur if i_{crit} is realized by the given flow situation.

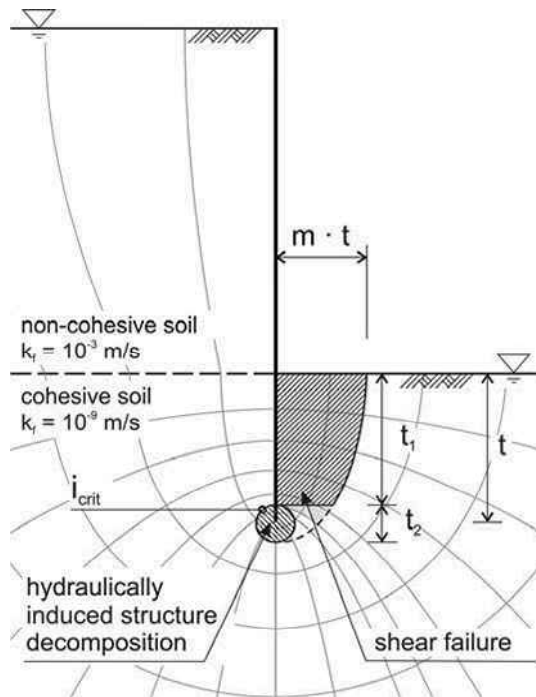


Figure 2. Areas of dominant shear failure and hydraulic induced structure loss at a flow around pit wall

4.2 Hydraulic induced soil structure decomposition

In the first step of safety analysis against hydraulic heave in cohesive soils, the maximum possible hydraulic gradient $\max i$ relevant for soil structure decomposition is estimated. $\max i$ is related to a construction stage and has to be compared with soil-specific i_{crit} .

The approach aims to identify a critical hydraulic loaded soil area. For the relevant part of the reference volume in the subsequent shear failure analysis no resistances are activated. Basic idea of the proposed design approach is the assumption of continuously decreasing shear strength of the soil in dependency of increasing hydraulic gradient. In the second step, the shear failure analysis, reduced shear resistances can be considered.

To analyze possible soil damage and soil structure decomposition, the following relationships have to be investigated.

Damage criterion

$$i_{crit} \geq \max i \geq i_{damage} = \frac{\gamma'}{\gamma_w} \quad (3)$$

Failure Criterion

$$\max i \geq i_{crit} \quad (4)$$

Soil damage starts when the effective stress disappears due to the hydraulic impact. This occurs first when $\sigma'_3 \leq 0$. Here the potentially shear failure body has to be identified. Depending on the size of $\max i$, i_{damage} and i_{crit} different results of the reference volume are possible.

4.3 Shear failure

As second part of the design approach the analysis of the safety against shear failure must be considered. The analysis is performed on the soil body remaining after the possibly hydraulic structure decomposition of the soil continuum. For the calculation a time-consistent failure state is valid, i.e. it is necessary to provide a strict separation of drained and undrained soil properties and flow conditions.

The stability of the reference volume to shear failure is determined by the following equation.

$$S \leq G' + C_v + F_v \quad (5)$$

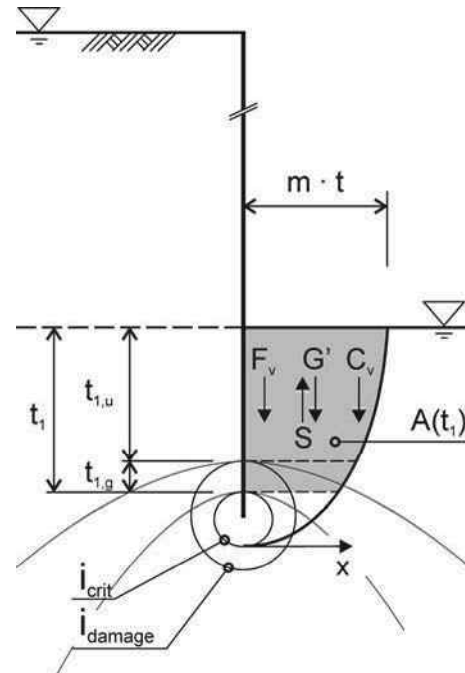


Figure 3. Parameter definition – shear failure

The limit state analysis is carried out as a balance of vertical forces of the flow impact and the acting resistances at the reference volume (Figure 3). Resistances are represented by the buoyant soil weight G' , the shear resistance acting at the shear surface C_v and the bracing acting at the pit wall F_v .

The resistance of soil weight is determined by the submerged unit weight of the soil γ' and the area of the shear body $A(t_1)$. To determine the soil weight a consideration of soil damage is generally not required. As a conservative assumption a consideration of hydraulic resistances from damaged soil areas can be dispensed. For the calculation a shear body height of $t_{1,u}$ is decisive. For example in a construction state with no filter

load, the equivalent resistance can be determined according to Equation (6). In stratified soil conditions, a layer-related determination of the area proportions of the shear body is relevant.

$$G' = \gamma' \cdot A(t_1) \quad (6)$$

In the excavation area near to the sheet pile after a definite construction stage, there will be bulging of the surface (uplift effect), this leads to activation of shear resistances in dependency of soil shear parameters. Damaged and undamaged soil areas have to be distinguished considering failure velocity depending resistance activation. Damaged soil areas, which have a comparatively lower shear resistance, are considered by available hydraulic gradient $i \geq \gamma' / \gamma_w$ (see Equation (3)).

The permeability of soil and the excavation velocity respectively load relieving governs the resistance activation of the soil during hydraulic heave. Due to reduction of soil load in excavation area, the pore water pressure is released slowly so that the undrained shear parameters are relevant. In contrast, drained conditions are relevant for long phases without excavation progress and relatively permeable soils. In the stability analysis for drained conditions the effective cohesion c' and for undrained conditions the undrained cohesion c_u must be considered. The friction of soil should be neglected.

For calculating the shear resistance, shear body height t_1 is divided into two different heights depending on the soil structure (Equation (7) and Figure 3). For the height of damaged area, the relevant shear property and for undamaged area its shear resistance must be assigned.

$$t_1 = t_{1,u} + t_{1,d} \quad (7)$$

$$C_v = C_{v,u} + C_{v,d} = c_u \cdot t_{1,u} + c_{u,d} \cdot t_{1,d} \quad (8)$$

$C_{v,u}$ and $C_{v,d}$ represent the activated shear resistances in undamaged and damaged areas. Equation (8) shows in this case a soil with undrained conditions.

Additionally, the soil bracing, supporting system of the pit wall and other ground supports can be considered as a further resistance. The consideration of abutment as an additional resistance effect can be described as a vertical force which is transmitted by the sheeting system. In accordance with DIN 1054:2010 this represents the difference between the stress state without soil deformation and the actual or planned stress at the excavation abutment.

The consideration of abutment resistance requires the stability of sheeting system. It must be ensured that the construction has sufficient safety to permit a deformation free transferring of vertical forces at sheeting foot.

Transferring of vertical forces by the sheet pile is depending on frictional component of shear strength. The drained or undrained shear properties must be assigned in relation to the anticipated load conditions. A definite amount of earth pressure can be transferred to sheeting foot by consideration of contact friction between soil and sheeting. The vertical load component of the active earth pressure is essential for the determination of the resistance and generally should be estimated conservatively. If the hydraulic impact does not lead to hydraulic soil damage the abutment effect F_v corresponds to the vertical load of the active earth pressure.

$$F_v = E_{a,v} \quad (9)$$

The approach for determining the resistance of the abutment effect of sheeting represents a general solution to the problem, and requires the full activation of the passive earth pressure in the pit. The most liable solution can be achieved by determining the difference stress distribution between an unstressed, not deformed soil and the stress condition of the planned construction at the excavation abutment.

According to equation (10), the flow force S is determined depending on area of the shear body $A(t_1)$, the available mean hydraulic gradient i_M at the bottom side of $A(t_1)$ and the weight of water γ_w .

$$S = i_M \cdot \gamma_w \cdot A(t_1) \quad (10)$$

5 CONCLUSIONS

The presented design approach for the stability analysis against hydraulic heave in cohesive soils explicitly considers the available resistance due to cohesion. In addition, the consideration of a ground support effect acting at the sheeting is integrated.

The design approach is divided in two steps. A first step considers the hydraulic induced structure decomposition of the cohesive soil beneath the foot of the sheet pile. The condition is essential to determine the reference volume of the failure area. The critical hydraulic gradient i_{crit} depends on the cohesion, density and stress history of the soil. i_{crit} can be determined experimentally or might be estimated by experience.

In the second step the equilibrium of the vertical forces acting at the reference volume is analyzed. In addition to the current design approach (DIN EN 1997-1:2009), side forces and cohesion of the appropriate shape of the failure area are taken into account. The second part of the design approach corresponds to the verification of uplift failure.

During hydraulic induced structure decomposition of the soil the flow conditions will change. Thus an update of the distribution of pore water pressure should be taken into account.

The assumption of resistances from the weight and the available shear resistance in accordance with current design practice (DIN EN 1997-1:2009) leads to a possible classification as design situations HYD and GEO. The assignment of coherent partial factors is still an open question under discussion.

6 REFERENCES

- DIN 1054:2010. Baugrund – Sicherheitsnachweise im Erd und Grundbau – Ergänzende Regelungen zu DIN EN 1997-1.
- DIN EN 1997-1:2009. Eurocode 7: Geotechnical design, Part 1: General rules; German version EN 1997-1.
- Wudtke, R.-B. (2013). Hydraulischer Grundbruch in bindigem Baugrund. Dissertation, Bauhaus-Universität Weimar. Weimar.
- Wudtke, R.-B. and Witt, K. J. (2006). A Static Analysis of Hydraulic Heave in Cohesive Soil. 3rd International Conference on Scour and Erosion, Amsterdam, 1 - 3 November 2006, 251.
- Wudtke, R.-B. and Witt, K. J. (2010). Hydraulischer Grundbruch im bindigen Baugrund – Schadensmechanismen und Nachweisstrategie. 9. Geotechnik-Tag in München – Wechselwirkungen Boden - Wasser - Bauwerk, München, 19.02.2010, 33 - 44.

Evaluation of sample disturbance due to the exsolution of dissolved gas in the pore water of deep lake bottom sediments

Évaluation du remaniement des échantillons dû à l'exsolution de gaz dissous dans les eaux interstitielles des sédiments de fond de lacs profonds

Yamashita S., Miura R.

Kitami Institute of Technology, Kitami, Japan

Kataoka S.

Hakodate National College of Technology, Hakodate, Japan

ABSTRACT: Core samplings were conducted in the Lake Baikal, Russia, where shallow gas hydrates existed. To examine the mechanical properties of lake-bottom sediments, the handy vane shear and cone penetration tests were performed for the core samples on board. To evaluate the sample disturbance due to the exsolution of dissolved gas during sampling, the relations between the concentration of dissolved gas in the pore water and the strength were also examined. Moreover, laboratory tests which simulate the stress relief from bringing the samples to the lake surface were also performed. Test results showed that the strength of soils becomes lower with the increase in degree of gas concentration on both on-board and laboratory tests. It was also showed that the in-situ strength can be estimated roughly from the disturbed samples.

RÉSUMÉ : Un prélèvement d'échantillons stratigraphiquement représentatifs (carottes) a été mené dans le lac Baïkal, en Russie, où il existe des réservoirs d'hydrates de gaz peu profonds. Afin d'examiner les propriétés mécaniques des sédiments de fond de lac, on a procédé à des essais de cisaillement avec un scissomètre portatif et à un test de pénétration au cône sur ces échantillons stratigraphiquement représentatifs, ceci sur place. Dans le but d'évaluer le degré de remaniement des échantillons dû à l'exsolution de gaz dissous au cours du prélèvement, on a également examiné le lien entre la concentration de gaz dissous dans les eaux interstitielles et la résistance de l'échantillon. On a, en outre, procédé à des tests en laboratoire qui simulent le relâchement de contraintes résultant de l'apport des échantillons à la surface du lac. Les tests menés sur place tout comme ceux menés en laboratoire, indiquent que la résistance des sols diminue plus le degré de concentration en gaz augmente. Ils ont également démontré que l'on pouvait obtenir une estimation approximative de la résistance in situ du sol à partir de l'échantillon remanié.

