Geotechnical and Geophysical Site Characterisation 5 – Lehane, Acosta-Martínez & Kelly (Eds)
© 2016 Australian Geomechanics Society, Sydney, Australia, ISBN 978-0-9946261-2-7

Characterisation of a lateritic soil using laboratory and in-situ tests

W. Shi

Changzhou Key Laboratory of Structure Engineering &Material Properties, Changzhou Institute of Technology, Changzhou, China

B.M. Lehane

School of Civil, Environmental and Mining Engineering, University of Western Australia

A. Gower

Water Corporation, Western Australia

S. Terzaghi

Arup Pty Ltd, Sydney

ABSTRACT: This paper extends our understanding of the characterisation of residual soils by presenting results from a field and laboratory investigation of a laterite soil from the site of Millstream Dam in southern West Australia (WA). The laboratory tests were conducted on intact (block) and reconstituted samples to allow the effects of structure on the mechanical characteristics of the soil to be examined. It is shown that structural effects contributing to the strength of the intact soil are relatively small but that this soil has a more open, permeable structure than equivalent reconstituted soil. Cone Penetration Tests are shown to provide a reasonable means of assessing the undrained strength of this particular soil (which is close to full saturation), despite the permeability being about 10 times higher than a typical sedimentary clay.

1 INTRODUCTION

Residual soil is typically found between the tropics of Cancer and Capricorn and is the weathered material remaining in-situ after soluble parent rock constituents have been removed. Recent large infrastructure developments in Northern Australia have revealed significant shortcomings in the characterisation and assessment of geotechnical design parameters for the residual soils present at these developments. Such shortcomings provide the motivation for this paper which aims to illustrate particular distinguishing features of one residual soil type and to examine the impact of these features on the derivation of parameters from the Cone Penetration Test (CPT).

Having formed in-situ, residual soils possess characteristics which are distinct from soils produced by erosion, transport and deposition. Laterites are residual soils formed by intensive and long lasting tropical weathering of the underlying parent rock. This tropical weathering is a prolonged process of chemical weathering which produces a wide variety in the thickness, grade, chemistry and ore mineralogy of the resulting soils (e.g. Geological Society 1997). The leaching and re-deposition of ions also causes a change in mineralogy and often produces a high void ratio and permeability. The high sensitivity to sample disturbance is a well known characteristic of residual soils (e.g. Leroueil & Vaughan 1990, Wesley 1990, 2010) and has been a significant impasse to the development of parameter interpretation techniques by geotechnical practitioners. The bonded structure of residual soils, which essentially provides a highly important c' component of strength, is damaged to varying degrees by conventional sampling techniques (i.e. tube and rotary cored sampling). As a consequence, correlations drawn between the laboratory test results on samples and in-situ test data (from CPTs and Standard Penetration Tests, SPTs) are poor and unreliable. Such poor reliability has discouraged Industry from specifying laboratory tests and has led to a reliance on design correlations for residual soils with in-situ test results. However, the (only available) correlations in use are those derived for un-structured soil, which are clearly not appropriate for residual soils.

This paper extends our understanding of the characterisation of residual soils by presenting results from a field and laboratory investigation of a laterite soil from the site of Millstream Dam in southern West Australia (WA). The laboratory tests were conducted on intact (block) and reconstituted samples to allow the effects of structure on the mechanical characteristics of the soil to be examined.

2 SOIL CHARACTERISATION

2.1 *Site conditions and sampling*

The first phase of the study involved the retrieval in 2012 of block samples of residual (lateritic) soil from the site of Millstream Dam, WA. The samples were obtained from a 9m deep benched excavation that had been created by the WA Water Corporation as part of its activities to increase the height of the dam (which was originally constructed in 1962). The block samples were cut from a depth of 4.5m below original ground level into 250×250×250mm cubes, which were subsequently wrapped in polythene and waxed (with the location and orientation clearly labelled). The samples were stored in specially fabricated wooden boxes and in a temperature and humidity controlled room at the University of Western Australia (UWA). A CPT had been performed in 2008 prior to any excavation at the location of the block samples.

