

CHAPTER 7

CONCLUSIONS

7.1 Summary of the Findings

A research program was initiated in the National University of Singapore to investigate the engineering properties of Singapore Old Alluvium (OA) as part of a bigger project to characterize all the natural soils in Singapore. Extensive laboratory tests and some field tests were carried out on OA during the process. Topics covered in this thesis include areas like sampling disturbance, in-situ stress state (K_o), strength and stiffness of OA. As the result of the studies conducted, the following conclusions can be drawn:

- 1) As the Old Alluvium was deposited by a braided river system, the material is very heterogeneous in both vertical and horizontal direction. Using a simple dispersion test, the material can be classified into cemented and uncemented OA. Using the Particle Size Distribution (PSD) test, the uncemented OA can be further classified into different sub-groups. The majority of OA is found to be uncemented, clay and sand mixture with low fines content.
- 2) Disturbances due to ‘perfect sampling’ and ‘ideal tube sampling’ were investigated using triaxial simulation on remoulded clay sand samples. Clay and over-consolidation helps soil samples to retain initial effective mean stress p' in ‘perfect sampling’ process. Over-consolidated remoulded clayey sand samples

- have strong resistance to ideal tube sampling disturbance. The in-situ p' value can be well retained though some of the structure may be destroyed.
- 3) In practice, the thin-wall sampler is not strong enough to sample Singapore OA. Comparing the shearing behaviour of OA samples taken using thick-wall, Mazier and block sampling, it is found that the thick-wall sampler imposes considerable disturbances to the retrieved OA sample. Therefore, only Mazier and Block samples can be regarded as 'undisturbed' samples of OA. Several methods may be taken to reduce the thick-wall sampling disturbances, such as to sharpen the cutting edge taper angle, to shorten the length of sampling tube and the usage of inside clearance of thick-wall sampler.
 - 4) One dimensional K_o consolidation tests were carried out using the triaxial apparatus and the oedometer cell. For loose, normally consolidated OA subjected to first time loading, the K_o value (K_{onc}) is constant and comply with Jaky's equation $K_o = 1 - \sin\phi'$. ϕ' is the critical state friction angle. Clay content up to 20% does not change ϕ' at critical state. Therefore it does not change K_o value.
 - 5) For over-consolidated OA, K_o values can be expressed by $K_o = K_{onc} OCR^a$. Parameter a is related to clay content. K_o values of over-consolidated soil are also upper bound by K_p . For over-consolidated OA in reloading, K_o values depend on when the reloading starts. If the soil is unloaded to very low stress level before reloading, K_o can be less than K_{onc} . K_o values are also lower bound by K_a and the reloading line will emerge with first time loading line when preconsolidation pressure is met.

- 6) In-situ horizontal stress was estimated using pressuremeter tests. Parameters of K_{onc} and a obtained in laboratory tests, together with the stress history of the site, can be used to give a rough indication of the in-situ K_o values.
- 7) The shear strength of OA is studied using triaxial CIU tests on intact Mazier and Block samples. For sand mixed with fines, void ratio e is no longer the governing factor of shearing behaviour. The concept of granular void ratio e_g is better than e but e_g does not differentiate contribution of plastic and non-plastic fines, and of the relative size of fines to the pore size.
- 8) The equivalent granular void ratio, e_{ge} , allows for different contribution factors to be assigned to account for the different fines and relative size. For normally consolidated sand mixture with plastic fines, the contribution factor of plastic fines is generally negative and the clay at best acts like voids. Over-consolidation will force the clay minerals out of sand-to-sand contact and generally prevent the fines to destabilize the structure and to cause the negative contribution. For non-plastic fines, it is generally positive and at worst act like voids. The contribution factor, b , has a physical basis for non-plastic fines as it reflects the relative size of the pore size to the non-plastic fines. The higher the ratio, which is big pores with small fines particles, the more likely the fines will not contribute to the strength or the smaller is the contribution factor. Over-consolidation has little effect on non-plastic fines.
- 9) The concept of e_{ge} was successfully applied to intact Singapore OA. Comparing to void ratio e and granular void ratio e_g , equivalent granular void ratio e_{ge} is superior in reducing the scattering of strength data. OA samples fall in a narrow

band in p' - q - e_{ge} space. Normalized Young's modulus of OA also generally decreases with the increase of e_{ge} .

- 10) Structure (packing and fabric) of uncemented OA has a strong influence on the small strain stiffness of OA. The relationship of small strain stiffness with the equivalent granular void ratio e_{ge} and the structure of the soil is not clearly understood. The shearing behaviour of cemented OA is different from uncemented OA and the concept of e_{ge} does not apply to cemented OA.

7.2 Recommendations for Future Research

In the present research project, the concept of equivalent granular void ratio e_{ge} was successively applied to characterize the shear strength of OA. However, during the process it is found that several aspects of OA are still unknown and seemingly not governed by e_{ge} . It would be interesting to further extend the research into the following areas:

- 1) The consolidation behaviour of OA. In Chapter 6 it is found that the shear strength of OA at steady state relates to the e_{ge} value after the consolidation is complete. At the same time, the volume change during the consolidation before undrained shearing remained an unsolved problem. It is known that for a clean sand, the volumetric strain during consolidation is usually little and there is no unique consolidation line in the ordinary pressure range. For uncemented OA, since the material contains sand, silt and clay, the behaviour can be more complex. If the consolidation behaviour of OA is to be understood better, the engineers and researchers would be better informed to predict the behaviour of OA.

- 2) The strains to failure of OA samples. It is admitted in Chapter 6 that the strain to failure ε_f values of OA samples were not predicted correctly and caused experimental errors to a certain degree. Obviously, the ε_f value is not governed by e_{ge} alone. Some axial strain is first needed to drive the clay minerals out of the force-carrying skeleton, then more axial strain is required to fully mobilize the force-carrying skeleton and to reach the steady state. If the ε_f values of OA samples can be known more precisely before shearing starts, a more consistent relationship of shear strength S_{us} and equivalent granular void ratio e_{ge} can be obtained.