KEYWORDS: dissolved gas, gas hydrate, sample disturbance, lake bottom sediment

1 INTRODUCTION

Gas hydrates (GH) are attracting attention as a next-generation energy source. In Japan, survey and test drilling of GH for resource development have been conducted in around the Nankai Trough. On the other hand, GH are also attracting attention in relation to the global environment, because methane gas contained in the GH has approximately 20 times the greenhouse effect of carbon dioxide. There are concerns that dissociation of GH and exsolution of dissolved gas, from the GH distributed in submarine surface layers, due to rising ocean temperatures or leakage at recovery of the hydrates for energy, may contribute to global warming. Moreover, these phenomena or seismic activities can reduce the stability of seabed and may induce seafloor landslides. Therefore, it is necessary to clarify the mechanical properties of the GH-bearing ground and the strength change of seabed sediments due to the dissociation or exsolution.

In generally, to assess the in-situ strength, laboratory tests on samples retrieved from the site or in-situ tests have been performed. However, it is difficult to perform the in-situ test in the deep lake or seabed. On the other hand, samples for laboratory tests taken from lake or seabed soils in deep waters are subjected to a large stress (back water pressure) relief. Even a small amount of gas dissolved in the pore water will come out of solution and cause disturbance to the soil structure due to the stress relief. As a result, laboratory tests may not give appropriate results for the in-situ soil conditions.

In this study, to evaluate the effects of the exsolution of dissolved gas in the pore water on the strength properties, core samplings were conducted in the Lake Baikal, Russia, shallow GH province, and some kinds of on-board tests were performed.

Laboratory tests which simulate the stress relief from bringing the samples to the lake surface were also performed.

2. SURVER AND SOIL SAMPLING

The sampling of lake-bottom sediments contained GH were conducted in the Lake Baikal, Russia, September 2010, by the survey ship 'Vereshchagin' of the Russian Limnological Institute. It is already reported that GH in the Lake Baikal is formed at immediately beneath lake-bottom of the mud volcano that is observed the eruption of cool spring water contained gases by the echo sounder (e.g., Matveeva et al. 2003). Thus, at locations where mud volcanoes were confirmed by the echo sounder images, lake-bottom sediments were collected by using a gravity core sampler (sampler length is about 5 m, diameter is 110 mm and weight is about 700 kg). Sampling sits are Novosibirsk site (water depth is about 1450 m) and Kukuy site (water depth is about 800 m) at the central parts of the lake, as shown in Figure 1. The lake-bottom sediments cores were retrieved 40 sample cores (24 cores in Novosibirsk site, 16 cores in Kukuy site).

After core samplings, these cores were cut into 1-m interval. Then, each section was cut up longitudinally on two parts for subsequent processing. To measure the strength of sediments immediately after recovery, the handy vane shear test (blade diameter is 10 mm, height is 20 mm; 10 to 40 cm interval) and cone penetration test (diameter is 9 mm, length is 16.8 mm, apex angle is 30 degrees; 10 to 40 cm interval) were performed on board. For measurement of the water content, 10 to 40 cm interval samples were taken. Additionally, the dissolved gases in the pore water were taken by the headspace gas method.

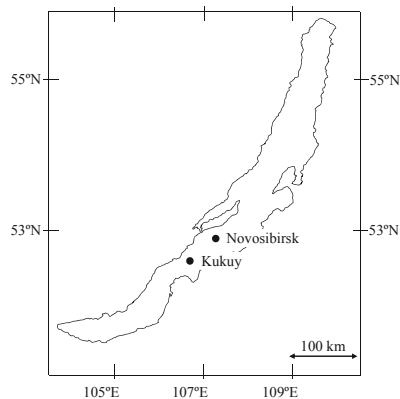


Figure 1. Survey and sampling sites in the Lake Baikal.

3 ON-BOARD TEST RESULTS

Figure 2 shows the on-board test results (vane shear strength, τ_v , and cone penetration resistance, q_c), together with the profiles of water content, w , with depth. In the samples retrieved from Novosibirsk site, the water contents in the upper strata of samples that were able to collect GH in lower part of the core (with GH) are lower than those of the samples that were not able to collect GH (without GH), and the strengths of samples with GH are higher than those of samples without GH, as shown in Figure 2(a). On the other hand, in the samples retrieved from Kukuy site, although the water contents of samples with GH are lower than those of samples without GH as same as Novosibirsk samples, the strengths are similar, as shown in Figure 2(b). Thus, the relations between the strength and the water content are different from the sampling site. The reason seems to be that the difference of degree of sample disturbance due to the exsolution of dissolved gas in the pore water caused by the decrease of pressure during the pulling up of core, because the concentrations of methane gas dissolving in pore water may be different from site. Then, the concentrations of methane gas dissolving in the pore water of sediments were measured, and the effects of the concentrations of gas on the strength of samples were examined.

Figure 3 shows the relations between the depth and the concentrations of dissolved methane gas, CH_4 , per sediment of one liter (Hachikubo et al. 2010). In the case of Novosibirsk samples, the gas concentrations are high from surface layer, irrespective of with/without GH. On the other hand, in the case of Kukuy samples, although the gas concentrations of samples with GH are high from surface layer, those of samples without GH are low at surface layer and increase suddenly from a certain depth.

Figure 4 shows the relations between the vane shear strength, τ_v , and the water content, w . It is found that the difference of strengths in Novosibirsk samples is not recognized irrespective of with/without GH, as shown in Figure 4(a). It would seem that the degree of sample disturbance due to the exsolution of dissolved gas in the pore water are similar, because the gas concentrations are high from surface layer irrespective of cores, as shown in Figure 3(a). On the other hand, in Kukuy samples, the strengths of samples with GH are lower than those without GH. It would seem that this is because the gas concentrations of samples with GH are high from surface layer as shown in Figure 3(b), so that the sample disturbance due to the exsolution of dissolved gas became large, and the strength became low. Thus, the correlation between the strength and the water content is recognized, considering the sample disturbance due to the difference of the gas concentration. The similar test results were obtained on the sea-bottom sediments retrieved from offshore Sakhalin Island, Sea of Okhotsk (Yamashita et al. 2011).

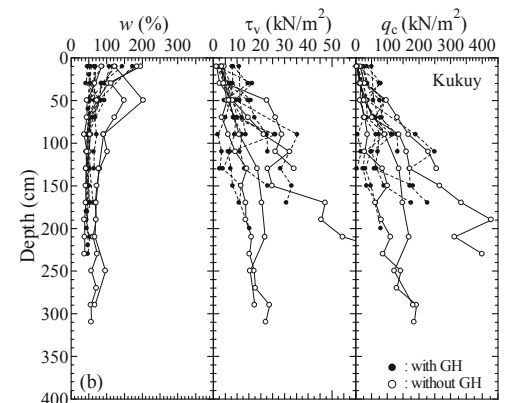
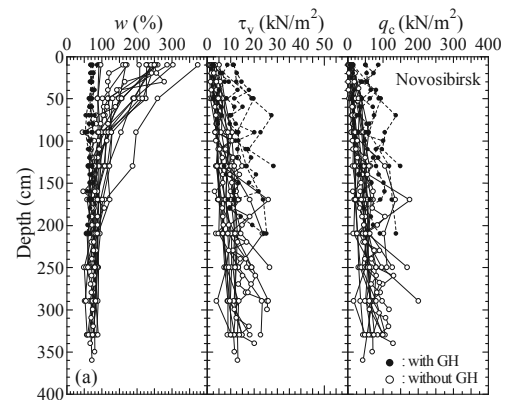


Figure 2. On-board test results; (a) Novosibirsk site, (b) Kukuy site.

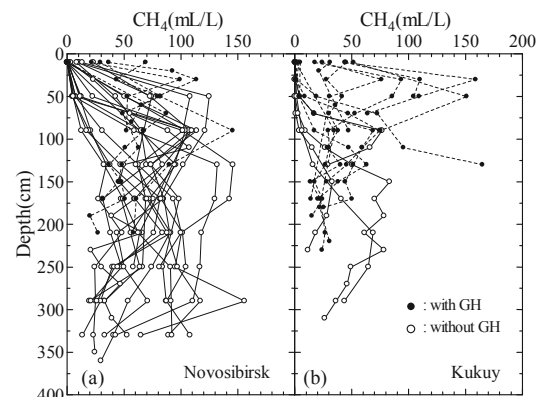


Figure 3. Concentration of methane gas with depth; (a) Novosibirsk site, (b) Kukuy site.

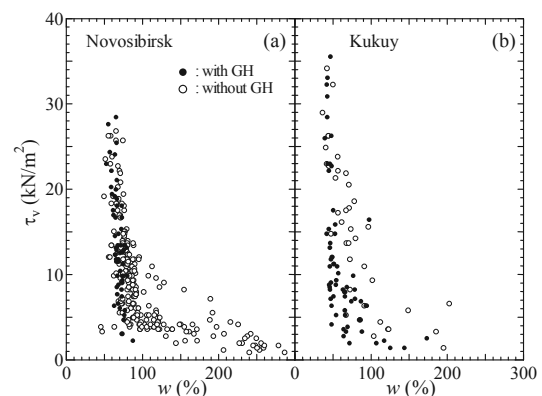


Figure 4. Relations of vane shear strength and water content; (a) Novosibirsk site, (b) Kukuy site.

4 LABORATORY TEST

From the on-board test results and the measurement of gas concentrations, it was guessed that the strength of samples retrieved from GH-bearing ground is decreased by the sample disturbance due to the exsolution of dissolved gas during the sampling. In order to clarify this, the strength change by the sample disturbance due to the exsolution of dissolved gas in the pore water was evaluated by the laboratory tests which simulate the stress relief from bringing the samples to the lake surface.

4.1 Test apparatus and test method

Figure 5 illustrates the oedometer apparatus using the simulating laboratory test. Used sample is a mixed Baikal lake-bottom sediment ($\rho_s = 2.720 \text{ g/cm}^3$, $w_L = 70.1\%$, $I_p = 41.9$, clay content is 58 %, silt content is 40 %, sand content is 2 %) retrieved from the Kukuy site at 2005 and 2006 (Kataoka et al. 2009). The mixed sample is slurry state having an initial water content of 1.6 times the liquid limit. Used gas is carbon dioxide (CO_2) instead of methane (CH_4), because CO_2 gas has high solubility in comparison with CH_4 gas. For example, the solubility of CH_4 gas under water temperature of four degrees centigrade and water depth of 1000 m is almost same to that of CO_2 gas under water temperature of 20 degrees and back water pressure of 500 kPa.

The laboratory tests were conducted on three test conditions as shown in Table 1 and Figure 6. The consolidation time is 24 hours for each consolidation stage of 20, 50, and 100 kPa. The back pressure was applied after end of consolidation of 20 kPa. In the case of Test Case 1, the back pressure of 500 kPa was applied by air pressure. On the other hand, in the cases of Test Case 2 and 3, the back pressure was applied by CO_2 gas pressure. The back pressures of Case 2 and 3 were 100, 300 and 500 kPa, respectively. In the Case 1, deaired water was permeated through the sample after end of consolidation of 100 kPa. In the Case 2 and 3, CO_2 gas dissolved water was permeated. The permeated time is 10 days. The volume of permeated water is similar to the volume of sample.

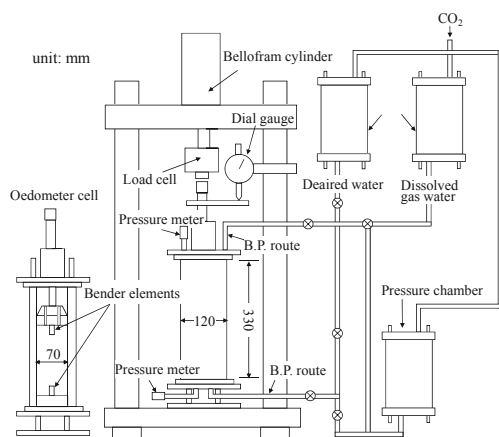


Figure 5. Schematic diagram of laboratory test apparatus.

