EXPERIMENTAL STUDY ON SETTLING CONSOLIDATION BEHAVIOR OF HONG KONG MARINE DEPOSITS

F. Tong[1] and J. H. Yin[2]
[1] Department of Civil and Structural Engineering, The Hong Kong Polytechnic University
Email: 08901828r@polyu.edu.hk
[2] Department of Civil and Structural Engineering, The Hong Kong Polytechnic University

ABSTRACT

The time-dependent feature of soft soils has gained intensive attraction in recent years. Due to the high water content and viscous property, the Hong Kong marine deposit (HKMD) frequently poses challenge to geotechnical practice, particularly to the reclamation in Hong Kong. A key issue regarding reclamation design, foundation construction and maintenance is excess settlement/deformation of the ground. Formation of HKMD typically goes through sedimentation and self-weight consolidation. A series of one-dimensional settling column consolidation tests are conducted on the HKMD. Results reveal that the settling curve and settling rate are significantly affected by the sediment concentration. Further study is conducted on the viscous behaviour in self-weight consolidation. Related parameters from sketches are evaluated for a better interpretation of the whole settling and consolidation behaviors.

KEYWORDS

Time-dependent, HKMD, Sedimentation, Self-weight consolidation, Settling column, Settling rate, Creep

INTRODUCTION

Construction on marginal land has gained increasingly attraction for economical and environmental purposes. Soft clays, mainly termed as marine clays in these constructions typically have quite high water content, high compressibility, and high swelling capacity. Appropriate prediction of soil behavior regarding consolidation, swelling and deformation has critical impacts on the design, construction, and maintenance. These behaviors are particularly time-dependent, requiring reliable soil description and understandings.

Sedimentary soils are formed initially through a process of sedimentation either in sea or in fresh water. Three main sedimentation stages are defined with respect to the concentration degree as Clarification regime, Zone-settling regime, and Compression regime. Fitch (1983) modified Kynch's theory by considering the presence of compaction zone at the column bottom. Next, the interface settles at a constant rate. In consolidation settling, no discrete flocs are formed. Actually the settling rate of dredged clay is viewed a key factor in land reclamation projects. The settling types of the dredged clay in seawater are mainly of zone settling and consolidation settling (Imai, 1980).

The fundamental mechanism of self-weight consolidation was successfully interpreted by Mikasa (1965). He defined the volume ratio f as f= l+e, where e is the void ratio. The relationship between the volume ratio f and effective stress p is needed for the estimation of the final settlement, and the relationship between the coefficient of permeability k and volume ratio e is needed for the estimation of the settling rate. Nishimura et al. (1992) reported that these parameters significantly influence the numerical results in consolidation analysis. The f-p and the k-f relationships, for numerical analysis of the self-weight consolidation, can be determined by the hydraulic consolidation test (Imai, 1981).

The self-weight consolidation recently becomes a major challenge in many engineering problems on the seashore (Mcvay et al., 1986). The settling type of marine clays in seawater shifts from zone settling to self-weight consolidation when the initial water content decreases to less than about 1000% (Imai, 1980). The zone settling rate of a clay suspension with high water content is so rapid thus makes the estimation difficult. However, it has been commonly accepted that the traditional consolidation theories are inadequate to explain the self-weight consolidation (Sills, 1981). In this consolidation field, two hypothesizes exist. One claimed such

consolidation started at the base, which was supported by majority of researchers including Imai (1981); the other considered contradictively that the consolidation commenced at the drainage surface-the top, which was approved by Li and Williams (1995).

Secondary compression or creep effect, on the other hand, can be important for high water content soils, especially those that contain organic matter. Settlement behavior referred to "tertiary compression" is also observed. This creep-like behavior has been attributed to biodegradation and gas generation. However recent studies have illustrated that treatment of the specimens for bacterial activity does not entirely remove the effect.

Consolidation testing of marine clays using traditional geotechnical testing methods is not practically possible. Among the conventional consolidation apparatuses, the settling column test is suitable in this study for sedimentation and self-weight consolidation analysis. The prime objective is to examine the settling consolidation behavior of Hong Kong marine deposits in self-weight condition. Applicable interpretation of consolidation of HKMD in wide stress range is critical to model this type of soils and field applications.

