Development of transparent clay for laboratory model tests

N. Mohidin & Marolo C. Alfaro
Department of Civil Engineering- University of Manitoba, Canada, Winnipeg, Manitoba, Canada
M. I. M. Masirin
Department of Geotechnical and Transportation Engineering-Universiti Teknologi Tun Hussien Onn Malaysia Parit Raja, Batu Pahat, Johor, Malaysia

ABSTRACT
Peat and soft clay deposits have created challenges and risks in construction of road embankments. They can read in excessive deformations and failures of foundations. One of the solutions to reduce deformations and prevent failures can be the use column-supported and geocell reinforced embankments. A research program is being carried out to understand the mechanisms of deformations and failures using physical and numerical model tests. The physical model tests in the laboratory use artificial transparent soils to simulate foundation clays and embankment fill materials. This will allow the use of visual techniques to determine strains in the foundation soil. This paper presents the development of synthetic transparent clay, including determination of its geotechnical properties from oedometer and triaxial tests.

RÉSUMÉ
Tourbe et dépôts argileux soft ont créé des défis et risques de construction de route talus. Ils peuvent entraîner de déformations excessives et les échecs de fondations. Une des solutions pour réduire les déformations et prévenir les défaillances est à utiliser colonne-prise en charge et les talus geocell renforcé. Un programme de recherche est en cours de comprendre les mécanismes des déformations et des échecs à l'aide des essais sur modèle physique et numérique. Les modèle physique de tests en laboratoire artificielles des sols transparents permet de simuler la Fondation argiles et de matériaux de remplissage du talus. Cela permettra l'utilisation de techniques visuelles pour obtenir les contraintes et les déformations dans le sol de Fondation. Ce livre présente le développement d'argile transparent synthétique, y compris la détermination de ses propriétés géotechniques d'oedometer et tests accéléromètre triaxial.

1 INTRODUCTION

There are various methods used for soft soils ground improvements to overcome the problems related to constructions on this soil. The design concerns are related to bearing capacity failure, intolerable total and differential settlement and lateral movements of the soil.

One of the methods is stone column which is also known as rockfill column. A diameter column was drilled and then filled with course aggregates such as sands, gravels or stones. This method had improved stability and bearing capacity of the soil which results in limiting settlements that could occur to the structure built. Rockfill column also has secondary function as vertical drain that provides excess pore water pressure. Unlike traditional method of using pile, rockfill columns are installed in large size so no pile caps needed. Therefore the construction cost is reduced. The performance of rockfill column are reported by Alfaro et al. 2009, Thiessen et al. (2010), Han et al (2007), Mckelvey et al. (2003) and Lee and Pande (1998).

The other option is to use geosynthetics reinforced pile where layers of geosynthetics are used above the piles. These layers of geosynthetics create a stiffened platform that spans the soft ground and prevent deflection between columns from being reflected to the surface. In addition to no pile caps needed, this method uses fewer piles since the spacing between columns are maximised. Thus, this method is faster, produce less disturbance to the existing structures nearby and more cost efficient compared to traditional and rock fill columns (Filz and Smith 2006). Han et al. 2006 reported that geosynthetic reinforcements are often employed in bridging layer to enhance load transfer to the column and reduce total and differential settlements. Then, Abdullah and Edil (2007) reported that the beam effect was effective if three or more layers of geosynthetic reinforcements were applied, if less than that the catenary behaviour was observed.

In this study, geocell is proposed to be used in place of planar layers geosynthetics. Geocell is a three-dimensional, polymeric, honeycomb-like structure welded or tied together at joints and filled with soil. Tafresi and Dawson (2010) indicated that, the geocell system behaves much stiffer and able carries larger loading and settles less than the equivalent geosynthetic planar reinforcement system. This benefit can be used to enhance the system. Therefore, geocell reduces the need of time-consuming compacted fills used in multiple geosynthethics reinforced pile. As a result, this method is expected to be faster and cost less than geosynthethics reinforced pile method.