The regional geology of the Millstream dam area is discussed in detail by Gordon (1984), and others. Weathering during the Tertiary period led to extensive laterisation of the in-situ quartz meta-gabbro and gneissic parent bedrock resulting in a well developed laterite soil profile (regolith). The CPT end resistance (q_c) and friction ratio profile at the block sample location is provided in Figure 1 along with the interpreted soil profile. The upper part of the profile comprises pisolitic or nodular gravel clasts in a sandy matrix and indurated pisolitic or massive ferricrete horizons.The block samples were obtained from the underlying, pale coloured (grey) lower pallid zone, which had neither the relic fabric of the underlying saprolite nor the significant development of secondary segregations such as mottles, nodules or pisoliths typical of other mottled zones within the profile. The maximum previous overburden pressure at this location is unknown.

2.2 *General soil classification tests*

The Robertson (2009) soil behaviour type (SBT) deduced form the CPT data at 4.5m on Figure 1 classifies the soil as a 'silty clay to clay' with a SBT index (I_c) of approximately 3. This classification is a little at variance with the hydrometer and sieve particle size distribution (psd) analyses which indicated that the material has a fines content of only 52% and a clay fraction (CF) of 18%. Liquid and plastic limits of the material are 62% and 35% respectively and these limits plot just below the A-line on Casagrande's plasticity chart in the high plasticity region (plasticity index, I_p=27%). The mean bulk density, specific gravity and degree of saturation of material from the block samples were 1560 kg/m^3, 2.65 and 84% respectively. The in-situ water content was 44 % (implying a liquidity index of 0.67) and the in-situ void ratio was 1.4.

The groundwater regime at the dam site is complex and influenced by both the reservoir water and water infiltration from the abutments. Based upon the available information, it is assumed that total and effective stresses are approximately the same at a depth of 4.5m (i.e. ambient pore pressure close to zero); the high degree of saturation of the block samples suggests that this is a reasonable assessment.

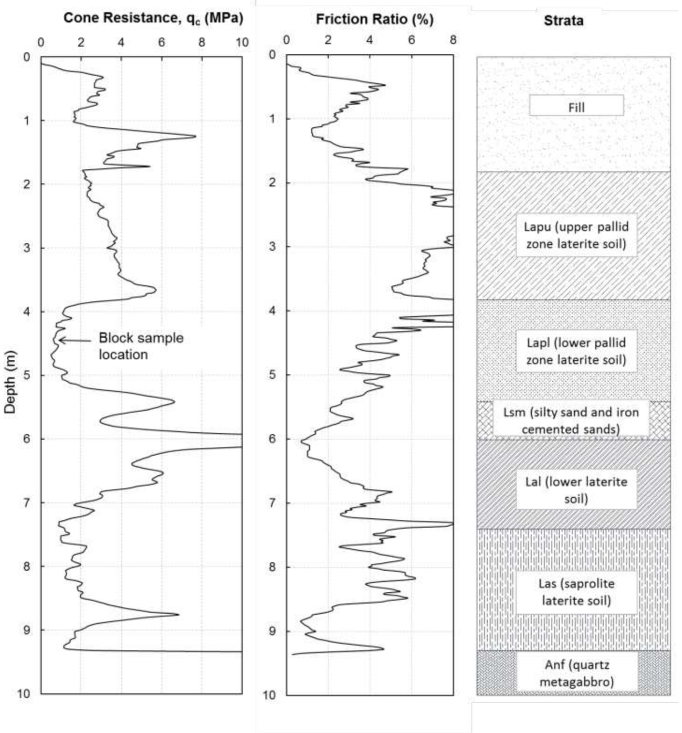

Figure 1. The in-situ CPT test results and soil profile.

2.3 *General soil classification tests*

X-ray diffraction analyses indicated that approximately 50% of the clay particles were kaolinite with some halloysite. Microcline and Muscovite (directly from the parent rock) made up 15% and quartz contributed another 15%. The material has evidently been leached of much of its iron content with only 4% of the clay fraction comprising iron rich compounds (ilmenite was dominant). The analyses could not distinguish about 15% of the constituents, labelling this component as 'amorphous content'.

The psd and Atterberg data imply a relatively high activity (I_p/CF) of about 1.5 which is more consistent with a clay mineral with higher activity than kaolinite; this may be due to the presence of particularly fine kaolinite crystals such as those described by Drummond *et al*. (2001).

2.4 *Behaviour in 1D compression*

Following on from the work of Burland (1990), and others, structural effects in the intact block samples were investigated by comparing the response of the intact soil in 1-D compression to the response of reconstituted soil. Reconstituted soil slurry with a water content of 62% (equal to the liquid limit) was prepared by adding water to oven dried soil that was ground down using a mortar and pestle. After de-airing using a vacuum, the slurry was consolidated one-dimensionally over a period of 10 days in 72mm diameter, 200mm high tubes (with top and bottom drainage) to a vertical effective stress of 56 kPa.

