Chapter 5

Analysis and Discussions

5.1 BACKGROUND

The purpose of this chapter is to analyse and interpret the data obtained from the experimental work. The hypothesis for this research states that **accurate simulation of the behaviour of gold tailings under laboratory conditions requires appropriate replication of the material fabric.** All tools and methods required for the analysis are discussed. Results from the preliminary testing are also included in this chapter.

5.2 PRELIMINARY TESTING RESULTS

Preliminary tests were included to classify the material and to aid in the sample preparation for the main tests. In situ conditions were determined from undisturbed triaxial samples and are summarized in *Table 5-1*.

	In situ moisture content	Void ratio	Deg. of saturation
Upper beach	12.73	0.61	0.57
Middle beach	17.32	1.09	0.43
Pond	48.69	1.42	0.94

Table 5-1. In situ conditions for the three sampled positions.

The in situ conditions are in agreement with those presented in the literature. The moisture content and void ratio of tailings in general increase down the beach of a tailings impoundment due to an increase in the amount of fine particles. The degree of saturation, however, may seem high for a dam which has been decommissioned for 10 years. The high degree of saturation is substantiated by the high rainfall in the summer season and periodic watering for dust prevention purposes. It was reported that watering occurred every three months.

5.2.1 Particle density

Particle density or specific gravity was determined using the density bottle method according to BS 1377: Part 2:1990:8.3. According to Vermeulen (2001), gold tailings have a typical specific gravity of 2.74Mg/m³, irrespective of sampling location or sample size and shape. *Gs* results from the density bottle test are given in *table 5-2.*

	Gs (Mg/m³)				
	Individual results				Average
Pond	2.739	2.745	2.754		2.75
Middle beach	2.687	2.692	2.691	2.685	2.69
Upper beach	2.709	2.728	2.723	2.727	2.72

Table 5-2. Specific gravity results from density bottle test.

The *Gs* results are consistent with those indicated by Vermeulen (2001) in that tailings samples from different positions have similar *Gs* values. This may be explained if the tailings are derived from the same parent rock.

5.2.2 Particle size distribution

Grading of the pond, MB and UB material was determined using the wet-sieving procedure followed by hydrometer sedimentation. Sedimentation makes use of Stoke's law, which assumes spherical particles, and according to

Vermeulen (2001), the method may yield slightly finer grading results due to the elongated or flattened shape of gold tailings particles.

Figure 5-1 shows that the middle and upper beach samples have very similar grading curves, despite the middle beach samples being positioned much lower down the beach. This is typical of open-ended discharge impoundments, where the slurry flows in channels down the beach, resulting in pockets of coarse material alone the beach. The pond sample was, however, significantly finer. It is interesting to note that, despite the MB and UB samples having similar grading, the MB samples have a significantly higher void ratio of 1.09 compared with a void ratio of 0.61 for UB samples. It is speculated that the in situ void ratio is less dependent upon the grading and more sensitive to the depositional environment.

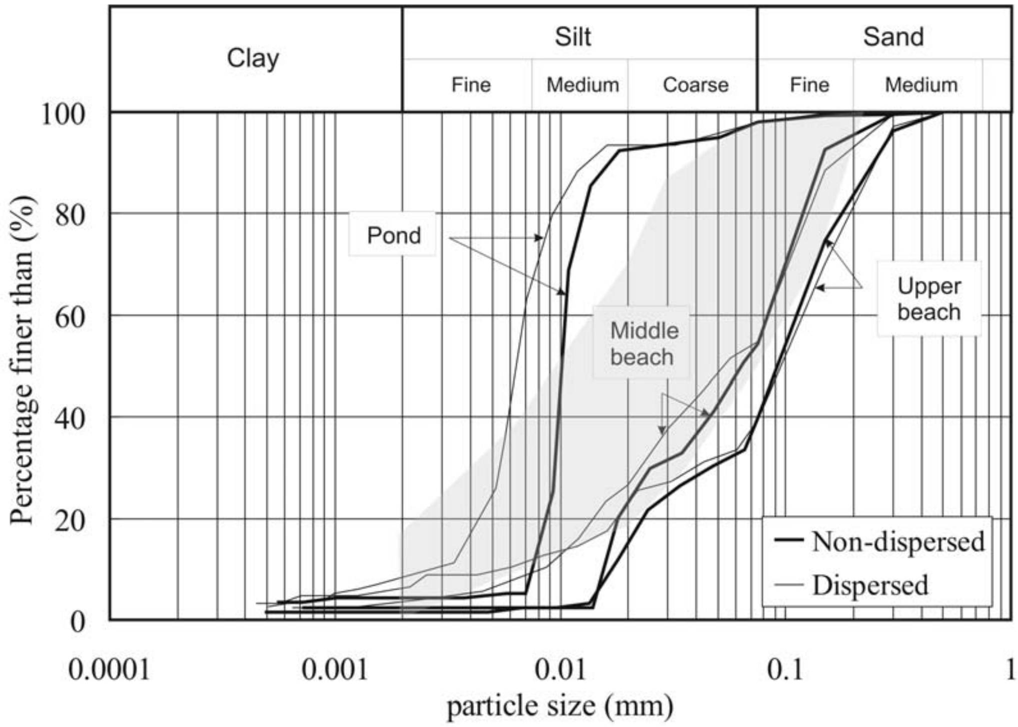

Figure 5-1. Grading of the pond, middle and upper beach material.

The grading curve can generally be described by properties such as fines content *FC* (< 0.075mm), clay content *CC* (<0.002mm), coefficient of uniformity C_u, coefficient of curvature C_Z and D_{50}. For references purposes, grading curve properties have been included in *Table 5-3*.

	Pond		Middle beach		Upper beach	
	Non-Disp.	Disp.	Non-Disp.	Disp.	Non-Disp.	Disp.
FC (%)	98	98	54	55	40	40
CC (%)	4	8	2	3	2	6
C_u	1.5	2.6	5.3	10.5	6.7	24.1
C_Z	1.0	1.6	0.5	0.8	0.9	2.2
D_{50} (mm)	0.010	0.006	0.064	0.053	0.091	0.095

Table 5-3. Grading curve properties for the three test materials.

Grading curves for the fine (pond) and the coarse (middle and upper beach) material approximately coincide with the grading envelope of typical South African gold tailings (Blight and Steffen, 1979) shown in the shaded area in *Figure 5-1*. The material tested in this thesis is thus a good representation of typical South African gold tailings.

Values of coefficient of uniformity for all materials tested are generally smaller than 36, which is the value for the 'ideal' Fuller Curve (Fuller and Thompson, 1907). The Fuller Curve describes the uniformity properties of spherical particles for the densest possible state of packing. A sample of spherical particles with a C_u of less than 36 has an abundance of fines that hold the coarse particles apart. The sample will also be less dense than the maximum density. If C_u is greater than 36, then the voids between the coarse particles are not completely filled with fines. In the case of the gold tailings tested, C_u values are significantly smaller than 36, indicating that the coarse particles are not all in contact with each other, and may be held apart by the finer particles. This is also confirmed by the SEMs presented in the previous chapter. It is also consistent with the results that C_u decreases down the beach as the material becomes finer. The Fuller Curve is, however, based on the arrangement of spherical particles, while tailings fines are predominantly platy. It is unclear how particle shape will influence the Fuller Curve, but it is suspected that an increase in the proportion of platy particles may result in increased difficulty for samples to reach the densest state.

A well-graded soil has a coefficient of curvature C_Z between 1 and 3 (Craig, 1997). C_Z values for the gold tailings material used in this thesis range between 0.5 to 2.2 and can therefore be considered well-graded.

5.2.3 Atterberg limits

For the purpose of the project, only the liquid and plastic limits were determined. The plastic limit for the MB and UB materials could not be determined as the material crumbled before a three millimetre diameter 'worm' could be obtained. Liquid and plastic limits for the three test materials are summarized in *Table 5-4.*

	Pond	Middle beach	Upper beach
Liquid limit (LL)	51	30	25
Plastic limit (PL)	39	NA	NA
Plasticity index (PI)	12	NA	NA

Table 5-4. Summary of Liquid and plastic limits for the three test materials.

The Atterberg limits shown in *Table 5-4* correspond to the grading observed in *Figure 5-1*. The middle and upper beach samples have similar grading and therefore similar liquid limits. The coarseness of these two material types prevents the formation of a 3mm 'worm' and thus any estimates of plasticity. The pond material, however, has a significantly higher *LL* and some plasticity.

According to the Unified Soil Classification System, the pond material can be classified as ML-MH while the MB and UB materials are classified as SM.

5.2.4 Limiting density

Limiting density tests were done at various moisture content to estimate the range of void ratios that can be expected for the material. It was also assumed that the closer the target (in situ) void ratio is to the minimum void ratio, the less the volumetric collapse would be during flushing. In this way, the damage to the initial fabric of the sample could be minimized. Results for the maximum and minimum density tests are shown in *Figure 5-2.* It should be noted that the limiting densities determined in this thesis can only be used as a guideline, and not for reference purposes, as standard BS and ASTM methods specify the use of dry material while the minimum and maximum densities of gold tailings were determined at various moisture content (as described in section 3.4.4). Furthermore, the maximum density of dry samples could not be determined as material was blown out between the weight and the mould by pore air pressure during compaction. The moisture content used for moist tamped samples is also plotted against the target (in situ) void ratios for each material. These are shown in full circles.

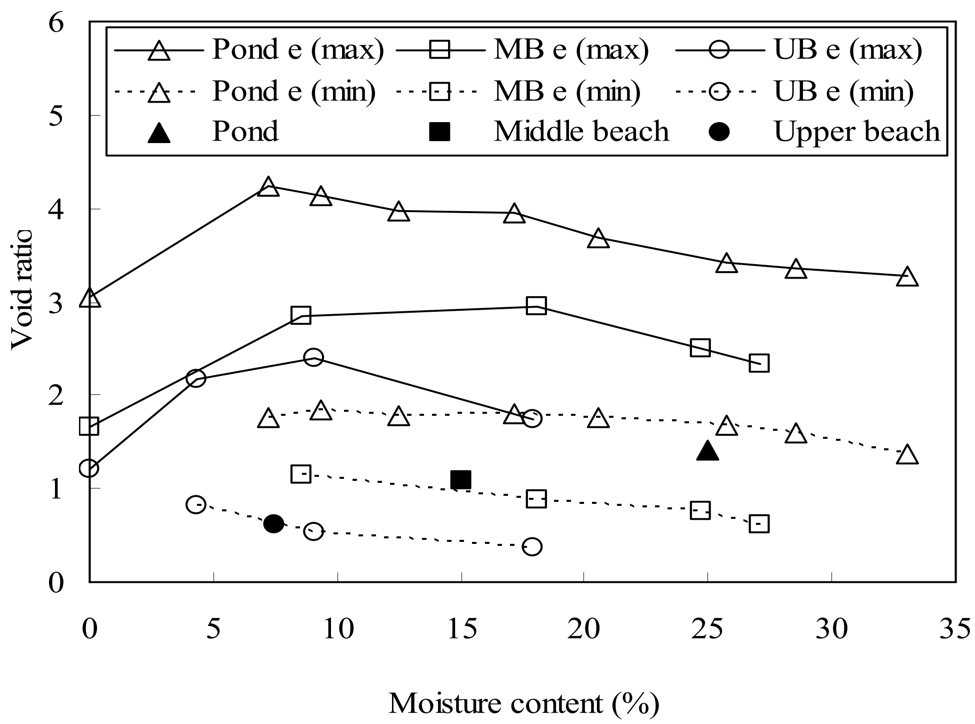

Figure 5-2. Limiting density tests for the three materials.

The void ratio of a soil, relative to the maximum and minimum obtainable void ratios can be expressed as the density index, I_D:

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}} \qquad \textit{Equation 5-1}$$

where e_{max} and e_{min} are the maximum and minimum void ratios respectively and e is the in situ or target void ratio. The density indices at preparation moisture content are 1.17, 0.95 and 1.03 for pond, middle and upper beach material respectively. It should be noted that these values cannot be compared with those derived from standard maximum and minimum density tests as the standard tests only involve the use of dry material.

The results show that the maximum void ratio increases drastically when the moisture content is increased by a few percent from the dry condition. A peak is reached after which further addition of moisture results in a decrease in the maximum void ratio. It is interesting to note that the peaks for all three material types were found in a relatively narrow range of 7% to 9% moisture, despite the differences in the material type.

The minimum void ratio for all three material types appears flat with a gradual decrease with increasing moisture content. This implies that the in situ void ratio would lie close to or below the minimum void ratio line for almost the entire range of moisture content tested. It was initially decided that the choice of preparation moisture content for moist tamped samples would be based on the relative position of the in situ void ratio and the minimum void ratio at specific moisture content. As changes in minimum void ratio were small with increasing moisture content, it was decided that the preparation moisture content for moist tamped samples would be based on the appearance of the sample (to prevent the formation of visible lumps) and the strength (during handling) as described in section 3.6.2.

The target (in situ) void ratio for the pond material is below the minimum void ratio obtained for the same material at the preparation moisture content (25%

for pond material). This implies that significant preparation energy, such as using the hydraulic jack, is required to construct the moist tamped samples at the target void ratio. The force required decreased as the target void ratio increased in comparison with the minimum void ratio, as seen with the middle and upper beach samples at the preparation moisture content (15 and 7.5% for middle and upper beach samples respectively).

It is interesting to note that the target void ratios for all three material types were close to the minimum void ratio curve shown in *Figure 5-2*. It appears that the in situ depositional environment has similar compaction effort as the vibratory table method described in 3.4.4 for the determination of minimum void ratio. As all samples were surface samples with little or no surcharge load, it is reasonable to assume that this reduction in void ratio is a direct result of suction pressures between the particles during desiccation. As estimated by Westraad (2004), suction pressures on the beach may be of the order of 150kPa.

5.2.5 Sedimentation tests

Sedimentation tests were done using dispersant and flocculent to increase the void ratio obtainable for slurry samples. During the testing programme, it was discovered that the MB and UB slurry samples cannot be constructed at the target void ratio using tap water alone. UB slurry samples needed to be at a void ratio of 0.6, but could only be prepared at a maximum void ratio of 0.5. The target void ratio of MB material was approximately 1.1, but slurry samples could only be prepared at a maximum void ratio of 0.8. Slurry samples for pond material could be prepared at the target void ratio of 1.4 with relative ease. As slurry samples for the middle and upper beach material could not be prepared at the target (in situ) void ratio, dispersant and flocculent were used to increase the initial void ratio of the slurry samples.

The effect of various concentrations of dispersant and flocculent on void ratio was thus investigated in sedimentation tests. The effect of dispersant or flocculent content can be seen in *Appendix E figures E-1* to *E-6* which plots void ratio against time. Void ratio was calculated from the height of the soil column in the cylinder. The final void ratios (recorded after 5000 minutes) of the three materials are shown in *Figure 5-3*.