Table 1. Test conditions.

Test Case	Consolidation stress (C.S.) (kPa)	Back pressure (B.P.) (kPa)	C.S. at B.P. reduction (kPa)
1	100	500	20
2	100	100, 300, 500	100
3	100	100, 300, 500	20

Thereafter, in the Case 2, the back pressure was decreased to atmospheric pressure under a consolidation stress of 100 kPa. On the other hand, in the Case 1 and 3, it was decreased after the consolidation stress was decreased to 20 kPa. Therefore, it would seem that the effects of the sample disturbance in Case 3

are larger than those in Case 2, because the vertical stress of Case 3 at stress release is lower than that of Case 2.

Unconfined compression tests (sample diameter is 50 mm, height is 100 mm, loading rate is 1 mm/min) were performed on the specimens prepared by above procedure.

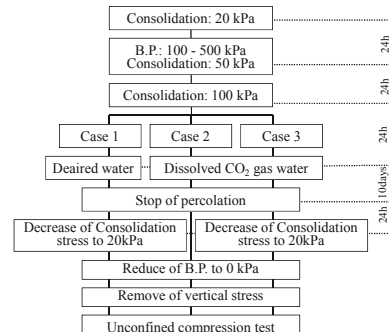


Figure 6. Test process of laboratory test.

4.2 Unconfined compression test results

Figure 7 shows the stress strain relations of unconfined compression tests on all specimens. Figure 8 shows the relations between the unconfined compression strength and the back pressure at consolidation. It is found that the strengths in Case 2 and 3 permeated CO_2 gas dissolved water are lower than those in Case 1 permeated deaired water. It is also found that the strengths in Case 2 and 3 decrease with the increase of back pressure.

Figure 9 shows the relations between the deformation modulus, E_{50} , and the back pressure. Although E_{50} in Case 2 and 3 on back pressure of 100 kPa has some scatter because the degree of CO_2 gas dissolution is low, E_{50} decreases with the increase of back pressure due to the exsolution of dissolved gas in the pore water in the case of the back pressure of 300 and 500 kPa. However, the strengths in Case 3 had not become lower than those in Case 2.

Figure 10 shows the typical time histories of vertical stress, back pressure and axial displacement during the stress release. It is found that the change of axial displacement is not recognized during the decrease of vertical stress and back pressure in all test cases. On the other hand, the axial displacement increases after the release of vertical stress and back pressure in the Case 3. Although the data recording is stopped halfway in the Case 2, the increase of axial displacement was recognized after the release of vertical stress and back pressure. Therefore, it would seem that the difference of strength between the Case 2 and the Case 3 is not recognized, because the sample disturbance was produced after the stress release. It is said that the occurrence of the sample disturbance by the swelling or cracking was delayed by the effect of the cohesion of sample having much clay content. In actuality, when the sampling core was retrieved from the deep lake bottom, the swelling or cracking of core surface is observed after the time of some extent passed. Thus, it is found that the strength of sample becomes low due to the effect of the exsolution of dissolved gas on both on-board and laboratory tests.

Next, the relations between the reduction of strength and the water depth (pressure) are compared. Figure 11 shows the relations between the strength ratio of Case 1 to Case 2, 3 and the water depth converted the solubility of CO_2 gas into that of CH_4 gas. In this figure, unconfined compression test results using the intact samples retrieved from the Lake Baikal (Kataoka et al. 2009) and the triaxial compression test results using intact Liestranda and Bothkennar clays (Lunne et al. 2001) were also plotted.

Test results of Kataoka et al. (2009) are for samples retrieved from the different water depth areas in the Lake Baikal, and the strength ratio is average value of the mud volcano samples

(high gas concentration) and the reference samples (low gas concentration) on same area. On the other hand, test results of Lunne et al. (2001) are for samples permeated CH₄ gas dissolved water under the different pressure (0 to 15 MPa), as similar to this study method.

It is found from this figure that the strength ratios decrease with the increase of water depth. In other words, it is found that the effects of the sample disturbance become large with the increase of gas concentration (water depth). Although the results of Kataoka et al. have some scatter, the reduction of strength with depth is larger than that in this study. This reason seems that the effects of the sample disturbance in intact samples are larger than those in reconstituted samples. On the other hand, the reduction of strength in the results of Lunne et al. is lower than in this study. It would seem that the effects of the sample disturbance were decreased by the reconsolidation during the triaxial test. From these test results, it would seem that the in-situ strength can be estimated roughly from the disturbed samples retrieved from deep lake bottom by the measuring the concentration of dissolved gas.

5 CONCLUSIONS

The strength of samples retrieved from GH-bearing ground is decreased by the sample disturbance due to the exsolution of dissolved gas during the sampling. The reduction of strength becomes large with the increase of dissolved gas concentration.

From the laboratory tests which simulate the stress relief from bringing the samples to the lake surface, it was recognized that the effects of the sample disturbance become large with the increase of gas concentration. It was also showed that the in-situ strength can be estimated roughly from the disturbed samples.

6 ACKNOWLEDGEMENTS

The authors would like to thank Dr. Oleg Khlystov of the Russian Limnological Institute, Prof. Hitoshi Shoji, Dr. Hirotsugu Minami and Dr. Akihiro Hachikubo of Kitami Institute of Technology for their cooperation in the survey.

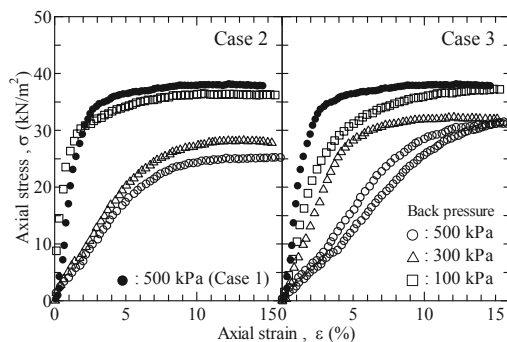


Figure 7. Unconfined compression test results.

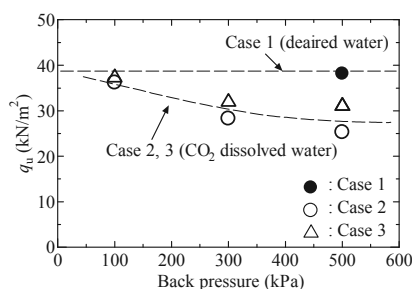


Figure 8. Relations of unconfined compression strength and back pressure.

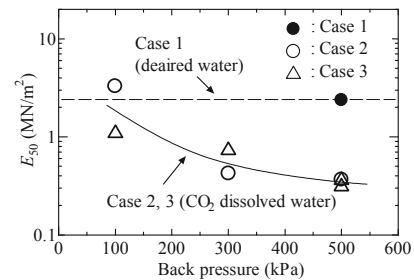


Figure 9. Relations of deformation modulus and back pressure.

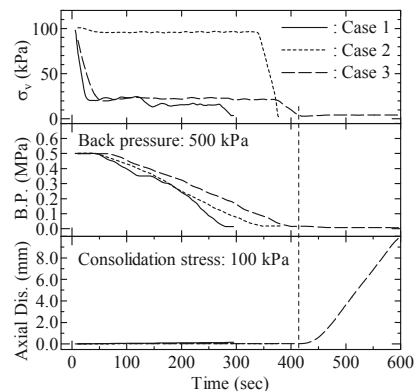


Figure 10. Time histories of vertical stress, back pressure and axial displacement.

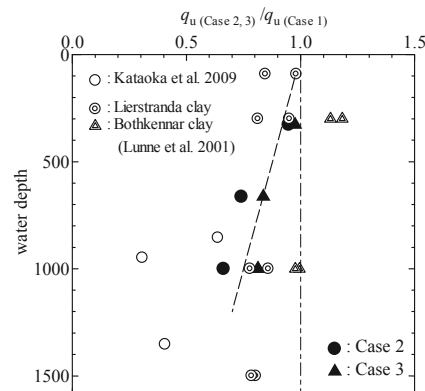


Figure 11. Relations of strength ratio and water depth.

7 REFERENCES

- Matveeva, T.V., Mazurenko, L.L., Soloviev, V.A., Klerkx, J., Kaulio, V.V. and Prasolov, E.M. 2003. Gas hydrate accumulation in the subsurface sediments of Lake Baikal (Eastern Siberia), *Geo-Marine Letters*, 23(3-4), 289-299.
- Kataoka, S., Yamashita, S., Kawaguchi, T. and Suzuki, T. 2009. The soil properties of lake-bottom sediments in the Lake Baikal gas hydrate province, *Soils and Foundations*, 49(5), 757-775.
- Hachikubo, A., Khlystov, O., Sakagami, H., Minami, H., Yamashita, S., Takahashi, N., Shoji, H., Kalmychikov, G. and Poort, J. 2010. Isotopic fractionation of hydrate-bound hydrocarbons in the subbottom sediments of Lake Baikal, *2010 AGU Fall Meeting*, San Francisco, USA.
- Lunne, T., Berre, T., Strandvik, S., Andersen, K.H. and Tjelta, T.I. 2001. Deepwater sample disturbance due to stress relief, *Proc. Geotechnical, Geological, and Geophysical Properties of Deepwater Sediments*, Texas, USA, NGI Rapport 521671-10.
- Yamashita, S., Moriwaki, T., Hachikubo, A., Minami, H., Shoji, H., Kawaguchi, T. and Kataoka, S. 2011. Strength change of seabed soils due to the vaporization of dissolved gas in the pore water, *Proc. of the Inter. Sym. on Deformation Characteristics of Geomaterials*, Seoul, Korea, 905-910.

Fabric and critical state of granular materials

La structure et l'état critique des matériaux granulaires

Yan W.M., Zhang L.
The University of Hong Kong

ABSTRACT: Critical state of particulate materials traditionally refers to a state where the material undergoes continued distortion at constant volume and constant stresses. By saying so the internal state of the material at the critical state is described solely by an isotropic (scalar-valued) parameter – the void ratio. Advances in modern laboratory tests have initiated the discussion of the effect of fabric on critical state and thus its uniqueness. More recently Li and Dafalias (2012) have shed light on the uniqueness of critical state from a thermodynamics perspective. This study uses the discrete element approach to investigate the fabric evolution of idealized two-dimensional assemblages having different initial fabrics subject to numerical biaxial shearing. The current paper focuses on the orientation of particles and void spaces at very large strains. It is shown that a unique fabric of particle orientation and void space is achieved at very large strains where the granular assemblage distorts continuously at constant density and stresses.

RÉSUMÉ : L'état critique des matériaux particulaires réfère normalement à un état où le matériau est soumis à un cisaillement continu sous volume et contraintes constants. Cela signifie que l'état interne des matériaux à l'état critique est décrit seulement par un paramètre (scalaire) isotrope – l'indice des vides. Avec les avancées en essais en laboratoire modernes a commencé la discussion de l'effet de la structure sur l'état critique et son unicité. Li & Dafalias (2012) ont étudié récemment l'unicité de l'état critique par la thermodynamique. Cette étude utilise une approche aux éléments discrets pour examiner l'évolution de la structure d'assemblées idéalisées en deux dimensions ayant des structures initiales différentes et soumises à un cisaillement bi-axial numérique. L'objet de cet article est l'orientation des particules et espaces des vides aux grandes déformations. On montre qu'aux grandes déformations une structure unique faite de l'orientation des particules et des vides est obtenue quand l'assemblée granulaire est cisailée de façon continue sous densité et contraintes constantes.

KEYWORDS: anisotropy; critical state; discrete element analysis, fabric; microstructure.