63.5mm diameter specimens for oedometer tests were extracted from the block samples and reconstituted tube samples using a cutting ring. The water bath was filled after application of the anticipated in-situ vertical effective stress of ~60 kPa. A comparison of the void ratio (e) - vertical effective stress (σ'_v) relationship obtained for both specimen types in standard oedometer tests (load increments held for 24 hours with a load increment ratio of 1) is provided on Figure 2. As is typical of structured soils (e.g. see Leroueil & Vaughan 1990), it is seen that the curve for the intact soil falls above that of the reconstituted soil (i.e. it exists at a higher void ratio at a given σ'_v). The intact sample indicates a vertical yield stress (σ'_{vy}) of about 125 kPa, which is 3 to 4 times the estimated in-situ effective stress at the block sample location. The compression index (C_c) for the reconstituted soil is 0.33 and significantly lower than the post-yield C_c value of 0.55 shown by the intact sample.

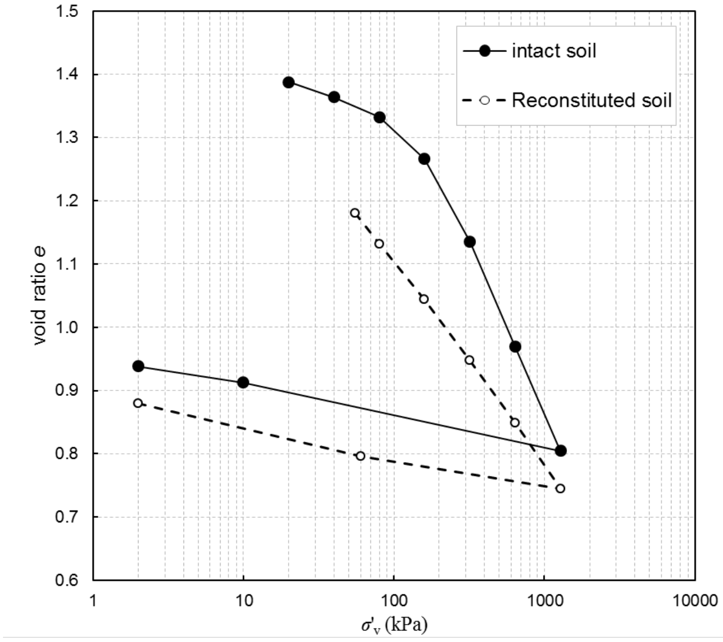

Figure 2. 1D compression curves for the intact & reconstituted soil.

The oedometer tests revealed far more rapid rates of consolidation for the intact sample. The coefficients of consolidation (c_v) derived from the settlement data are plotted against σ'_v on Figure 3 and indicate c_v values for the intact soil at the in-situ stress level over 100 times greater those for the reconstituted soil. The difference between the respective c_v values reduces as the stress level increases above the yield stress with c_v for the intact soil being 10 times the reconstituted value at σ'_v ~1 MPa. The values of vertical permeability (k) inferred from these c_v values around the in-situ stress range are 5×10^{-8} m/s and 5×10^{-10} m/s for the intact and reconstituted soil respectively. The relatively high c_v and k values for the intact soil are typical of residual soils and arise due to their distinctive formation process.