BASIC PROPERTIES OF HKMD

Two typical marine clays (12 and 17) with viscosity are adopted from around Nam Sang Wai and Mai Po, Hong Kong for self-weight settling and oedometer analysis. Before self-weight consolidation, basic properties including water content and specific gravity are determined and shown in Table 1.

Table 1 Basic properties of settled HKMD samples

Sample	Water Content (%)	Bulk Density (g/cm^3)	Specific Gravity (g/cm^3)
12	94.6	2.03	2.54
17	68.8	2.01	2.62

METHEDOLOGY AND TESTING PROCEDURAL

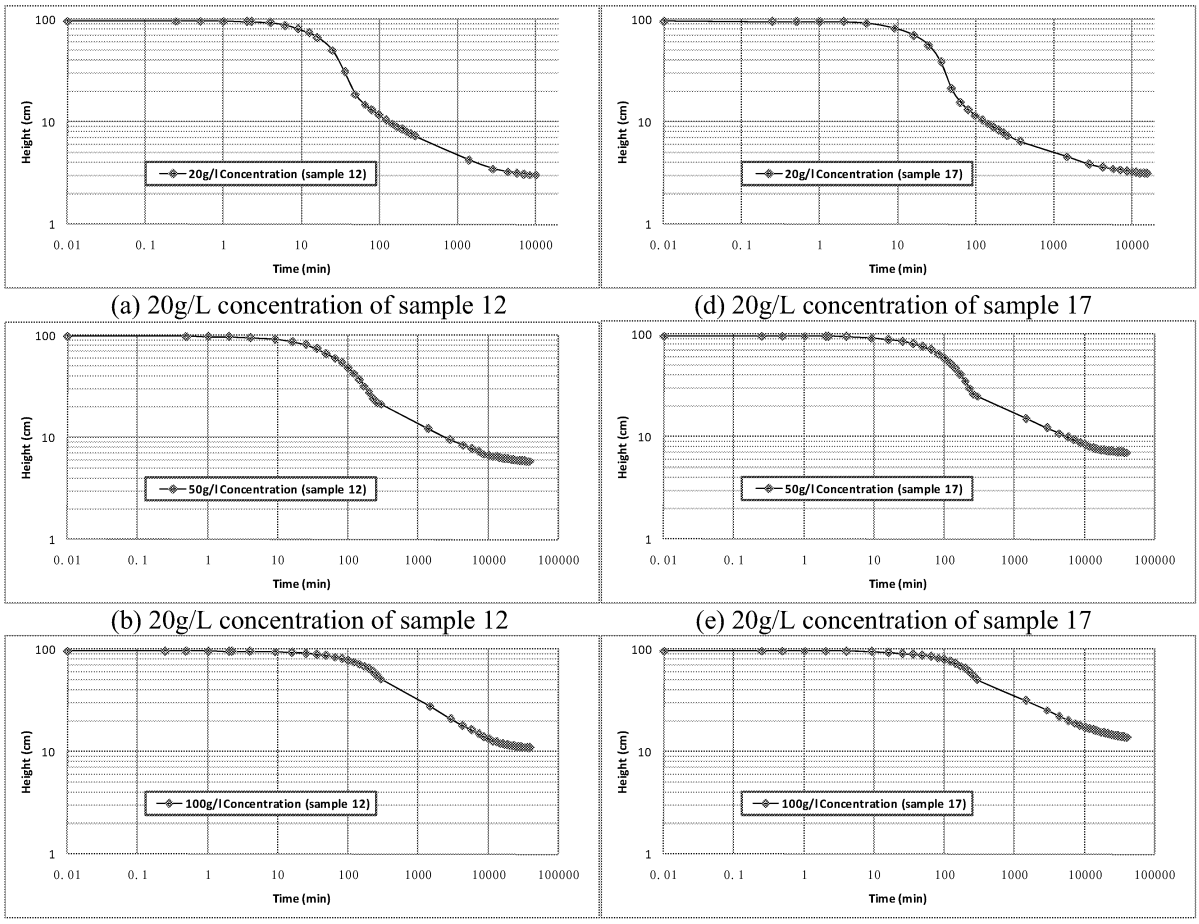
The experiments were conducted using site water with a pH of about 7.2. Two soil samples in different colors with site water were tested. Glass jars of 130 mm internal diameter were used for the tests. The raw soil of known water content and weight was mixed with the water to form slurry, and then the water content was increased to desired value. A high-speed rotary mixer is used to prepare slurry. The soil-water mixture was thoroughly mixed and transferred to the test jar and allowed settle. The soils were artificially sedimented at different initial water contents in order to obtain both segregated and homogeneous sediments. In settling column consolidation, initial water content of the soil-water suspension is one of the important factors controlling the settling behavior of soils (Imai 1980). The soil slurry is prepared with water content sufficiently high, yet maintaining the homogeneity (Sheeran and Krizek, 1971). At very high initial water contents, grain size sorting takes place, resulting in segregated sediments. In contrast, when the initial water content is below a limiting value, mutual mechanical interaction among the settling soil particles/flocs dominate, which results in homogeneous sediments. All the tests were carried out under the constant laboratory temperature of 20±1°C. Result from the settling tests are presented and discussed in the following section.