1 INTRODUCTION

The concept of critical state is important to soil mechanics. It defines the existence of a unique state where the particulate material exhibits constant volume shearing at constant effective stresses under continuous distortion. The state is often obtained phenomenologically. Over decades many constitutive models are formulated in accordance with this concept since the pioneering work by Roscoe and Schofield (1963) and Roscoe and Burland (1968). On the one hand, those models successfully capture the key mechanical behavior of many geomaterials subject to compression and shear. On the other hand, with the advances in modern laboratory testing techniques the influence of initial fabric on the material's stress-strain-strength responses have received much attention and the uniqueness of critical state has been great challenged (Vaid et al. 1985, Negussey and Islam 1994, Mooney et al. 1998, Finno and Rechenmacher 2003, etc). Herein fabric is a collective term to describe the geometric arrangement of grains and the associated voids, and the distribution of inter-particle contact forces. Material anisotropy has often believed to be the prime reason for the observation of a non-unique critical state. The critical state is often represented by two projection lines (critical state line, CSL) in the q - p' and e - p' (or v - p') plane, in which $q = \sqrt{3}\mathbf{s} : \mathbf{s} / 2$, $p' = \text{tr}(\boldsymbol{\sigma}') / 3$ where $\mathbf{s}$ is the deviatoric part of the effective stress tensor $\boldsymbol{\sigma}'$; e is the void ratio and $v = 1 + e$ is the specific volume. Internal state of the soil at the critical state is solely described by a scalar quantity of density (e or v) which implicitly shows that any anisotropic information of the material cannot be properly

addressed. However, one may doubt why the material state remains (or becomes) isotropic at the critical state where the imposed stress is anisotropic ($q / p' = M$ where M is the critical stress ratio). Besides, while density and stress are uniquely related, should there be a unique particulate fabric at the critical state?

More recently, Li and Dafalias (2012) revisited the critical state concept and proposed an anisotropic critical state theory (ACST) by considering the role of the particulate fabric. From a thermodynamics perspective and based on the Gibbs stability requirement, uniqueness of the critical state line (CSL) has been proved. Furthermore, they also concluded that a unique fabric as a function of the loading direction must exist.

This paper investigates the evolution of the fabric of a two-dimensional idealized granular assemblage subject to numerical biaxial shearing. It aims to shed light on ACST proposed by Li and Dafalias (2012) from a numerical perspective using the discrete element approach.

2 NUMERICAL SPECIMEN AND TEST

Two-dimensional mono-sized non-crushable pill shape rigid particles with length-to-width ratio of 1.5 (width = 1 mm) are generated with the built-in clump function in PFC2D (Itasca 2008). The linear contact model between particles is adopted. The particles are then rained into a model container under gravity fields of different directions (see Figure 1). By doing so, assemblages formed by particles having different average

preferential orientations could be expected. Generally speaking, by adopting this approach particles will have their long axis roughly perpendicular to the gravity field direction (Yan and Zhang 2013). Another approach is used to create assemblies of particles having random orientations. In this case, particles with random orientations are generated inside the model container and numerical iterations are allowed to achieve an equilibrium of the assemblage.

A square assemblage having 80 mm in each side is then extracted from the model container as shown schematically in Figure 1. An isotropic confining pressure is applied to the boundary walls of the specimen right after the body force field is turned off. Isotropic pressure of three different magnitudes is modeled: 50, 100 and 200 kPa. Attempts are made by controlling the inter-particle friction coefficient and selecting appropriate location of the square such that specimens confined at the same pressure show essentially identical void ratios but different initial fabrics prior to shear. The difference in void ratio in all cases is less than 0.0006.

A specimen is labeled by its initial stress, details of specimen generation method and void ratio prior to shearing. For instance, I100D90_219 denotes a specimen confined at 100 kPa, formed by depositing particles from a direction perpendicular to (i.e., 90°) to the major principal stress direction and had an initial void ratio of 0.219 before shearing. By the same token, I50Ran_219 represents a specimen confined at 50 kPa, having an initial void ratio of 0.219 with particles generated with random orientations.

To simulate a constant lateral stress biaxial compression, the top and bottom walls are moved simultaneously and slowly inwards while the horizontal position of the left and right lateral walls is continuously adjusted according to a servo-controlled mechanism to keep the lateral confining pressure at the initial value. It is to ensure that a quasi-static equilibrium is maintained throughout the analysis. A compression strain rate of $2.5 \times 10^{-5}\%$ per computational step is used and all the specimens are sheared to 80% axial strain.

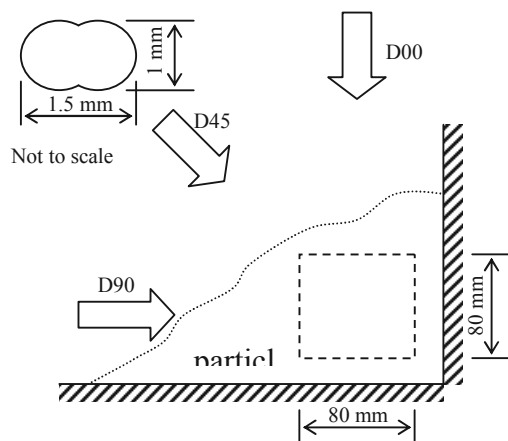


Figure 1. Specimen generation.

3 PARTICULATE FABRIC

Following the pioneer work presented by Oda et al. (1985), three major sources of fabric are considered in the study. They are (i) particle orientation, (ii) contact normal; and (iii) void distribution which are denoted by superscripts p , fn , and v respectively. Directional information of each fabric parameter is represented statistically by a rose diagram (Yan and Lin 2013). Two scalar quantities, a and θ_a are used to describe the distribution density $f(\mathbf{n})$ of the fabric (Rothenburg 1981, see Equation (1)).

$$f(\mathbf{n}) \approx [1 + a \cos 2(\theta - \theta_a)] / 2\pi \quad (1)$$

where $\mathbf{n}$ is a unit vector, a is called the coefficient of anisotropy that characterizes the degree of anisotropy and $0^\circ \leq \theta_a < 180^\circ$ describes the preferred direction of the anisotropy given that $\oint f(\mathbf{n}) d\mathbf{n} = 1$. Clearly $a = 0$ indicates an isotropic distribution and the degree of anisotropy increases with a . In this study, $\theta_a = 0^\circ$ denotes a direction perpendicular to the major principal stress. Details of the fabric description could be found in Yan and Zhang (2013).

4 RESULTS

4.1 Influence of initial fabric

This paper focuses on the evolution of fabric, in particular the particle orientation and void space, during biaxial shearing. The stress-strain-strength behavior of the assemblages is not shown due to the page limitation. It is found that the stresses and volumetric strain become essentially steady beyond 60% axial strain. Figure 2 and 3 show the evolution of the fabric of specimens having identical initial density but different initial fabric. As shown in Figure 1, specimens having different initial particle orientations have been created by different methods of generation. Specimens generated by the particle deposition method generally show an average particle orientation highly correlated to the deposition direction. For instance specimen I100D90_219 shows $\theta_a^p \approx 70^\circ$ before shear, which indicates that the particles tend to align with its long axis perpendicular to the deposited direction (D90). Specimens generated by the deposition method exhibit a higher initial degree of anisotropy as compared to the one with particles randomly generated. Upon shearing, particles gradually rearrange in a way that their long axis becomes perpendicular to the principal stress direction (i.e., the shearing direction). Furthermore, a unique fabric of very similar pair of a^p and θ_a^p is shown at large strains ($a^p \approx 0.55$ and $\theta_a^p \approx 0^\circ$).

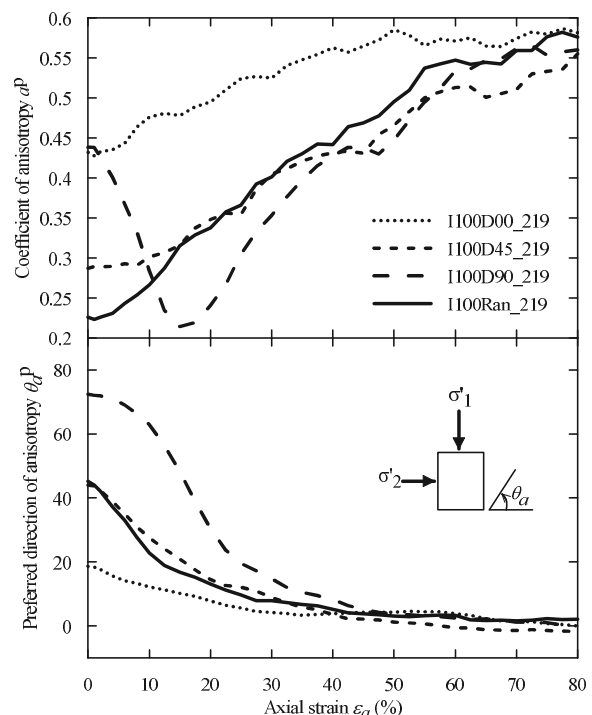


Figure 2. Evolution of particle orientation fabric – the influence of initial fabric.

A void space is defined following Li and Li (2009). Figure 3 shows its evolution with shear. It can be seen from the figure that the void space evolves from a highly isotropic distribution prior to shear commencement to an anisotropic one at large strains with a preferential orientation in the direction of the major principal stress ($\theta_a^v \approx 90^\circ$). Substantial amount of anisotropy is developed within 10% axial strain. Initial fabric seems to have very little influence on the evolution of void fabric. Like particle orientation, a unique void fabric is exhibited at large strain ($a^v \approx 0.3$ and $\theta_a^v \approx 90^\circ$).

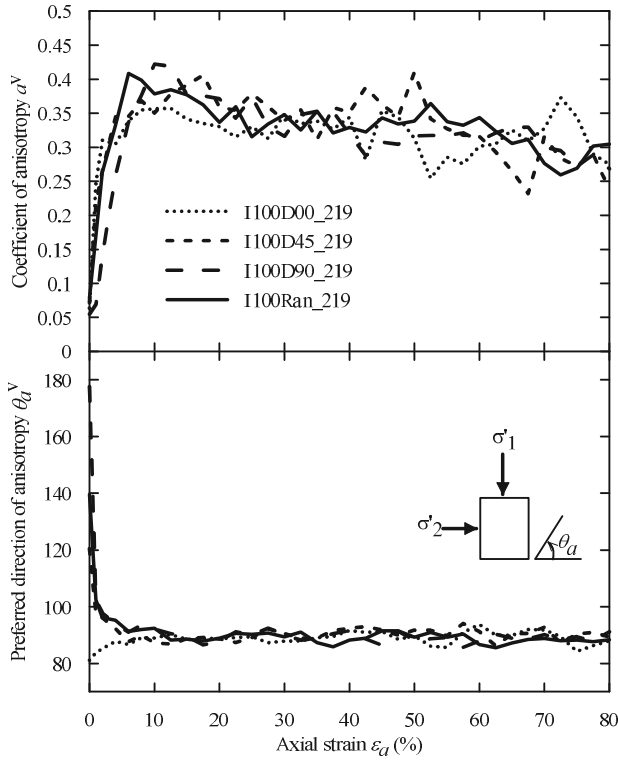


Figure 3. Evolution of void space fabric – the influence of initial fabric.

4.2 Influence of initial density

Figure 4 shows the effect of initial density on the evolution of particle orientation fabric. Deposited assemblages (D45) confined at 100 kPa having three different initial densities (initial void ratio from loose to dense = 0.237, 0.219 and 0.212, respectively) are investigated. Firstly, the particle orientation fabric changes gradually with increasing axial strain. The coefficient of anisotropy a^p increases gradually with axial strain while θ^p changes from its initial value (45°) to 0° . It means that the particles align with their long axis perpendicular to the loading direction when subject to prolonged shearing. A noticeably anisotropic and unique fabric can be observed at large strains. The effect of initial density is minimal. The evolution of void space fabric exhibits very similar behavior as the one shown in Figure 3, regardless the initial density of the assemblage.