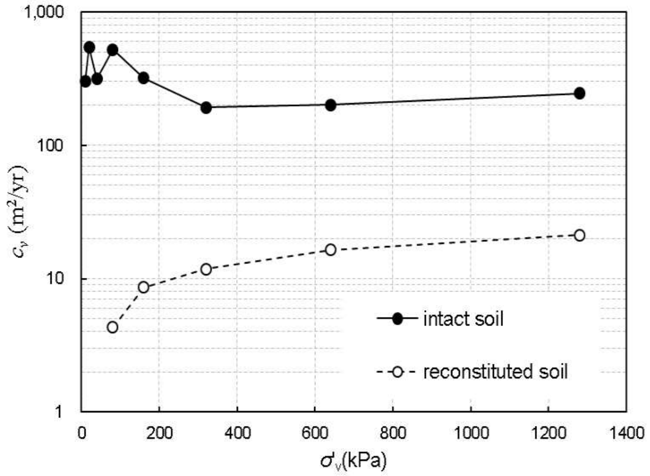

Figure 3. c_v values for intact and reconstituted samples

2.5 *Behaviour in triaxial compression*

Specimens for triaxial testing with a diameter of 72 mm and length of 150 mm were carved from the block samples and from samples extruded from the tubes used to prepare reconstituted samples for oedometer tests (discussed previously). The specimens were saturated and consolidated isotropically to mean effective stresses (p'_0) of 60 kPa, 120 kPa and 300 kPa before being sheared undrained. The variations of the deviator stress (q) with p' measured during the undrained shearing phase are shown on Figure 4.

These show a marked contrast between the stress paths recorded by the intact and reconstituted soil after consolidation to 60 kPa but generally similar paths after consolidation to the higher stress levels of 120 kPa and 300 kPa. The mean undrained strength ratio (s_u/p'_0) of the three tests on the (normally consolidated) reconstituted soil is 0.44. This relatively high ratio (compared to a more typical value of 0.25 to 0.3