This study involves laboratory model tests to understand the performance of geocell-reinforced embankments by simulating four model test set-ups namely: i) embankments without reinforcement (control condition (ii) with geocell (iii) with geocell and rockfill columns and (iv) with geosynthetics and rockfill columns. These four conditions enable the comparison between existing methods with proposed new method. The use of see through artificial clays in laboratory model tests in this study offers more accurate determination of deformation patterns of the foundation during the application of embankment loading. This paper is focused on preparation procedures of producing the synthetics soil and the results of consolidation and shear strength characteristics of the synthetics transparent clay soil.

2 PROPERTIES OF MATERIAL USED

Amorphous silica and pore fluid were two key materials to produce the transparent clay. A commercially available powder product of amorphous silica (Fig.2) was used. The amorphous silica consists of ultra fine particle with individual diameter of 0.02 μm and surface area of 170 - 230 m^2/g. The refractive index, bulk density and tampered density for the silica powder determined in this study are 1.46, 20-130 kg/m^3 and 40 g/l, respectively. The specific gravity value around 2.1 was reported by McKelvey, (2002) and Gill and Lehane, (2001). According to Iskander et al. (2002) and Liu (2003), amorphous silica powder is well-matched for producing transparent clay-like material for the reason that (i) it has hygroscopic property; therefore it has ability to adsorb pore fluid and replacing the air, (ii) large surface area similar to clays. Consequently, silica has special characteristic that combines microparticles together to form larger aggregates of similar size with natural clay particles.

The internal porosity of the silica is high and therefore void ratio. This high void ratio is not representative of typical natural clays. Iskander et al. (1994) suggested using the interaggregate void ratio instead to represent the void ratio in geotechnical applications. The interaggregate void ratio can be determined by equation proposed by Liu et al. (2002) as follows.

$$e_i = \frac{e_t - \alpha\gamma_s}{1 + \alpha\gamma_s} \quad [1]$$

where, e_i is interaggregate viod ratio, e_t is total void ratio, γ_s is the unit weight of solids and α is the adsorption factor.(is defined as the volume of pore fluid absorbed per unit weight of solids). The specific gravity for this material has been reported to be around 2.1 (McKelvey, 2002; Gill and Lehane, 2001). The amount of absorbed oil was estimated by Mannheimer and Oswald, (1993) to be 2.1 cm3 of pore fluid per gram of amorphous silica.

The pore fluid was prepared from the blend of a colourless mineral oil and normal paraffinic oil as solvent. Both fluids were blended at 50:50 ratios by weight. The blend oil has a refractive index 1.464 at room temperature (24^{o}C). The viscosity and density of the oil blend were 0.0548 Pa.s (54.8 cP) and 868 kg/m3, respectively. The combination gave the clearest end product of transparent synthetic soil.

3 SYNTHETIC SOIL PREPARATION

The production of transparent materials can be customized to meet model test requirements is the necessary foundation for utilizing optical techniques to study spatial deformation patterns (Liu 2003). Matching the material with suitable liquid is the key factor to make good transparent slurry. This produces the right transparent mixture. The production of transparent soil for present study was made by matching the refractive indices of selected pore fluid and amorphous silica in order to ensure that refraction is avoided, both materials used must have appropriate matching indices. Liu et al., (2002) and Iskander et al. (1994) indicated two main factors that contribute towards the clarity of this synthetic soil are: (i) the right match of the refractive indices of silica and fluid used, and (ii) the absence of entrapped air and impurities. If any of those factor are not fulfilled, partially transparent or an opaque soil will be obtained.

The first step to prepare the sample is to dissolve amorphous silica into blended pore fluid at the concentrations of 5% by weight. The right percentage need to be determined to maintain workability of slurry. Then, the suspensions were de-aired to remove all entrapped air by applying a vacuum. This process continues until the silica slurry became transparent. The larger the aggregates size, the longer time is needed (Iskander et al. 2002). The slurry is then moved into a modified one-dimensional consolidometer.

Two different modified one-dimensional consolidometers were fabricated to prepare specimens for triaxial and oedometer tests. A pressure of 100 kPa was applied using dead loads for both specimens during the sedimentation phase. In producing the oedometer specimens, modified consolidometer consists of two mesh wires and filter papers on top and bottom inside of a plexiglass mould. The mould produce specimen of 64 mm in diameter, close to the size of specimens used in consolidation test. Triaxial test specimens was prepared using a modified consolidometer that made from split steel mould extended with PVC pipe. It consists of two mesh wire and filter papers on top and bottom inside the mould. The mould produced specimens with dimensions of 50 mm diameter and 100 mm height. Fig. 3(a) and 3(b) show the end product demonstrating the transparency of the artificial clay specimens for triaxial and oedometer tests, respectively.