Results of the sedimentation tests showed that the addition of dispersant had a varied effect on the final void ratios obtained. Pond material showed an initial decrease followed by an increase in final void ratio with the increase in dispersant content. For middle and upper beach material, an initial increase is observed followed by a decrease in the final void ratio with increasing dispersant content. No consistent explanation was reached for the observed behaviour.

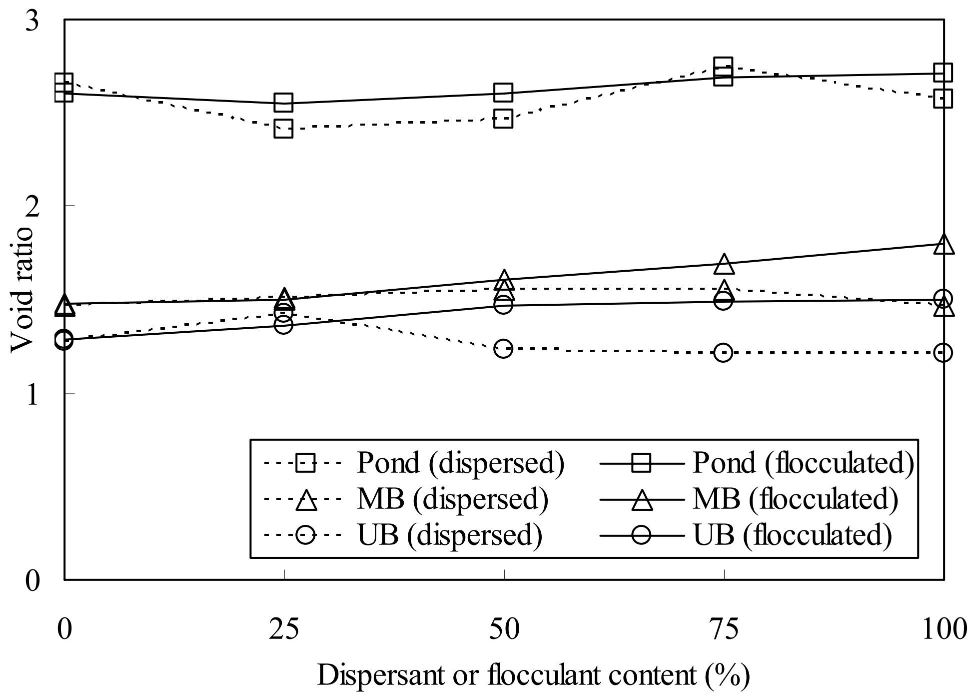

Figure 5-3. Final void ratio of the sedimentation test.

Although no conclusion could be made with regard to the final void ratio, something interesting can be observed from the plots of void ratio with time attached in *Appendix E*. For both pond and middle beach dispersed, a gradual reduction in void ratio with time is observed. For upper beach material, however, there seems to be a limiting dispersant concentration before which the samples become effectively dispersed. The void ratio of a well dispersed sample appears to increase with time due to the settlement of the platy fines.

The void ratio of samples not fully dispersed decrease with time, similar to that observed in pond and MB samples.

The addition of flocculent, however, results in an increase in the final void ratio for all materials. There also seems to be a limiting flocculent content where after further addition of flocculent will have no effect on the final void ratio. This limiting flocculent content seems to vary with material type, but appears to be in the range of 75-100%. The addition of flocculent also has the effect of increased rate of settlement for all materials.

The target void ratio for middle beach slurry samples was obtained with the addition of flocculent and target void ratio for upper beach slurry samples was achieved with the addition of dispersant.

5.3 VOLUME CHANGE BEHAVIOUR

Volume change during undrained triaxial testing includes volume changes during flushing, consolidation and creep. All volume changes were calculated in terms of axial deformation measured from the local LVDT outputs under isotropic conditions from *Equation 5-2*.

$$\varepsilon_v = (1- \varepsilon_a)^3 \qquad \textit{Equation 5-2}$$

where ε_v and ε_a are volumetric and axial strains respectively.

5.3.1 Volume change during flushing

Collapse during flushing occurs when water is introduced into a soil with a metastable fabric. On the other hand, swell is initiated by the reduction of effective stress by unloading and/or the adding of water. Volume changes were monitored for all test samples and are summarized in *Tables E-1, E-2* and *E-3* in *Appendix E* for pond, middle beach and upper beach materials respectively. The volume changes in the samples cannot be compared directly, as the initial target void ratio varied to accommodate the difference in consolidation behaviour. Some observations can, however, be made with regards to the volume change behaviour during flushing.

For all three material types, the undisturbed consolidation and shear samples showed volumetric collapse in the order of 2% during flushing.

Moist tamped samples may be constructed to void ratios well above the standard maximum value due to capillary effects between the grains (Zlatovic and Ishihara, 1997). Yi (1991) demonstrated that silty soils prepared at a loose state may display a considerable decrease in volume due to loss of the capillary forces upon wetting. This is observed to some extent in the volumetric behaviour of gold tailings during flushing. The volume change can be related directly to the effort of sample preparation for moist tamped samples. The amount of collapse is indirectly proportional to the effort required for sample preparation. The samples may even swell, as seen in the pond samples, when high preparation effort is required. Preparation of moist tamped samples involves the compaction of aggregates with some strength. For the pond material, a significant amount of effort was required, resulting in an increasingly stable fabric with significant stresses being locked in the sample. The stress is released with the addition of water during flushing, resulting in the volumetric swell observed. Middle and upper beach samples, however, require far less preparation effort. The result is a metastable fabric which collapses during flushing.

No significant volume change was expected for slurry samples, as the samples were prepared in water. There was, however, a significant decrease in volume observed in the middle beach slurry samples prepared at very high void ratios with the addition of flocculent. The addition of flocculent creates an aggregated fabric at high void ratio which cannot sustain the initial cell pressure. Volume changes in slurry samples can thus not be classified as collapse or swell, as the samples were prepared to near full saturation.

5.3.2 Consolidation behaviour

Consolidation was analysed for the shear-200 samples as well as the consolidation samples. It should be noted that as the consolidation behaviours differ, the initial void ratio before consolidation for the shear-200 samples was not constant for each material type. They, however, arrived at the same void ratio after the consolidation and creep, and could thus be sheared at the same state.

Consolidation characteristics for the three materials could be estimated from the shear-200 samples, which underwent standard isotropic triaxial consolidation. The coefficient of consolidation, C_v, can be determined using Taylor's well-known root time method:

$$C_v = \frac{0.848d^2}{t_{90}} \qquad \textit{Equation 5-3}$$

where d is the drainage path length and t_{90} is the time to 90% consolidation. *Equation 5-3* is based on Terzaghi's theory of one-dimensional consolidation, and application to isotropic triaxial consolidation is questionable. The issue is complex and is not discussed in this thesis. Readers can refer to the work of Biot (1941) for generalization of Terzaghi's one-dimensional consolidation theory to three dimensions. The coefficients of consolidation values for all shear-200 samples are summarized in *Table 5-5.* C_v values are expressed in m^2/year.

	Coefficient of consolidation, C_v (m^2/year)		
	Pond	Middle beach	Upper beach
Undisturbed	96	697	2862
Slurry	228	2222	9402
Moist tamped	180	1367	3339

Table 5-5. Coefficient of consolidation for shear-200 samples.

Upon first inspection of the results in *Table 5-5*, it appears that the C_v values are high. Although high C_v values can be expected for coarse materials such as the middle and upper beach, the validity of Darcy's law in Terzaghi's one-dimensional consolidation theory may be questionable. At high hydraulic gradients, flow may be turbulent if pore sizes and flow rates are sufficiently great, and Darcy's law (which assumes laminar flow) may not apply. Grading results in *Figure 5-1* and *Table 5-3* indicate that the test material is generally silt or clay-sized, with few sand-sized particles. The SEM images also showed that pore size may be a maximum of 100µm. The combination of small particle and pore size for gold tailings suggests that turbulent flow is not present and Darcy's law is still valid.

A practical aspect with regard to triaxial consolidation of highly permeable materials with high C_v values is that as consolidation occurs quickly, the accuracy of volume gauge reading is reduced. Estimation of C_v values from consolidation data may also be difficult and more subjective.

The high hydraulic gradient may, however, affect the fabric of the samples observed from the SEM images. As discussed in section 4.3.2, platy particles or flocks which exist in the voids between the rotund particles (*Figure 4-8*) are passive and non-load-bearing. Under high hydraulic gradients, these passive particles may migrate to create a non-uniform fabric. Migration of fine particles within a soil matrix may also block or unblock interconnected voids and affect the result of consolidation (Mitchell and Soga, 2005).

Table 5-5 indicates that the C_v values were in the same range as those summarized in *Table 2-7* for gold tailings. It can also be seen that C_v increases up the beach as would be expected as the material becomes coarser.

The coefficient of consolidation is of the same order of magnitude for the pond samples. This is validated by the similar fabric observed for pond material. As no side drains were used, the only drainage path in all the triaxial samples was vertical, and the horizontal particle orientation observed in the P-I-200 sample may have caused a slightly lower C_v value.

The aggregated fabric observed may have contributed to the high C_v values in moist tamped samples of middle and upper beach material. Aggregation results in a high connectivity inter-aggregate pore system to facilitate drainage. High permeability coupled with the unstable R-P-R particle contact resulted in the observed high coefficient of consolidation.

Cell pressure for the consolidation tests was ramped at a rate of 10s/kPa. Consolidation was limited to 1000 or 900kPa effective stress (depending on back pressure used) due to the cell capacity of 1300kPa. Reconstituted samples were prepared to the same void ratio as the undisturbed sample after saturation. In this way consolidation samples could be consolidated at the same initial void ratio and the effect of fabric on consolidation could then be investigated. Void ratios for all consolidation samples before consolidation are summarized in *Table 5-6.* The difference is defined, in percentage, as the difference between the reconstituted (moist tamped and slurry) samples and the undisturbed samples. As mentioned in section 3.2, consolidation samples were prepared to a target void ratio difference, Δ, of two percent.

	Pond		Middle beach		Upper beach	
	Void ratio	Δ (%)	Void ratio	Δ (%)	Void ratio	Δ (%)
Undisturbed	1.374		1.097		0.628	
Slurry	1.377	0.4	1.105	0.7	0.636	1.2
Moist tamped	1.376	0.3	1.076	1.9	0.633	0.7

Table 5-6. Void ratios before consolidation of all consolidation samples.

Consolidation results for the three materials with different fabrics are shown in *Figure 5-4.* A distinct change of slope can be observed in the compression curve for the undisturbed pond sample, clearly defining the pre-consolidation pressure. The compression curves of undisturbed middle and upper beach samples show gentle curvature over the test pressure range. This is characteristic of sand behaviour where the transition from elastic to plastic deformation is gradual, and relatively high confining stresses are required to locate the normal consolidation line.

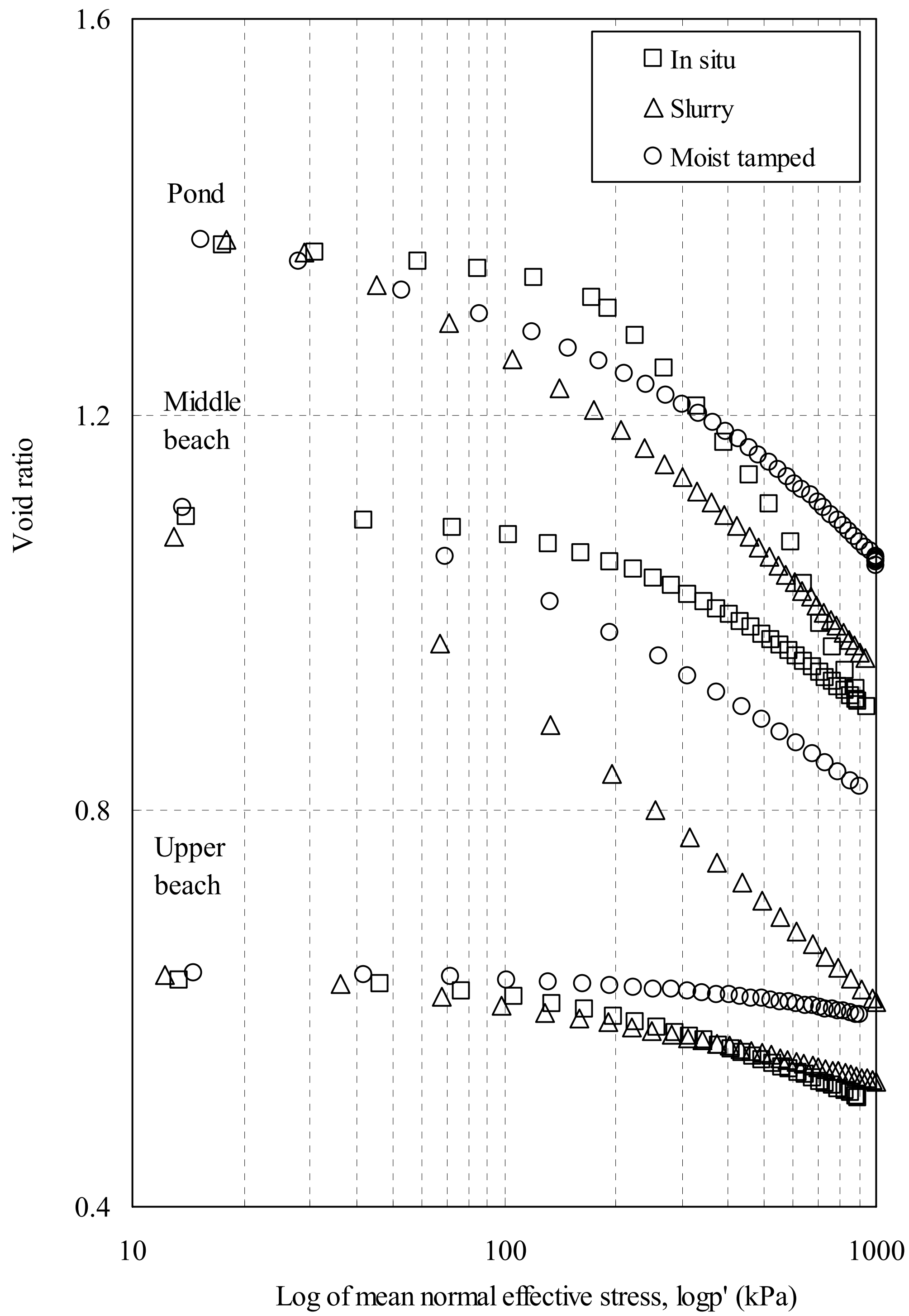

Figure 5-4. Consolidation results for the three materials.