The time scale to reach full consolidation depends on the initial concentration of the sample used to fill the column. By doubling the concentration, the duration also (nearly) doubles, requiring a four times longer consolidation time. After free-settling for sufficient time, the water above the sediment was siphoned out and a specially made spatula (bent horizontally at end) was inserted for sampling layer by layer starting from the top of the sediment over the entire depth of the sediment.

The final thickness of the soil varies from 38mm to 146mm. A special transparent plastic tube with an internal diameter of 3mm and very thin thickness (0.05 mm) is used to suck soil at the certain depth into the tube. The weight the tube is measured so that the density is obtained. The soil is pushed out and put into a glass container. The contained with the wet soil is weighed to get the wet mass. The contained with the wet soil is then out in an oven to dry for 24 hours at the standard temperature l05°C and weighed to get the dry mass. In this way, the water content is determined and the void ratio profiles over the entire depth of the sediment were obtained.

SETTLING RESULTS AND DISCUSSIONS

With settling-column technique, one can get the compression curve over a very low stress range (Imai 1981; Umehara and Zen 1982; Scully 1984). Typical settling results for each sample with different concentration degree (20g/l, 50g/l and 100g/l, respectively) are illustrated in Figure 1 and Figure 2.Data can be used to plot the compression curve (i.e., e-logσ' or w%-logσ') where w%, e, and σ' are the water content, void ratio, and the effective overburden pressure at any depth measured from the top of the soil sediment, respectively.



(a) 20g/L concentration of sample 12 (d) 20g/L concentration of sample 17

(b) 20g/L concentration of sample 12 (e) 20g/L concentration of sample 17

(c) 20g/L concentration of sample 12 (f) 20g/L concentration of sample 17

Figure 1 Relationship of vertical strain and elapse time in settling column tests

When the stress is low, the e-logσ' curve indicates the extent of grain size sorting takes place during settling. After particle segregation, the self-weight consolidation takes place. Flatter curve means more homogeneity. In this sense, the absolute case without self-weight consolidation will produce a horizontal line.

All the test results are proved to be reasonable. According to Merckelbach (2000) and Merckelbach and Kranenburg (2004a, b), the settlement of the interface h (t) may be expressed as:

$$h(t) = \left(\xi \frac{2-n}{1-n}\right)^{\frac{1-n}{2-n}} \left[K_K \frac{\rho_s - \rho_w}{\rho_w}(n-2)\right]^{\frac{1}{2-n}} t^{\frac{1}{2-n}} \tag{1}$$

where ρ_s is the density of the solid particles; ρ_w is the density of pore water and can be taken as 1.0 Mg/m^3; ζ is the material height, m; K_K is the permeability parameters; n=2/(3-D), D is the fractal dimension.

The above equation can be rewritten in normal log scale as

$$\log h(t) = \log\left[\left(\xi \frac{2-n}{1-n}\right)^{\frac{1-n}{2-n}} \left(K_K \frac{\rho_s - \rho_w}{\rho_w}(n-2\right)^{\frac{1}{2-n}}\right] + \frac{1}{2-n}\log t \tag{2}$$