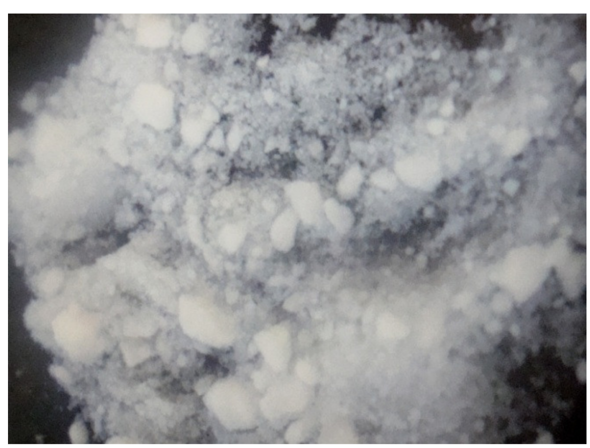
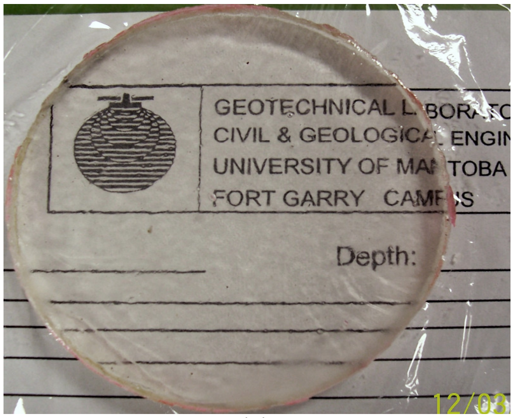
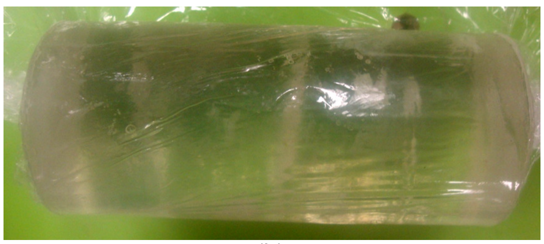


Fig. 2. Amorphous silica powder



(a)



(b)

Fig. 3. (a) Artificial transparent clay specimens for triaxial tests (b) artificial transparent clay specimens for oedometer tests.

4 LABORATORY TESTING PROGRAM

Consolidation and triaxial compression test were performed on specimens from the modified consolidometer. One-dimensional consolidation tests were carried out in accordance with ASTM D2435-03 (2003), Test Method for One-Dimensional Consolidation Properties of Soil to determine the consolidation properties of the synthetic soil. Specimen height is 63 mm, while the diameter 15 mm. The test was conducted using conventional odeometer apparatus, using loading increment ratio (LIR) of unity. The applied pressures were in the range of 50 kPa to 800 kPa. Filter papers were positioned on the top and bottom creating separation between specimens and porous stones. This prevents clogging from occurring.

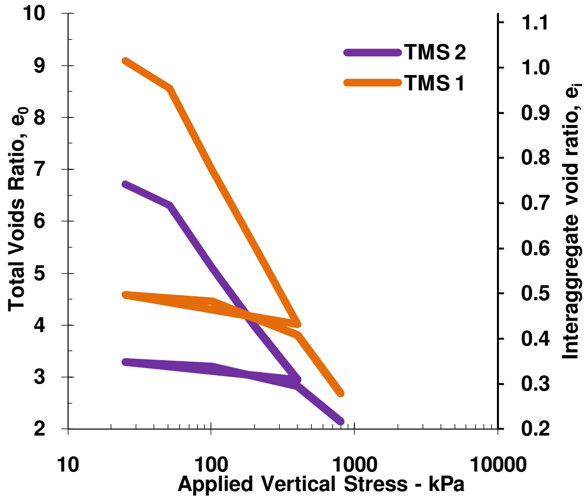
Shear strength properties of synthetic soil was determined by performing the conventional consolidation isotropic undrained (CIU) triaxial test. The test was performed according to ASTM D4767-02 (2003) Standard Test Method for Consolidated Undrained Triaxial Compression Test on Cohesive Soil. Normally consolidated specimens were tested using pressure raging from 50 to 100 kPa. Specimens 100 mm height and 50 mm diameter were extracted from the mould and trimmed for this study. During the saturation phase, approximately 400 kPa was applied as back pressure for all samples. The shear rate was 0.0055 mm per minutes. The failure is define base on maximum deviatoric stress as proposed by Liu et al. (2002).