Two parameters were used to quantify and compare the consolidation behaviour of gold tailings: the 'initial' slope and the compression index, *Cc*. The 'initial' slope was estimated from the first 5 data points on the consolidation curve and is an indication of the initial behaviour of the samples. *Cc* was estimated from last 5 data points of the consolidation curve, and is the best estimation of the linear slope of the normal consolidation line from the available data. The 'initial' slopes and the *Cc* values of the test samples are summarized in *Table 5-7*.

	Pond		Middle beach		Upper beach	
	Initial	*Cc*	Initial	*Cc*	Initial	*Cc*
Undisturbed	0.040	0.656	0.028	0.279	0.023	0.157
Slurry	0.155	0.429	0.187	0.268	0.036	0.070
Moist tamped	0.104	0.370	0.095	0.342	0.010	0.099

Table 5-7. 'Initial' slope and Cc of the consolidation samples.

A consistent trend can be observed in the 'initial' slope of the gold tailings samples. Slurry samples showed the highest 'initial' slope while the undisturbed samples generally had the lowest 'initial' slope, with the exception of the UB-I-400. This implies that both the slurry and moist tamped samples have lower initial bulk stiffness in comparison with the undisturbed samples. Bulk and shear stiffness is discussed in section 5.4.

Isotropic compression of a soil can also be expressed in terms of critical state soil mechanics. The normal compression line can be defined in terms of critical state parameters λ and N in the *Equation 5-4:*

$$v = N - \lambda \ln p' \qquad \textit{Equation 5-4}$$

where λ defines the slope of the normal consolidation line NCL in the compression (v-lnp') plane and N defines the NCL at p' = 1kPa. Although from *Figure 5-4* it appears that the normal consolidation line was reached, values of λ were nevertheless estimated from the consolidation data available. Values of λ (obtained from the last 5 data points) are summarized in *Table 5-8.* The difference in C_c and λ values at the end of the test is in agreement with the findings from the SEM images that the fabric was not destroyed at large isotropic stresses and strains.

	Pond	Middle beach	Upper beach
Undisturbed	0.281	0.121	0.068
Moist tamped	0.154	0.116	0.031
Slurry	0.186	0.144	0.043

Table 5-8. Critical state parameters λ for gold tailings.

Vermeulen (2001) reported λ values for fine gold tailings to be in the range of 0.1 to 0.17 and for coarse and whole tailings to be in the range of 0.04 to 0.07. The λ values for pond and upper beach samples are consistent with those presented by Vermeulen (2001) for fine and coarse gold tailings.

The consolidation results indicate that the pond samples have the highest compression and the upper beach exhibits the lowest C_c and λ values. It can be expected that the pond samples, having a behaviour dominated by platy particles which can bend and slide, will yield the higher C_c and λ values than middle or upper beach samples. The behaviour of middle and upper beach samples are dominated by the rotund particle and thus yield a lower slope during isotropic consolidation.

Values from *Table 5-7* and *Table 5-8* show that on average, reconstituted samples exhibit 40% lower C_c and λ values than undisturbed counterparts, with the exception of the higher slope for the middle beach slurry sample (which may be the result of the flocculated fabric). Although the block samples appeared uniform, undisturbed samples may nevertheless contain some fissures or cracks which would significantly lower the bulk stiffness of these samples and result in a higher slope during consolidation. Moist tamped samples show slightly lower C_c and λ values than the slurry deposited counterparts. This may be as a result of the isotropic compression forces being normal to the orientation of the platy particles as suggested in section 4.3.2. C_c and λ values of the moist tamped samples were on average 25% lower than those of the slurry samples.

5.3.3 Secondary compression

The terms creep and secondary compression are often used interchangeably to describe the time-dependent strain at constant stress that develops in soils. According to Mitchell and Soga (2005), however, creep is associated with the time-dependent shear and/or volumetric strains while secondary compression refers to a special case of constrained creep which follows primary consolidation. For this thesis, both terms are used to describe the same time-dependent volumetric strains that occur after primary consolidation. Secondary compression in shear samples was monitored to ensure that the rate was small when compared with the subsequent shear rate. Jardine (1995) suggested a creep rate to shear strain rate ratio of less than 1% before shear could begin, to avoid measurement errors during shear. All shear samples were rested for approximately 24 hours before shearing. Volume changes were monitored for 10 minutes prior to shear to ensure that the creep rate to shear strain rate ratio was within the recommended one percent. Creep rate for all samples, except MB-S-400, were well within the recommended range after 24 hours. Creep rate *CR* for all tests are summarized in *Table 5-9.*

SR refers to the shear rate at the beginning of shear, calculated from the machine rate of 0.103mm/min and the height of the sample at the beginning of shear. It should be noted that shear-200 samples underwent standard consolidation procedures while the shear-400 samples were ramped at 10s/kPa. Consolidation and creep data for the two should thus not be compared directly.

200 samples	Creep rate	*CR/SR*	400 samples	Creep rate	*CR/SR*
	(%/min)	(%)		(%/min)	(%)
P-I-200	0.0005	0.46	P-I-400	0.0004	0.34
P-MT-200	0.0004	0.35	P-MT-400	0.0001	0.11
P-S-200	0.0003	0.29	P-S-400	0.0004	0.41
MB-I-200	0.0005	0.47	MB-I-400	0.0010	0.97
MB-MT-200	0.0006	0.59	MB-MT-400	0.0008	0.78
MB-S-200	0.0010	0.91	MB-S-400	0.0016	1.43
UB-I-200	0.0004	0.42	UB-I-400	0.0002	0.16
UB-MT-200	0.0000	0.03	UB-MT-400	0.0001	0.08
UB-S-200	0.0002	0.23	UB-S-400	0.0003	0.31

Table 5-9. Summary of creep rates before shear for shear-200 and shear-400 samples.

The creep of soils can also be expressed in terms of the secondary compression index, C_α, defined as the slope of the creep line on the *e*-log*t'* graph. Secondary compression index for the shear-200 and shear-400 samples are summarized in *Table 5-10.*

Pond		Middle beach		Upper beach	
I-200	0.0068	I-200	0.0079	I-200	0.0033
MT-200	0.0044	MT-200	0.0056	MT-200	0.0003
S-200	0.0040	S-200	0.0125	S-200	0.0024
I-400	0.0075	I-400	0.0094	I-400	0.0010
MT-400	0.0050	MT-400	0.0054	MT-400	0.0005
S-400	0.0012	S-400	0.0147	S-400	0.0023

Table 5-10. Secondary compression indexes for shear-200 and shear-400 samples.

Graphs for the creep data of individual samples are attached in *Appendix E.* It should be noted that the consolidation process for the shear-200 and shear-400 samples were different. Shear-200 samples were consolidated using standard consolidation while the shear-400 samples were ramp consolidated. The effect of the consolidation process on the consequential C_α values is unclear.

To the knowledge of the author, very little work on creep of gold tailings is available in the literature. Values for the coefficient of secondary compression have, however, been reported by various authors in the literature for natural soils, and may be used as reference. Leroueil and Marques (1996) reported values of between 0.029 and 0.059 for inorganic clays. As seen in *Table 5-10*, values of C_α for gold tailings range between 0.0003 and 0.0147 with an average of 0.005. This is significantly lower than values presented for natural soils.

Furthermore, the ratio of C_α/C_c has also been reported in the literature. Mesri and Godlewski (1977) reported C_α/C_c in the range of 0.025 to 0.075 for inorganic clays and silts, and values between 0.035 and 0.1 for organic clays and silts. Mesri et al. (1990) suggested that for all practical purposes, the ratio C_α/C_c for sands can be considered constant and equal to 0.02. The values for the ratio C_α/C_c for gold tailings ranged between 0.004 and 0.037 with an average value of 0.018. This is consistent with the C_α/C_c value of 0.02 suggested by Mesri et al. (1990). The values are also on the low side of the values reported by Mesri and Godlewski (1977) for inorganic clays and silts. This indicates that creep behaviour of tailings is similar to that of sands, showing low C_α values. The platy particles seem more stable than natural clays in terms of time-dependent straining due to particle rearrangement.

From the results, it seems that the coefficient of secondary compression is dependent upon the particle shape as well as on the void ratio in relation to the minimum void ratio of the sample. Although middle and upper beach material have similar particle characteristics, the C_α for middle beach samples is higher in comparison with that of the upper beach samples, probably due to the fact that the upper beach samples are at a lower relative density than the middle beach samples. Comparison of the pond and upper beach samples, which are at similar relative densities, show that a higher C_α is observed in the pond material, probably due to the slippage and realignment of platy particles. It also appears that creep for moist tamped samples may be lower than for undisturbed and slurry samples, but this is not always the case. Assuming a consistent C_α/C_c ratio, moist tamped samples which show lower C_c values should in theory also yield lower C_α values.

5.4 STIFFNESS

The stiffness of the samples was determined from the two local LVDTs. Both the bulk stiffness and the shear stiffness were investigated. The bulk stiffness was determined from the consolidation results of the consolidation samples (samples isotropically consolidated to 1000kPa), while the shear stiffness was determined from the shear samples. Both the bulk and shear modulus were determined as the secant value as demonstrated in *Figure 5-5*. Bulk modulus K was determined as the secant value shown by the slope of the secant line on the p'- ε_v plot as shown in *Figure 5-5a* at various mean normal effective stress increments. The secant Young's modulus E was defined as the slope of the secant line on the $q' - \varepsilon_a$ graph as shown in *Figure 5-5b*.

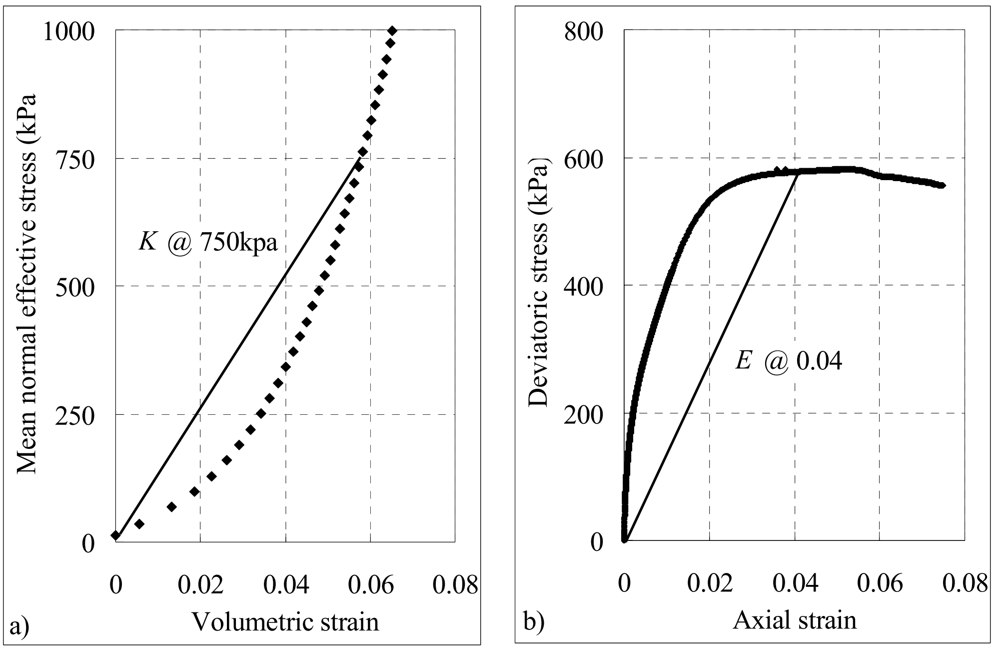

Figure 5-5. Graphic illustration of secant K and E interpretations.

5.4.1 Bulk modulus

As mentioned in section 2.5.6, the bulk stiffness is a measure of the soil's ability to resist volumetric strains under an applied pressure. The bulk modulus determined from *Equation 2-3* for all consolidation samples at effective confining stresses of 250, 500 and 750kPa are shown in *Table 5-11*.

	Bulk modulus at various confining stresses (MPa)			
p_o' (kPa)		250	500	750
Pond	Undisturbed	5.08	4.51	4.33
	Slurry	2.55	3.64	4.56
	Moist tamped	3.73	5.21	6.33
	Average	3.79	4.45	5.07
Middle beach	Undisturbed	8.02	8.49	9.34
	Slurry	1.81	2.74	3.59
	Moist tamped	3.41	4.75	5.97
	Average	4.41	5.33	6.30
Upper beach	Undisturbed	8.44	9.84	11.38
	Slurry	6.96	10.10	12.73
	Moist tamped	24.34	29.70	33.01
	Average	13.25	16.55	19.04

Table 5-11.Bulk stiffness of gold tailings at various confining stresses.

Significant variations in bulk stiffness behaviour can be observed at low confining stresses of up to 100kPa, where after the bulk modulus increases with increasing confining stress. Initial variations in behaviour may be a result of several factors such as over-consolidation ratio and specimen uniformity. Cracks and fissures may also result in a constant bulk modulus as demonstrated by the undisturbed pond sample. In homogeneous samples pores close up as consolidation progresses resulting in a denser configuration and an increase in the bulk stiffness. In cracked or fissured samples, however, consolidation results in the closing of cracks and fissures and constant bulk stiffness are observed. Once the cracks and fissures are closed the homogeneous sample then displays an increase in bulk stiffness upon further consolidation.

Comparison of the bulk modulus values shown in *Table 5-11* and those obtained by Vermeulen (2001) indicate that the pond samples which have similar initial void ratios (of around 1.4) have similar bulk modulus. Middle beach samples also have similar initial void ratios (of around 0.8) and bulk modulus when compared with Vermeulen's results for coarse tailings (2001). The upper beach samples are however significantly stiffer than those obtained by Vermeulen for coarse tailings. This may be explained by the upper beach samples having a lower initial void ratio (around 0.5) than a void ratio of 0.8 for Vermeulen's coarse tailings.

With regard to sample preparation/fabric effects on bulk modulus, it appears that pond undisturbed, moist tamped and slurry samples exhibit similar values of K for the stress range investigated. This further validates the similar fabric observed from the SEM images of the pond samples. The bulk stiffness of middle beach samples was, however, significantly affected by the fabric. K values for the slurry samples were in the range of 25% to 30% of the undisturbed counterparts. It is speculated that the flocculated fabric may have significantly lowered the stiffness of the slurry samples. Bulk stiffness of the moist tamped samples was approximately half that of the undisturbed counterpart. The bulk stiffness of the undisturbed and slurry upper beach samples was similar, but significantly lower than that of the moist tamped samples. It is unclear why the moist tamped upper beach samples showed significantly higher bulk stiffness than the undisturbed and slurry samples, but it is speculated that the moist tamped sample was constructed at a lower void ratio or that larger rotund particles were included in the sample.