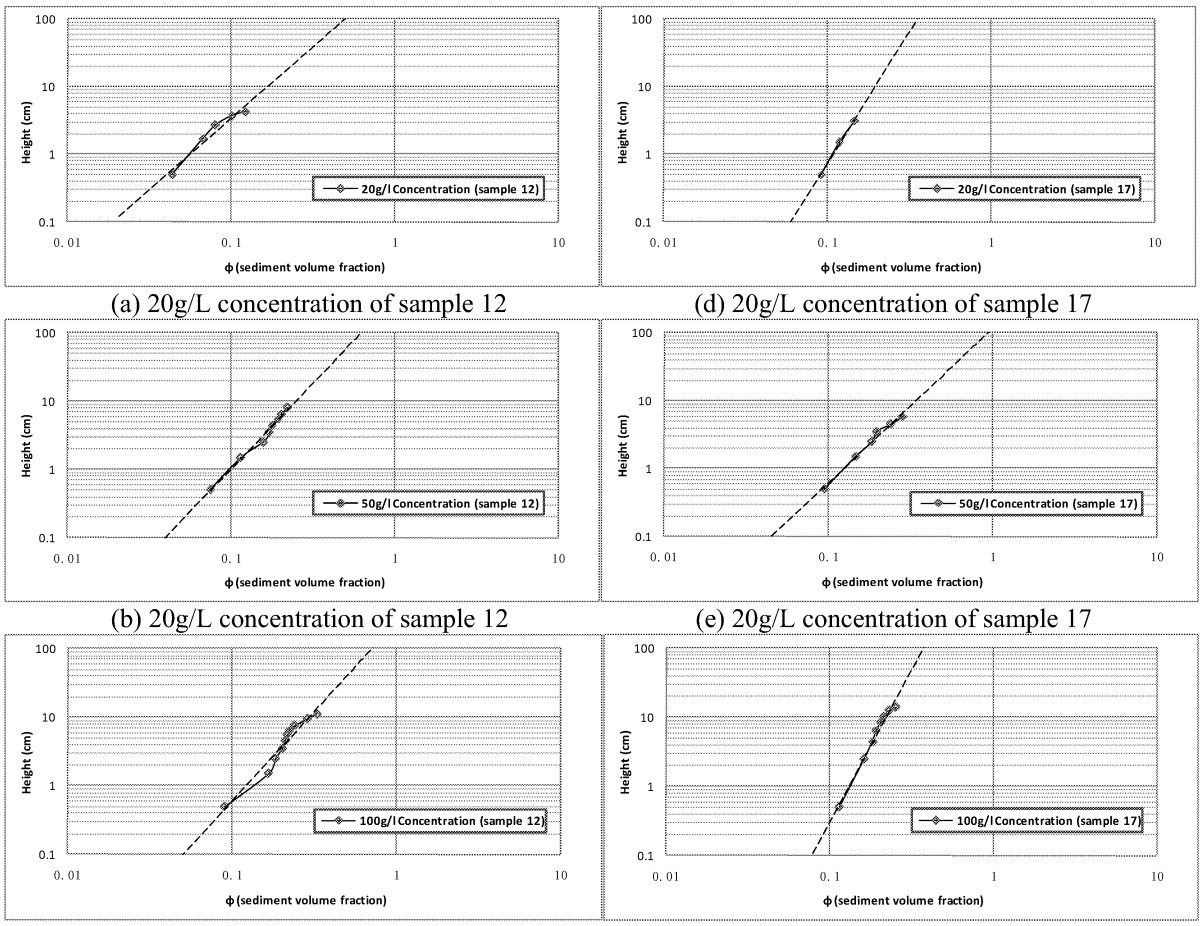
As demonstrated in equation (2), the parameters for settling interface are to be determined. In all the settling progress, the parameter K_K is a key factor governing permeability, which can further affect settling rate.

$$k = c_v m_v \gamma_w = K_k \cdot \phi^{\frac{-2}{3-D}} \tag{3}$$

where k is the permeability; φ is sediment volume fraction, namely the volume of the solid particles with respect to the total volume.