5 CONSOLIDATION PROPERTIES

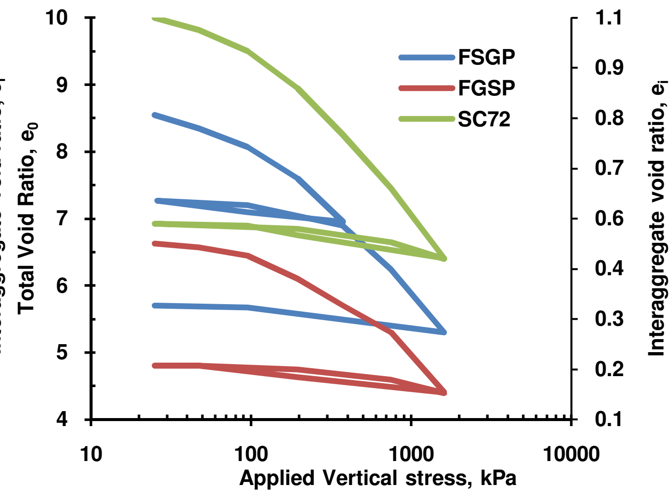
Typical relationship between total void ratio and applied stress e-log p curves from odeometer tests are shown in Fig. 4(a). The results are comparable with those for typical clays and typical e –log p curves from separate test results of Liu (2003) as demonstrated in Fig. 4(b). In that Fig., the total void ratio is shown on the left vertical axis and the interaggregates on the right.

Table 1 Summary of consolidation properties of transparent clay.

Parameter	Compression Index C_c	Recompression Index C_r	C_r/C_c Ratio	Hydraulic Conductivity k (cm/s)	Total Void Ratio, e_0	Interaggregate Void Ratio e_i	Coefficient of Consolidation C_v (cm^2/s)
Present Study	4.6 to 3.8	0.28	0.09	8.5×10^{-6} to 9.5×10^{-6}	9.5 to 7.0	0.87	1.1×10^{-4} to 2.3×10^{-4}
McKelvey et al. 2004	4.8	-	-	2.3×10^{-7} to 2.5×10^{-5}	39 to 11	-	2.54×10^{-4} to 3.8×10^{-4}
Liu 2003	1.6 to 3.0	0.16 to 0.3	0.1	2.3×10^{-7} to 2.5×10^{-5}	10.55 to 6.65	0.85 to 0.2	0.78×10^{-3} to 1.66×10^{-3}
Iskander et al. 2002	1.6 to 3.0	0.15 to 0.3	0.1	-	10.6 to 6.7	0.8 to 0.2	0.58×10^{-3} to 1.66×10^{-3}
Sadek et. al. 2002	1.6 to 3.0	0.15 to 0.3	0.09 to 1.0	2.3×10^{-7} to 2.5×10^{-5}	8.5	0.8	1.0×10^{-3} to 2.0×10^{-3}
Gill and Lehane 2001, 2004	0.34	-	-	1.0×10^{-10}	12.1	2.6	2.95×10^{-4}
Iskander et al. 1994	2.35	0.23	0.1	2.3×10^{-7} to 2.5×10^{-5}	6.6	3.4	1.0×10^{-3} to 2.0×10^{-3}



(a)



(b)

Fig. 4 (a) Typical e-log P curves from current study, (b) Replotted e-log P curve from Liu et al.(2003).