5.4.2 Young's modulus

According to Heymann (1998), stiffness may be plotted against the logarithm of strain to give equal prominence to stiffness at all strain levels. Consider a typical force time plot for the sample P-I-400 shown in *Figure 5-6.* After the creep rate had been checked, the ram was lowered to a height slightly above the sample, and the machine was started. Due to the limited flow rate of the digital pressure controller, it was important to lower the ram slowly enough to prevent an increase in cell pressure. Data logging was initiated at time zero before the ram touched the top cap. The start of shear was identified by observing lift-off of the measurement as shown in the *Figure 5-6*.

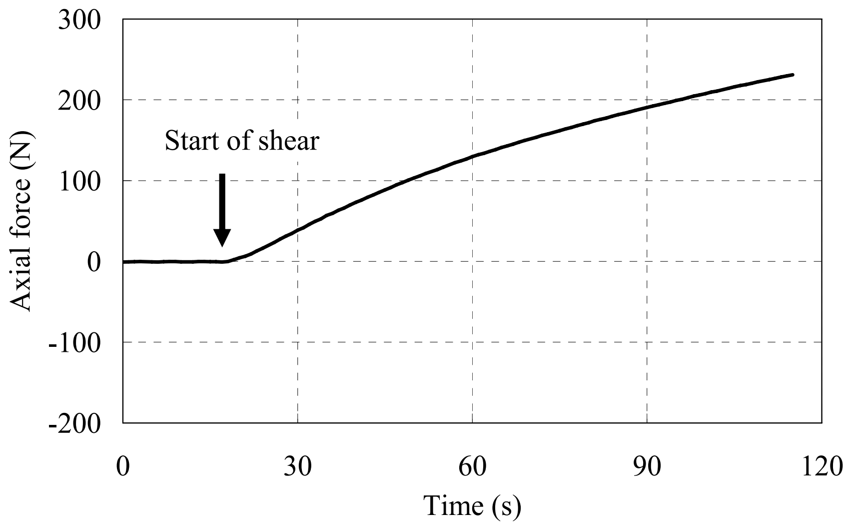

Figure 5-6. Force time plot used to identify the start of shear.

The secant Young's modulus is defined in *Figure 5-5b* and is determined using *Equation 5-5*:

$$E = \frac{q' - q'_0}{\varepsilon_a - \varepsilon_{a0}}$$ *Equation 5-5*

where E is the stiffness, q' the deviatoric stress and ε_a the axial strain. q'_0 and ε_{a0} are points defining the origin.

Small strain stiffness

The small strain stiffness is a measure of soil stiffness in the elastic range. Stiffness of soils in the elastic range is constant and using high accuracy interferometers for local strain measurements, Heymann (1998) was able to demonstrate that the linear plateau of stiffness for soils and weak rocks is in the range of 0.002% strain. The results show that there is significant below the 0.001% range, indicating that the LVDTs used may not be capable of accurately identifying the linear plateau. The small strain stiffness for gold tailings can, nevertheless, be estimated. It was decided that the small strain stiffness would be taken as the average value of the stiffness values in the range of 0.0001% to 0.001% strain. The scatter of values below 0.0001% strain may be extremely large (positive or negative) and may influence the average value considerably. Values above 0.001% strain may be out of the linear plateau and are thus not included. An example for P-U-200 is shown in *Figure 5-7*.

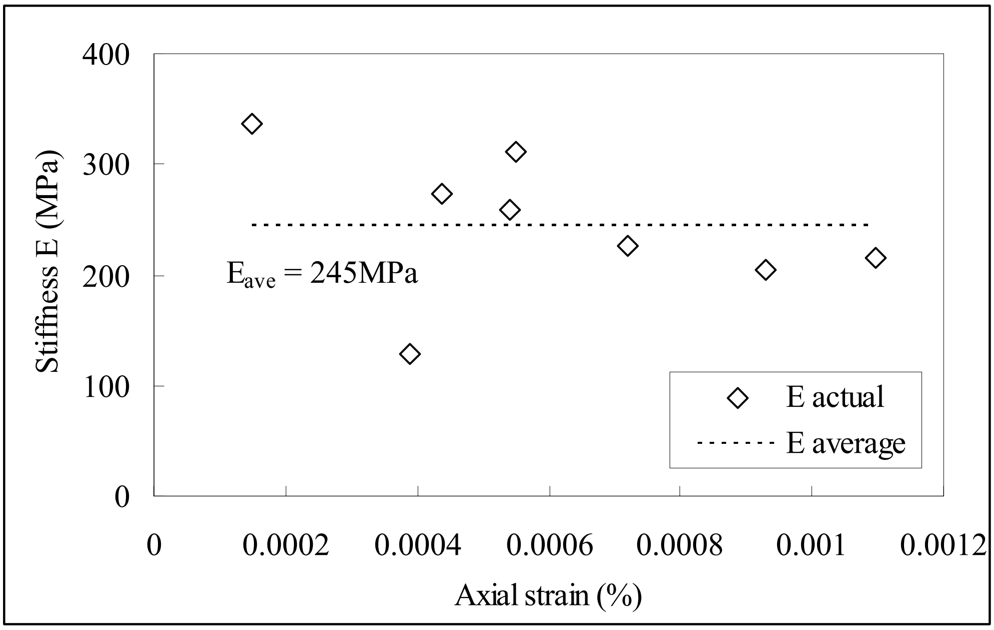

Figure 5-7. Example of small strain stiffness derivation (P-U-200)

The small strain stiffness values for the samples tested are summarized in *Table 5-12*. Theron, Heymann and Clayton (2004) presented small strain shear stiffness data of slurry prepared gold tailings based on bender element measurements. Small strain stiffness of gold tailings is in the order of 100MPa and 175MPa for confining stresses of 200 and 400kPa respectively. G_{max} of moist tamped gold tailings samples determined from shear wave velocities obtained using bender elements (Chang, 2004) were in the order of 60MPa and 100MPa for samples consolidated to 200kPa and 400kPa respectively.

Small strain Young's modulus, *E* (MPa)						
	Pond		Middle beach		Upper beach	
p_o' (kPa)	200	400	200	400	200	400
Undisturbed	245	336	237	451	174	470
Moist tamped	226	332	135	280	234	374
Slurry	301	383	251	460	180	447

Table 5-12. Small strain Young's modulus values for gold tailings.

Under isotropic conditions, the Young's modulus E can be related to the shear modulus G via *Equation 5-6:*

$$G = \frac{E}{2(1+\nu)} \qquad \textit{Equation 5-6}$$

where ν is the Poisson's ratio. Under undrained conditions, ν equals 0.5 and the equation can be simplified to:

$$E_u=3G. \qquad \textit{Equation 5-7}$$

The undrained Young's modulus from Theron, Heymann and Clayton's results for gold tailings can then be approximated to 300MPa and 525MPa for confining stresses of 200kPa and 400kPa respectively. Results of Chang's bender element tests yield a Young's modulus of 180MPa and 300MPa for confining stresses of 200kPa and 400kPa respectively. This is consistent with the results of this section, as Theron, Heymann and Clayton (2004) tested slurry samples and the samples of Chang (2004) were moist tamped.

It appears that the small strain stiffness results shown in *Table 5-12* are in between those obtained by Theron, Heymann and Clayton (2004) and Chang (2004). The difference between the results of Theron, Heymann and Clayton (2004) and Chang (2004) may partly be a result of the preparation method and thus the fabric.

From *Table 5-12*, it can be seen that fabric may have a significant effect on the small strain stiffness of gold tailings. All but one slurry sample show higher small strain stiffness when compared with the undisturbed counterparts. With the exception of UB-MT-200, all moist tamped samples show lower small strain stiffness than the undisturbed samples. The difference may be minimal, as in the case of the pond samples, or significant, as in the case of middle beach samples. Comparison of small strain stiffness of undisturbed and reconstituted samples is summarized in *Table 5-13*. Differences are shown as percentages and negative values indicate a value lower than the undisturbed stiffness.

	Pond		Middle beach		Upper beach	
p_o' (kPa)	200	400	200	400	200	400
Moist tamped	-7.8%	-1.2%	-43%	-37.9%	34.5%	-20.4%
Slurry	24.8%	14.2%	10.4%	3.2%	2.6%	-6.1%

Table 5-13. Differences in E values between undisturbed and reconstituted gold tailings.

The small strain stiffness of soils is significantly affected by the homogeneity of the sample. Triaxial tests conducted on reconstituted Laval and Sherbrooke samples indicate that stiffness increases as the material becomes more homogeneous (Clayton et al., 1992). This is consistent with the small strain stiffness results observed for gold tailings. Slurry samples represent samples with the highest degree of homogeneity and thus exhibited the highest small strain stiffness values. Undisturbed samples generally contain small cracks and fissures which can significantly lower the homogeneity and thus the small strain stiffness of the sample. According to the particle contact model postulated in section 4.3.2, shear force transfer through the aggregates is accomplished through the parallel orientated platy particles, and this may significantly reduce the initial stiffness of the sample in the small strain range.

Stiffness degradation

It is generally recognized that soils stiffness decreases with increasing strain (e.g. Simpson, 1979; Jardine et al., 1984; Clayton and Khatrush, 1986; Tatsuoka, 1988). An idealized stiffness degradation curve such as the one shown in *Figure 5-8* includes an initial linear plateau at small (<0.002%) strains followed by continued degradation at intermediate and large strains.

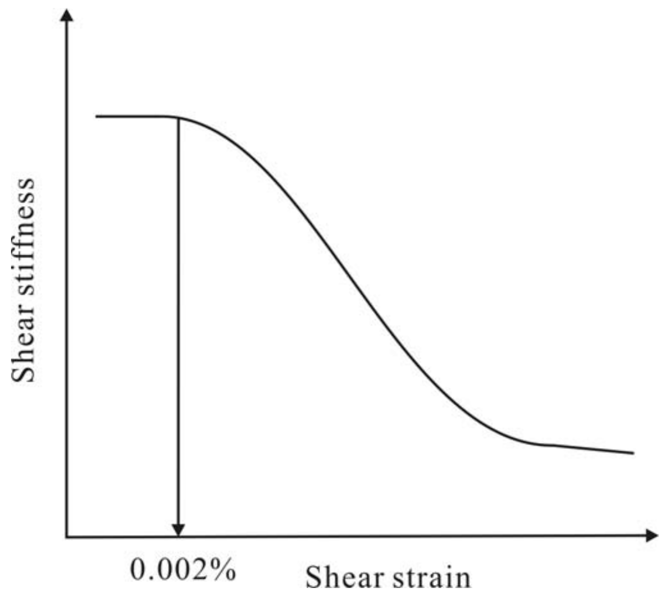

Figure 5-8. Idealized stiffness degradation for soils.

The stiffness of the pond, middle and upper beach samples are attached in *Figures E-10, E-11* and *E-12* respectively in *Appendix E*. The entire stiffness degradation curve can generally be described by values of stiffness at various strain levels and the ratios of these stiffness values. Stiffness of gold tailings at strain levels of 0.001, 0.01 and 0.1 and 1% strain and the stiffness ratios are summarized in *Table E-4*.

From the figures, it can be seen that there is convergence of the data points at higher strain levels. For pond and upper beach samples, no significant differences can be observed in the stiffness of the shear samples at the same effective stress. It is expected that pond material will have similar stiffness values, as the behaviour is dominated by the matrix of platy particles and flocks. The aggregated fabric observed in the middle and upper beach moist tamped samples implies that moist tamped samples should have a lower stiffness than the undisturbed and slurry counterparts.

For the middle beach samples, the stiffness for moist tamped 200 and 400 samples are considerably lower than those for the undisturbed and slurry counterparts, as expected from the aggregated fabric. Contrary to the author's expectations, the stiffness values of upper beach samples are similar. It may be that the higher relative density has suppressed the instability of the unstable contacts in the aggregated fabric.

The shape of the stiffness degradation curves, described by the ratio of the stiffness at various strain levels seems to be similar for all samples. Average values for these ratios for gold tailings are summarized in *Table 5-14*.

Stiffness ratio	Pond	Middle beach	Upper beach
$E_{0.01}/E_{0.001}$	0.78	0.84	0.80
$E_{0.1}/E_{0.001}$	0.34	0.35	0.30
$E_{1.0}/E_{0.001}$	0.06	0.07	0.05

Table 5-14. Average stiffness ratios gold tailing of varying material type.

Stiffness ratios shown in *Table 5-14* indicate that the rate of degradation is not significantly affected by the material type. These ratios are, however, lower than values presented by Heymann (1998) for natural geomaterials. The effect of sample preparation method (fabric) on the stiffness degradation of gold tailings is demonstrated in *Figure 5-9*.

Average stiffness degradation compared on the basis of preparation method shows that moist tamped samples display lower rates of degradation than the undisturbed and slurry samples. It should be noted that a higher stiffness ratio indicates a lower rate of degradation, suggesting that moist tamped samples degrade at a slower rate than the undisturbed and slurry samples. The effect of fabric on stiffness of gold tailings decreases as the sample straining is continued. It seems that the stiffness tends to a unique value dependent on the material type only. As stiffness tends to a constant value, the higher stiffness ratio (lower rate of stiffness degradation) of moist tamped samples may be a function of the lower small strain stiffness observed in the moist tamped samples.

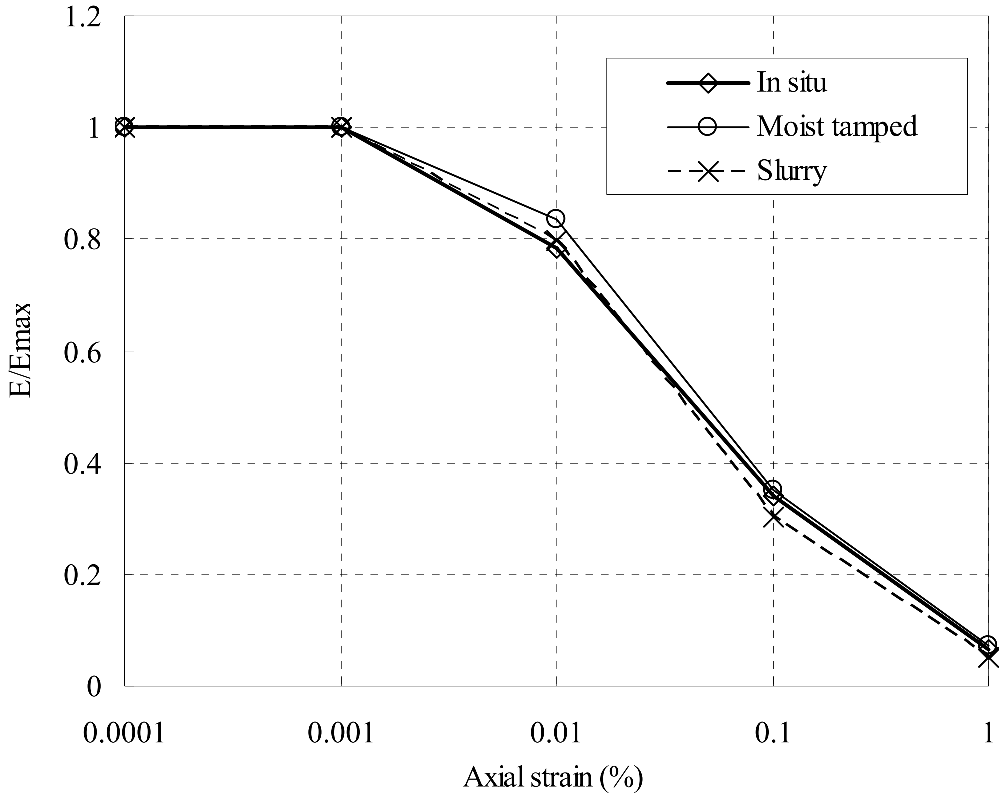

Figure 5-9. Average stiffness ratios gold tailing for varying preparation method.