4.3 Critical state

Figure 5 shows the initial and critical state of the assemblages in an $e-s'$ space where $s' = (\sigma'_1 + \sigma'_2)/2$. Initial state of the specimen includes various void ratios and fabrics confined at different pressures. It can be seen that a unique line can be used to describe the critical state. Together with the findings as

revealed from Figure 2-4, it is known that a critical state can be achieved at very large strains where not only the density and mean stress reaches a steady value but also a unique fabric is obtained. Furthermore, the fabric at critical state is correlated to the loading direction. The results echo the theory proposed by Li and Dafalias (2012).

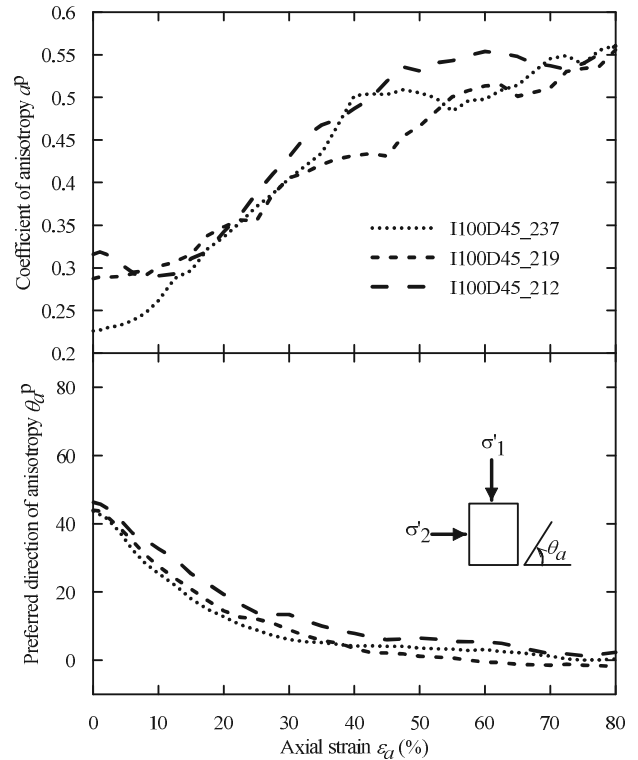


Figure 4. Evolution of particle orientation fabric – the influence of initial density.

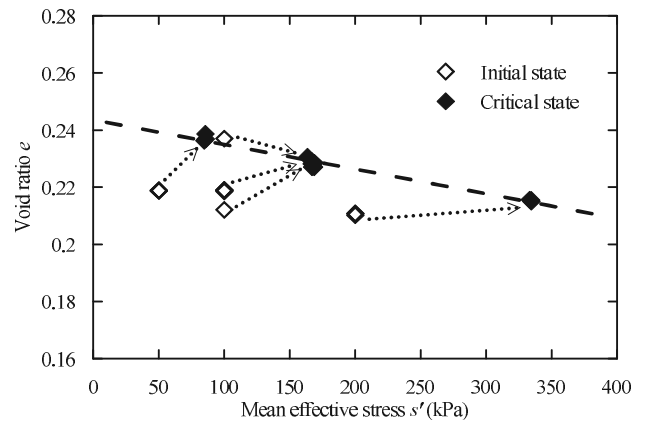


Figure 5. Initial and critical states of the study assemblages.

4.4 Limitations

The findings of unique fabric at critical state are obtained by conducting numerical tests limited to monotonic biaxial shearing of specimens composed of idealized mono-sized pill shape particles. Tests with continued changing of loading direction are not considered herein.

5 CONCLUSIONS

A series of numerical biaxial compression tests is undertaken on idealized two-dimensional granular specimen having various

initial particulate fabrics in terms of particle and void space spatial arrangement to investigate the uniqueness of the critical state. It is found that a unique fabric is obtained at large strains where the stresses and volume of the assemblages are essentially constant. Fabric at critical state is correlated to the loading direction. This study is only limited to the case whereas the loading direction remains the same.

6 ACKNOWLEDGEMENTS

The authors would like to acknowledge the research fund provided by the University of Hong Kong (Project Code: 200908159002).

7 REFERENCES

- Finno, R. J. and Rechenmacher, A. L. (2003). Effects of consolidation history on critical state of sand. *J. Geotech. Geoenviron. Engng.* 129(4), 350-360.
- Itasca Consulting Group, Inc. (2008). *PFC2D user's manual, version 4.0*. Itasca Consulting Group.
- Li, X. and Li, X. S. (2009). Micro-macro quantification of the internal structure of granular materials. *J. Engng Mech.* 135(7), 641-656.
- Li, X. S. and Dafalias, Y. F. (2012). Anisotropic critical state theory: role of fabric. *J. Eng. Mech.* 138(3), 263-275.
- Mooney, M. A., Finno, R. J. and Viggiani, M. G. (1998). A unique critical state for sand? *J. Geotech. Geoenviron. Engng.* 124(11), 1100-1108.
- Negussey, D. and Islam, M. S. (1994). Uniqueness of steady state and liquefaction potential. *Can. Geotech. J.* 31(1), 132-139.
- Oda, M., Nemat-Nasser, S. and Konishi, J. (1985). Stress-induced anisotropy in granular masses. *Soils Found.* 25(3), 85-97.
- Roscoe, K. H. and Burland, J. B. (1968). On the generalized stress-strain behaviour of wet clays. In *Engineering plasticity* (eds. J. Heyman and F. A. Leckie), pp. 535-609. Cambridge: Cambridge University Press.
- Roscoe, K. H. and Schofield, A. N. (1963). Mechanical behaviour of an idealized "wet" clay. *Proc. 2nd Eur. Conf. Soil Mech. Foud. Engng., Wiesbaden* 1, 47-54.
- Rothenburg, L. (1981). *Micromechanics of idealized granular system*. PhD thesis, Carleton University, Ottawa, Canada.
- Vaid, Y. P., Chung, E. K. F. and Kuerbis, R. H. (1990). Stress path and steady state. *Can. Geotech. J.* 27(1), 1-7.
- Yan, W. M. and Zhang, L. (2013). Fabric and the critical state of idealised granular assemblages subject to biaxial shear. *Comp. Geotech.* 49, 43-52.

Study on New Method of Accelerated Clay Creep Characteristics Test

Étude d'une nouvelle méthode d'évaluation accélérée des caractéristiques de fluage des argiles

Ye Y., Zhang Q., Cai D., Chen F., Yao J., Wang L.

Railway Engineering Research Institute, China Academy of Rails Science, Beijing, 100081

State Key Lab for Track Technology of High-speed Railway, China Academy of Rails Science, Beijing, 100081

ABSTRACT: Long-term creep tests, dynamic triaxial tests and corresponding scanning electron microscope (SEM) test were carried out for remolded saturated clay to study the way of rapid acquisition of the creep characteristics. The creep deformation and permanent deformation under the cyclic loading of soil with same initial state were analyzed. Based on the principle of static stress and dynamic stress equivalent, and the creep time and the cyclic number equivalent, a method using the relationship between stress-strain-cyclic number to predict the creep deformation was established. The SEM test results showed that the clay microstructure changing trend of creep test and dynamic triaxial test were similar. The multilevel stress and long-term creep deformation can be predicted by the proposed method.

RÉSUMÉ : Des essais de fluage à long terme, des essais triaxiaux dynamiques ainsi que des observations correspondantes faites au microscope électronique à balayage (MEB) ont été réalisés sur une argile saturée remaniée pour étudier la possibilité d'acquérir rapidement les caractéristiques de fluage du matériau. On a analysé comparativement les déformations de fluage et les déformations permanentes obtenues sous chargement cyclique à partir du même état initial. En se basant sur le principe d'équivalence entre contrainte statique et contrainte dynamique, ainsi que sur l'équivalence entre le temps de fluage et le nombre de cycles, une méthode de prévision des déformations de fluage a été établie, basée sur la relation entre contrainte, déformation et nombre de cycles. Les images obtenues au MEB ont montré que les évolutions de microstructures obtenues dans les essais de fluage et dans les essais triaxiaux dynamiques étaient similaires. Les déformations de fluage à long terme peuvent donc être évaluées à partir de la méthode proposée.

KEYWORDS: Remolded Saturated Clay; Creep; Acceleration; Equivalent efficiency; Dynamic triaxial test

MOTS-CLÉS : argile saturée remaniée, fluage, accélération, efficacité équivalente, essai triaxial dynamique.

1 INTRODUCTION.

Because of the great deep and long draining path, there are the characteristics of chronicity and complexity in composite foundation substratum deformation. So the deformation of substratum is the main resource of foundation deformation. The creep has been the key problem of the infrastructure with high standard including high-speed railway (Liu Junfei Zhao Jian Zhao Guotang et al. 2011, Cai Degou Ye Yangsheng Yan Hongye et al. 2010). For the purpose of ensuring foundation stability and meeting the requirement of engineering design, it is very vital to research the method which can obtain creep characteristic of foundation soil quickly.

According to related reference, there are two reasons for creep. One is that deviator stress leads to viscous shear flow; the other is that spherical stress leads to viscous body flow (Li Xiao 2011). Clay grains are connected by hydrated film, so it has inherent cohesion and rheological behavior. Only the hydrated film is extruded under low stress level, so the deformation is elastic. When the stress level is raised, stress concentrations will happen among grains. Soil grains will contact directly dislocation and rearrange. So the permanent deformation generates (He Kaisheng and Shen Zhujiang 2003). The deformation of soil under load is the synthetic result of microstructure change, such as structure bond failure, dislocation and porosity change (Zhang Xianwei Wang Changming and Li Junxia 2010).

The evident characteristics of soil stress-strain are nonlinear, hysteretic and cumulative under dynamic load. When dynamic stress level is low, elastic deformation is dominating. With the increasing of dynamic stress level, the permanent deformation increases gradually. Under different cyclic number and dynamic stress level, the soil grain adjusts and rearranges to some degree. In nature, there is correlation between microstructure change and macrostructure deformation (Liu Sha 2008). It is concluded that the deformation in macrostructure is a pattern of appearance of microstructure change.

2 CREEP AND DYNAMIC TRIAXIAL TEST AND RESULTS ANALYSIS

2.1 Parameters of Soil

Remolded saturated clay is used to experiment. Its parameters can be seen in Table 1 and Figure 1.

Table 1. Parameters of Soil

Specific gravity	Liquid limit (%)	Plastic limit (%)	Plasticity index
2.70	45.5	23.0	22.5

The samples used to creep and cyclic triaxial test were prepared as follows: Clay with moisture content over liquid limit was consolidated under certain load until 50kPa. That can ensure soil with the same stress history and the same consistency. The samples ($\Phi 39.1$ (mm) $\times$ H80 (mm)) used to creep test and $\Phi 50$ (mm) $\times$ H100 (mm) used to dynamic triaxial test) were consolidated isotropically in chamber under 50kPa. The chamber pressure during test is 50kPa. Creep soil samples were sheared by respective loading in triaxial creep apparatus under drained and undrained condition. Dynamic triaxial test samples were sheared under undrained and 5Hz condition.

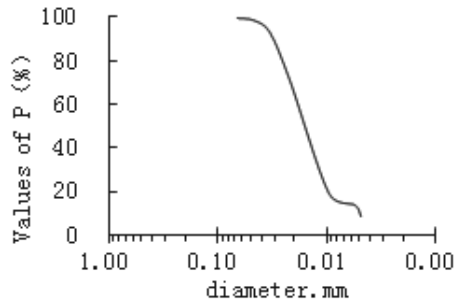


Figure 1. Semilogarithmic plot of results of grain analysis

Note: P means percentage by weight of grains smaller than the denoted size.

Creep test and dynamic triaxial test samples parameters can be seen in Table 3 and Table 4. The drained and undrained strength were 52.0kPa and 38.9kPa respectively under 50kPa chamber pressure.