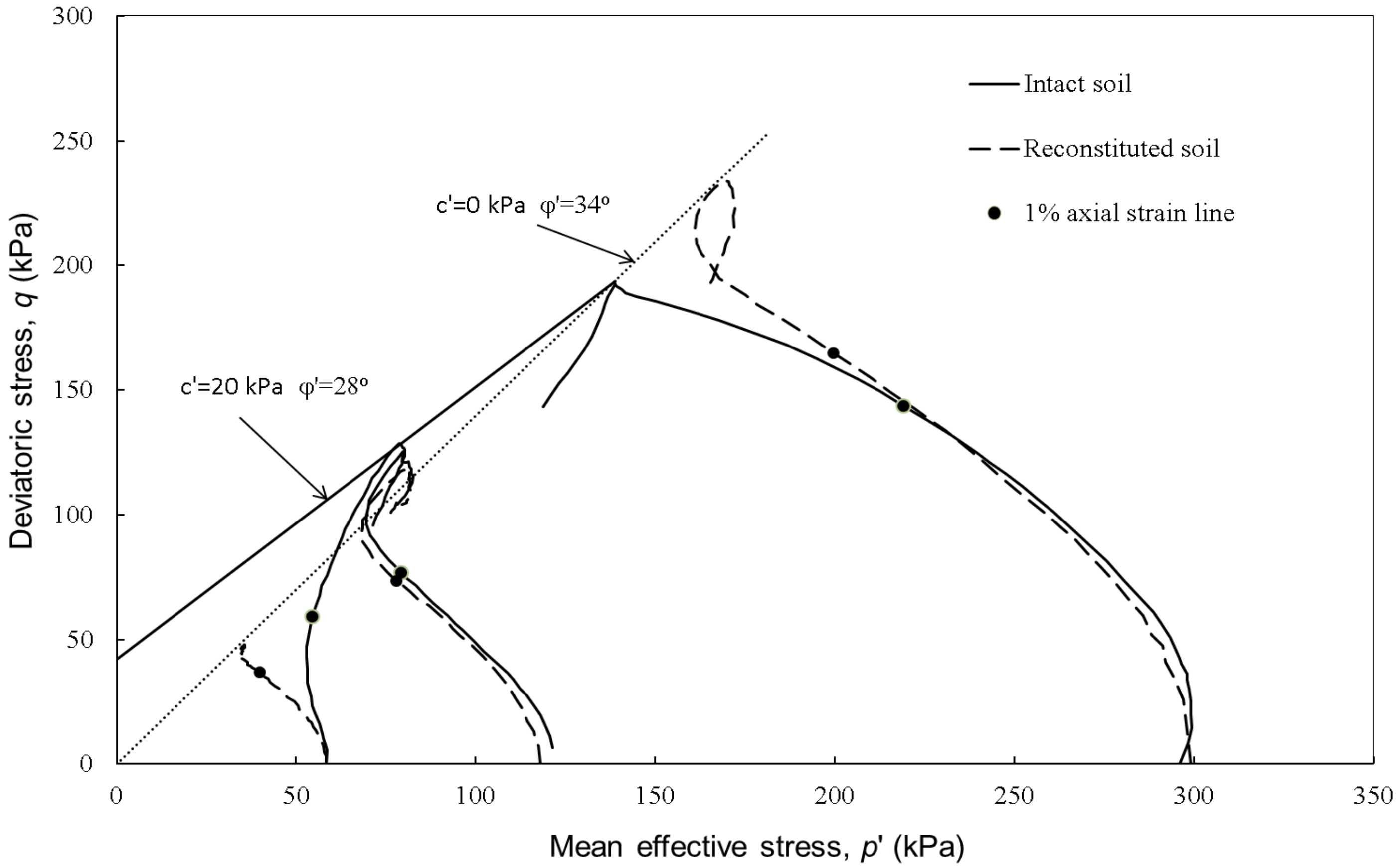

Figure 4. Stress paths in q-p' space during undrained triaxial compression tests on intact samples and reconstituted (normally consolidated) samples.

for alluvial clays) is attributed to the large coarse fraction and consequent tendency for a reduced level of contraction (or greater dilation) under shear compared to more fine grained deposits.

The high undrained strength developed by the sample consolidated to 60 kPa (s_u/p'_0 ~1.2) cannot be explained solely by overconsolidation (given that the other stress paths suggest that the maximum pre-consolidation isotropic stress is about 120 kPa) and must also reflect some structural effects which appear to be progressively destroyed with the imposition of high stresses. Although the behaviour of the intact specimens with the reconstituted specimens for $p'_0 \geq 120$ kPa is comparable, the effects of structure are clearly still important as the intact samples are at a considerably higher void ratio than their reconstituted equivalents (e.g. see Figure 2).

Mohr Coulomb parameters of c'=20 kPa and φ'=28° provide the best fit to the peak strength envelope for the intact soil. The best-fit friction angle of 34° for the reconstituted soil (for which c'=0) is relatively high compared to that of the intact soil and possibly reflects differences in the way the respective soil fabrics were created.

2.6 *Small strain stiffness*

Small strain shear moduli (G_{max}) were derived from the shear wave velocities deduced from bender elements located at the top and bottom of the triaxial specimens. Surprisingly, G_{max} values for the reconstituted soil were found to the greater than those of the intact soil. When G_{max} values are normalised by the void ratio (e) function, $F(e)=e^{-1.3}$, proposed by Lo Presti (1994), intact and reconstituted values exhibit the same relationship with mean effective stress, as shown on Figure 5. The best fit equation for G_{max}, which uses atmospheric pressure (p_{atm}) as a reference stress, is as follows. This reveals a square root dependence of G_{max} on the effective stress level, which is typical of coarse grained soils (Tatsuoka et al 1997):

$$\frac{G_{\max}}{e^{-1.3}p_{\text{atm}}} = 900\left(\frac{p'}{p_{\text{atm}}}\right)^{0.5} \qquad (1)$$

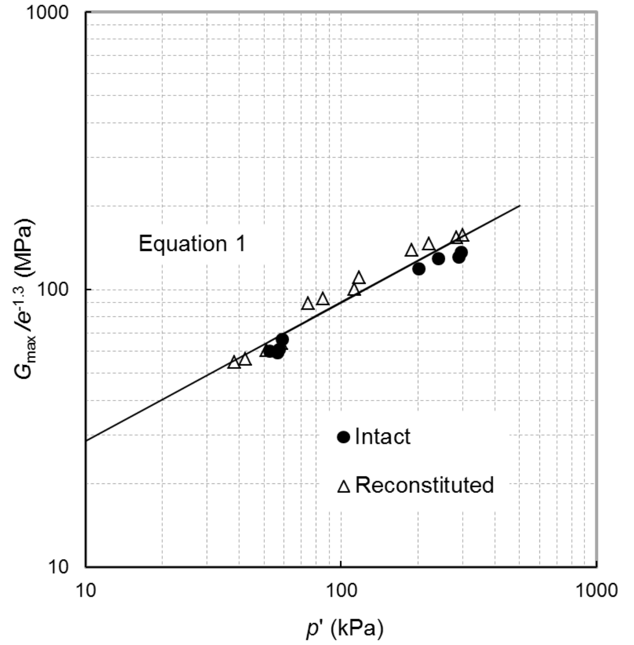

Figure 5. Relationship between maximum small strain shear modulus and mean effective pressure.

The applicability of Equation (1) to both the intact and reconstituted soils suggests that bonding within the intact soil is not significant.