Normalized stiffness

The stiffness may also be normalized against the current mean effective stress *p'* to eliminate the effects of *p'*. The normalization by division by *p'* is strictly only applicable for large strains and it is generally more conventional to divide by $(p')^n$ where *n* is a function of, among other things, the strain level (Porovic, 1995). As the value of *n* for gold tailings is unclear, it is assumed to be unity. The normalized stiffness of gold tailings is shown in *Figure 5-10.*

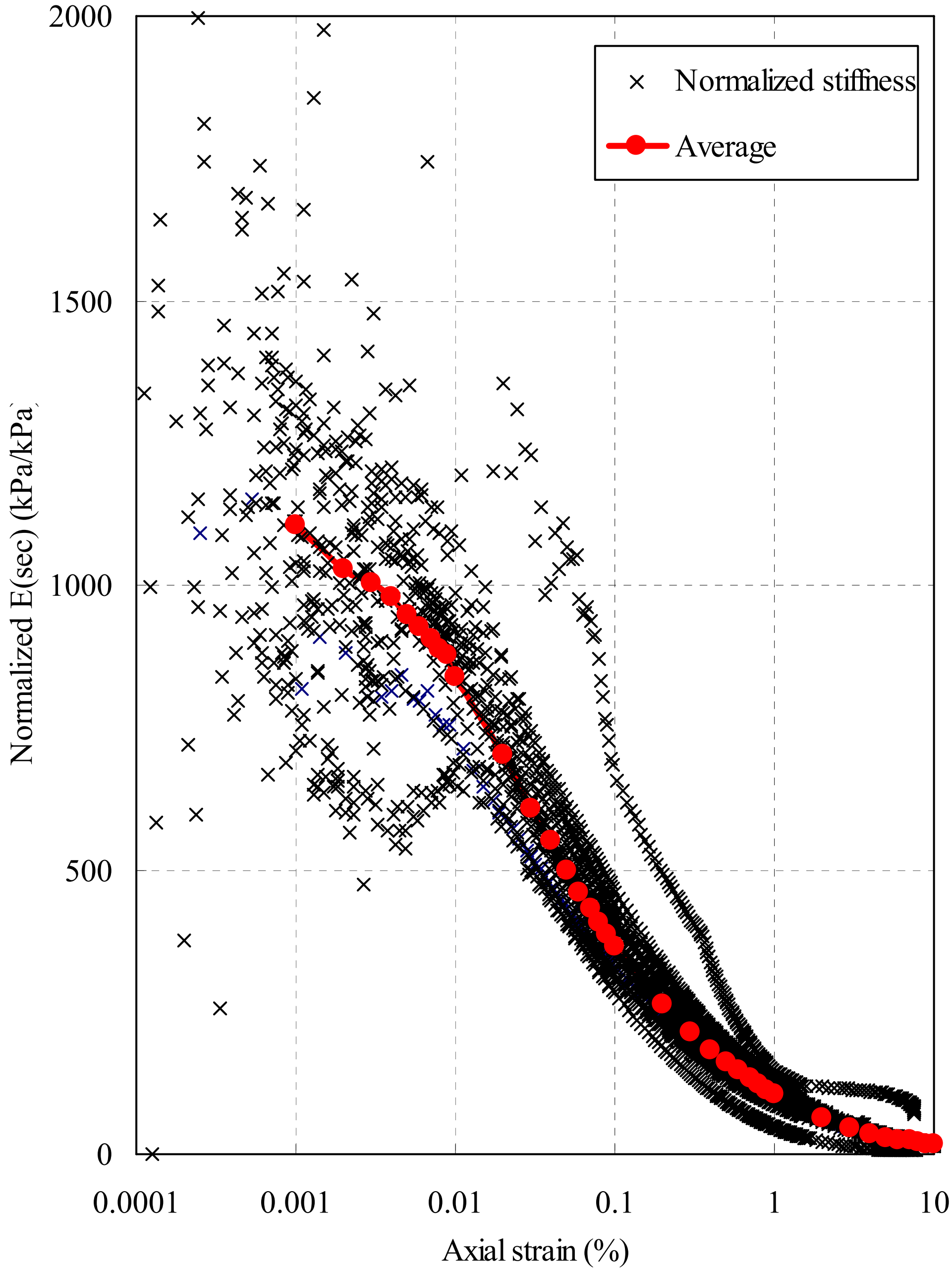

Figure 5-10. Normalized (against current p') stiffness of gold tailings.

The figure shows that there is considerable scatter in the normalized stiffness values, particularly in the small to medium strain range. There is, however, increasing convergence at larger strains. From the figure, is can be concluded that the normalized small strain stiffness (against *p'*) of gold tailings is generally in the range of 1000 to 1500kPa/kPa, with the exception of the two moist tamped samples which show significantly lower values (in the order of

750kPa/kPa). Average values have been plotted as red points. The average was determined for every log scale increment from 0.001 to 10. The two moist tamped samples were also included in the calculation for completeness. The trend for the average curve shows that gold tailings has a normalized small strain stiffness of around 1200kPa/kPa and appears to tend to a linear plateau below the 0.001% strain level.

The figure also shows that the fabric has no significant effect on the stiffness of gold tailings, again with the exception of the two moist tamped samples. It seems that the effect of fabric on small strain stiffness is also a function of the relative density state of the sample.

5.4.3 Stiffness anisotropy

Sample stiffness anisotropy can also be investigated from the stress paths in *p'-q'* space. According to Graham and Houlsby (1983), stiffness anisotropy, can be expressed by the initial slope of the stress path in *p'-q'* space. A soil which is stiffer horizontally will display an initial decrease in *p'* while the stress paths of a soil which is stiffer vertically will move initially to the right displaying an initial increase in the mean effective stress. An isotropic soil will display a near vertical initial stress path. The initial slope of the stress path in *p'-q'* space has been summarized in *Table 5-15* in terms of the initial slope angle θ at 0.05% axial strain following *Equation5-8*:

$$\theta = \tan^{-1}\left(\frac{\delta p_{0.05}}{\delta q_{0.05}}\right) \qquad \textit{Equation 5-8}$$

Assuming 0° at the vertical, a positive θ indicates a positive slope (initially to the right, stiffer vertically) and a negative θ suggests a negative slope (initially to the left, stiffer horizontally).

	Pond		Middle beach		Upper beach	
p_o' (kPa)	200	400	200	400	200	400
Undisturbed	-4.6	-1.9	-2.6	-2.0	0.2	2.9
Moist tamped	1.1	4.5	-3.6	0.3	3.2	4.6
Slurry	4.2	3.9	-3.6	-0.4	11.3	2.4

Table 5-15. Initial stress path slope θ of gold tailings.

From the stress paths, it can be seen that most of the samples show a near vertical initial stress path, with the exception of UB-S-200 which displays strong anisotropy in the vertical direction, probably as a result of the initial one-dimensional consolidation of the slurry sample which resulted in a higher vertical stiffness. It is surprising, however, that the other slurry samples did not show some anisotropy. Moist tamped samples are expected to be isotopic and this is confirmed by the near vertical initial stress paths. It is also surprising that the undisturbed samples were also isotropic. Horizontal layering seen in tailings generally yields a sample which is stiffer in the horizontal direction, and would display an initial decrease in the mean normal stress *p'*. The isotropy observed in the undisturbed sample indicates that layered blocks were avoided during block sampling.

5.5 SHEAR BEHAVIOUR

Gold tailings have generally been regarded as material which shows phase transfer dilation upon undrained shear (Vermeulen, 2001). To the author's knowledge, there has been limited evidence, besides the Merriespruit samples (Wagener et al., 1998), that undisturbed gold tailing samples actually contract and strain-soften during static loading in a triaxial test. This fact has been confirmed to some degree by the outcome of the shear tests. The results of the undrained shear tests are attached in *Appendix E* and are presented in the form of stress-strain relationships of mean normal effective stress *p'* (*Figures E16-E18*), deviatoric stress *q'*(*Figures E13-E15*) and excess pore pressure u_e

(*Figures E19-E21*). Stress paths plotted in *s'-t'* and *p'-q'* space for all three material types have also been attached in *Appendix E* (*Figures E22-E27*).

5.5.1 Available shear strength

The shear behaviour of the undisturbed samples is generally characterized by an increase in deviatoric stress and the generation of excess pore pressure. This is accompanied by a decrease of mean normal effective stress to constant values. Stress paths of undisturbed gold tailings samples indicated initial contraction followed by dilation after phase transfer. Failure generally occurred close to the point of phase transformation without significant dilation occurring. Sample UB-U-400, however, showed a significant increase in deviatoric stress accompanied by a decrease in excess pore pressure and an increase in mean normal effective stress, indicative of strong dilative behaviour.

The shear behaviour of slurry samples is generally similar to the undisturbed counterparts, exhibiting initial contraction followed by phase transfer dilation. Slurry samples, however, showed stronger dilative response than the undisturbed samples. This can be observed by the increase in both deviator stress and mean normal effective stress accompanied by a decrease in excess pore pressure in the pond and upper beach samples. This strong dilative behaviour can also be seen in the stress paths where, following the point of phase transformation, dilation continues substantially before failure occurs.

The shear behaviour of some moist tamped samples was significantly different from that observed for the undisturbed and slurry deposited samples. Moist tamped pond samples showed almost identical behaviour to the undisturbed and slurry deposited counterparts, with initial contraction followed by phase transfer dilation. The shear behaviour of the moist tamped middle and upper beach samples was, however, significantly different to that of the undisturbed and slurry deposited samples. The moist tamped samples demonstrated contractive and strain-softening behaviour as suggested by a decrease in deviatoric stress to a residual value.

Difference in the stress path of undisturbed, moist tamped and slurry samples have significant implications in terms of the available shear strength in the gold tailings samples. The difference in available shear strength is not only between the strain-hardening undisturbed and slurry samples and the strain-softening moist tamped samples, but also between the strain-hardening undisturbed and slurry samples. This is exemplified by the stress paths of middle beach shear-400 samples shown in *Figure 5-11*.

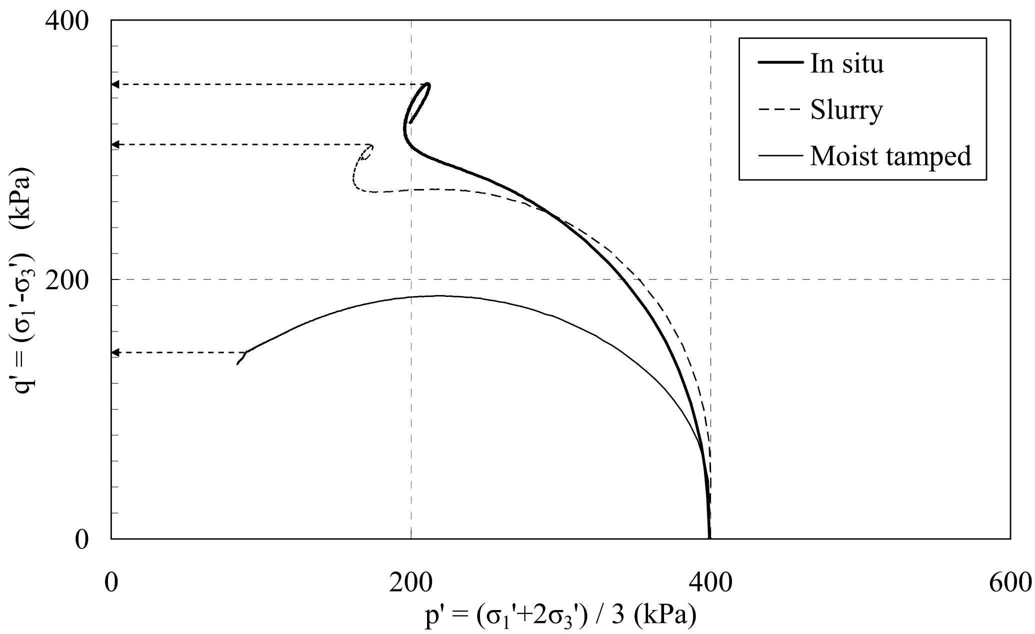

Figure 5-11. Difference in available shear strength between middle beach shear-400 samples.

It can be observed that, although both undisturbed and slurry samples exhibit phase transfer dilation behaviour, the available shear strength between the two samples is still different. The strain-softening moist tamped sample, however, had significantly lower shear strength than the undisturbed and slurry counterparts. The strain-softening behaviour of the moist tamped samples may have been caused by a tendency of the aggregates to move into a denser configuration during shear, resulting in an increase in excess pore pressure and a decrease in effective stress.

The available shear strength of gold tailings samples are summarized in *Table 5-16.*

Available shear strength (kPa)						
	Pond		Middle beach		Upper beach	
p_o' (kPa)	200	400	200	400	200	400
Undisturbed	248.2	365.8	187.8	386.0	142.2	1021.2
Moist tamped	197.5	595.4	82.5	160.9	113.4	542.2
Slurry	313.2	589.7	161.4	354.5	270.9	686.8

Table 5-16. Available shear strength of gold tailings samples.

From these values, it is clear that although the moist tamped pond samples demonstrated strain-hardening behaviour, the available shear strength of moist tamped samples was in general lower (approximately 25%) than those of the undisturbed samples. Comparison between the undisturbed and slurry samples was indecisive, as in some cases the undisturbed sample had higher shear strength and in other cases the slurry sample had higher strength.

5.5.2 Liquefaction behaviour

The difference in stress paths between gold tailings samples can also have significant implications in terms of the liquefaction behaviour of the sample. Liquefaction is a combined effect of contractive behaviour and strain-softening behaviour and this was observed in some of the moist tamped samples. As described in section 2.5.8, cohesionless soils may exhibit three types of behaviour, namely contractive (C), dilative (D) and limited liquefaction (*LL*). The behaviour types observed for gold tailings are summarized in *Table 5-17.*

From these results, it can be seen that none of the undisturbed or slurry samples strain-softened during shear. For these samples the deviatoric stress and excess pore pressure generally increases to a constant value. For the stress paths, *q'* or *t'* increased until failure while *p'* or *s'* first show a decrease and then an increase. It can further be observed that, with the exception of UB-U-400 and UB-S-400, the stress paths of undisturbed and slurry samples are almost identical. Although both UB-U-400 and UB-S-400 samples show different degrees of dilation, the general behaviour is the same.