The compression index (C_c) is in the range of 4.6 – 3.8, while recompression index (C_r) is about 0.28. These values are comparable with the range of montmorillonite clays. The coefficient of consolidation, C_v of the specimen ranged between 0.7x10^{-3} and 0.55x10^{-3} cm^2/s and the value values are within the range typically reported for highly plastic montmorillonite clays reported by Lambe and Whitman (1979). The ratio C_r/C_c is around 0.09, which is again comparable to values of 0.02 – 0.2 for natural clays reported by Terzaghi et al. (1996). The values for total void ratio were in the range of 9.5-7.0 and again compared well with typical very soft clays and peat soils (Head 1994 and Al-Khafji and Andersland 1992) High total void ratio is influence by internal porosity of silica aggregates (Liu 2003). The coefficient of volume change, m_v is 1.3 m^2/MN which is within the typical range of 0.3 – 1.5 for normally consolidated alluvial clays reported by Head (1994). The consolidation parameters obtained are summarized in Table 1 and compared with those by other investigators

6 SHEAR STRENGTH PROPERTIES

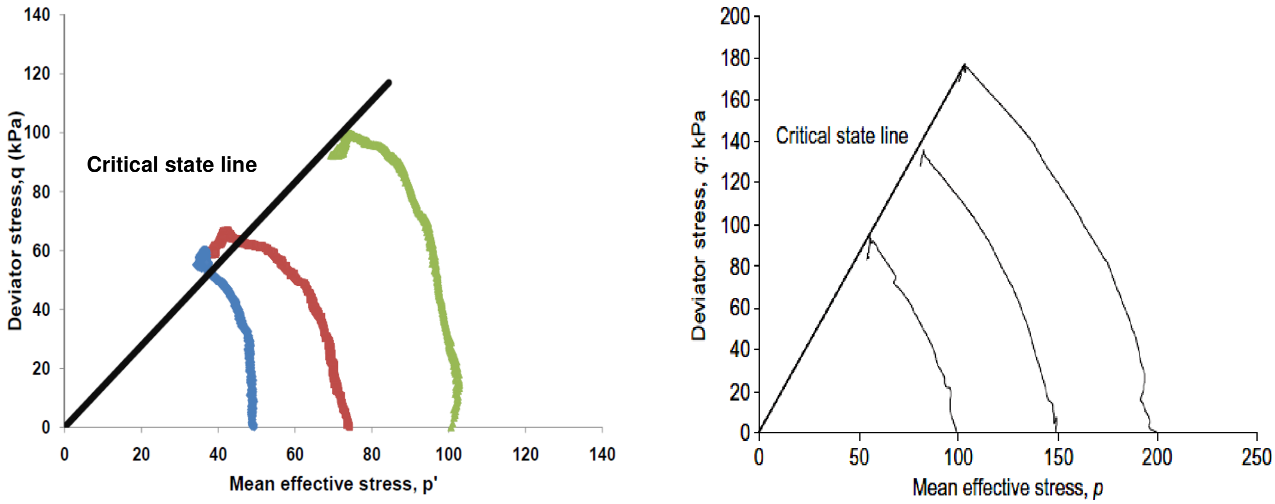
Typical stress strain and pore-pressure curves of undrained normally consolidated amorphous silica are shown in Fig. 6. Generally, the curves show gradual increment in deviator stress as the axial strain increases and no strain softening observed. The behaviour is consistent with typical of normally consolidated clay (Budhu 2007). The maximum excess pore pressure generated during shear reached 57 to 73% of the confining pressures.

Bishop and Henkel (1969) reveal that characteristic of soft clays can be confirmed when both the maximum excess pore pressure and the strain at which it occurred increased with confining pressure. This characteristic was also found to hold for amorphous silica in this study. Furthermore, results obtained are similar with characteristics reported by Iskander et al. (1994), Gill and Lehane (2001), Sadek et al. (2002) and Liu (2003). For 50 and 75 kPa samples, the test stopped at about axial strain of 20% but for 100 kPa sample the test had to be stopped at 9 % . at this point the sample had reached the shearing failure. Although it stopped on lower axial strain, the data obtained is adequate to determine the peak failure stress and pore pressure relationship for analysis.

The behaviour of the transparent material in the q–p' plane is shown in Fig. 5. The observed behaviour is very similar to that of normally consolidated clay. The slope of the critical state line, M, was found to be 1.39. This implies that the angle of internal friction and cohesion of the material is approximately 34^0 and 0, respectively. Internal friction angle and cohesion values for this study are similar with the value reported by McKelvey et al. (2004). This angle of internal friction appears to be high for a material that has many clay-like properties since similar observation were reported by previous researchers (Iskander et al. 1994, Gill and Lehane 2001, Sadek et al. 2002 and Liu 2003 and McKelvey et al. 2004).