3 DISCUSSION

The soil recovered in the block samples from 4.5m depth at Millstream classifies as 'silty clay to clay', according to the CPT soil behaviour type classification system with an I_c index of approximately 3. However, like many residual soils, the void space and permeability of the samples was relatively high compared with equivalent sedimentary soils (e.g. see Figure 2) with the consequence that there is a high chance of errors associated with conventional in-situ test interpretation due to effects of partial drainage.

Research on the effects of partial drainage during CPT penetration summarised by Suzuki (2014) indicate that the in-situ's material's coarse fraction of 48 % as well as its coefficient of consolidation (Figure 3) and interpreted k value (~5 $\times 10^{-8}$ m/s) imply that standard cone penetration (at 2cm/s) is essentially undrained with a likelihood of a small degree of partial drainage.

The laboratory tests have indicated that the undrained strength of in-situ material in triaxial compression (with an estimated K_0 value of ~0.8) is approximately 60 kPa. The average CPT end resistance (q_t) at the depth of the block samples was 730 kPa, leading to a backfigured cone factor, N_k, of approximately 11. This N_k value is similar to that inferred in many clays (Lunne *et al.* 1997) suggesting that partial drainage and suction effects are not significant. It is concluded, therefore, that the CPT is a reasonable means of assessing the in-situ undrained strength of the type of laterite encountered at the Millstream site.

4 CONCLUSIONS

Triaxial tests on high quality block samples from the lower pallid zone of a lateritic profile have shown a relatively low level of structure (consistent with a c' value of about 20 kPa) and a comparable friction angle to reconstituted material. The void ratio of the in-situ material, which is close to full saturation, is higher than equivalent reconstituted samples consolidated to the same effective stress level. The very small strain (elastic) shear stiffness of the intact and reconstituted soil are the same if the void ratio differences are accounted for; this observation also indicates that structural effects in the intact soil are not significant. The more open fabric in the intact soil gives rise to permeabilities and coefficients of consolidation which are an order of magnitude higher than the reconstituted material. Despite this characteristic and an estimated in-situ permeability of 5×10^{-8} m/s, standard cone penetration (at 2cm/s) in the material led to resistances consistent with undrained penetration. The CPT is shown to provide a reasonable means of assessing the in-situ undrained strength of the type of laterite encountered at the Millstream site.

5 ACKNOWLEDGMENTS

The support provided by Arup and Water Corporation, WA is gratefully acknowledged. Sponsorship of the first author by the Jiangsu Overseas Research & Training Program for University Prominent Young & Middle-aged Teachers and Presidents is also acknowledged. The project is also supported by National Natural Science Foundation of China (Grant No. 41302226). The authors acknowledge the excellent assistance provided by the laboratory staff at The University of Western Australia.

6 REFERENCES

Burland J.B. (1990) On the compressibility and shear strength of natural clays. *Geotechnique*, 1990, 40(3): 329-378.

Drummond A.F., Varagão C., Gilkes R.J. and Hart R.D. (2001). The relationships between kaolinite crystal properties and the origin of materials for a Brazilian kaolin deposit. *Clays and minerals*, 49(1),44-59.

Geological Society (1997).Tropical Residual Soils. A Geological Society Engineering report, edited by P.J. Fookes, Published by the Geologiocal Society of London, pp184.

Gordon F. R., 1984, The Laterite weathering profiles of precambrian igneous rocks at the Worsley Alumina Refinery site, South West Division, Western Australia. *Proc 4th Australia-New Zealand conference on geomechanics*, Perth, Western Australia, V1, P261-266.

Leroueil S. & Vaughan P.R. (1990). The general and congruent effect of structure in natural soils and weak rocks. *Geotechnique*, 40(3), 467-488.

Lo Presti (1994). Measurement of shear deformation of geomaterials in the laboratory. *Proc. 1st Int. Conf. Pre-failure deformation of geomaterials*, Sapporo, Japan, 2, 1067-1088

Lunne, T., Robertson, P. & Powell, J. (1997). Cone penetration testing in geotechnical practice. London: Blackie Academic & Professional.

Robertson, P. K. (2009). Interpretation of cone penetration tests - a unified approach. *Canadian Geotechnical Journal*, 46(11), 1337-1355.

Wesley L.D. (1990). Influence of structure and composition on residual soils. *J. Geotechnical Engineering Division*, ASCE, 116(4), 589-603.

Wesley L.D. (2010). Geotechnical Engineering in residual soils. Wiley, pp272.