	Pond		Middle beach		Upper beach	
p_o' (kPa)	200	400	200	400	200	400
Undisturbed	D	D	D	D	D	D
Moist tamped	D	D	C	C	C	D
Slurry	D	D	D	D	D	D

Table 5-17. Liquefaction behaviour type for gold tailings.

From the results, it can also be seen that moist tamped samples may strain-soften during undrained shear, as in the case of MB-MT-200, MB-MT-400 and UB-MT-200 samples. Brittleness index values I_B described in section 2.5.8 for the contractive samples are summarized in *Table 5-18*.

p_o' (kPa)	Middle beach	Upper beach
200	0.26	0.45
400	0.28	NA

Table 5-18. Brittleness index for strain-softening moist tamped samples.

Middle beach samples yielded a similar brittleness index irrespective of confining stress. Upper beach samples, however, displayed a brittleness index of approximately half. No conclusions can be reached with regard to brittleness index as only limited data is available. It can, however, be said that moist tamped samples may lose up to half their peak strength due to strain-softening.

The reason for the observed strain-softening behaviour may be the aggregated fabric described in chapter 4. During shearing, platy particles at the contact points tend to slide relative to each other and the result is a disintegration of the platy contacts between existing aggregates. This causes a tendency of the rotund particles to move into a denser configuration, resulting in an increase in pore-water pressure accompanied by a decrease in deviatoric stress (strength of the sample). This strain-softening behaviour was not visible in all aggregated samples. UB-MT-400 exhibited strong dilative response and this may be a result of dilation due to the sample being in a state of high relative density.

5.5.3 Shear strength

The shear strength of soils is generally described by a cohesion *c'* and a friction angle, *φ'*. The values of *c'* and *φ'* may be obtained from the stress paths in *s'*-*t'* space, as demonstrated in *Figure 5-12* for P-U-400.

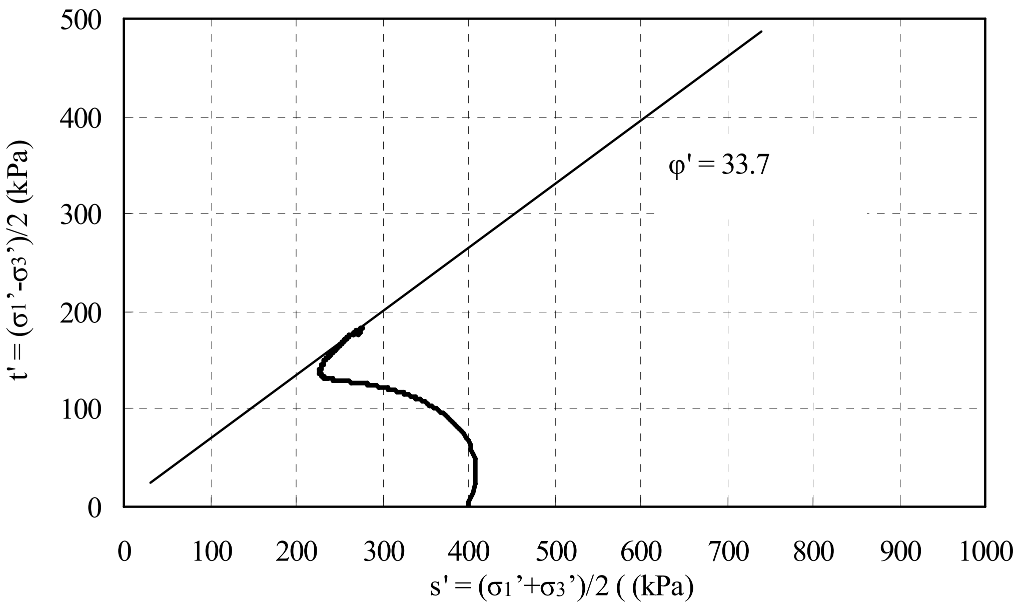

Figure 5-12. Friction angle of P-I-400 sample.

Since gold tailings are generally considered cohesionless, values for the friction angle have been determined based on the assumption that the material has no cohesion. Each sample was determined individually. The friction angles for the moist tamped middle beach samples were not determined as the sample strain-softened and the stress paths did not reach a constant slope at the end of the test. Values of the friction angle for all gold tailings samples are summarized in *Table 5-19.*

	Pond		Middle beach		Upper beach	
p_o' (kPa)	200	400	200	400	200	400
Undisturbed	31.6°	33.7°	32.6°	33.3°	30.9°	31.5°
Moist tamped	30.4°	30.2°	NA	NA	30.9°	30.5°
Slurry	31.6°	31.5°	34.3°	34.3°	31.5°	31.2°
Average	31.5°		33.6°		31.1°	

Table 5-19. Angles of internal friction for gold tailings.

From the results, it is apparent that the angle of friction is independent of the fabric of the sample as a similar trend can be observed for the same material type. It should be noted that visual determination of samples which failed before dilation or strained-softened is difficult and may cause some scatter in the results.

A better indication of the frictional strength is the critical state parameter *M* defining the slope of the stress path in *p'-q'* space. The value of *M* is defined as the constant to which the stress ratio (*q'/p'*) approaches at increasing axial strain. Plots of stress ratio against axial strain for all samples are attached in *Figure E-28, E-29* and *E-30.* Values of *M* are given in *Table 5-20.* It should be noted that the tests were generally terminated at 20% axial strain or when further straining could damage the LVDTs or the triaxial loading ram. Most samples reached the critical state, with stabilizing of the deviatoric stress q' and effective confining stress p' but some samples were terminated close to the critical state. As a result, the position of the critical state line for some samples could not be accurately identified.

	Pond		Middle beach		Upper beach	
p_o' (kPa)	200	400	200	400	200	400
Undisturbed	1.66	1.71	1.73	1.73	1.61	1.61
Moist tamped	1.54	1.48	1.76	1.69	1.61	1.44
Slurry	1.69	1.66	1.88	1.86	1.53	1.58
Average	1.62		1.78		1.56	

Table 5-20. Critical state parameter M for gold tailings.

The friction angle shown in *Table 5-19* is in the same range as those obtained by Vermeulen (2001). Critical state parameter *M,* however, is somewhat higher than Vermeulen's results.

From the stress ratio plots, it can be seen that all samples yield similar values for *M*. For the same material type, values of *M* also seem to be in a similar range despite some scatter. This is consistent with the findings of Zlatovic and Ishihara (1997) which suggested that the shear strength of soils is only governed by the particle characteristics and is independent of the initial fabric.

Values of friction angle and *M* for middle beach slurry material seem slightly higher than other samples in general. This may be a result of the addition of flocculent in the slurry samples. According to Horn and Deere (1962), the absorption of pore fluid may alter the coefficient of surface friction of the particle. This anti-lubricating effect is especially noticeable on quartz and less on muscovite.

An interesting fact was also visible for pond and upper beach samples. The materials have similar values for friction angle or *M* despite the significant difference in the particle properties. Similar results were published by Vermeulen (2001). As observed in the SEM images, pond material consists predominantly of fine platy particles while particles constituting upper beach material are mainly coarse and rotund.

5.6 THE EFFECT OF FLOCCULENT AND DISPERSANT

The addition of flocculent and dispersant was necessary to prepare slurry samples to the required void ratio. This however, changes the pore fluid chemistry and may result in a change in behaviour of the tailings. The effects of pore fluid chemistry will be both mechanical and physico-chemical, with one dominant mechanism (Olson and Mesri, 1970). Mechanical effects occur when the pore fluid absorbs into the particle surface, resulting in a change in the particle's coefficient of surface friction. This was substantiated by the slightly higher friction angle and M values observed for the middle beach slurry samples.

The physico-chemical effect of pore fluid chemistry was visible during sample preparation of the slurry samples. Sedimentation tests showed that the addition of flocculent to the slurry increased the rate of settlement, but also the final settled void ratio. This confirms the observations of Olson and Mesri (1970), who suggested that physico-chemical effects in the slurry influence the original soil structure.

5.7 SUMMARY

The results of the experimental programme have been presented and discussed in terms of the observed fabric of undisturbed and laboratory prepared gold tailings samples. The differences in behaviour have been explained in terms of the particle contact model proposed in Chapter 4. The model, is however, inadequate to explain the observed behaviour fully.

The results also indicate that a limit in density or relative density may exist for samples of different fabric to behave differently. Compaction above that density or relative density limit may destroy any significant fabric, and results in similar fabric and behaviour. Verification of this, however, will require a systematic experimental programme which is beyond the scope of this thesis.

Chapter 6

Conclusions

6.1 BACKGROUND

The fabric and behaviour of undisturbed and laboratory prepared gold tailings have been investigated. The hypothesis states that **accurate simulation of the behaviour of gold tailings under laboratory conditions requires appropriate replication of the material fabric.** An experimental programme was set up to investigate the difference in the fabric of undisturbed and laboratory prepared gold tailings samples and the subsequent fabric effects on the behaviour of gold tailings. Conclusions based on the results and discussions are presented in this chapter.

6.2 CONCLUSIONS FROM EXPERIMENTAL PROGRAMME

- A method of preparing slurry samples was proposed. Traditional methods result in an hourglass shaped sample. The proposed method allows some vertical settlement and one-dimensional consolidation to take place under low suctions. This method, however, may result in some anisotropy in the sample.

- Middle and upper beach slurry samples could not be prepared to the target (in situ) void ratio using tap water alone. Flocculent and dispersant were required to increase the initial void ratio of the slurry samples.

- The addition of dispersant and flocculent affects the amount of settlement for gold tailings in a sedimentation test. The effect of increased dispersant concentration on the settlement rate and amount of settlement is not clear. Increased flocculent concentration increases the rate of settlement, but also decreases the amount of settlement (i.e. settles to a higher void ratio).

- The fabric of gold tailings can be classified into four levels:

  Level 1: non-aggregated and non-orientated.

  Level 2: aggregated but non-orientated.

  Level 3: orientated but non-aggregated.

  Level 4: aggregated and orientated.

  These four levels were used to describe the differences in the fabric of gold tailings, but may be inadequate to describe soils in general. According to the proposed classification system, difference in the fabric can be observed in undisturbed and laboratory prepared gold tailings samples.

- The initial fabric observed before consolidation is not destroyed at large isotropic stresses and strains.

- A particle contact model was postulated based on the observed fabric of gold tailings. According to the model, isotropic compression forces normal to the parallel alignment of the platy particles of the aggregate contact point results in a higher bulk modulus in aggregated gold tailings samples. The transfer of shear forces through the aggregated fabric during initial stages of shear is dampened by the ineffectiveness of the parallel orientated platy particles to transfer forces tangential to the platy particle orientation. At large strains, the platy contact is destroyed. This results in a tendency of the aggregates to collapse into a denser configuration which causes an increase in excess pore pressure accompanied by strain-softening behaviour.

- Volume changes occurred during sample flushing. All undisturbed samples show collapse of the order of 2%, irrespective of material type. Collapse of moist tamped samples is dependent on the relative density state. Samples may show collapse up to 10% at low relative density and may even swell at high relative density.

- The coefficient of consolidation, C_v is a function of the material type as well as the fabric. C_v decreases down the beach, due to a decrease in particle size or a change in particle shape from predominantly rotund to predominantly platy. C_v is significantly increased due to aggregation. Undisturbed C_v values are approximately a third that of moist tamped samples and half that obtained for slurry samples.

- Compression index C_c and critical state parameter λ estimated from the last five data points of consolidation data is dependent on both particle characteristics and fabric. C_c and λ both increases down the beach with a decrease in particle size and an increase in platy particle content. C_c and λ values for the reconstituted moist tamped and slurry samples were on average 40% lower than those for the undisturbed samples. The fabric was not destroyed at large isotropic stresses and strains.

- Coefficient of secondary compression, C_α, for gold tailings ranges between 0.0003 and 0.147 with an average of 0.005. C_α is a function of relative density and particle shape. The value increases with decreasing relative density and increasing platy content. C_α is also lower for moist tamped samples.

- The bulk modulus of gold tailings is dependent on the sample fabric. At low confining stresses, undisturbed samples exhibit higher bulk modulus values than the reconstituted samples and at higher confining stresses the moist tamped samples produce the highest K values. Slurry samples generally produce similar or conservative K values compared with the undisturbed counterparts.

- The small strain stiffness, E_{max}, of gold tailings is a function of the fabric. Slurry samples show higher E_{max} values (approximately 10%) than the undisturbed samples while moist tamped sampled yield lower E_{max} values (approximately 15%) than the undisturbed counterparts. Both laboratory preparation methods may yield erroneous results, but moist tamped samples yield conservative results.

- Moist tamped samples show a lower rate of stiffness degradation at small strains as a result of the lower small strain stiffness values. The samples, however, tend to the same stiffness value upon continued shearing to strains in excess of 10%.

- Gold tailings have normalized (against the current mean normal effective stress) small strain stiffness in the range of 1000 to 1500kPa/kPa with an average value of around 1200kPa/kPa and tend to a linear plateau below the 0.001% strain level.

- Undisturbed and slurry samples at the target (in situ) void ratio only show dilating behaviour. Moist tamped samples at the same state may exhibit contractive and strain-softening behaviour. The available shear strength of slurry samples are in general 20% higher than those for the undisturbed samples while the available shear strength of moist tamped samples are on average 25% lower in comparison with the undisturbed counterpart.

- The angle of friction and *M* are independent of the fabric. The friction angle and *M* may however be influenced by changes in pore fluid chemistry.

- *Table 6-1* shows the recommended laboratory sample preparation method for testing of gold tailings specimens based on the behaviours in comparison with undisturbed samples.

Behaviour type or engineering parameter	Moist tamped	Slurry
Collapse/Swell	👎	NA
Coefficient of consolidation, C_v	👎	👎
Compression Index C_c and λ	👎	👎
Coefficient of secondary compression, C_α	👎	👎
Bulk modulus, K	👎	👍
Small strain stiffness	👎	👎
Medium to large strain stiffness	👍	👍
Friction angle and M	👍	👍
Liquefaction potential	👎	👍
Total	2/9	4/8

👍 Reconstitution method yields similar results in comparison with undisturbed samples

👎 Reconstitution method yields different results in comparison with undisturbed samples

Table 6-1. Recommended laboratory preparation method for triaxial testing of gold tailings.