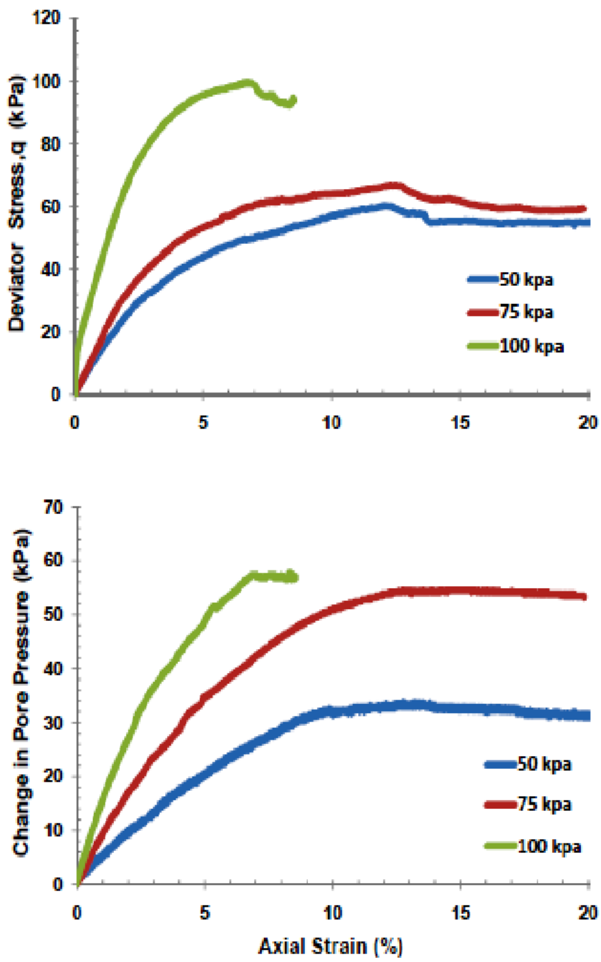
On the other hand, these values are still in the range of properties reported by Kenny (1959), and Bjerrum and Simons (1960) for many types of clay. The increased strength is expected considering that transparent soils are made of silica and possesses low plasticity (Bjerrum and Simons 1960).

Ladd and Lambe (1963) reported that normally consolidated natural clay demonstrates a unique relationship between normalized stress-strain and pore pressure. The synthetics transparent amorphous silica illustrates the same observation. Curves obtained from this study for both normalized pore pressure and deviatoric shear stress are comparable with those results obtained by Liu 2003 as demonstrated in Fig. 7. In that figure, pore pressure (u) and deviatoric shear stress (σ_1-σ_3) are normalized by the consolidation pressure (σ_c). Where (σ_1 and σ_3 are representing major and minor principle stresses, respectively.



(a) (b)

Fig. 5. Relationship between deviator stress and mean effective stress from (a) current study (b) McKelvey et al. (2004).