Table 2. creep test samples index

No	Drainage lines	Deviator stress (kPa)	Density (g/cm ³)	Water content (%)
1	open	14.9	1.81	39.2
2	open	29.5	1.79	39.6
3	open	36.8	1.81	40.1
4	open	41.0	1.81	39.7
5	closed	14.9	1.85	39.8
6	closed	31.0	1.82	42.3
7	closed	40.5	1.84	42.3
8	closed	42.5	1.83	41.9

Table 3.. Dynamic triaxial test samples index

No.	Drainage lines	Deviator stress σ_d (kPa)	Density (g/cm ³)	Water content (%)
1	closed	20.0	1.80	42.1
2	closed	30.0	1.81	42.1
3	closed	35.0	1.80	43.0
4	closed	40.0	1.82	39.7
5	closed	45.0	1.81	40.7

2.2 Test results analysis

In creep test axial strain is generally denoted as the function stress level and time, for example . Exponential or hyperbolic function is used to express the influence of stress level. Power, logarithm or hyperbolic function is used to express the influence of time(Singh A. and Mitchell J.K. 1968, Mesri G. and Rebres-Cordero E. 1981, Wang BinWang ChangMingand Zhang Xianwei et al,2008, Zhang Wang 2009). It can be seen that logarithm function is reasonable for the correlation between strain and time (see Figure 3 and Figure 4). In Figure 3 and Figure 4 the origin is the deformation of 1h. Test data is expressed in the form of the dots of all kinds of shapes. The logarithm fitting curves are the black solid line.

The samples used to creep and cyclic triaxial test were prepared as follows: Clay with moisture content over liquid limit was consolidated under certain load until 50kPa. That can ensure soil with the same stress history and the same consistency. The samples ($\Phi 39.1$ (mm) $\times$ H80 (mm)) used to creep test and $\Phi 50$ (mm) $\times$ H100 (mm) used to dynamic triaxial test) were consolidated isotropically in chamber under 50kPa. The chamber pressure during test is 50kPa. Creep soil samples

were sheared by respective loading in triaxial creep apparatus under drained and undrained condition. Dynamic triaxial test samples were sheared under undrained and 5Hz condition.

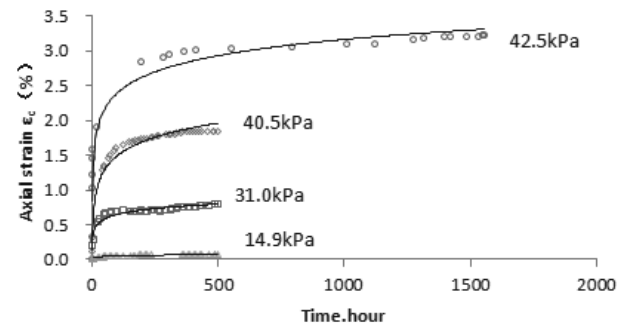


Figure 2. Undrained Creep Curve

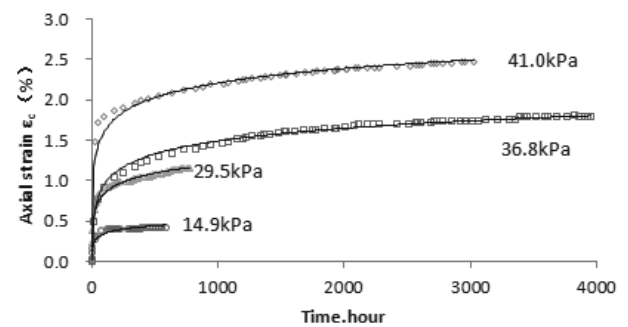


Figure 3. Drained Creep Curve

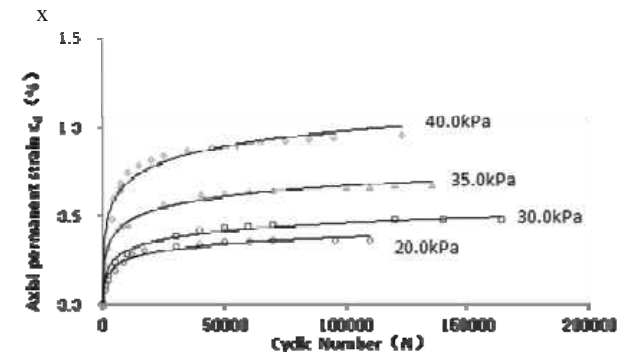


Figure 4. Dynamic Triaxial Test Curve

In this dynamic triaxial test logarithm function is used to express the correlation between axial strain and time. $f(D)$ is used to express the correlation between axial strain and stress level, (see Eq. 1).

$$\varepsilon_d = f(D) \ln N \quad (1)$$

ε_d —axial strain in dynamic triaxial test;
 $D = \sigma_d / \sigma_3$, σ_d —deviator stress, σ_3 —chamber stress.

A fitted coefficient under every stress level can be obtained by fitting the test data. $f(D)$ can be solved by analyzing the correlation between the coefficient and stress level.

$$\varepsilon_d = 0.022e^{2.04D} \ln N \quad (2)$$

Because of the similarity between creep deformation and permanent deformation in dynamic triaxial test, it is raised that the stress-strain-cyclic number formula in dynamic triaxial test can be used to predict the creep deformation. In order to depict the influence on the difference between static and dynamic load and drainage condition, firstly $\psi(D_r)$ is introduced.

$$\varepsilon_c = \psi(D_r) 0.0226 e^{2.04D} \ln N \quad (3)$$

Secondly, assume that static and dynamic stress level is equivalent. Static stress parameters are as follows:

$D_r = q/q_f$, $q = \sigma_1 - \sigma_3$, σ_1 —axial load, q_f —failure strength.

Thirdly, assume that time and cyclic number is equivalent.

$N = t$, N —cyclic number, t —creep time (h).

Finally, dynamic parameters are instead of static parameters. $\psi(D_r)$ is introduced. Then the creep strain formula can be obtained:

$$\varepsilon_c = \psi(D_r) 0.0226 e^{2.04D} \ln t \quad (4)$$

$D = q/\sigma_3$

$\psi(D_r)$ is a function of stress level. By analyzing the test data, the formula under different condition can be fitted as follows:

$$\text{Drained condition: } \psi(D_r) = 1.52 e^{0.67D_r} \quad (5)$$

$$\text{Undrained condition: } \psi(D_r) = 0.07 e^{3.61D_r} \quad (6)$$

The strain under different stress level and drainage condition can be calculated by Eq. 4, Eq. 5 and Eq. 6. The calculation and test results can be seen in Figure 6 and Figure 7.

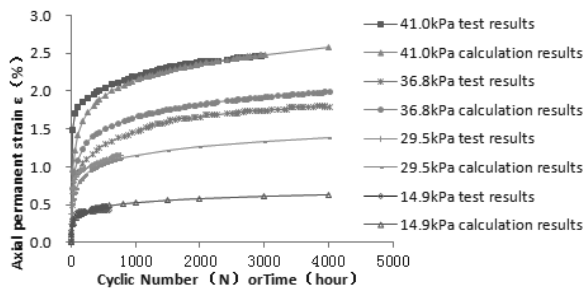


Figure 5. Creep Test and Calculation Results under Drained Condition

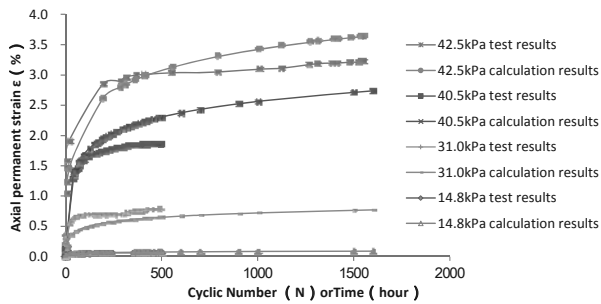


Figure 6. Creep Test and Calculation Results under Undrained Condition

The creep time under drained condition is longer than undrained. The developing trend and magnitude of strain which is calculated are in accordance with test results and the long-term creep deformation is predicted. The calculation results of 31.0 kPa and 40.5 kPa deviate from test results under undrained condition. The reason may be that the function $\psi(D_r)$ is an approximate expression which comes from limited test data. There might be other factor or correlation. It is the next work to solve.

3 MICROSTRUCTURE ANALYSIS

In order to explicit the microstructure change after the creep and dynamic triaxial test, the SEM test was carried out in low vacuum mode. The SEM test instrument is Quanta400 scanning electron microscope. The soil sample was cut to a cube of 1.5cm×1.5cm×1.5cm by wire saw. Both vertical and horizontal plane was test. The porosity parameters of soil including plane

porosity, anisotropy and directional probability entropy were counted to analyse the microstructure change. The image of soil was magnified 2000 times, and the microstructure photographs before and after loading were shown as Figure 8 and Figure 9.

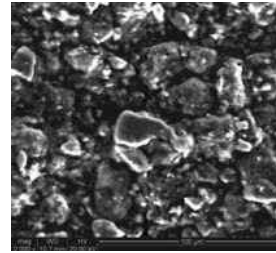


Figure 7. Unloaded soil SEM photograph

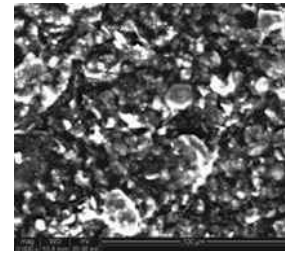


Figure 8. Soil SEM photograph after creep test

Figure 10, Figure 11 and Table 5 shows the microstructure parameter of different test. Both the vertical and horizontal plane porosity decreased after creep and dynamic triaxial test. The increasing anisotropy of porosity means that the porosity's shape trends to oval. The porosity directional probability entropy increases after creep and dynamic triaxial test. When the soil samples failure, the porosity directional probability entropy of creep soil sample is similar with dynamic triaxial test.

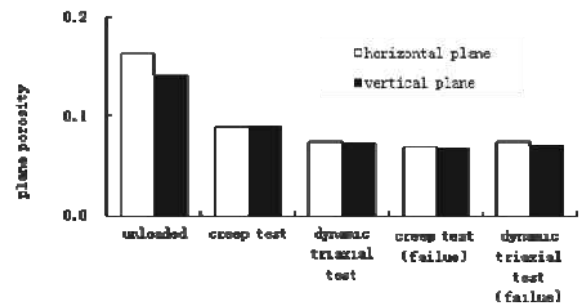


Fig 9. soil sample plane porosity of different test

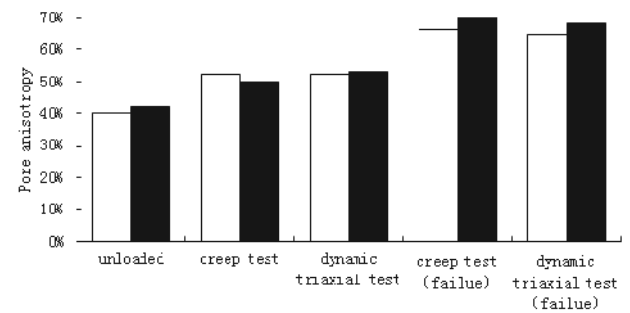


Fig 10. Porosity anisotropy of different test

Table 4. directional probability entropy of Porosity

unloaded	Creep test	dynamic triaxial test	Creep test (failure)	dynamic triaxial test (failure)
0.982	0.91	0.86	0.81	0.823

4 CONCLUSIONS

1. The creep deformation and permanent deformation of dynamic triaxial test are nonlinear. The deformation in macrostructure is a pattern of appearance of microstructure change.

2. The results of creep and dynamic triaxial test show that it is reasonable to use logarithmic function to describe the axial strain of creep and dynamic triaxial sample.

3. Based on the principle of static stress and dynamic stress equivalent, the stress-strain-cyclic number formula in dynamic triaxial test can be used to predict the creep deformation.

4. The SEM test results show that the change trend of plane porosity, anisotropy and directional probability entropy of both creep and dynamic triaxial test are similar. The microstructure status with equivalent condition is close.

5. The creep deformation under multilevel stress can be obtained with few samples by use of the method. The long-term creep deformation can be obtained within shorter time.