In conclusion, it appears that neither moist tamping nor slurry deposition can fully replicate the behaviour of undisturbed samples. It is recommended that moist tamping be used when the friction angle of gold tailings is required. The method may also be used for small strain stiffness determinations as the results are conservative. Slurry samples generally replicate the behaviour of undisturbed samples better than moist tamped samples do, but may yield higher C_v, and small strain stiffness values which may be un-conservative. It should be emphasized that the recommendations shown in the table are based on general application and may not be applicable to special cases.

Based on the conclusions presented in this chapter, the hypothesis: **Accurate simulation of the behaviour of gold tailings under laboratory conditions requires appropriate replication of the material fabric,** is accepted.

6.3 RECOMMENDATIONS

Recommendations are made based on the findings of this thesis.

- The effect of additives (dispersant and flocculent) on the behaviour of gold tailings requires further investigation. The assumption was made that the influence of additives on the general behaviour of gold tailings will be minimal, but this assumption needs to be validated.

- During the course of this research, it has become increasingly apparent that the effect of fabric is significantly dependent on the relative density of the samples. For this research the in situ density (or void ratio) was used as a benchmark against which all samples were compared. Research may be required to quantify the correlation between relative density and fabric and its effects on the behaviour of gold tailings.

- This research provides a first step in validating that the observed differences in behaviour is in fact due to a difference in fabric. A more comprehensive method for quantifying or classifying soil fabric may need to be developed in order to investigate the relationship between soil fabric and soil behaviour in more detail.

- The proposed particle contact model needs to be validated or refined. The model may provide explanation for some of the observed similarities or differences in behaviour, but is inadequate to model comprehensively the behaviour of gold tailings.

BLIGHT, G.E. (1969) Shear stability of dumps and dams of gold mining waste, *Transactions of the South African Institute of Civil Engineers*, 11(3), 49-54.

BLIGHT, G.E. (1976) Mitigation of subgrade salts damages thin pavements, *ASCE Transportation Engineering Journal*, 102(TE4), 779-791.

BLIGHT, G.E. (1980) Properties of pumped tailings fill, *Journal of the South African Institute of Mining and Metallurgy,* 79(15), 446-453.

BLIGHT, G.E. (1981) Assessment for environmentally acceptable disposal of mine waste, *The Civil Engineer in South Africa*, 23(10), 480-499.

BLIGHT, G.E. (1997) Destructive mudflows as a consequence of tailings dyke failures, *Proceedings of the Institute of Civil Engineers: Geotechnical Engineering*, 125(1), 9-18.

BLIGHT, G.E. and STEFFEN, K.H. (1979) Geotechnics of gold mine waste disposal, *Current Geotechnical Practice in Mine Waste Disposal,* New York: ASCE, 1-53.

BLIGHT, G.E., VORSTER, K. and THOMPSON, R.R. (1985) Profiles of hydraulic-fill tailings beaches, and seepage through hydraulically sorted tailings, *Journal of the South African Institute of mining and Metallurgy,* 85(5), 157-161.

BLIGHT, G.E. and DU PREEZ, J. (1997) The escape of salt pollution from decommissioned gold residue deposits in South Africa, *Journal of the South African Institute of mining and Metallurgy,* July/August, 201-204.

BREWER, R. and SLEEMAN, J.R. (1988) *Soil Structure and Fabric,* Adelaide: CSIRO.

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

CAFARO, F. and COTECCHIA, F. (2001) Structural degradation and changes in the mechanical behaviour of a stiff clay due to weathering, *Géotechnique*, 51(5), 441-453.

CASTRO, G. and POULOS, S.J. (1977) Factors affecting liquefaction and cyclic mobility, *ASCE Journal of the Geotechnical Engineering Division*, 103(GT6), 501-516.

CETIN, H. (2004) Soil-particle and pore orientations during consolidation of cohesive soil, *Engineering Geology*, 73, 1-11.

CHANG, H.N. (2004) *The Relationship between Shear Wave Velocity and Void Ratio of Gold Tailings*, Thesis (Meng), University of Pretoria.

CHEN, H.W. and VAN ZYL, D.J.A. (1988) Shear strength and volume change behaviour of copper tailings under saturated condition, *In*: D.J.A. VAN ZYL and S.G. VICK (ed.) *Hydraulic Fill Structures*, New York: ASCE, 430-451.

CHILLARIGE, A.R.V., MORGENSTERN, N.R., ROBERTSON, P.K. and CHRISTIAN, H. (1997) Seabed instability due to flow liquefaction in the Fraser River delta, *Canadian Geotechnical Journal*, 34, 520-533.

CLAYTON, C.R.I. and KHATRUSH, S.A. (1986) A new device for measuring local axial strains on triaxial specimens, *Géotechnique*, 36(4), 593-597.

CLAYTON, C.R.I., HIGHT, D.W. and HOPPER, R.J. (1992) Progressive destructuring of Bothkennar clay: Implications for sampling and reconsolidation procedures, *Géotechnique*, 42(2), 219-240.

CLAYTON, C.R.I., MATTHEWS, C. and SIMONS, N.E. (1995) *Site Investigation, 2nd edition*, Oxford: Blackwell Science.

COLLETT, C.V. and HOPE, A.D. (1983) *Engineering Measurements*, Bath: Pitman Books.

COLLINS, K. and MCGOWN, A. (1974) The form and function of microfabric features in a variety of natural soils, *Géotechnique*, 24(2), 223-254.

COTECCHIA, F. and CHANDLER, R.J. (1997). The influence of structure on the pre-failure behaviour of a natural clay, *Géotechnique*, 47(3), 523-544.

CRAIG, R.F. (1997) *Soil Mechanic, 6th edition*, New York: E&FN Spon.

DEWHURST, D.N., CLENNELL, M.B., BROWN, K.M. and WESTBROOK, G.K. (1996) Fabric and hydraulic conductivity of sheared clay, *Géotechnique*, 46(4), 761-768.

DOEBELIN, E.O. (1990) *Measurement systems- Application and Design, 4th edition*, New York: McGraw-Hill.

DYVIK, R. and HØEG, K. (1999) Comparison of tests on undisturbed and reconstituted silt and silty sand, *In* P.V. LADE AND J.A. YAMAMURO (ed.) *Physics and Mechanics of Soil Liquefaction*, Rotterdam: AA Balkema, 159-168.

EAST, D.R., RANSONE, J.W. and CINCILLA, W.A., 1988. The use of the electric piezocone for mine tailings deposits, *In:* J.DE RUITER (ed.) *Penetration Testing 1988,Vol 2*, Rotterdam: AA Balkema, 2, 745-750.

FEARON, R.E. and COOP, M.R. (2000) Reconstitution: What makes an appropriate reference material? *Géotechnique*, 50(4), 471-477.

FEDA, J. (1982) *Mechanics of Particulate Materials*, Amsterdam: Elsevier.

FREDRICH, J.T., MENENDEZ, B. and WANG, T.F. (1995) Imaging the pore structure of geomaterials, *Science,* 268, 276-279.

FROST, J.D., JANG, D-J., CHEN, C-C. and PARK, J-Y. (1998) Quantitative characterization of microstructure evolution, *In*: P.V. LADE and J.A. YAMAMURO (ed.) *Physics and Mechanics of Liquefaction,* Rotterdam: AA Balkema, 169-178.

FULLER, W.B. and THOMPSON, S.E. (1907) The laws of proportioning concrete, *Transactions of the American Society of Civil Engineers, paper no. 1053,* 67-172.

GARGA, V.K. and MCKAY, L.D. (1984) Cyclic triaxial strength of mine tailings, *ASCE Journal of Geotechnical Engineering,* 110(8), 1091-1105.

GARGA, V.K. and ZHANG, H. (1997) Volume change in undrained triaxial tests on sands, *Canadian Geotechnical Journal,* 34, 763-772.

GOWEN, M.J. and WILLIAMSON, J.R.G. (1987) A review of tailings deposition techniques in South Africa and appropriate selection of application, *In:* J.A. WATES and D. BRINK (ed.) *Mining and Industrial Waste Management,* Johannesburg: SAICE, 81-87.

GRAHAM, J. and HOULSBY, G.T. (1983) Elastic anisotropy of a natural clay, *Géotechnique,* 33(2), 165-180.

HAMEL, J.V. and GUNDERSON, J.W. (1973) Shear strength of Homestake slimes tailings, *ASCE Journal of the Soil Mechanics and Foundation Division,* 99, 427-431.

HARTGE, H. and RATHE, I. (1983) Schrumpf- und Scherrisse-Labormessungen, *Geoderma,* 31, 325-336.

HEYMANN, G. (1998) *The Stiffness of Soils and Weak Rocks at Very Small Strains,* Thesis (PhD), University of Surrey.

HEYMANN, G. and CLAYTON, C.R.I. (1999) Block sampling of soils: some practical considerations, *In:* G.R. WARDLE, G.E. BLIGHT and A.B. FOURIE (ed.) *Geotechnics of Developing Africa,* Rotterdam: AA Balkema, 331-339.

HØEG, K., DYVIK, R. and SANDBAEKKEN, G. (2000) Strength of undisturbed versus reconstituted silt and silty sand specimens, *Journal of Geotechnical and Geoenvironmental Engineering,* 126(7), 606-616.

HORN, R., BAUMGARTL, T., KAYSER, R. and BAASCH, S. (1995) Effect of aggregate strength on strength and stress distribution in structured soils, *In:* K.H. HARTGE and B.A. STEWART (ed.) *Soil Structure, its Development and Function,* Florida: CRC Press, 31-52.

IMAI, G. (1981) Experimental studies on sedimentation mechanisms and sediment formation of clay materials, *Soils and Foundations,* 21(1), 7-20.

INGLES, O.G. and LAFEBER, D. (1966) The influence of volume defects on the strength and strength isotropy of stabilized clays, *Engineering Geology,* 305-310.

ISHIHARA, K. (1993) Liquefaction and flow failure during earthquakes, *Géotechnique,* 43(3), 351-415.

JARDINE, R.J. (1995) One perspective of the pre-failure deformation characteristics of some geomaterials, *In:* M. B. JAMIOLKOWSKI, R. LANCELLOTTA AND D. LO PRESTI (ed.) *Prefailure deformation of Geomaterials,* Rotterdam: AA Balkema, 885-886.

JARDINE, R.J., SYMES, N.J. and BURLAND, J.B. (1984) The measurement of soil stiffness in the triaxial apparatus, *Géotechnique,* 34(3), 323-340.

JOHNS, R.A., STEUDE, J.S., CASTANIER, L.M. and ROBERTS, P.V. (1993) Nondestructive measurements of fracture aperture in crystalline rock cores using X-ray computed tomography, *Journal of Geophysical Research*, 98(B2), 1889-1900.

KEALY, C.D. and BUSCH, R. (1979) Evaluation of mine tailings disposal, *In:* ASCE Committee on Embankment Dams and Slopes of the Geotechnical Engineering Division (ed.) *Current Geotechnical Practice in Mine Waste Disposal*, New York: ASCE, 181-201.

KUERBIS, R. (1989) *The drained loading response of sands*, Thesis (Msc), University of British Columbia.

KUERBIS, R. and VAID, Y.P. (1988) Sand sample preparation- the slurry deposition method, *Soils and Foundations*, 28(4), 107-118.

KUERBIS, R., NEGUSSEY, D. and VAID Y.P. (1988) Effect of gradation and fines content on the undrained response of sands, *In*: D.J.A. VAN ZYL AND S.G. VICK (ed.) *Hydraulic Fill Structures*, New York: ASCE, 330-345.

LADD, R.S. (1974) Specimen preparation and liquefaction of sands, *ASCE Journal of the Geotechnical Engineering Division*, 100(GT10), 1180-1184.

LADD, R.S. (1977) Specimen preparation and cyclic stability of sands, *ASCE Journal of the Geotechnical Engineering Division*, 103(GT6), 535-547.

LADE, P.V. and YAMAMURO, J.A. (1997) Effects of non-plastic fines on the static liquefaction of sands, *Canadian Geotechnical Journal*, 34(6), 918-928.

LAMBE, T.W. and WHITMAN, R.V. (1969) *Soil Mechanics*, Ney York: Wiley, 71.

LAWRENCE, G.P. (1978) Stability of soil pores during mercury intrusion pososimetry, *Journal of Soil Science*, 29, 299-304.

LEFEBVRE, G. and DELAGE, P. (1986) The use of mercury intrusion porosimetry for the analysis of soft clay microstructure, *In:* A.S. BALASUBRAMANIAM, S. CHANDRA, and D.T. BERGADO (ed.) *Recent Developments in Laboratory and Field Tests and Analysis of Geotechnical Problems,* Rotterdam: AA Balkema, 31-42.

LEROUEIL, S. and MARQUES, M.E.S. (1996) Importance of strain rate and temperature effects in geotechnical engineering, *In:* T.C. SHEAHAN and V.N. KALIAKIN (ed.) *Measuring and Modeling Time Dependent Soil Behaviour,* New York: ASCE, 1-60.

LUCIA, P.C., DUNCAN, J.M. and SEED, H.B. (1981) Summary of research on case histories of flow failures of mine tailings impoundments, *Mine Waste Disposal Technology,* Washington, DC: US Department of Interior, Bureau of Mines, 46-53.

LUPINI, J.F., SKINNER, A.E. and VAUGHAN, P.R. (1981) The drained residual strength of cohesive soils, *Géotechnique,* 31(2), 181-213.

MAHMOOD, A. and MITCHELL, J.K. (1974) Fabric-property relationships in fine granular materials, *Clays and Clay Minerals*, 22, 397-408.

MAO, X. and FAHEY, M (1999) A method of reconstituting aragonite soil using a synthetic flocculant, *Géotechnique*, 49(1), 15-32.

MATYAS, E.L., READES, D.W. and WESCH, D.E. (1984) Geotechnical parameters and behaviour of uranium tailings, *Canadian Geotechnical Journal*, 21(3), 489-504.

MCPHAIL, G.I. and WAGNER, J.C. (1989) Disposal of residues, *In:* G.G. STANLEY (ed.) *The Extractive Metallurgy of Gold in South Africa, volume 2*, Johannesburg: The Chamber of Mines of South Africa, 655-707.

MESRI, G. and GODLEWSKI, P.M. (1977) Time and stress-compressibility interrelationship, *ASCE Journal of the Geotechnical Engineering Division*, 105(1), 106-113.

MESRI, G., FENG, T.W. and BENAK, J.M. (1990) Post densification penetration resistance of clean densified sands, *ASCE Journal of Geotechnical Engineering*, 116(7), 1095-1115.

MITCHELL, J.K. (1956) The fabric of natural clays and its relation to engineering properties, *Proceedings of the Highway Research Board*, 35, 693-713.

MITCHELL, J.K. (1976) *Fundamentals of Soil Behaviour*, New York: Wiley.

MITCHELL, J.K. (1993) *Fundamentals of Soil Behaviour*, *2nd edition*, New York Wiley.