Note that equation (2) is a straight line, only valid for $t1<t<t_2$. At $t=t_1$, the interface becomes clear and a single discontinuity remains. At $t=t_2$, the effective stress takes effect, and the assumption in equation (3) does not work anymore. The final density profile after self-consolidation exhibited in Figure 2 can be adopted to compute the effective stress parameter K_σ in equation (4). In Figure 2, the slope of soil height over volume fraction becomes steeper once the soil concentration increases.

$$\log h(t-z) = \log\left(\frac{2K_\sigma}{(D-1)(\rho_s-\rho_w)g}\right) + \frac{D-1}{3-D}\log\phi \tag{4}$$



(a) 20g/L concentration of sample 12 (d) 20g/L concentration of sample 17

(b) 20g/L concentration of sample 12 (e) 20g/L concentration of sample 17

(c) 20g/L concentration of sample 12 (f) 20g/L concentration of sample 17

Figure 2 Relationship of sediment height and volume fraction after consolidation

Then the effective stress can be determined.

$$\sigma_v^{'} = K_\sigma \cdot \phi^{\frac{2}{3-D}} \tag{5}$$

Additionally incorporating equation (4), the coefficients of consolidation c_v can be determined and summarized in Table 2. Value of c_v is obtained based on equation (6).

$$c_v = -\frac{k(1+e)}{\rho_w g}\frac{d\sigma^{'}}{de} = \frac{2}{3-D}\frac{K_\sigma K_K}{\rho_w g}\frac{1+e}{e} \tag{6}$$

It can be seen from Table 2 that the coefficient of consolidation increases gradually with reduction of HKMD concentration. The fractal dimension however increases once increasing the concentration degree. Another fact can be found that sample 17 consolidates "faster" than the sample 12, which means sample 17 exhibits more viscous property. Surely the value of c_v will slightly vary depending on the void ratio and thickness of soil in column.

Extensive studies have been conducted on the relationship of strain and effective stress, particularly on the relevant C_c, C_e indexes. Early study by Skempton (1944) indicates the both the initial water content and the liquid limit W_L has a great impact on the consolidation behavior. Many researchers have already incorporated the W_L concept into compressibility estimation (Skempton, 1944; Terzaghi and Peck, 1967; Yoon et al., 2004).

Table 2 Summary of parameters from the settling tests

ID	Concentration	h (cm)	ξ (cm)	D	K_k (10^{-10} m/s)	K_σ (kPa)	C_v ($10^{-2}m^2$/yr)
12	20g/L	96.55	1.19	2.52	11.00	16.78	4.17
	50g/L	98.10	2.97	2.60	6.19	24.31	4.00
	100g/L	96.08	5.94	2.63	1.92	53.78	3.03
17	20g/L	95.91	1.15	2.52	11.20	40.03	9.80
	50g/L	96.40	2.88	2.62	4.09	64.72	6.86
	100g/L	96.57	5.75	2.63	1.14	106.46	3.21

Butterfield (1979) assumes that the plot of logV (V=1+e) and logp shows a linear relationship, when the initial water content is fairly high. Then the Basic Compression Line is defined as

$$\log_e V = \log_e V_1 - \frac{\lambda}{V}\log_e p \tag{7}$$

Where p is the consolidation pressure; V_1 is the specific volume at p=1kPa and λ/V is slope of the compression line expressed by Yin and Graham (1989) as

$$\frac{\lambda}{V} = \frac{\Delta\varepsilon_z}{\Delta\ln\sigma_z^{'}} = \frac{-\Delta e}{V\Delta\ln\sigma_z^{'}} = \frac{-\Delta e\times 0.434}{V\Delta\log\sigma_z^{'}} = \frac{0.434}{V}\frac{-\Delta e}{\Delta\log\sigma_z^{'}} = \frac{0.434}{V}C_c \tag{8}$$

By differentiation of the above equation, a linear fundamental law between stress and strain in equation (9) can be obtained.

$$\frac{dV}{V} = \frac{\lambda}{V}\frac{dp}{p} \tag{9}$$

However in Figure 1, the settling continues with time after the "linear region", which indicates that the structure of HKMD forms during free settling process (deposition), thus making the void ratio smaller than that on the compression curve, which further causes creep (secondary compression) during the whole consolidation.

CONCLUSIONS

The global consolidation behaviors of Hong Kong Marine Deposits both in settling column condition and in oedometer condition are studied in the paper. Conclusions can be drawn as follows.

Typical settling curves of HKMD are obtained. The e-logt curves obtained due to self-weight consolidation are use to evaluate the self-weight consolidation behavior. The void ratio of the sediment surface at the end of settling is found to be 9.8~11.6 for sample 12; 10.2~12.3 for sample 17.

The settling curves indicate a constant settling rate at zone settling, and then attain transition points, thereafter decreasing. Soils even exhibit viscosity in self-weight consolidation. After the primary self-weight consolidation, the settling rate is further reduced in the "secondary self-weight consolidation".