5 REFERENCES

- [1] Singh A., Mitchell J.K. General stress-strain-time function for clay[J]. Journal of the clay mechanics and foundation division, ASCE, 1968; 94(SM1): 21-46
- [2] Mesri G., Rebres-Cordero E., Shields D. R. and Castro A. Shear stress-strain-time behaviour of clays[J]. Geotechnique, 1981, 31(4): 537-552
- [3] LI Xiao. Analysis on Accelerated Creep Characteristics of Littoral Soft Soil [D]. TianJin University, 2011.
- [4] HE Kai-sheng, SHEN Zhu-jiang. Study on micro-deformation and mechanism of structural clay [J]. Journal of Hohai University(Natural Sciences), 2003 , 31(2): 162 - 165.
- [5] ZHANG Xian-wei, WANG Chang-ming, LI Jun-xia et al. Variation characteristics of soft clay micropore in creep condition [J]. Rock and Soil Mechanics, 2010, 31(4): 1061-1067.
- [6] Liu Sha. Study on the rheological properties of saturated muddy soft clay around the tunnel under subway loading [D]. Shanghai, Tongji University, 2008.
- [7] Dingqing Li, Ernest T. Selig. Cumulative Plastic Deformation For Fine-Grained Subgrade Soils[J]. Journal of Geotechnical engineering, 1996,12.
- [8] Experimental Study on Dynamic Characteristic of Subgrade Soil [R]. Beijing, China Academy of Railway Science. 1986,12.
- [9] Kong Lingrong. Microstructure Characteristic and elastic plastic Micro-constitutive model of Saturated Soft Clay [D]. Shanghai, Tongji University, 2007.
- [10] Wang Bin, Wang Changming, Zhang Xianwei et al. Experimental research on creep characteristics of Zhang zhou soft soil [J]. Journal of Engineering Geology, 2008, 16(Suppl.):645-649.
- [11] Zhang Wang. Experimental Study on Creep Characteristic of Remolded Saturated Soft Clay [D]. Nan Jing, Nanjing Hydraulic Research Institute, 2009.
- [12] LIU Junfei, ZHAO Jian, ZHAO Guotan, et al. Additional Stress Analysis of Upper Soil between Piles in Rigid Pile- Net Composite Foundation[J]. CHINA RAILWAY SCIENCE, 2011.3,32(2):15-19.
- [13] CAI Degou, YE Yangsheng, YAN Hongye, et al. HE Huawu Numerical Analysis on the Mechanical Properties of Geosynthetic Reinforced and Pile Supported Embankment[J]. CHINA RAILWAY SCIENCE, 2010.5,31(3):1-7.

Constitutive model and simulation of non-segregation freezing and thawing in soils

Modèle de comportement et simulation du gel et le dégel des sols sans ségrégation

Zhang Y., Michalowski R.L.

Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, USA

ABSTRACT: An elasto-plastic constitutive model for non-segregation freezing and thawing soils is developed to capture the deformation behavior and strength evolution of the soil subjected to arbitrary loading as well as temperature changes. The model is based on the critical state framework, with the yield condition as in the modified Cam clay model. It uses the pore ice ratio as a scalar parameter to describe the evolution of the yield condition due to freezing and thawing. The model has been implemented in the finite element system ABAQUS. A thermal and mechanical process in a soil column was simulated to illustrate the response of the model.

RÉSUMÉ : On présente un modèle élasto-plastique de comportement pour les sols soumis à des cycles de gel-dégel, ségrégation exclue. Ce modèle représente la réponse en déformation et l'évolution de la résistance du sol sous n'importe quel chargement mécanique, avec variations de température. Il s'inscrit dans le cadre de la théorie de l'état critique, avec une fonction de charge du type Cam Clay modifié. Un paramètre scalaire appelé pore ice ratio est défini, qui régit l'évolution de la fonction de charge sous l'effet du gel et du dégel. Le modèle a été implémenté dans le code de calcul aux Eléments Finis ABAQUS. La réponse du modèle est illustrée par la simulation d'une colonne de sol soumise à un chargement thermique et mécanique.

KEYWORDS: non-segregation soil freezing, pore ice, soil thawing, constitutive model

1 INTRODUCTION

Various soils experiencing freezing and thawing may exhibit dramatically different behaviors. For a non-frost-susceptible soil, such as medium sand, no ice segregation will take place during freezing. The pore water and pore ice will co-exist while the temperature drops below freezing point. An increase in strength occurs as the soil freezes; weakening is expected during the melting process. The bulk volume of the soil expands and contracts little due to the phase change. The same may occur in frost susceptible soils subjected to quick freezing. In this paper, both of these conditions will be considered as non-segregation cases in which no ice lenses are generated during freezing.

Elasto-plastic models and models accounting for viscous properties of ice (creep), as well as ice melting, have been developed (Lai *et al.*, 2009, Wei *et al.*, 2011). Most of these models are based on continuum approach, and are suited for one type of the frozen soil. Changes of the soil components upon freezing and thawing, and the corresponding changes in strength, are not addressed by these models. Some effort was made to include the governing parameters of soil freezing into the constitutive model (Shastri and Sanchez, 2012), but its applicability is yet to be assessed.

This paper focuses on developing an elasto-plastic constitutive model including freezing and thawing, to capture the deformation behavior and strength evolution of the soil subjected to loading and temperature changes. The model is based on the critical state framework and it is formulated by introducing the influence of ice ratio into the modified Cam clay model. The model developed is suited for non-segregation freezing and thawing soils (no ice lens formation). In non-segregation soils, the strength of the soil upon thawing will return to the strength prior to freezing, i.e., if the soil only experiences freeze-thaw cycle, but no change in loading, no thaw-settlement will take place. This is because the soil volume variations during freezing/thawing are only due to the phase

change of pore water. Creep effect of frozen soil is not considered in this paper.

2 THE MODEL

The model developed is temperature-dependent, with the *ice ratio* e_i being the key parameter that describes the influence of the ice content. The ice ratio is defined as

$$e_i = \frac{V_i}{V_s} \quad (1)$$

where V_i is the volume of ice and V_s is the volume of the solid constituent (skeleton). e_i is related to the unfrozen water content in the frozen soil.

2.1 Constitutive model

Compression tests on frozen soils indicate that the behavior of frozen soil in the v, p -plane (specific volume, isotropic stress) can be represented by normal compression line (NCL) and unloading-reloading line (URL) (Qi *et al.*, 2010, Lee *et al.*, 2002). The slopes of these lines vary, however, depending on the extent of freezing. The slopes for the two lines are defined by λ and κ for unfrozen soil, and λ_f and κ_f for frozen soil, respectively. The specific volume upon isotropic compression of frozen soil is given by

$$v = v_f - \lambda_f \ln p \quad (2)$$

and elastic behavior in unloading-reloading regime is given by

$$\delta v^e = -\kappa_f \frac{\delta p}{p} \quad (3)$$

Both λ_f and κ_f are functions of ice ratio e_i . Soils become stronger upon freezing, which is characterized by higher preconsolidation stress p_o , and the slopes NCL and URL lines become flatter (Qi *et al.*, 2010). A reasonable relative position of normal compression lines for a soil in both unfrozen state and frozen state is shown in Figure 1(a). The relationship between preconsolidation stress and ice ratio is illustrated in Figure 1(b). The yield surface has elliptical shape in q, p -plane (deviatoric, isotropic stress), and its evolution, as function of e_i , is illustrated in Figure 1(b). p_o is the preconsolidation stress for unfrozen soil, and p_o^f is the preconsolidation stress for the same soil in frozen state with the ice ratio of $e_i = e_{iC}$ (p_o^r is the reference stress).

Consider freezing and loading path B-C-D as illustrated in Figure 1a. A virgin compressed unfrozen soil (saturated) at point B is subjected to freezing under a constant isotropic stress state. The specific volume increases due to the volumetric expansion upon phase change. Then, isotropic load is added at constant temperature until point D is reached on the isotropic stress yield line for the frozen soil. During the isotropic loading process the ice ratio remains constant, $e_i = e_{iC}$. This is because the ice ratio follows the unfrozen water content curve in non-segregation freezing process, thus having a unique relationship to the temperature. The unfrozen water content will be discussed in the next section.

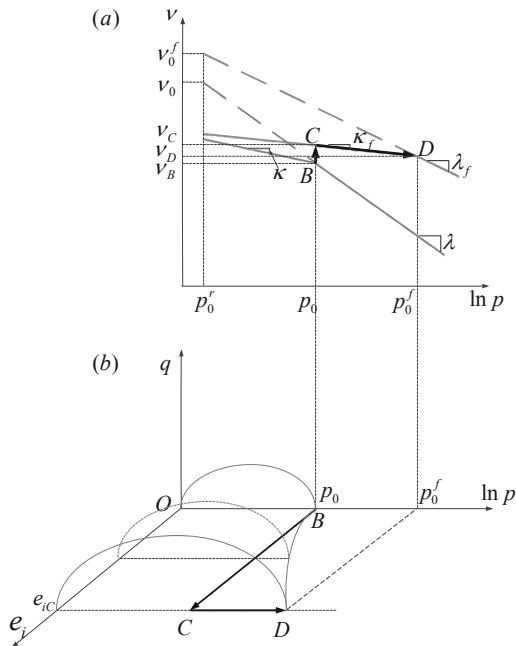


Figure 1 (a) freeze-thaw thermal loading path in v - $\ln(p)$ plane; (b) "pseudo preconsolidation pressure" changing along BD due to increasing pore ice ratio

Points B and D belong to the same yield curve on p, e_i -plane, and yield stresses p_o^f for frozen states with different ice ratio e_i during freezing from point B to C are located on curve BD (Figure 1(b)).

The specific volume at the final point D can be described as

$$v_D = v_B + \Delta v_{BC} - \Delta v_{CD} \quad (4)$$

where Δv_{BC} and Δv_{CD} are the increments due to initial freezing and due to subsequent loading.

The expansion upon freezing is related to phase change and is treated as reversible, because it reversed during thawing. This volume change can then be calculated as

$$\Delta v_f = 0.09 e_i \quad (5)$$

Substituting equations (2), (3), and (5) into (4) renders the following equation

$$v_0^f - \lambda_f \ln \frac{p_0^f}{p_0^r} = (v_0 - \lambda \ln \frac{p_0}{p_0^r}) + 0.09 e_i - \kappa_f \ln \frac{p_0^f}{p_0} \quad (6)$$

where p_0^r is the reference pressure to locate the virgin compression lines for both soil states. Consequently, the increase in isotropic yield stress caused by the ice ratio increase is found as

$$\frac{p_0^f}{p_0^r} = e^{\frac{v_0^f - v_0 - 0.09 e_i}{\lambda_f - \kappa_f}} \left(\frac{p_0}{p_0^r} \right)^{\frac{\lambda - \kappa_f}{\lambda_f - \kappa_f}} \quad (7)$$

The slopes of the NCL and URL of frozen soil are both functions of e_i . This path of derivation was followed earlier by Alonso *et al.* (1990) in the context of unsaturated soils. Because the frozen soil has limited tendency for compression (pores are filled with ice), slopes λ_f and κ_f drop dramatically compared to those for the unfrozen soil. The following relationships are postulated

$$\lambda_f = \lambda \exp(-\alpha_1 e_i) \quad (8)$$

$$\kappa_f = \kappa \exp(-\alpha_2 e_i) \quad (9)$$

where α_1 and α_2 are soil constants.