MITCHELL, J.K. and SOGA, K. (2005) *Fundamentals of Soil Behaviour, 3rd edition*, New York: Wiley.

MITCHELL, J.K., CHATOIAN, J.M. and CARPENTER, G.C. (1976) *The influence of sand fabric on liquefaction behaviour*, *Report No. TE 76-1*, Vicksburg, MS: Department of Civil Engineering, University of California, Berkley.

MITTEL, H.K. and MORGENSTERN, N.R. (1975) Parameters for the design of tailings dams, *Canadian Geotechnical Journal*, 12, 277-293.

MIURA, S. and TOKI, S. (1982) A sample preparation method and its effect on static and cyclic deformation-strength properties of sand, *Soils and Foundations*, 22(1), 61-77.

MOORE, C.A. (1971) Effect of mica on k_o compressibility of two soils, *Journal of Soil Mechanics and Foundation Division*, ASCE 97(SM9), 1275-1291.

MORGENSTERN, N.R. and TCHALENKO, J.S. (1967) Microscopic structures in kaolin subjected to direct shear, *Géotechnique*, 17, 309-328.

MULILIS, J.P., SEED, H.B., CHAN, C.K., MITCHELL, J.K. and ARULANANDAN, K. (1977) Effects of sample preparation on sand liquefaction, *ASCE Journal of the Geotechnical Engineering Division*, 103(GT2), 91-108.

ODA, M. (1972) Deformation mechanism of sands in triaxial compression tests, *Soils and Foundations*, 12(4), 45-63.

ODA, M. and KAZAMA, H. (1998) Microstructure of shear band and its relation to the mechanics of dilatency and failure of granular soils, *Géotechnique*, 48(4), 465-481.

ODA, M., KOISHIKAWA, I. and HIGUCHI, T. (1978) Experimental study of anisotropic shear strength of sands by plane strain test, *Soils and Foundations*, 18(1), 25-38.

ODA, M., TAKEMURA, T. and TAKAHASHI, M. (2004) Microstructure in shear band observed by microfocus X-ray computed tomography, *Géotechnique*, 54(8): 539-542.

OLSEN, R.E. and MESRI, G. (1970) Mechanisms controlling compressibility of clays, *ASCE Journal of the Soil Mechanics and Foundation Division*, 96, 1853-1878.

PAN, Y-W. and DONG, J-J. (1999) A micromechanics-based methodology for evaluating the fabric of granular material, *Géotechnique*, 49(6), 761-775.

PAPAGEORGIOU, G. (2004) *Liquefaction Assessment and Flume Modelling of the Merriespruit Gold and Bafokeng Platinum Tailings*, Thesis (PhD), University of Witwatersrand.

PENMAN, A.D.M. (1994) Tailings dams, some aspects of their design and construction, *In:* K.R.SAXENA (ed.) *Geotechnical Engineering: Emerging Trends in Design and Practice*, Rotterdam: AA Balkema, 247-277.

PETTIBONE, H. and KEALY, D. (1971) Engineering properties of mine tailings, *ASCE Journal of the Soil Mechanics and Foundation Division*, 97, 1207-1225.

POROVIC, E. (1995) *Investigations of soil behaviour using a resonant column torsional shear hollow cylinder apparatus*, Thesis (PhD), Imperial College of Science, Technology and Medicine, University of London.

QIU, Y. and SEGO, D.C. (2001) Laboratory properties of mine tailings, *Canadian Geotechnical Journal*, 38(1), 183-190.

RAD, N.S. and TUMAY, M.T. (1987) Factors affecting sand specimen preparation by raining, *ASTM Geotechnical Testing Journal*, 10(1), 31-57.

RIPLEY, E.A., REDMANN, R.E. and MAXWELL, J. (1982) Environmental impact of mining in Canada, *The National Impact of Mining Series*, Kingston: Center for Resource Studies.

RIPPA, F. and PICARELLI, L. (1977) Some considerations on index properties of southern Italian shales, *Proceedings, International Symposium on the Geotechnics of Structurally Complex Formations*, 19- 21 September, 1977, Capri, 401-406.

RITCEY, G.M. (1989) *Tailings Management, Problems and solutions in the mining industry,* Excerpts: Elsevier.

ROBERTSON, A.M. (1987) Alternative acid mine drainage abatement measures, *Proceedings, Mine Land Reclamation Confrence,* Campbell River, British Columbia.

ROBRINSKI, E.I. (1975) Thickened discharge- A new approach to tailings disposal, *The Canadian Institute of Mining and Metallurgy Bullitin,* 68, 47-59.

ROWE, P.W. (1972) The relevance of soil fabric to site investigation practice, *Géotechnique,* 22(2), 195-300.

RUEHRWEIN, W.A. and WARD, D.W. (1952) Mechanism of clay aggregation by polyelectrolytes, *Soil Science,* 73, 485-492.

SANTAMARINA, J.C. and CASCANTE, G. (1996) Stress anisotropy and wave propagation: a micromechanical view, *Canadian Geotechnical Journal,* 33(5), 770-782.

SCHIFFMEN, R.L., VICKS, S.G. and GIBSON, R.E. (1988) Behaviour and properties of hydraulic fills, *In*: D.J.A. VAN ZYL AND S.G. VICK (ed.) *Hydraulic Fill Structures*, New York: ASCE, 166-202.

SEED, H.B. and DE ALBA, P. (1986) Use of SPT and CPT tests for evaluating the liquefaction resistance of sands, *In:* S.P. CLEMENCE (ed.) *Use of in situ tests in geotechnical engineering,* Philadelphia: ASTM, 281-302.

SEED, H.B., MITCHELL, J.K. and CHAN C.K. (1962) Swell and swell pressure characteristics of compacted clays, *Highway Research Board Bullitin,* 313, 12-39.

SHERARD, J.L., DUNNIGAN, L.P. and TALBOT, J.R. (1984) Basic properties of sand and gravel filters, *ASCE Journal of Geotechnical Engineering*, 110(6), 700-718.

SHERWOOD, P.T. and RYLEY, D.M. (1968) *An examination of cone-penetrometer methods for determining the liquid limit of soils, TRRL Report LR 233*, Crowthorne: Transport and Road Research Laboratory.

SIMPSON, B., O'RIORDAN, N.J. and CROFT, D.D. (1979) A computer model for the analysis of ground movements in London Clay, *Géotechnique,* 29(2), 149-175.

SLADEN, J.A., D'HOLLANDER, R.D. and KRAHN, J. (1985) The liquefaction of sands: a collapse surface approach, *Canadian Geotechnical Journal*, 22(4), 564-578.

SMITH, E.S. (1972) Tailings disposal- Failures and lessons, *In:* C.L. APLIN and G.O. ARGALL (ed.) *Tailings Disposal Today*, Arizona, Miller Freeman, 356-376.

STANLEY, G.G. (ed.) (1987) The Extractive Metallurgy of Gold in South Africa, *The S.A. Institute of Mining and Metallurgy Monograph Series M7,* Johahannesberg: The Chamber of Mines.

STONE, K.J.L., RANDOLPH, M.F. TOH, S. and SALES, A.A. (1994) Evaluation of consolidation behaviour of mine tailings, *ASCE Journal of Geotechnical Engineering,* 120(3), 473-490.

SULLY, J.P. (1985) Geotechnical aspects of remedial design for a gold tailings dam, *International Journal for Numerical and Analytical Methods in Geomechanics,* 9(6), 589-598.

SYDENHAM, P.H. (1982) *Standardization of Measurements: Fundamentals and Practice,* Chichester: John Wiley and Sons.

TATSUOKA, F. (1988) Some recent developments in triaxial testing systems for cohesionless soils, *Advanced Triaxial Testing of Soils and Rocks*, Philadelphia: ASTM, 7-67.

TATSUOKA, F., OCHI, K., FUJII, S. and OKAMATO, M. (1986) Cyclic undrained triaxial and torsional shear strength of sands for different sample preparation methods, *Soils and Foundations*, 6(3), 23-41.

TERZAGHI, K. (1925) Zur charakteristik der bausande, *Zeitschrift des Oesterreichischen Ingenieur und Architekten Vereines*, 33-34.

TERZAGHI, K. and PECK, R.B. (1948) *Soil Mechanics in Engineering Practice, 2nd edition*, New York: John Wiley and Sons.

THEVANAYAGAM, S., SHENTHAN, T., MOHANS, S. and LIANG, J. (2002) Undrained fragility of clean sands, silty sands and sandy silts, *Journal of Geotechnical and Geoenvironmental Engineering*, 128, 849-859.

THERON, M. (2004) *The Influence of Fine Platy Particles on the Behaviour of a Rotund Sand*, Thesis (PhD), University of Southampton.

THERON, M., HEYMANN, G. and CLAYTON, C.R.I. (2004) The small strain stiffness of gold tailings, *In:* A.V. DA FONSECA and P.W. MAYNE (ed.) *Geotechnical and Geophysical Site Characterization, volume 1*, Rotterdam: Millpress, 575-580.

VAID, Y.P. and NEGUSSEY, D. (1984) Relative density of air and water pluviated sand, *Soils and Foundations*, 24(2): 101-105.

VAID, Y.P. and NEGUSSEY, D. (1988) Preparation of reconstituted sand specimens, *Advanced Triaxial Testing of Soils and Rocks*, Phiadelphia: ASTM, 405-417.

VAID, Y.P. and PILLAI, V.S. (1992) Critical state of sands, *Géotechnique*, 42(4), 655-663.

VAID, Y.P. and SIVATHAYALAN, S. (1996) Static and cyclic liquefaction potential of Fraser Delta sand in simple shear and triaxial tests, *Canadian Geotechnical Journal*, 33, 281-289.

VAID, Y.P. and SIVATHAYALAN, S. (2000) Fundamental factors affecting liquefaction susceptibility of sands, *Canadian Geotechnical Journal,* 37: 592-606.

VAID, Y.P., CHUNG, E.K.F. and KUERBIS, R.H. (1990) Stress path and steady state, *Canadian Geotechnical Journal,* 27: 1-7.

VAID, Y.P., SIVATHAYALAN, S. and STEDMAN, D. (1999) Influence of specimen reconstituting method on the undrained response of sands, *Geotechnical Testing Journal*, 22(3): 187-195.

VAN ZYL, D. (1993) Mine waste disposal, *In*: D.E. DANIEL (ed.) *Geotechnical Practice for Waste Disposal,* London: Chapman Hall, 269-286.

VAUGHAN, P.R. (1985) Mechanical and hydraulic properties of in situ residual soils, *Proceedings, 1st International Conference on Geomechanics in Tropical, Lateritic and Saprolitic Soils,* February, 1985, Brazil, 231-263.

VAUGHAN, P.R. (1997) Panel discussions: Sedimentation of tailings, *Proceedings, 14th International Conference on Soil Mechanics and Foundation Engineering*, September, 1997, Hamburg, Germany, 2561-2562.

VAUGHAN, P.R., MACCARINI, M. and MOKHTAR, S.M. (1988) Indexing the engineering properties of residual soils, *Quarterly Journal of Engineering Geology,* 21(1): 69-84.

VERMEULEN, N.J. (2001) *The Composition and State of Gold Tailings*, Thesis (Ph.D), University of Pretoria.

VICKS, S.G. (1983) *Planning, Design and Analysis of Tailings Dams*, New York, Wiley.

VICKS, S.G. (1990) *Planning, Design and Analysis of Tailings Dams,2nd edition*, New York, Wiley.

WAGENER, F.M., CRAIG, H.J., BLIGHT, G.E., MCPHAIL, G., WILLIAMS, A.A.B. and STRYDOM, J.H. (1998) The Merriespruit tailings dam failure- A review, *Proceedings of the 5th International Conference on Tailings and Mine Waste*, Fort Collins, USA, 925-952.

WAGENER, F. and JACOBSZ, S.W. (1999) Penstock failure on gold tailings dams in South Africa, *Proceedings, 12th Regional Conference for Africa on Soil Mechanics and Geotechnical Engineering*, October, 1999, Durban, South Africa, 95-100.

WESTRAAD, D. (2004) *Suction induced shear strength of gold mine tailings*, Thesis (Meng.), University of Pretoria.

WOOD, F.M. (1999) *Influence of Specimen Reconstitution Method on the Undrained Response and Microstructure of Silty Sands*, Thesis (Meng.), Clarkson University.

WOOD, F.M. and YAMAMURO, J.A. (1999) The effect of depositional method on the liquefaction behaviour of silty sands, *Proceedings, 13th ASCE Engineering Mechanics Conference*, June, 1999, Baltimore,

YAMAMURO, J.A. and LADE, P.V. (1997) Static liquefaction of very loose sands, *Canadian Geotechnical Journal*, 34: 905-917.

Chapter 7

References

AKERS, R.J. (1975) *Flocculation*, London: The Institute of Chemical Engineers.

ANANDARAJAH, A. and KUGANENTHIRA, N. (1995) Some aspects of fabric anisotropy of soils, *Géotechnique*, 45(1), 69-81.

ANANDARAJAH, A. and ZHAO, D. (2000) Triaxial behaviour of kaolinite in different pore fluids, *Journal of Geotechnical and Geoenvoironmental Engineering*, 126(2), 148-156.

ARCH, J. and MALTMAN, A.J. (1990) Anisotropic permeability and tortuosity in deformed wet sediments, *Journal of Geophysical Research*, 95(B6), 9035-9047.

BABEL, U., BENECKE, P., HARTGE, K.H., HORN, R. and WEICHMANN, H. (1995) Determination of soil structure at various scales, *In*: K.H. HARTGE AND B.A. STEWART (ed.) *Soil Structure, its Development and Function*, Florida: CRC Press, 1-10.

BIOT, M.A. (1941) General theory of three-dimensional consolidation, *Journal of Applied Physics*, 12, 155-164.

BISHOP, A.W. (1967) Progressive Failure- with special reference to machanism causing it, *In: Proceedings of the Geotechnical Conference, Oslo, Norway*, 2, 142-150.

YI, F. (1991) Behaviour of compacted collapsible soils subjected to water infiltration, Thesis (Dr. Eng.), University of Tokyo.

YOSHIMI, Y. TOKIMATSU, K. and HOSAKA, Y. (1989) Evaluation of liquefaction resistance of clean sands based on high quality undisturbed samples, *Soils and Foundations,* 29(1): 3-104.

ZLATOVIC, S. and ISHIHARA, K. (1997) Normalized behaviour of very loose non-plastic soils: effects of fabric, *Soils and Foundations,* 37(4): 47-56.

ZLATOVIC, S. and ISHIHARA, K. (1998) Flow failure- Some data on onset conditions, *In:* B. MARIC, Z. LISAC and A. SZAVITS-NOSSAN (ed.) *Geotechnical Hazards, Proceedings, 11th Danube-European Conference on Soil Mechanics and Foundation Engineering,* May, 1998, Croatia.

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