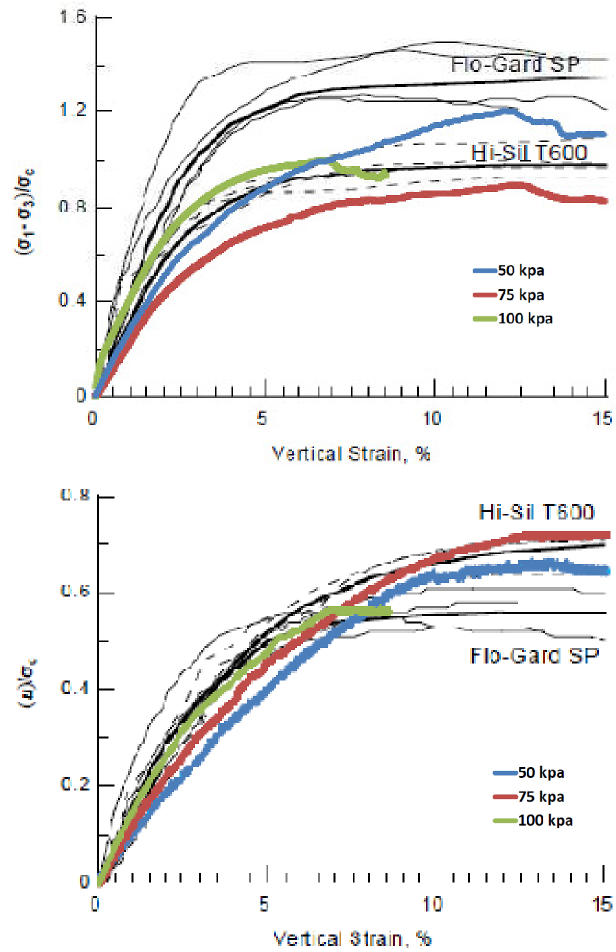


Fig. 6. Typical stress-strain and pore-pressure diagrams of undrained normally consolidated amorphous silica

Fig. 7. Comparison of normalized stress-strain and pore pressure curved between current study and results from Liu (2003).

7 CONCLUSIONS

The synthetic transparent soil produced in this study exhibits consolidation and shear strength characteristics similar to varies soft clays. Although the mechanical and flow properties of the natural clays depend on mineral contents that may not be found in artificial clays, transparent synthetic soils will allow simulating natural clays in laboratory model tests. With the use of digital imaging techniques, the patterns of deformation of foundation with and without reinforcements can be investigated. This offers invaluable potential for future simulations of soil deformations and developing design guidelines and construction methods of mitigation works required for embankments on soft clay foundations.

ACKNOWLEDGEMENTS

The first author is grateful to the Ministry of Higher Education (MoHE), Malaysia and Universiti Tun Hussein Onn Malaysia (UTHM) for funding his studies. The funding for this research is provided by the Natural Sciences and Engineering Research Council (NSERC) of Canada. The authors are grateful to the laboratory technicians, Kerry and Tai, for their help and assistance

REFERENCES

Abdullah, C.H.and Edil, T.B. 2007. Behaviouir of geogrid-reinforced load transfer platform for embankment on rammed aggregate piers. Geosynthetics International **14**:141-153

Alfaro, M.C., Blatz, J.A., Abdulrazaq, W.F. and Kim, C.S. 2009. Evaluating shear mobolization in rockfill columns. Canadian Geotechnical Journal 46(8): 976-986.

Al-Khafji A.W. and Andersland B.O.1992. Geotechnical Engineering and Soil Testing, Sounders College Publishing, U.S.A

ASTM D2435-03 2003.Standard Test Method for One-Dimensional Consolidation Properties of Soils, Annual Book of ASTM Standards, ASTM Vol.04.08,Philadelphia, PA.

ASTM D4767-02 2003. Standard Test Method for Consolidated Undrained Triaxial Compression Test on Cohesive Soil. Annual Book of ASTM Standards, ASTM Vol.04.08, Philadelphia, PA.

Bathurst R.J. and Karpurapu R. 1993; Large Scale Traxial Compression Testing of Geocell Reinforced Granular Soils, Geotechnical Testing Journal **16**(3):296-303

Bjerrum, L., and Simons, N.E. 1960. Comparison of shear strength characteristics of normally consolidated clays. *In* Proceeding of the 1st ASCE Specialty Conference: Shear Strength of Cohesive Soils,: 711–726.

Bishop, A. W. and Henkel, D. J. 1969. The measurement of soil properties in the triaxial test, 2d ed., Edward Arnold, London

Budhu M. (2007); Soil Mechanics and Foundation 2nd. Edition John Wiley and Sons, Inc. USA

Bush D.I., McCombie P.F., and Smith I.M. 1990. An examination of method of analysis and prediction of performance of the geocell mattress, *In* Proceeding of the International Reinforced Soil Conference, Glasgow :329-333

Filz, G. and Smith, M. 2006,Design of bridging layer in geosynthetic - reinforced, column supported embankments. Virginia Transportation Research Council. Virginia USA.

Gill, D. and Lehane, B. 2001. An optical technique for investigating soil displacement patterns. ASTM Geotechnical Testing Journal, GTJODJ, **24**(3): 324-329.

Gill, D. and Lehane, B. 2004. Displacement fields induced by penetrometers installation in an artificial soil. International Journal of Physical Modelling and Geotechnics **1**: 25-36.