In order for equation (7) to yield p_0 for unfrozen soil when $e_i = 0$, we postulate the following linear law defining the shift of v_0 to v_0^f as function of e_i

$$v_0^f = v_0 - (\beta - 0.09) e_i \quad (10)$$

where β is a soil constant. Equation (7) can, therefore, be written as

$$\frac{p_0^f}{p_0^r} = e^{\frac{-\beta e_i}{\lambda_f - \kappa_f}} \left(\frac{p_0}{p_0^r} \right)^{\frac{\lambda - \kappa_f}{\lambda_f - \kappa_f}} \quad (11)$$

An illustration of how the preconsolidation stress p_o^f changes in soil with progressively increasing ice ratio e_i is shown in Figure 2, for three cases, each starting from a different preconsolidation stress p_o for unfrozen soil (the following parameters were used: $\alpha_1 = \alpha_2 = 0.2$, $\beta = 0$, $\lambda = 0.35$, $\kappa = 0.07$)

The plastic volumetric strain due to mechanical load and thermal process can be calculated as

$$d\varepsilon_v^{pl} = \frac{\lambda_f - \kappa_f}{v} \frac{dp_0^f}{p_0^f} = \frac{\lambda - \kappa_f}{v} \frac{dp_0}{p_0} \quad (12)$$

The yield function in a p, q -plane is

$$f = q^2 - M^2 p(p_0 - p) = 0 \quad (13)$$

An associate flow rule is assumed, i.e., and the plastic potential $g = f = 0$.

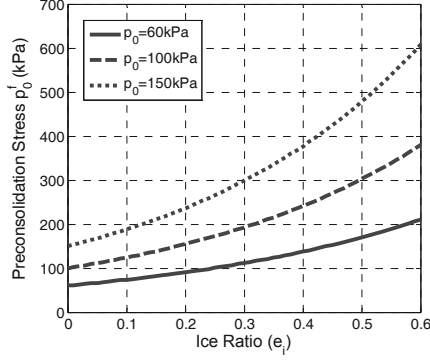


Figure 2 Yield stress for frozen soil with different ice ratio e_i

Thawing of a frozen soil will decrease the amount of ice in its pores, reducing “ice cementation” and consequently, reducing strength. For non-segregation frozen soils, the strength after complete thawing tends to move back to the same envelope as before freezing. There will be no further weakening since no thermally induced mass migration occurred during freezing, and the change of the soil fabric during the thermal process is not significant.

2.2 Unfrozen water content

The unfrozen water curve can be used to identify the amount of pore ice formed during freezing. In most of the soils other than coarse granular ones, not all water freezes at the freezing point. A 3-parameter function (Michalowski and Zhu, 2006) was developed to describe the unfrozen water content for most soils (see Figure 3). This function has the following form:

$$w = w^* + (\bar{w} - w^*) e^{a(T - T_0)} \quad (14)$$

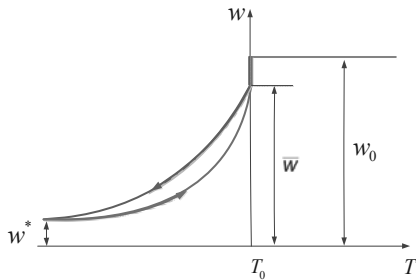


Figure 3 Unfrozen water content curve

At freezing point T_0 , a portion of the water freezes and the rest remains in the liquid state ($\bar{w}$). The liquid moisture content reduces with the decrease in the temperature to reach w^* at some low reference temperature. Parameter a describes this reduction rate.

The ice ratio e_i can be calculated from the unfrozen water content as

$$e_i = (w_0 - w) \cdot \frac{\rho_s}{\rho_w} \cdot 1.09 \quad (15)$$

3 THERMAL-MECHANICAL LOADING PROCESS

To better illustrate the model, a freeze-thaw cycle along with an external load applied is shown in Figure 4. The thermal process is depicted with the blue (freezing) and red (thawing) lines, whereas the loading-unloading is depicted by the green lines.

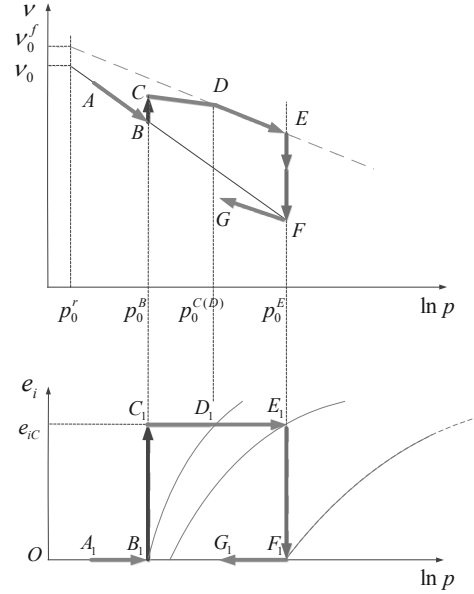


Figure 4 Freeze-thaw cycle and load path with corresponding yield curves

A saturated soil specimen is preconsolidated under isotropic compression from point A to B. Then, under constant stress, the freezing process takes place (B-C), and the ice ratio at point C is e_{ic} . In this process, the isotropic yield stress (apparent preconsolidation stress for corresponding frozen soil) increases from B_1 to D_1 following equation (7), and it has a value of p_0^D at point D_1 (the soil has the same apparent preconsolidation stress after it had been frozen at point C_1). The below-freezing temperature is then maintained, and isotropic compression is increased to reach the normal compression line for frozen soil at pressure p_0^D ; the load is then continued along the NCL to reach p_0^E . During this loading, the void ratio e is changing while the ice ratio remains constant. This is based on an assumption that ice and soil skeleton are both incompressible. From C to D, the frozen soil experiences elastic behavior, whereas from D to E it behaves plastically.

Thereafter, the soil is thawed at constant stress p_0^E . Once ice starts melting, the soil can no longer sustain load $p = p_0^E$, and the process of consolidation will start, moving the soil to the normal compression line for the unfrozen soil (point F). Unloading from F results in an elastic rebound along the URL for unfrozen soil to G.

4 APPLICATION AND FINAL REMARKS

The constitutive model was implemented into the FE system ABAQUS using subroutine UMAT and UEXPAN to solve boundary value problems. A soil column subjected to both the mechanical load and a thermal process was simulated.

The parameters and initial values used in the simulation are listed in Table 1. λ and κ are the slopes for NCL and URL in compression plane. M is the slope of the critical state line in p - q space. p_0 and e_0 are the initial values for the pre-consolidation pressure and the void ratio. w^* , $\bar{w}$, and a are parameters for

unfrozen water content function; this function was calibrated with the data for Fairbanks silt, Figure 5. The curve fitting function is represented in Figure 5 by the continuous curve (red). The unfrozen water curve is quite steep, and it is assumed that the functions for freezing and thawing process are the same.

Table 1. Parameters and initial values in simulation

λ	κ	Initial p_0	M	α_1	α_2
0.35	0.1	60 (kPa)	1.0	0.2	0.2
e_0	w^*	$w = w_0$	a	μ	β
0.85	0.08	0.325	6.0	0.3	0

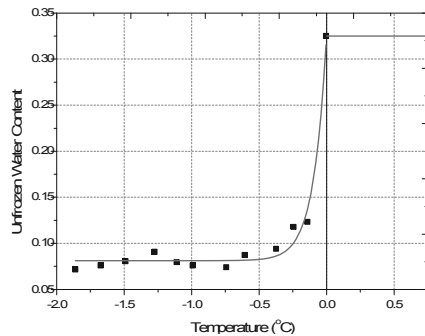


Figure 5 Unfrozen water content curve (experimental points from Huang *et al.*, 2004)

The soil column is 1 m tall and 0.05 m in width. The walls of the column are adiabatic and rigid. The initial uniform temperature is 1°C, and the initial vertical and horizontal compressive stresses are 20 kPa and 10 kPa, respectively.

Initial and boundary conditions in terms of load and temperature at the top of the column are shown in Figure 6. The bottom of the column is fixed and the temperature is maintained at 1°C throughout the process.

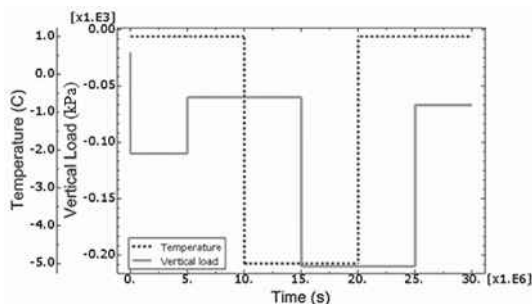


Figure 6 Boundary condition on top of the soil column

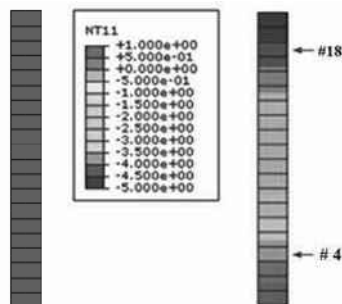


Figure 7 Temperature distribution at $t=0$ and $t=1.3e7s$

The relationship between void ratio and the vertical stress is shown in Figure 8 for element #18 and #4. For element #18, the temperature is about -4.5°C when the column reaches steady state after freezing, having more pore ice and being stronger than element #4 whose steady state temperature is around

-0.3°C. There is a substantial difference in the behavior of the two elements during the loading segment from 60 to 210 kPa: while element #18 behaves elastically, element #4 is elastic only until the load reaches 160 kPa, and becomes elasto-plastic afterward. Both elements experience additional settlement due to thawing.

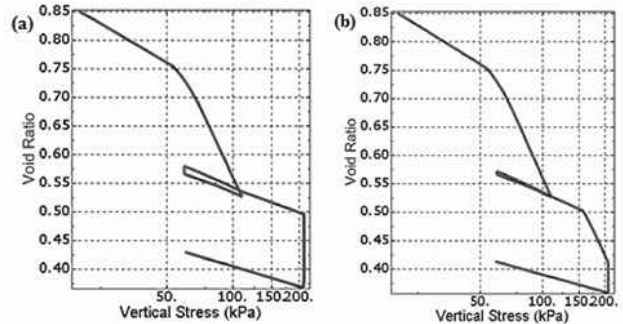


Figure 8 Compression for element (a) #18 and (b) #4

The model using the pore ice ratio as model parameter captures freezing and thawing process for non-segregation soils well. The parameter is related to the unfrozen water content curve and is easy to obtain from tests. This constitutive model is convenient to use, and it has been implemented in the FE system. The model will be calibrated based on the test data available, and it will be applied to solve practical boundary value problems.

The model can be used as a tool to predict the behavior of soil subjected to freezing and thawing as long as no ice lenses are formed. Such problems include, for instance, construction using artificial ground freezing. The model will be extended to include ice lens formation in frost-susceptible soils.

5 ACKNOWLEDGEMENTS

The work presented in this paper was supported by the Army Research Office, grant No. W911NF-08-1-0376. This support is greatly appreciated.

6 REFERENCES

- Alonso E.E., Gens A. and Josa A. 1990. A constitutive model for partially saturated soils. *Geotechnique*, 40 (3), 405-430.
- Huang S.L., Bray M.T., Akagawa S., and Fukuda M. 2004. Field investigation of soil heave by a large diameter chilled gas pipeline experiment, Fairbanks, Alaska. *Journal of Cold Regions Engineering*, 18 (1), 2-34.
- Lee M.Y., Fossum A., Costin L.S. and Bronowski D. 2002. Frozen soil material testing and constitutive modeling. *Sandia National Laboratories Report* 2002-0524.
- Michalowski R.L. and Zhu M. 2006. Frost heave modeling using porosity rate function. *International Journal for Numerical and Analytical Methods in Geomechanics*, 30 (8), 703-722.
- Qi J., Hu W. and Ma W. 2010. Experimental study of a pseudo-preconsolidation pressure in frozen soils. *Cold Regions Science and Technology*, 60, 230-233.
- Roscoe K.H. and Burland J.B. 1968. On the generalized stress strain behavior of wet clay. In *Engineering Plasticity*, eds. J. Heyman and F.A. Leckie; Cambridge University Press, 535-609.
- Shastri A. and Sanchez M. 2012. Mechanical modeling of frozen soils incorporating the effect of cryogenic suction and temperature. *GeoCongress 2012: State of the Art and Practice in Geotechnical Engineering*, 2492-2501.
- Wei L., Hong F. and He W. 2011. Research on visco-elastic-plastic creep model of artificially frozen soil under high confining pressures. *Cold Regions Science and Technology*, 65 (2), 219-225.