REFERENCES

Abu-Hejleh, A. N., Znidarcic, D. and Barnes, B. L. (1996). "Consolidation characteristics of phosphatic clays", *Journal of Geotechnical Engineering, ASCE* 122:4, 295-301.

Been, K., and Sills, G. C. (1981). ”Self-weight consolidation of soft soils: An experimental andtheoretical study”. *Geotechnigue*, 31(4):519-535

Butterfield, R. (1979). ”A natural compression law for soil (an advance on e-logp’)”, *Geotechnique*, 29(4), 469-480.

Fitch (1983). ”Kynch theory and compression zones”, *AIChE J.* 29(1983), 940-942.

GEO (2001). "GEOSPEC 3-Model Specification for Soil Testing". Geotechnical Engineering Office (GEO)

Yoon, G.L., Kim, B.T., and Jeon, S.S. (2004). ”Empirical correlations of compression index for marine clay from regression analysis”, *Can. Geotech. J.* 41(6), 1213–1221.

Imai, G. (1980). ”Settling behaviour of clay suspension”. *Soils and Foundations* 20(2), 61-77

Imai, G. (1981). ”Experimental studies on sedimentation mechanism and sediment formation of clay minerals”. *Soils and Foundations* 21(1), 7-20.

Park, J.H., Watanabe, K. and Seguchi, M. (1992). ”Experimental investigation on the settling properties of mud”, Bull. Fac. Agr. Saga. Univ., 73, 129-147.

Lee, K. and Sills, G.C. (1981). ”The consolidation of a soil stratum, including self-weight effect and large strains”. *Int. J. Num. Anal. Meth. Geomech*., 5, 405-428.

Li, H. and Williams, D.J. (1995). ”Numerical modeling of combined sedimentation and self-weight consolidation of an accreting coal mine tailing slurry”, *Proc. of compression and consolidation of clayey soils conference*, Balkema, Rotterdam, 441-452.

Liu, J.C. and Znidarcic, D. (1991). ”Modeling one dimensional compression characteristics of soils”, *J. Geotech. Eng.*, 117(1), 162-169.

Mcvay, M., Townsend, F. and Bloomquist, D. (1986). ”Quiescent consolidation of phosphatic waste clays”. *J. Geot. Engi., ASCE*, 112, 1033-1049.

Merckelbach, L. M. (2000). ”Consolidation and strength evolution of soft mud layers”. PhD thesis, Delft University of Technology.

Merckelbach, L. M. & Kranenburg, C. (2004a). ”Equations for effective stress and permeability of soft mud-sand mixtures”. *Geotechnique*, 54(4), 235-243.

Merckelbach, L. M. & Kranenburg, C. (2004b). ”Determining effective stress and permeability equations for soft mud from simple laboratory experiments”. *Geotechnique*, 54(9), 581-591.

Mikasa, M. (1965). ”The Consolidation of soft clay, a new consolidation theory and its application”, *Japanese Society of Civil Engineers*, 21-26.

Scully, R.W. (1984). ”Determination of consolidation properties of phosphatic clay at very high void ratios”, MSc Thesis, University of Colorado.

Sheeran, D.E. and Krizek, R.J. (1971). ”Preparation of homogeneous soil samples by slurry consolidation”, *J. of materials, ASTM*, 6(2), 356-373.

Shimada, K., Fujii, H., Nishimura, S. and Tajiri, N. (1992). ”Plane strain finite element analysis for consolidation settlement in soft ground improved with packed sand drains”, *Trans. of JSIDRE*, 162, 1-7.

Skempton, A.W., (1944). ”Note on the Compressibility of Clays”, *Q. J. Geological Society*. 100, 119-135.

Tan, T.S., Yong, K.T., Leong, E.C., and Lee, S.L. (1990a). ”Sedimentation of clayey slurry”, *J. Geotech. Eng.*, 116(6), 885-898.

Terzhagi, K.and Peck, R. B. (1967). ”Soil mechanics in engineering practice”, *2nd ed. John Wiely and Sons*, New York.

Yin, J.H. & Graham, J. (1989). “Viscous-elastic-plastic modeling of one dimensional time-dependent behaviour of clays”. *Canadian Geotechnical Journal*. 266, 199-209.