Han J., Oztoprak S., Robert L.P., Huang J. 2007. Numerical analysis of foundation columns to support widening of embankments Computers and Geotechnics **34**: 435–448

Head 1982. Manual of Soil Laboratory Testing, Volume 2 Pentech Press, London.

Hinchberger S.D. and Rowe R.K. 2003. Geosynthetic reinforced embankments on soft clay foundations: predicting reinforcement strains at failure. Geotextile and Geomembranes, **21**: 151-175.

Iskander M. G. 1997. Transparent soils to image 3D flow and deformation, imaging technologies : techniques and applications in Civil Engineering. Proceeding of the Second International Conference, CRESTA Switzerland, : 255-264.

Iskander, M., Lai, J., Oswald, C., and Mannheimer, R. 1994. Development of a transparent material to model the geotechnical properties of soils. ASTM Geotechnical Testing Journal, GTJODJ, **17**(4): 425–433

Iskander M., Sadek S., and Liu J. 2003. Soil structure interaction in transparent synthetic soils using digital image correlation. TRB 2003 Session on Recent Advances in Modeling Techniques in Geomechanics (Committee A2K05) TRB 2003 Annual Meeting CD-ROM

Konagai, K., Tamura, C., Rangelow, P., and Matsushima, T. 1992. Laser-aided tomography: a tool for visualization of changes in the fabric of granular assemblage. *In* Proceedings Japan Society of Civil Engineers (JSCE) No: 455 I-21. Structural Engineering – Earthquake Engineering **9**(3):193–201

Kenny, T. C. 1959. Discussion of Geotechnical properties of glacial lake clays. Journal of Soil Mechanics Found. Division, ASCE, **85**(SM3): 67–79

Ladd, C. C., and Lambe, T. W. 1963. The strength of undisturbed clay determined from undrained tests. Laboratory shear testing of soils, ASTM STP No. 361, West Conshohocken, Pa. :342–371.

Lambe, T.W. and Whitman, R.V. 1979. Soil Mechanics, SI Version, Wiley, New York.

Lee, J.S. and Pande, G.N. 1998. Analysis of stone column reinforced foundations. International Journal of

Numerical and Analytical Methods in Geomechanics, 22: 1001-1020.

Liu J. 2003. Visualization of 3-D deformation using transparent soil models, PhD Dissertation, Polytechnic University.

Madhavi G.L., and Rajagopal, K. 2007. Parametric study finite element analyses of geocell supported embankment, Canadian Geotechnical Journal, **44**: 917-927

Magnan, J. 1994. Methods to reduce the settlement of embankments on soft clay: A review. Vertical-Horizontal Deformations of Foundations and Embankments, ASCE, Geotechnical Special Publication No. 40: 77–91.

Mannheimer, R.J., and Oswald, C. 1993. Development of transparent porous media with permeabilities and porosities comparable to soils, aquifers, and petroleum reservoirs. Ground Water, **31**(5):781–788.

McKelvey D. 2002. The performance of vibro stone column reinforced foundations in deep soft soil, PhD Thesis, The Queen`s University of Belfast

Rajagopal K., Krishnaswamy N.R., Madhavi G.L., 1999. Behaviour of sand confined with single and multiple geocells, Geotextiles and Geomembranes **17**: 171-184

Sadek, S., M. Iskander and J. Liu 2002. Geotechnical properties of transparent material to model soils. Canadian Geotechnical Journal., **39**:111-124.

Tafresi S.N. and Dawson A.R. 2010. Comparison of bearing capacity of strip footing on sand with geocell and with planar forms of geotextile reinforcement. Geotextile and Geomembranes **28(**1):72-84

Terzaghi, K., Peck, R.B. and Mesri, G. 1996. Soil Mechanics in Engineering Practice, 3rd ed.,Wiley.

Thiessen, K.J., Alfaro, M.C., and Blatz, J.A. 2010. Measuring the load –deformation response of rockfill column by full-scale test on riverbank Canadian Geotechnical Journal -*In Review*.

Welker, A., Bowders, J., and Gilbert, R. 1999. Applied research using transparent material with hydraulic properties similar to soil. Geotechnical Testing Journal, GTJODJ**, 22**(3): 266–270.